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A PROCEDURE FOR THE ASSESSMENT OF THE BEHAVIOUR FACTOR FOR STEEL MOMENT RESISTING FRAME SYSTEMS BASED ON PUSHOVER CURVES

A Dissertation Presented

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Dedicated To

My Father, who took keen interest in my work and guided me all along, till the completion of the book by providing all the necessary information for developing a good system.

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Finally, to my incredible **Wife** and partner in everything where words fail me, but I will try anyway. Having her in my life has been the most precious gift that anyone could have ever asked for.

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ABSTRACT

The purpose of this dissertation was to calculate/obtain the behaviour factor for steel moment resisting frame systems by means of re-analysis of a pushover curve which can be conveniently applied in everyday practice due to its simplicity. Presently, FEMA P-695 provides a procedure for the definition of the behaviour factor by re-analysis of the push over curve. In Europe, such a method is not yet proposed by EC8 and only ECCS provides some recommendations. Furthermore, the reference parameters to obtain the behaviour factor are defined by each code in a different way which results in a wide variety of possible choices and resulting output. In order to overcome this kind of problem, first, all possible definitions of the reference overstrength and ductility parameters which may be used for all types of structures will be discussed in this research. Then, standard re-analysis procedure of the results of the pushover analysis will be introduced.

The influence of different choices of such parameters in the assessment of the behaviour factor will be investigated in 102 case studies of different composite steel-concrete *MRF* buildings, each designed with increasing values of the behaviour factor from 2.0 to 7.0. The case studies employ 96 conventional steel-concrete *MRF* buildings and 6 non-conventional *MRF* structures. Nonlinear static analysis (*PushOver*) is conducted for each type of structure to provide an estimate of overstrength and the ductility factor based on different possible selection of the reference parameters defined in the current seismic codes. The re-analysis of the obtained results is presented and discussed.

On the other hand, the current structural design procedure, however, doesn't assure that the "*actual*" behaviour factor (= the actual ductility of the structure) will coincide with the "*assumed*" one (= behaviour factor given by the codes), so that most of the time results in over-designing the structures. As a result, design engineers never tend to optimize their design using more advanced procedures, due to the complexity of these methods. This might be overcome through the definition of a new step in the design scenario before seismic analysis in order to identify the real behaviour factor based on the proposed methodology.

In the same way, the initial construction cost of a building has been always an important parameter. The design optimization objective is to minimize the initial structural cost/weight. It is evident that the lower the actions in a structure, the less need for materials required for a

strength based design of the structure. But, it is not always so, due to other factors, such as the limits imposed by the codes and allowable inter-storey drift, which will prevent reduction of member's sizes and stiffness. The current research will present a novel strategy for an optimal design of steel structures in high seismic zones through balancing the initial cost and the lifetime seismic damage. The investigation will be performed in order to explore the relationship between the expected initial material cost and the design behaviour factor.

Incremental Dynamic Analysis is subsequently performed to obtain a refined representation of response throughout the desired range of seismic intensity measure. The so-called average spectral acceleration is used to illustrate the severity of the ground motions. The dynamic analysis results for the considered modes of failure are conveniently summarised into fragility functions, which are further convoluted with the seismic hazard function in order to derive the associated mean annual frequency of exceedance. Further the Ballio-Setti's methodology is investigated to approve the behaviour factor obtained by means of re-analysis of the pushover curve.

The results introduce an optimal method to define a consistent behaviour factor for moment resisting frame systems based on the re-analysis of the pushover curves. The results are also approved by the incremental dynamic analysis. Finally, a novel strategy for an optimal design of steel structures in high seismic zones through balancing the initial cost and the lifetime seismic damage is presented.

Keywords: Behaviour Factor, Pushover, Nonlinear Analysis, Incremental Dynamic Analysis, MRF Systems and Behaviour Factor Assessment

SOMMARIO

Scopo di questa tesi è la valutazione del fattore di struttura (q-factor) per strutture a telaio in acciaio mediante una rielaborazione della curva pushover che possa essere convenientemente applicata nella pratica quotidiana grazie alla sua semplicità.

Ad oggi, solo la normativa FEMA P-695 fornisce una procedura per la definizione del fattore q basata sulla rielaborazione della curva pushover. In Europa, un metodo simile non è previsto dalll'EC8, mentre l'ECCS fornisce solo alcune raccomandazioni in merito.

Inoltre, le varie Normative antisismiche più recenti definiscono in modo differente i parametri di riferimento per la valutazione del fattore di struttura. Questo si traduce in un'ampia varietà di scelte possibili, che vengono lasciate al progettista; conseguenza di ciò è che, nella pratica progettuale, il q-factor per una stessa struttura può essere attualmente stimato in modo differente da differenti professionisti, a seconda della Normativa adottata e della combinazione dei parametri di riferimento scelti.

Per superare questo problema, in questa ricerca verranno discusse tutte le possibili definizioni dei parametri di sovra-resistenza e duttilità, proposti da varie Normative, che possono essere utilizzati per vari tipi di strutture. Quindi, verrà introdotta una procedura standard di rielaborazione dei risultati dell'analisi pushover.

L'influenza delle diverse scelte di tali parametri nella valutazione del fattore di comportamento è stata studiata in 102 casi di studio di diversi edifici con struttura intelaiata tipo MRF composta acciaio-calcestruzzo, ciascuno progettato con valori crescenti del fattore di struttura, da 2.0 a 7.0. I casi studio analizzati comprendono 96 edifici MRF a struttura acciaio-calcestruzzo convenzionale e 6 strutture MRF non convenzionali con collegamenti dissipativi / fusibili strutturali.

L'analisi statica non lineare (PushOver) è stata condotta per le varie strutture, ottenendo, in funzione delle diverse possibili combinazioni dei parametri di riferimento, una stima della sovraresistenza e del fattore di duttilità.

D'altro canto, la pratica corrente non assicura che il fattore di comportamento "effettivo" (= la duttilità effettiva della struttura) coinciderà con quello "assunto" in fase di progetto (= fattore di comportamento derivato dalle indicazioni di normativa).

Ciò crea una serie di incertezze in merito al dimensionamento delle strutture, che potrebbero essere superate solo a patto di adottare metodi di progettazione avanzati, che sono molte volte incompatibili con la normale pratica progettuale, e che sono adatti esclusivamente a scopo scientifico.

La proposta sviluppata in questo lavoro di tesi mira al superamento di tale situazione mediante una stima dell'effettivo valore del coefficiente di struttura da effettuarsi mediante un'analisi di push-over sul modello strutturale, a valle del dimensionamento per carichi gravitazionali, ma prima di eseguire l'analisi sismica.

Ovviamente, l'individuazione della "combinazione ottimale" dei parametri di riferimento può essere effettuata solo sulla base di un confronto (in termini di q-factor) tra i risultati ottenibili con l'analisi semplificata (push-over) e quello "effettivo" della struttura stessa. Quest'ultimo valore di confronto, in questa tesi è stato ottenuto mediante una serie di Analisi Dinamiche Incrementali (IDA), considerando i possibili effetti non-lineari, in termini di comportamento dei materiali e di geometria delle strutture in esame.

Dal confronto tra i risultati ottenuti mediante analisi push-over e IDA è stato possibile identificare il metodo di combinazione dei parametri di riferimento che porta ad una minimizzazione dell'errore in termini di fattore di struttura.

Infine, viene presentata una nuova strategia per una progettazione ottimale delle strutture in acciaio in zone sismiche attraverso un bilanciamento tra costo iniziale e danno conseguente ad eventi sismici.

Il costo iniziale di costruzione di un edificio è sempre stato un parametro importante. L'obiettivo di ottimizzazione del progetto è la minimizzazione del costo / peso strutturale iniziale. È evidente che minore sono le azioni che impegnano una struttura, minore è la "richiesta" in termini di sezioni portanti (=peso dei materiali) necessari secondo una progettazione in termini di resistenza.

Altri fattori, quali ad esempio le limitazioni imposte dalla normativa agli spostamenti di interpiano, impediscono una riduzione delle dimensioni e della rigidezza delle membrature.

L'ultima parte di questa tesi presenta una nuova strategia per una progettazione ottimale delle strutture in acciaio in zone ad elevata sismicità, attraverso un bilanciamento del costo iniziale e del danno sismico nel corso della vita utile. L'indagine verrà condotta al fine di esplorare la relazione tra il costo del materiale iniziale previsto e il fattore di struttura adottato in fase di progetto.

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List of Symbols and Abbreviations

q	behaviour factor
q_{des}	design behaviour factor
MRFS	moment resisting frame system
V	seismic design base shear
Z	seismic zone factor
C	building’s natural period of vibration
W	building’s weight
S	spectral response acceleration
F	site coefficient
I	importance factor
R	response modification factor also called force reduction factor
T	natural period of vibration
EC8	EN 1998-1:2004
FEMA	federal emergency management agency
AIJ	seismic loading—strong motion prediction and building response
q_{μ}	ductility factor
q_{Ω}	overstrength factor
F_e	strength force if the structure remains elastic during the earthquake
F_1	significant yield strength
q_{ξ}	damping dependent factor
d_m	displacement corresponding to the maximum strength
d_y	displacement corresponding to the knee-point of the idealized bilinear elastic-plastic behavior curve
Fy	knee point of the idealized bilinear yield strength
μ	system ductility
SDOFS	single degree of freedom system
MDOFS	multiple degree of freedom system
$\overline{a}_{max}$	maximum peak ground acceleration
α_d	maximum design acceleration
I_D^L	dynamic linear elastic analysis

I_D^{NL}	dynamic non-linear analysis
$\in_{Pi}$	plastic deformation
$\in_y$	yield deformation
A	area of the cross section
ECCS	European convention for constructional steelwork
IDA	incremental dynamic analysis
a_g	peak ground acceleration
$AvgSa$	average spectral acceleration
IM	intensity measure
T_{Ri}	can be selected as linearly spaced within the range of [t_l, t_h].
T_L	low period near the minimum second mode of the structure
T_H	high period which is close to 1.5 times of the maximum of the first mode period
S_a	spectral acceleration
EDP	engineering demand parameter
GC	global collapse limit state
LS	life safety limit state
ASCE	American society of civil engineers
NC	near collapse
SD	significant damage
MAF	mean annual frequency
λ_{DSlim}	maximum allowable *maf* limit whose exceedance signals violation of the damage state
K_x	standard normal variants associated with a confidence level of x%, kx=φ-1(x)
k	local slope of the hazard curve in log-log space
β_u	total dispersion due to uncertainty, assuming log-normality $$\beta_u = \sqrt{\beta_{TD}^2 + \beta_{DR}^2 + \beta_{AS}^2 + \beta_C^2}$$
β_{TD}	test data quality rating
β_{DR}	design rules quality rating
β_{AS}	archetype sample size
β_C	element capacity test dispersion

m	mass of structure
R(T)	normalized spectrum $R(T) = \frac{R_0}{(T/_{T_0})^k}$
R_0, T_0 and k	parameters that define the design spectrum
α	ground acceleration
α_1	ground acceleration in which the first yield or plastic mechanism occurs (local plasticization)
CBFs	braced frame system
H	structural height
B	structural width
HSFG	high strength friction grip
N_{Ed}	normal design force
N_{Rd}	design values of the resistance to normal forces
$M_{y,Ed}$	design bending moment in y-axis
$M_{y,Rd}$	design values of the resistance to bending moment in y-axis
$M_{z,Ed}$	design bending moment in z-axis
$M_{z,Rd}$	design values of the resistance to bending moment in z-axis.
Nb,Rd	design buckling resistance of the compression member $N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$
χ	reduction factor for the relevant buckling mode
M_{Ed}	design value of the moment
$M_{b,Rd}$	design buckling resistance moment
h	storey height
ν	reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.
d_r	design inter-storey drift
P_{tot}	total gravity load at and above the storey considered in the seismic design situation
V_{tot}	total seismic storey shear
ΣM_{Rc}	sum of the design values of the bending resistance of the columns framing the joint
ΣM_{Rb}	sum of the design values of the bending resistance of the beams framing the joint
RSA	response spectrum analysis

t_f	flange thickness of the beam splices
L_0	free buckling length
M_p	elastic moment of the rectangular cross-section of the plate $$M_p = \frac{b_f * t_f^{\ 2}}{4} * f_y$$
b_f	width of the flange plate
$M^{+}_{Rd,fuse}$	sagging resistant moment of the bolted beam splice
f_{yd}	design yield strength of the structural steel according to en1993-1-1
z	distance between the flange plate and the center of gravity of the rebar layers $z = h_a + h_p + \frac{h_c}{2}$
h_a	height of the steel beam
h_p	height composite beam
h_c	height of concrete slab
V_{Ed}	total shear force $V_{Ed} = V_{Ed,M} + V_{Ed,G}$
$V_{Ed,M}$	ear force due to moment resistance of the fuse $V_{Ed,M} = \frac{M^{+}_{fuse,Rd} - M^{-}_{fuse,Rd}}{d}$
$V_{Ed,G}$	shear force due to gravity loads
d	distance between the fuses.
$G_{k,j}$	gravity load effects in seismic design situation
$Q_{k,i}$	movable load effects in seismic design situation
E	effect of the seismic action including accidental torsional effects
ABSSUM	absolute sum
SRSS	square root of sum of squares
CQC	complete quadratic combination
S	soil factor
β	lower bound factor for the horizontal design spectrum
η	damping correction factor
ULS	ultimate limit state
SLS	serviceability limit state

Chapter 1

INTRODUCTION

1 INTRODUCTION

The research presented in this thesis is motivated by the disparity between the vast volume of academic literature in the field of structural design, especially in the part of the force reduction factor (=behaviour factor or response modification factor) and the modern seismic design provisions in building design practice. The core research objective is therefore to contribute towards reducing the gap between the research and industry. The accompanying central hypothesis is to define a procedure for the assessment of the behaviour factor for steel moment resisting frame systems based on pushover curves which can be successfully and appropriately applied in practice. The research objective is achieved through the investigation of 90 possible definitions of reference parameters to define the behaviour factor based on Static Nonlinear Analysis (*Pushover*) cross checking with the results of Incremental Dynamic Nonlinear Analysis "*IDA*" (30 records), all applied to 102 conventional and non-conventional steel moment resisting frame systems. Significant research contributions are made in each of these studies, as stated in section 1.5.

This introductory chapter begins to explore a very short introduction on the behaviour factor, the problem statement, the research goals and the proposed methodology to achieve the research goals. Followed by the structure of the thesis with an overview of each subsequent chapter.

1.1 A Short Introduction on the Behaviour Factor

Design for seismic resistance has been undergoing a critical reappraisal in recent years, with the emphasis changing from "strength" to "performance". For most of the past 80 years, strength and performance have been considered to be synonymous. However, over the past 30 years there has been a gradual shift from this position with the realization that increasing strength may not enhance safety, nor necessarily reduce damage. For instance, Park and Paulay in 1975 [1] recognised that a frame building would perform better under seismic attack if it could be assured that plastic hinges would occur in beams rather than in columns (weak beam-strong column mechanism), and if the shear strength of members exceeded the shear corresponding to flexural strength. This can be identified as the true start of performance based seismic design, where the overall performance of the building is controlled as a function of the design process.

Based on this idea, in the early 80s, a new generation of structural design codes was issued all over the world, considering two alternative methods for structural analysis:

i) Nonlinear dynamic analysis
ii) Linear elastic response spectrum analysis

The use of the first method is motivated by particular cases related to the importance of the building or its functionality, however, its application is unusual in everyday practice because of its complexity. Hence, seismic design codes define a procedure in which the nonlinear behaviour of the structure can be predicted by a linear design procedure (the second method) using a single value called behaviour factor.

According to the modern seismic codes in the design scenario, to avoid explicit inelastic structural analysis, the capacity of the structures to resist the ground motion forces is taken into account by performing an elastic analysis with the use of a seismic design load reduction factor which is the so called behaviour factor “q”. In other words, the behaviour factor modifies the linear elastic spectra to the nonlinear inelastic spectra in order to obtain an approximation of the nonlinear dynamic response of the structure through a linear structural model (see Figure 1-1). The behaviour factor (also called “q-factor” according to the European Standard [2]) can be defined as the ratio of the peak ground acceleration producing collapse of the structure to that at which the first yielding occurs (where the response of any structural members is no longer linear). Hence, the behaviour factor should reflect the capability of the structure to dissipate seismic energy through inelastic behaviour. Thus, the force reduction factor should take into account the actual structural behaviour, the system ductility and the collapse criteria under the earthquake loading. The system ductility could also depend on structural configurations, the ductility of the material, the second order effects and fragile mechanisms [2]–[4]. Further, it may depend moderately on the period of vibration and on the hysteretic model [5], [6].

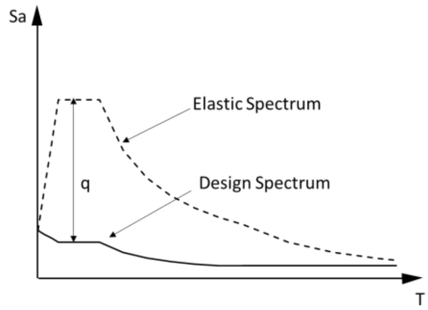


Figure 1-1 Behaviour Factor Definition

1.2 Problem Statement

A precise estimation of the behaviour factor for a given structure is very complex since it depends on many factors such as: the accelerogram acting at the base of the structure, the structural ductility, the system configuration, degree of redundancy, local and global buckling effects. Therefore, in the 80's, generic estimates of the q-factor have been provided by extensive research carried out all over the world [3], [4], [15]–[23], [7]–[14]. For example, in Europe the behaviour factor was introduced in the first publication of Eurocode 8 (May 1988). The behaviour factors defined in this edition remain nearly unchanged. A comparison between the two editions of Eurocode 8 is shown in Figure 9-1 in APPENDIX-A [24].

The following statements define the research problem in three stages:

- **Stage 1**

Modern seismic codes present the *value* of the behaviour factor, without even mentioning how they evaluated them or how to evaluate them, in a table which specifies a number for each structural typology. Interestingly, this value of the behaviour factor varies from code to code as shown in Eurocode [2], FEMA [25] and AIJ [26], with a large discrepancy. Table 9-1 in APPENDIX-A shows the value of the behaviour factor in 27 different seismic codes provision for MRF systems. EC8 [2], for example, defines the q-factor for steel moment resisting frame structures as 4.0 for medium class ductility and 5.5 to 6.5 for high class ductility and to the maximum value of 8.0 when nonlinear analysis is performed. On the contrary, this value, in the world, is felt between a min of 2 for the Philippines and a maximum of 12 for Bangladesh (only 8.0 in UBC [27] and only 4.0 in Japanese seismic code (*AIJ*) [6], see APPENDIX-A, Table 9-1).

The large scatter of the defined behaviour factor might be arisen from the assumption of the different approaches by the various codes which may not be acceptable from the scientific and practitioners point of view. In other words, different codes define different physical procedures in a different way which results in a wide variety of possible choices by selecting the reference parameters. For instance, EC8 uses the first significant yield strength α_1 for overstrength factor ($^{\alpha_{max}}/_{\alpha_1}$) rather than the design base shear V_{design} employed by US codes.

In particular, the economic implementations related to the fact that the same structure designed by various codes (that in principle should provide the same safety requirements) results in a different weight of the materials, is barely acceptable.

Additionally, the number of code-approved structural systems in Eurocode is limited to only four original systems, namely moment resisting frames (*MRF*), concentric or eccentric braced frames and concrete cores or walls that at best date back to the 1970's. Hence, newer seismic protection systems, can only be employed by experts and will remain out of reach of most professional scientists and practitioners. Unlike in the US, where the well-received FEMA P-695 [28] standard has settled this debate, Europe (EC8 [2]) has not formulated any standard methodologies to settle this kind of problem, just bearing some recommendations in ECCS [29].

The current stage made an attempt to propose a methodology for the assessment of quantitative values of the behavior factor "*q*" for steel *MRF* buildings that can be lead to a close approximation of the actual behaviour factor for any structure.

- **Stage 2**

In general, the usual process for the design of a non-statically determined structure requires three steps as follows; Figure 1-2 shows the general structural design process according to the modern seismic design codes.

Phase 1;

> When the engineer, mostly based on his/her experience and/or by "analogy" with other existing structures of similar dimensions and typology, identifies an initial "size" of the structural load-carrying members.

Phase 2;

> When the structural analysis is carried out, leading to the assessment of the "demand" (in terms of axial load, bending moments, shear and torsion and of displacements or rotations in the joints) in the various structural members under the external actions.

Phase 3;

> When the "capacity" of the load carrying structural members, of their connections and of the building foundations is verified (under the stresses induced by the internal actions derived from the structural analysis in phase 2) both in terms of "strength" and "ductility". The latter is extremely important and desirable, especially when designing a structure in an earthquake prone area, as it allows the structure to dissipate the energy through the development of plastic deformations in specific zones.

Figure 1-2 General Structural Design Process

In order to verify the global and local capacity of the structure in terms of ductility, the easiest way allowed by the modern software tools is to perform a push-over analysis. However, this doesn't assure that the "actual" behaviour factor (= the actual ductility of the structure) verified by the pushover analysis will coincide with the "assumed" one (= behaviour factor given by the codes). Indeed, several case studies analysing the ductility of the structures designed with a certain q-factor show that the real ductility is far from the assumed value. Of course, for an optimal design these two values should more or less coincide.

Design engineers, however, never tend to optimize their design using more advanced procedures, due to their complexity. Even if an engineer would like to use the current advanced design procedures (such as nonlinear pushover analysis), without "reliable" information on the nonlinear response of each single component of his/her structure, he/she cannot achieve "reliable" results.

In the same way, academic community agrees on the weakness of the available linear force-based design code procedures, and have been working on the more complicated procedures to provide tools for the optimized design of structures (such as displacement and performance-based design). As regards, apparently, there are not yet available tools that convince the design engineers to use such methods.

The author believes that the current linear seismic design procedure requires additional activities (additional step) so that the above problems can be overcome by performing a pushover analysis after phase 1, and assessing the capacity (in terms of strength and ductility) of the preliminary designed structure as well as its "actual" q-factor based on the results of such analysis. Subsequently, phase 2 and 3 can be performed, but in this case "assumed" and "actual" ductility are more or less coincident, and hence the design is optimized. Figure 1-3 shows the schematic view of the proposed new design procedure with augmented efficiency.

In order to do so, a standard re-analysis procedure of the results of the pushover analysis should be identified. Presently, only FEMA P-695 [28] provides a procedure for the definition of the q-factor (in [25], [27], [28] referenced to as "response modification factor") by re-analysis of the push over curve. In Europe, such a method is not yet proposed by EC8 and only ECCS [29] provides some recommendations. In any case, in order to re-analyse the push-over curve, the main parameters adopted by both FEMA and ECCS are defined in different ways which affects the assessment of the q-factor and hence, results in a different value of the q-factor (see Stage 1).

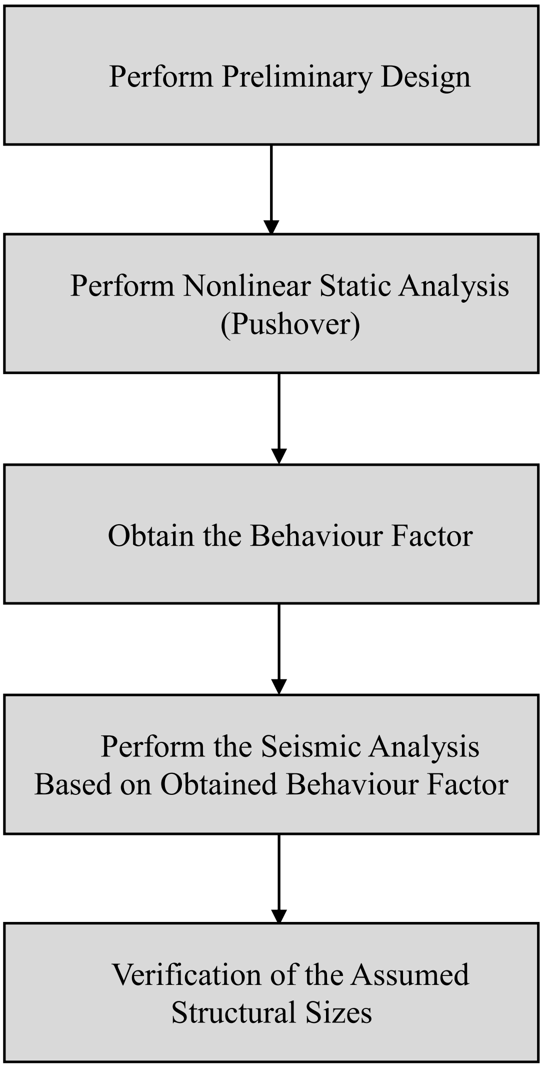


Figure 1-3 New Design Procedure with Augmented Efficiency

- **Stage 3**

The optimization of the structural design remains a paramount issue for the structural designers to achieve sustainable building with a low construction cost and minimum maintenance costs during the structure's life. Since the initial construction cost (cost of the material such as columns, beams, etc.) of a building is always an important parameter, the design optimization objective is to minimize the initial structural weight (or initial structural cost of the material) in a fully stressed state so that it fulfills the desired component reliability levels. The design of steel-based-structures in seismic areas is highly influenced by the seismic loads due to their considerable flexibility under the lateral loads. According to the code requirements, the capacity of the structures to resist the ground motion forces can be verified by performing an elastic analysis with the use of a seismic design load reduction factor which is so called behaviour factor "q". It is evident that the lower the actions in a structure, the less need for materials required for a strength/force-based design of the structure. In other words, when increasing the q factor, the global forces decrease, hence, a reduction of structural weight is expected. But, it is not always so, because of other factors, such as the limits imposed by the codes (allowable inter-storey drift, drift sensitivity "θ" and lower bound factor "β"), which will prevent reduction of member's sizes and both local and global structural stiffness. The current stage will present a novel strategy for an optimal design of steel structures in high seismic zones through balancing the initial cost and the lifetime seismic damage. The investigation will be performed in order to explore the relationship between the expected initial material cost and the design behaviour factor.

1.3 The Research Goals

The main aim of this research is to establish a consistent procedure for the assessment of the behaviour factor for MRF buildings.

The research goals can be identified as follows:

1) To propose a methodology for the assessment of quantitative values of the behavior factor "*q*" for steel MRF buildings that can be easily applied in everyday design practice, and can lead to a close approximation of the actual behaviour factor for each specific structure being analysed.

2) To show the generality of the proposed methodology by applying it to innovative structural systems such as building having dissipative elements that are more complex than the conventional structures, in order to show the ability of the method to provide a reasonable assessment of the behaviour factor for any newer innovative structures.

3) To achieve a balance between the initial material weight and lifetime seismic damage with respect to optimal behaviour factor which results in minimizing the weight/costs of the structure.

1.4 Proposed Methodology to Achieve the Research Goals

In this thesis, firstly, all the possible definitions of reference parameters to identify the behaviour factor based on reanalysis of a Pushover curve will be analysed. Combining these parameters in every possible way result in many different combinations/methods for the assessment of the q-factor. These methods/combinations hereafter will be applied to some case studies of steel MRF systems which will be designed with an initial value of the behaviour factor q=2, 3, 4, 5, 6 and 7. The case studies are different in the number of stories (2, 4, 8 and 12 storey). The number of bays are 3 and 4 with the length of the bays equal to 6m and 8m. Therefore 96 (6x4x2x2=96) buildings in total will be designed according to EN1993-1 [30], EN1998-1 [2] and EN1994-1 [31]. In addition to these case studies, other six buildings with innovative bolted and welded FUSEIS dissipative beam splices which makes the structure more complex than the conventional one [32]–[38]) will be investigated. Hereafter, parameter combinations/methods of reanalysis of the pushover curve are applied to each different structure. A comparison of the achieved results with the initial design value of the q-factor will be then presented in order to identify those methods which lead to q values close to the initial/assumed one, and those which lead the values far from it.

However, the *"optimal method"* can be chosen only after a validation of the results of the Incremental Dynamic Analysis (IDA) [39], [40]. To do so, 30 sets of ground motion records from 3 sites in Europe (i.e., Athens, Greece – Perugia, Italy – Focsani, Romania) characterized by high seismicity will be selected. Each ground motion record will be then scaled 12 times to cover the entire range of structural response such as elastic part, yield point and dynamic instability using more advanced algorithm (Hunt and Fill [39]) which minimizes the number of record time. Each structure that is designed according to the previous definitions might be re-analysed by nonlinear dynamic analysis for 360 times (30 records each scaled 12 times), hence, huge number of analyses will be performed for each structural typology which results in total of 36822 analysis. The incremental dynamic analysis results will be then evaluated on the basis of *AvgSa* as the geometric mean of 5% damped spectral acceleration ordinates within the period range of interest. The performance of each structure will be verified against life safety (*LS*) and global collapse (*GC*). Thereafter, the assumed design q- factor may accepted or rejected according to Cornell et al. [41] fragility-hazard convolution approach by determining the mean annual frequency (*MAF*) of damage exceedance of the defined limit states. In addition, a comparison is carried out between the results obtained by means of the proposed methodology and those obtained by the method proposed by Ballio [4] and Setti [10]. According to [4] and [10], the behaviour factor can be estimated as the intersection between the ductility demand curve (obtained by inelastic dynamic analysis) and a straight line (demonstrating the behaviour captured from an elastic dynamic analysis). By this definition the q-factor corresponds to the value beyond which a linear elastic analysis is no longer a safe solution, because the global ductility demand estimated by means of a non-linear analysis is larger than that estimated with a linear analysis. Figure 1-4 represents the research flowchart.

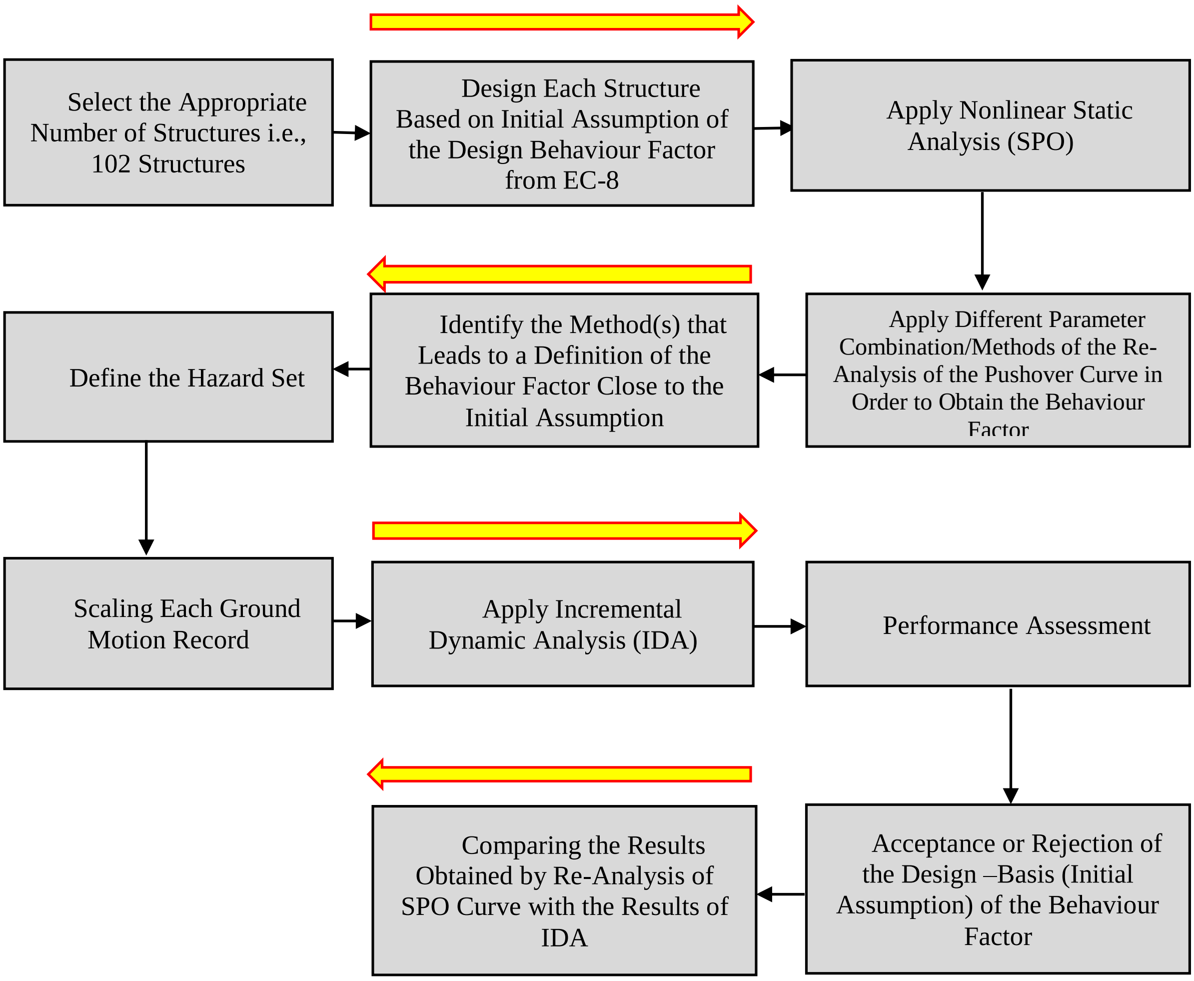


Figure 1-4 Research Flowchart

1.5 Organisation of Dissertation

The research dissertation might be organized as follows:

Chapter 1 provides a short introduction on the behaviour factor, the problem statement, research goals, methodology to achieve the research goals and organization of the dissertation.

Chapter 2 summarizes the historical perspective of the behaviour factor, the role of the behaviour factor in structural design, behaviour factor evaluation on modern seismic codes and behaviour factor assessment methods.

Chapter 3 presents the research methodology that proposes the behavior factor evaluation based on re-analysis of the pushover curve. In particular, this chapter introduces all the possible combinations and methods to define the reference parameters in order to calculate the behaviour factor based on the pushover curve. Afterwards, it addresses a procedure to reject or accept the initial assumption of the design behaviour factor calculated in the previous step through incremental dynamic analysis using fragility-hazard convolution approach to determine the mean annual frequency of the damage exceedance. And to cross check the behaviour factor obtained by re-analysis of the pushover curve with the results of incremental dynamic analysis (IDA).

Chapter 4 addresses the case studies consisting of 96 conventional structures which are designed based on different design behaviour factor (2/3/4/5/6 and 7), bay length (6m and 8m), number of bays (3 and 4) and the number of stories (2/4/8 and 12). The chapter also considers other 6 case studies (innovative dissipative bolted and welded beam splices) as non-conventional structures having 2/4 and 8 story-3 bays and 6 meter length of the bay.

Chapter 5 provides the results of nonlinear static analysis (Push-Over) together with the results of Incremental Dynamic Analysis (IDA). Further, a strategy for an optimal design of steel structures in high seismic zones through balancing the initial weight of the material and the lifetime seismic damage is introduced. The optimization of the design procedure is achieved by minimizing the initial structural weight based upon on the behaviour factor consideration. The chapter also represents the results of incremental dynamic analysis (IDA) which is used to estimate the structural performance under seismic loads. The 3 seismicity ground motion sets were selected across Europe representing high seismicity zones. Each ground motion record scaled to multiple level of interests (i.e., 12 runs) to produce the response of structure versus intensity measure. The final outcome of this chapter will be to reject or accept the behaviour factor obtained through the methodology introduced in chapter 3.

Chapter 6 summarizes the behaviour factor estimations/calculations using the results of re-analysis of the pushover curves based on methodology addressed in chapter 3. The chapter also introduces the *"optimal methods"* to define a consistent behaviour factor for moment resisting frame systems based on those methods, giving the value of the behaviour factor in closest agreement to the initial/assumed design behaviour factor. Eventually, a comparison is made between the results of IDA and re-analysis of the pushover curve for the *"optimal methods"*.

Chapter 7 provides conclusions and recommendations for further steps in advancing this research.

Chapter 2

STATE OF THE ART

2 STATE OF THE ART

2.1 Historical Perspective of the Behaviour Factor

According to the building codes, earthquake-resistant structures are intended to withstand the strongest earthquake with low damage having a certain probability of occurrence during their design lifetime. This means that the loss of lives of the occupants should be minimized by preventing the collapse of the buildings for rare earthquakes while the loss of the functionality should be limited to more frequent ones. Based on this idea, a group of engineers in the early 1900's after the San Francisco earthquake in 1906 started a study to observe the damage and to obtain a design solution to withstand this kind of unexpected natural phenomena. The progression of earthquake resistant design over the last 100 years can be subdivided into three major periods as follows:

Before the earthquake design and the concept of dynamic response of the buildings start, during 1800's until 1900's, the buildings were only designed based on the wind loads and static force concept approach.

The First Period;

- After the San Francisco earthquake in 1906, the dynamic response of the structure gained the attention of the researchers and designers of the time where the seismic load for the first time was considered to be applied at 10% of the structural weight, according to the provisions of a new building code [42].

The Second Period;

- Researchers from Stanford University in 1930 [42] started the first study on dynamic response and analysis of the structures where the design approach changed forever from the static load concept to the structural dynamic and the natural period of vibration of the structures [42], [43]. During 1930 and 1950 was introduced in the seismic design codes a relationship between the load and strength through the use of an equivalent lateral load procedure making reference to the design equation:

$$V = ZKCW \qquad \text{Eq. 2-1}$$

Where

V is the seismic design base shear

Z is the seismic zone factor

K is the building system type

C is the building's natural period of vibration

W is the building's weight

The Third Period or the Current Period;

- Earthquake resistant design (Housner [44]–[46] in 1952) introduced the spectral response acceleration. Since then, other innovative concepts were introduced in the subsequent generation of codes such as; soil interaction, force reduction factor (response modification factor, according to U.S codes [25], [27]) and the importance factor. The design Eq. 2-1 was then modified as follows:

$$V = \frac{2SFIW}{3RT} \qquad \text{Eq. 2-2}$$

Where

S is the spectral response acceleration

F is the site coefficient

I is the importance factor

R is the response modification factor also called force reduction factor

T is the natural period of vibration

V, W are the seismic design base shear and the building's weight, respectively.

By the time the analysis methods were being developed, designers needed additional knowledge of non-linear behavior of structural components. Thus, substantial testing of materials and connection assemblies to justify actual behavior were undertaken from 1950 to 1990.

2.2 The Role of the Behaviour Factor

In force-based seismic design procedures, behavior factor "q" in Eurocode 8 [2] also called response modification factor in NEHRP [25] or simply called as "R" coefficient in UBC [27] is a force reduction factor that modifies the linear elastic spectra to the nonlinear response spectra. In other words, behavior factor is a ratio in which the elastic response spectrum will be modified into an inelastic one (see Figure 1-1). The behaviour factor should take into consideration the actual structural behaviour, the ductility demand as well as the collapse criteria under the earthquake loading [47].

According to Eurocode 8 [2] for example, which is based on force-controlled and capacity design, the reduction of the elastic design seismic forces is evaluated on the basis of the behaviour factor which relies on the reserve of strength and ductility to improve the capability of the structure to absorb the ground motion forces [48]. The behaviour factor depends on the system ductility dependent component, strength dependent factor and cyclic excitation inputs. The system ductility also depends on structural configurations, the ductility of the material, the second order effects and fragile mechanisms. EC8 defines the q-factor for steel moment resisting structures as 4.0 for medium ductility and $5^{\alpha_u}/_{\alpha_1}$for high ductility which clearly depends on system ductility, redundancy and overstrength of the elements. The value of $^{\alpha_u}/_{\alpha_1}$is a ratio related to the first plasticity and ultimate plasticity capacity, which is related to the redundancy of the structure. When the multiplication factor $^{\alpha_u}/_{\alpha_1}$ has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate values of $^{\alpha_u}/_{\alpha_1}$may be used:

- One-storey buildings or industrial buildings: $^{\alpha_u}/_{\alpha_1}$=1.1;
- Multistorey, one-bay frames: $^{\alpha_u}/_{\alpha_1}$=1.2;
- Multistorey, multi-bay frames: $^{\alpha_u}/_{\alpha_1}$=1.3.

2.3 The Behaviour Factor Definitions on Modern Seismic Codes

As shown in Table 9-1 in APPENDIX-A with reference to the steel MRF systems, the values of the behaviour factor vary from code to code (e.g. EC8 [2], FEMA [25], AIJ [26], etc.), with a large discrepancy, mainly due to the different approaches assumed by various codes for its definition. More interesting than the prescribed values in themselves are the backgrounds that support or justify these values. Below, the overview of three different definitions of the seismic reduction factor of the seismic codes in Europe, in the United States, and in Japan is discussed.

2.3.1 European Approach

Figure 2-1 shows the schematic of a force-displacement response of an elastic and an inelastic system. The shear force reduction factor is mainly due to two factors the ductility factor "q_μ" and overstrength factor "q_Ω". The first one reduces the strength from the elastic demand forces (F_e), if the structure remains elastic during the earthquake, to the ultimate strength (F_y) or reducing the displacement corresponding to the elastic demand forces (d_m) to the displacement corresponding to the yield strength (d_y). The latter reduces the strength of the structure from the maximum strength (F_y) to the first significant yield strength (F_1). Hence, it can be concluded that the force reduction factor basically reduces the elastic demand base shear forces (F_e) to the level of first significant yield strength (F_1). It can be obtained by simply dividing F_e over F_1 i.e., $q = {}^{F_e}/_{F_1}$ or through the multiplication of ductility "q_μ" and overstrength "q_Ω" dependent factor $q_\mu . q_\Omega$[49].

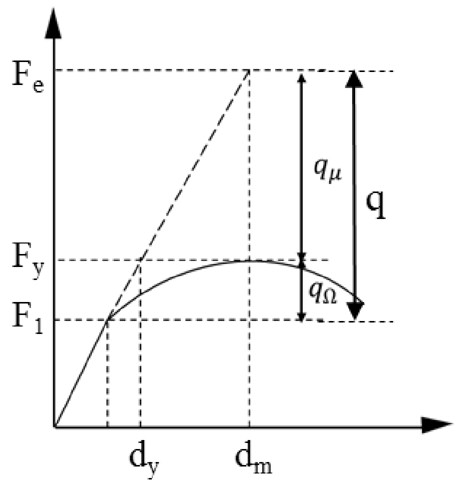


Figure 2-1 Force-Displacement Response of Elastic and Inelastic System

In general the force reduction factor or behaviour factor can be defined as follows:

$$q = \frac{F_e}{F_1} = q_\mu . q_\Omega . q_\xi \qquad \text{Eq. 2-3}$$

Where

F_e is the strength force if the structure remains elastic during the earthquake

F_1 is the significant yield strength

q_μ is the ductility dependent factor, a function of the displacement ductility

q_Ω is an over-strength dependent factor, a function of the non-linear structural response

q_ξ is a damping dependent factor, which is equal to 1.0 when assuming the same damping ratio holds for both elastic and inelastic analysis

2.3.1.1 Ductility Dependent Factor

The ductility reduction factor can be calculated as follows:

$$q_\mu = \frac{F_e}{F_y} = \frac{d_m}{d_y} \qquad \text{Eq. 2-4}$$

Where

Fy is the knee point of the idealized bilinear yield strength

d_m is the displacement corresponding to the maximum strength

d_y is the displacement corresponding to the knee-point of the idealized bilinear elastic-plastic behavior curve

Newmark and Hall [8], [50] in 1982 made the first attempt to relate the ductility dependent factor q_μ to the system ductility μ for a single degree of freedom system based on elastic-perfectly plastic curve. They found that for short period structures i.e., T < 0.03 Sec. the system ductility cannot reduce the response of the structure. On the contrary, when the period of the structure is between 0.03 Sec. and 0.5 Sec. i.e., 0.03 Sec. < T < 0.5 Sec. (medium period) the energy that can be absorbed by the elastic system at the maximum displacement is equal to the one of the inelastic system (equivalence of energy). For long period structure, i.e., T > 0.5 Sec. the maximum displacement sustained by an elastic system is equal to the one sustained by an inelastic system (equivalence of displacement). Figure 2-2 shows the structural ductility and the system ductility relationship according to Newmark and Hall definition [8], [50] for a) short period (equal acceleration) b) moderate period (equal energy) c) long period (equal displacement).

Hence, the ductility factor q_μ according to Newmark and Hall [50] can be expressed as a function of the system ductility μ, related to the natural period of vibration T, as follows:

$$\begin{aligned} q_\mu &= 1.0 && \text{(for T<0.03s)} \\ q_\mu &= \sqrt{2\mu - 1} && \text{(for 0.03s <T<0.5s)} \\ q_\mu &= \mu && \text{(for T>0.5s)} \end{aligned} \qquad \text{Eq. 2-5}$$

and

$$\mu = \frac{d_m}{d_y} \qquad \text{Eq. 2-6}$$

Where

T is the natural period of vibration of the structure

d_m is the displacement corresponding to the maximum strength

d_y is the displacement corresponding to the knee-point of the idealized bilinear elastic-plastic behavior curve.

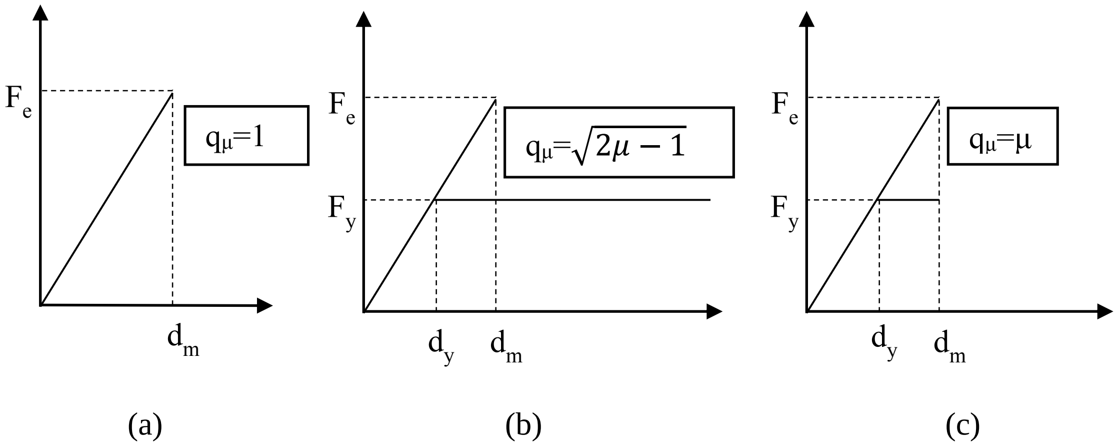


Figure 2-2 The Structural Ductility and the System Ductility Relationship According to Newmark and Hall Definition for a) Short Period (Equal Acceleration) b) Moderate Period (Equal Energy) c) Long Period (Equal Displacement)

2.3.1.2 Overstrength Dependent Factor

Any structure designed according to the concepts of earthquake resistant design (considering q>1) should resist the ground motion forces without collapse, but with some damages. In other words, after obtaining the first significant yield, the structure can still take further loads. The overstrength may be the result of 1) the higher strength of the material used in the construction phase than the one specified in design scenario; 2) a greater strength than the one required, if using the standard sections (i.e., greater member sizes) 3) lower gravity load than the one specified in design code and 4) the special ductility requirement such as strong column-weak beam mechanism. According to the definition and with reference to Figure 2-1, the overstrength reduction factor can be calculated as follows:

$$q_{\Omega} = \frac{F_y}{F_1} \qquad \text{Eq. 2-7}$$

Where

F_y is the strength corresponding to the knee-point of the idealized bilinear elastic-plastic behavior curve.

F_1 is the strength corresponding to the first significant yielding of the structure

2.3.2 American Approach

The approach in the United States is based on the assumption that the nonlinear design spectrum has a direct effect on structural performance. Numerical and experimental results with this approach show that the structural capacity is much higher than the one required [6]. Since the American approach is mainly based on the observation of the structural performance after an earthquake, ATC 10 (1982), for example, stated that "In numerous cases, buildings have sustained little or no damage even though the equivalent forces associated with the maximum amplitude of recorded peak horizontal ground acceleration were several times higher than the lateral forces used in building design". Hence, according to Sanchez-Ricart [23] the structural capacity is more related to the conceptual design of the structure than to the seismic action defined in the seismic code. Figure 2-3 shows the behaviour factor definition in terms of base shear and top roof displacement in the US.

The response modification factor in the United States UBC [4] and NEHRP [34] can be calculated as follows:

$$R = R_S . R_\mu \qquad \text{Eq. 2-8}$$

Where

$$R_S = R_\rho . R_\Omega = {}^{V_y}\!/_{V_d} \qquad \text{Eq. 2-9}$$

Redundancy factor R_ρ can be calculated as the ratio between the strength at the knee-point of the idealized bilinear elasto-plastic curve (V_y) and the strength at the first yield (V_1).

Overstrength factor R_Ω can be calculated as the ratio between the strength at the first yield (V_1) and the design base shear (V_d).

Ductility reduction factor R_μ can be obtained through a ratio between the structural elastic strength response (V_e) and the idealized yield strength (V_y) by the following equation:

$$R_\mu = {}^{V_e}\!/_{V_y} = {}^{\Delta_u}\!/_{\Delta_y} \qquad \text{Eq. 2-10}$$

The factor R_μ is a function of both structural specifications as well as the system ductility and the fundamental period of vibration (T) [50] in which for T > 0.5 s, R_μ is effectively equal to the ductility factor of the structure "μ" [50]. Nassar & Krawinkler [51] and Miranda &

Bertero [18] also proved that there is a fundamental a period of vibration (T)-dependence of R_μ for period greater than 0.5 sec. as well as that there is an influence of soil type of the values of ductility reduction factor. This lead to a different definition of R_μ factor as $R_\mu = R_\mu(T, \mu)$ [18], [52], [53].

Ductility demand ratio "μ" is the maximum structural drift over the knee-point of the idealized yield displacement according to the Eq. 2-6:

By the definition given above and using Eq. 2-8 the response modification factor "R" (=behaviour factor "q") can be written as follows:

$$R = \frac{V_e}{V_y}\frac{V_y}{V_1}\frac{V_1}{V_d} = \frac{V_e}{V_d} \quad \text{Eq. 2-11}$$

Where

V_e is the maximum elastic base share

V_y is the actual strength of the system

V_1 is the strength corresponding to the first significant yielding of the structure

V_d is the design base shear

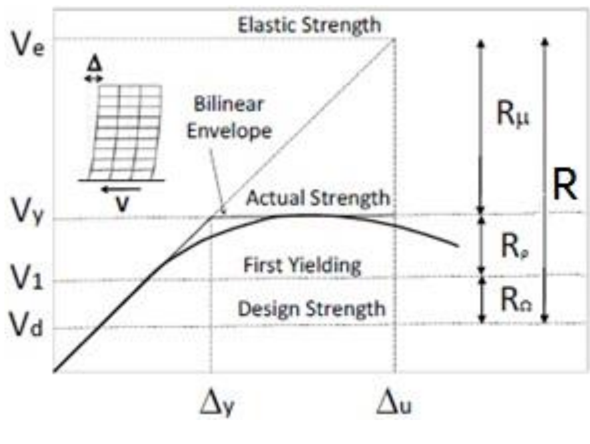


Figure 2-3 Base Shear vs. Top Roof Displacement in American Approach [54]

2.3.3 Japanese Approach

Japanese seismic code is perhaps one of the most conservative seismic codes and is based on the energy approach. The behaviour factor of the structure in Japanese seismic provision code is calculated as the square root of the dissipated energy and the stored energy [6], [55]. In

Japanese seismic provision, structural collapse is related to the hysteretic energy, regardless of the maximum plastic excursion. The behaviour factor is computed by the following formula:

$$q = \sqrt{1 + 4\, c_1 \alpha_1 \eta_1} \quad \text{Eq. 2-12}$$

Where

c_1 is the ratio of the elastic stored energy on the first storey over the elastic stored energy of the entire structure.

α_1 is the ratio of the plastic energy dissipation of the whole structure to the plastic energy dissipation of the first storey.

η_1 is the ratio of cumulated plastic ductility for the first storey to the elastic stored energy of the first storey. This can be assumed as a local plastic ductility factor.

For the steel MRF systems the behaviour factor can be obtained as 4.0

2.4 Behaviour Factor Assessment Methods

There are generally four general methodologies/theories to calculate the behaviour factor[5]:

I. Ductility-dependent factor theory
II. Extrapolation of inelastic dynamic response analysis of SDOF (single degree of freedom) systems
III. Energy approach
IV. Damage accumulation method

2.4.1 Ductility-Dependent Factor Theory

This method was first introduced by Ballio [4] and Setti [10] in 1985. In this method the behaviour factor can be estimated by performing non-linear time history analysis under increasing PGA (a procedure similar to IDA); plotting the maximum ductility demand normalized on the ductility demand at first yield ($^{\delta}/_{\delta_1}$) vs the PGA normalized on the PGA at first yield ($^{\alpha}/_{\alpha_1}$), and identifying the intersection of such curves with the straight line bisecting the first quadrant of such curves. By this definition the q-factor corresponds to the value beyond which a linear elastic analysis is no longer a safe solution, because the global ductility demand estimated by means of a non-linear analysis is larger than that estimated with a linear analysis. This method interprets the code but does not allow an assessment of the cumulative damage in the structural detail. Some years later, Sedlacek and Kuck [17] established the same procedure, but this time they also considered the second order effects.

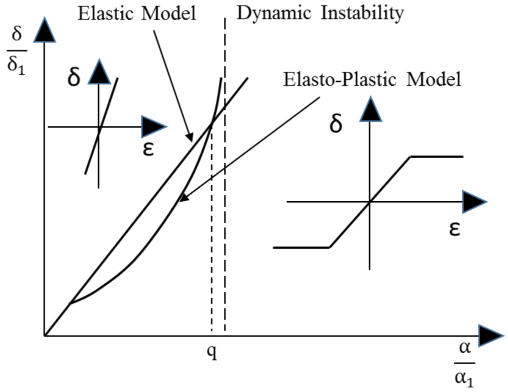


Figure 2-4 Evaluating of q Factor on The Basis of Ductility Factor Theory [22]

2.4.2 Extrapolation of Nonlinear Inelastic Dynamic Response Analysis of SDOFS

In this method, nonlinear spectra of SDOFS is determined by a single parameter, such as ductility. On the other hand, for a MDOF system, different types of yielding may correspond to the same maximum nonlinear displacement. Moreover, a MDOFS could experience very large axial forces (in nonlinear range) in interior columns, whereas these axial forces in a same range of plasticity obtained by modal analysis or a nonlinear spectrum may be less which leads to a reduction of plastic moment capacities while increasing the ductility requirement. On the other hand, SDOF systems, cannot allow for these effects [8]. Cosenza et al [57] combined the seismic response of a SDOF system to the static nonlinear response of steel structures. His method is on the basis of identification of the structure by means of an equivalent single degree of freedom model in which the parameters are characterized by load multiplier-max. roof displacement (α-δ) relationship also known as the behavioural factor of the structures.

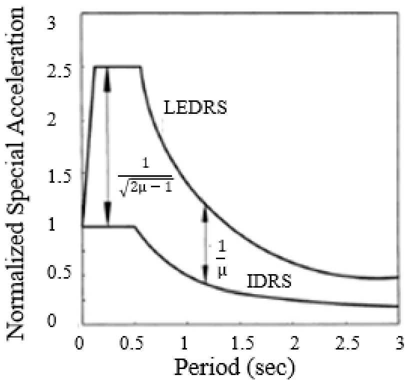
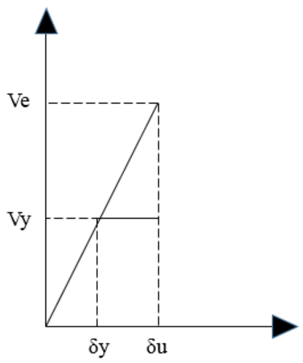


Figure 2-5 Evaluating of q Factor on The Basis of The Response of SDOF Systems

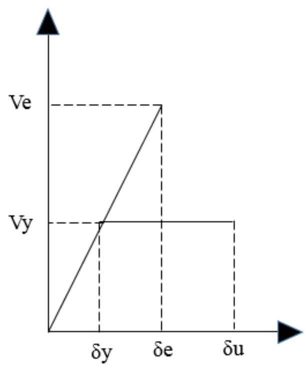
* IDRS represents the inelastic design response spectrum and LEDRS represents linear elastic design response spectrum [22].

2.4.3 Energy Approach

Energy approach was first showed by Como and Lanni [9]. They introduced a simplified model of the energy exchanges during the seismic excitation. In their method, the seismic action of a structure through a complicated procedure will be divided into a number of simplified cycles of energy. Each cycle is made up of a first phase of kinetic energy storing, during which the energy accumulated in the first phase is transformed into an elastic-plastic work. The kinetic energy in the energy dissipation step is neglected. The behaviour factor, then related to the linear strain energy of the system at the yield state and the total strain energy at the failure which may obtain from the energy equivalent formulation. There are some limitations in their method such as assumption of a global collapse mechanism and the need for structural regularity. On the other hand, it has the advantage of considering not only the maximum displacement, but also damage cumulated during seismic cycles. Kato and Akiyama [58] proposed an energy approach in which the elasto-plastic analysis is not required. They compared the structural energy dissipation capacity with the seismic energy input. This procedure must be applied at any single storey in order to assess the safety of the structures on the seismic events. Their method can be used only in a shear-type system such as weak column and strong beam frames.



a)



b)

Figure 2-6 A Comparison between a) Ductility Theory and b) Energy Approach

2.4.4 Damage Accumulations Method

This is a promising method, since the damage accumulations approach takes into account not only the displacement ductility but also the number of yield flows as well as the damage accumulation in the structure, [3], [5], [20], [21]. A method to evaluate the behaviour factor "*q*" is proposed by Castiglioni [3] based on the definition of the *q* factor [4] in which the collapse mechanism was used to evaluate the damage accumulation in the plastic hinges during seismic motion input. The Castiglioni's method was investigated both experimentally [14]–[16] and numerically [12], [59].

Castiglioni first, proposed a damage index obtained by a linear analysis "I_D^L" and another damage index which is obtained by a nonlinear analysis "I_D^{NL}" used to estimate the suitable behaviour factor based on cyclic test results on steel beams [3]. In his method, when the damage index of one of the elements of the structures which is calculated through a linear time history analysis, reaches the critical value as α, the structure considers as collapse.

$$\alpha = I_D^L \qquad \text{Eq. 2-13}$$

The optimal value of the q-factor can then be estimated as the ratio between the maximum peak ground acceleration $\overline{a}_{max}$ and the maximum design acceleration α_d corresponding to the equivalence of the damage accumulation indexes assessed by means of two analyses, dynamic linear elastic analysis (I_D^L) and dynamic non-linear analysis (I_D^{NL}). I_D^L and I_D^{NL} are defined by Miner as the rule of linear damage accumulation [7]. The behaviour factor corresponds to the following criterion:

$$I_D^L = I_D^{NL} \qquad \text{Eq. 2-14}$$

On the other hand, Calado and Azevedo [13] proposed a different method for the assessment of the damage accumulate by a structure up to failure using classical low-cycle fatigue model for steel members and the Miner's rule (linear damage accumulation). In their definition failure of the structure is assumed to occur when the number of plastic hinges characterized by accumulated damage equal to ∂ is enough to allow development of plastic mechanism. The damage accumulation up to failure depends only on the sum of the plastic deformation and the damage index ∂ which represents the characteristic value of the accumulated damage. This measures the ductile hysteretic capacity of the structural elements which might be used to indicate failure under cyclic loading; a limit to the recorded accumulated damage. In the Calado's method the most important parameter influencing ∂ is the level of axial load in the case of beam-columns, whereas in bracing elements, ∂ is mostly influenced by the slenderness. The accumulated damage D after L cycles of different amplitude is addressed by:

$$D = C \sum_{i=1}^{L} i^{a} (\Delta\xi_{Pi})^{c} \leq \partial \quad \text{Eq. 2-15}$$

Where

$$\Delta\xi_{Pi} = \frac{\int_{A} \Delta \in_{Pi} dA}{A \in_{y}} \quad \text{Eq. 2-16}$$

$\in_{Pi}$ is the plastic deformation

$\in_{y}$ is the yield deformation

A is the area of the cross section

A comparison between these two methods (Castiglioni and Calado method) [21] showed that, in general, the values of the behaviour factor calculated by these methods are almost similar, although, they are different on the basis of dynamic analyses (Castiglioni's method uses linear elastic dynamic analysis while Calado utilizes the nonlinear dynamic analysis).

Chapter 3

RESEARCH METHODOLOGY

3 RESEARCH METHODOLOGY

In order to achieve the research goals explained in chapter 1, this chapter is proposing a novel methodology for the assessment of a quantitative value of the behavior factor “*q*” for steel moment resisting frames based on the re-analysis of the nonlinear static analysis (*Pushover*) curve.

As explained in chapter 1, the current force-based design procedure is too simple and generalized, and most of the time results in over-designing the structures. The over-designing of the structures may arise from the difference between the actual behaviour factor (= the actual ductility of the structure) and the assumed one (= q-factor given by the codes). This problem can be outreached by performing a pushover analysis after the initial design of the structure and then assessing the capacity (in terms of strength and ductility) of the preliminary designed structure as well as its “*actual*” q-factor. If the actual behaviour factor and assumed one are found to be different, re-design as well as another cycle of comparison (= between the *initial* and *actual* behaviour factor) may be required. Although this cycle of iteration might seem time consuming, the subsequent linear dynamic response spectrum analysis will lead to an optimal design of the structure as the “*assumed*” and “*actual*” ductility are more or less coincident.

As shown in Table 9-1 in APPENDIX-A with reference to the steel MRF, the behaviour factor values vary from code to code (e.g. EC8 [2], FEMA [25], AIJ [26], etc.), with a large discrepancy, mainly due to the different approaches assumed by various codes for the definition of the q-factor. In Europe, generic estimates of the q-factor have been provided in the 1980’s by extensive research [3], [4], [15]–[23], [7]–[14] which led to the first publication of Eurocode 8 in May 1988 (see APPENDIX-A). The behaviour factors defined in this edition remained nearly unchanged. A comparison between two editions of Eurocode 8 is shown in Figure 9-1 in APPENDIX-A [24].

On the other hand, codes only provide a single upper bound value of the behaviour factor without even mentioning how they were evaluated or how they should be evaluated. In the EN1998-compliant document, for instance, there are no guidelines and procedures to calculate the behaviour factor based on the pushover curve, neither for conventional nor for the newer structural systems. Some recommendations are proposed in ECCS [29]. Hence, this research made an attempt to propose a methodology for the assessment of quantitative values of the behaviour factor based on re-analysis of the pushover curves for steel moment resisting frame systems.

3.1 Proposed Behavior Factor Evaluation Procedure Based on Re-Analysis of the Pushover Curve

The behaviour factor is a force reduction factor which implies the linear spectra to equivalent nonlinear spectra in order to account for the real behaviour of the structures under earthquake loads. The behaviour factor plays an important role in the evaluation of the design forces on a structure. The q-factor is directly related to the structural ductility, redundancy, viscous damping, and the structure's overstrength. These parameters have a great influence on the energy dissipation capacity of a structure.

In practice, the behaviour factor "q" is described as the ratio of the peak ground acceleration that induces collapse (α_{max}), which depends on the type of the collapse mechanism (see Figure 3-1), to the first significant yielding of the structure (α_1) where the global/local behaviour of the structure is no longer linear:

$$q = \frac{\alpha_{max}}{\alpha_1} \qquad \text{Eq. 3-1}$$

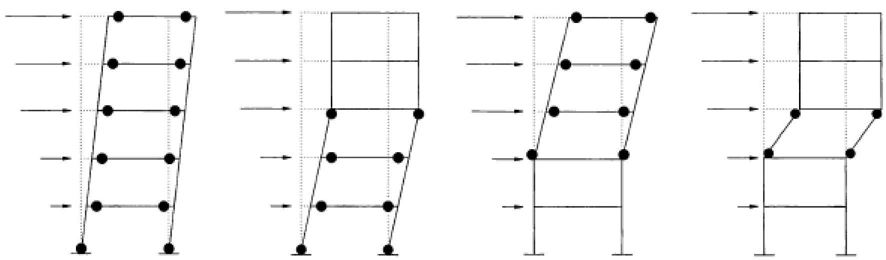


Figure 3-1 Collapse Mechanism Typologies for Moment-Resisting Frames under Seismic Horizontal Forces

Alternatively, According to ECCS [29] and FEMA P-695 [28] the behaviour factor can also be calculated as a product of overstrength q_Ω, ductility factor q_μ and redundancy q_ξ. Hence, as explained in chapter 2 a proper approximation of the force-reduction factor "q" may be calculated as in Eq. 2-3

3.1.1 Introduction

In the literature, various parameters are proposed for the re-analysis of a pushover curve, in order to define the structural ductility/behaviour factor. In what follows, all the possible definitions of these reference parameters are first introduced. The combination of these parameters results in many different methods for the re-analysis of the pushover curve. These methods are then applied to 102 case studies of MRF systems in order to identify those methods which lead to q values close to the initial one, and those which lead to values far from it.

The Incremental Dynamic Analysis (IDA) [39], [40] which, being a more sophisticated method, based on non-linear dynamic analysis, should lead to better assessment of the structural response under seismic loading, will be then applied to all the case studies to cross check with the behaviour factor calculated by means of the proposed simplified method based on the pushover curve. Figure 3-2 displays an overview of the behavior factor evaluation procedure.

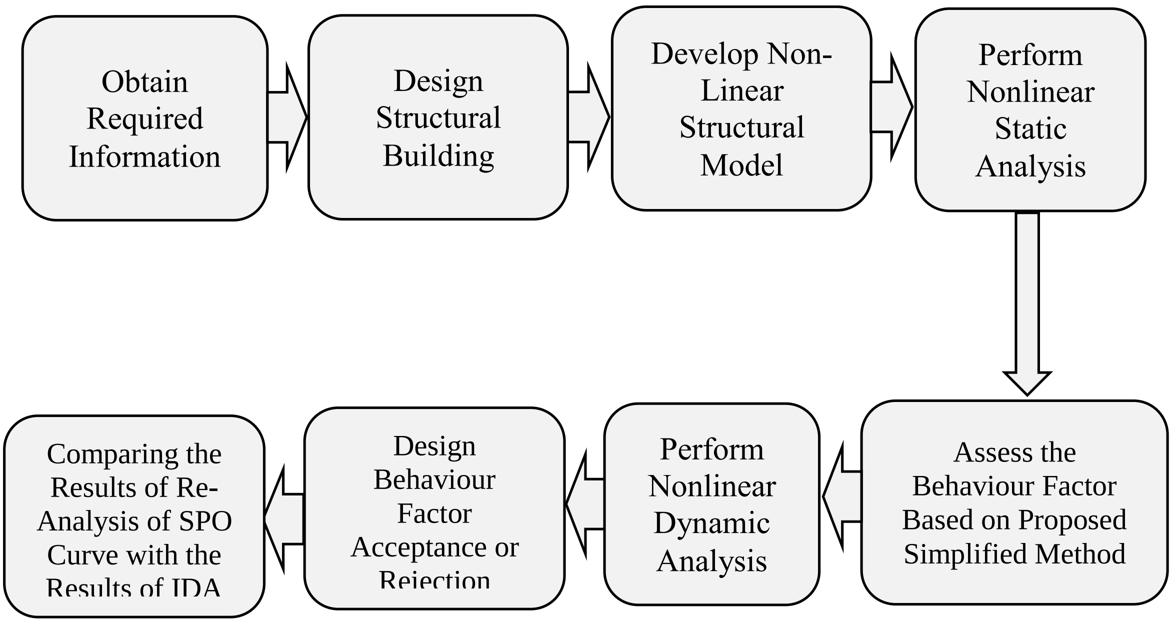


Figure 3-2 Behaviour Factor Evaluation Procedure

3.1.2 Definition of the Reference Parameters

For the assessment of the behaviour factor by means of re-analysis of the Push-Over Curve, various seismic design codes define the reference parameters in a different way [60]. FEMA P-695 [28], for example, defines the overstrength factor q_Ω as a ratio of the maximum actual strength of the structure to the design base shear V_y/V_d , whereas EC8 [2] refers to the first significant yield F_y/F_1 where Fy is the ultimate strength of the structure (see Figure 2-1 and Figure 2-2).

In the same way, to define the period-based ductility factor q_μ, FEMA P-695 [28] defines the maximum displacement Δ_u corresponding to 20% loss of strength of the structure in post hardening, while EC8 [2] defines the same parameter as the maximum displacement corresponding to the formation of the plastic mechanism (see Figure 2-1 and Figure 2-2).

Hence, in this section, all the possible definitions of these parameters and the effect of their combination on the assessment of the q-factor will be discussed.

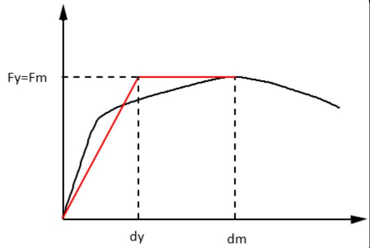
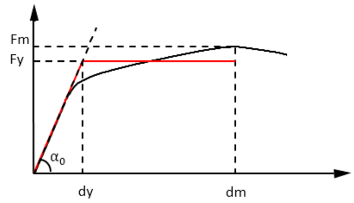
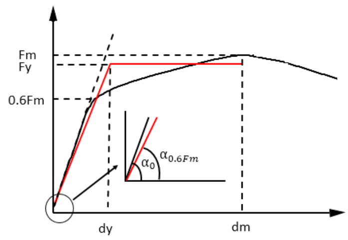
As explained in chapter 2, calculation of the behaviour factor is based on the 5 main parameters: *F1, Fy, Fm, dy* and *dm*. While *Fm* can be unequivocally defined, with a general agreement, as the maximum actual strength of the structure, the other parameters might be defined in different ways as shown in Tables 3-1, 3-2 and 3-3. Table 3-1 displays the possible definitions for maximum horizontal roof displacement (*dm*) corresponding to the maximum strength or/and in softening branch.

Table 3-1 Possible Definitions for Maximum Horizontal Roof Displacement (d_m) Corresponding to the Maximum Strength or/and in Softening Branch

Identifier	Definition
d_m-1	Horizontal roof displacement corresponding to the maximum structural strength
d_m-2	Horizontal roof displacement corresponding to 5% loss of structural load carrying capacity, in softening branch
d_m-3	Horizontal roof displacement corresponding to 10% loss of structural load carrying capacity, in softening branch
d_m-4	Horizontal roof displacement corresponding to 15% loss of structural load carrying capacity, in softening branch
d_m-5	Horizontal roof displacement corresponding to 20% loss of structural load carrying capacity, in softening branch

Table 3-2 shows the possible definitions for yielding point (F_y-d_y). The actual deformation energy for all methods present in Table 3-2 except F_y-d_y-*5* is assumed to be equal to the one obtained with reference to the idealized bilinear elasto-plastic curve. Table 3-3 illustrates the possible definitions for the first significant yielding (F_1). By combining the definitions of F_y and F_1 given in Table 3-2 and Table 3-3, respectively, with the five definitions of d_m, given in Table 3-1, 90 possible different definitions of the q-factor are obtained as shown in Table 3-4.

Table 3-2 Possible Definitions for Yielding Point (F_y-d_y)

Identifier	Definition	Equations	Schematic View
F_y-d_y-1	Knee point of the idealized bilinear elastic-perfectly plastic curve based on the equivalence of the area under both curves (capacity and bilinear curve) up to d_m, with $F_y = F_m$.	1) $F_y = F_m$ 2) $d_y = 2(d_m - E_m/F_y)$ E_m is the area under the capacity curve up to "d_m"	
F_y-d_y-2	Knee point of the idealized bilinear elastic-perfectly plastic curve based on the equivalence of the area under both curves (capacity and bilinear curve) up to d_m, where the initial stiffness of the idealized system (F_y/d_y) is equal to the initial stiffness of the capacity curve ($\tan\alpha_0$).	1) $F_y/d_y = \tan\alpha_0$ 2) $d_y = d_m - \sqrt{{d_m}^2 - \frac{2E_m}{\tan\alpha_0}}$ E_m is the area under the capacity curve up to "d_m"	
F_y-d_y-3	Knee point of the idealized bilinear elastic-perfectly plastic curve based on the equivalence of the area under both curves (capacity and bilinear curve) up to d_m, where the initial stiffness of the idealized system (F_y/d_y) is equal to the secant stiffness of the capacity curve at $0.6F_m$ ($\tan\alpha_{0.6Fm}$).	1) $F_y/d_y = \tan\alpha_{0.6Fm}$ 2) $d_y = d_m - \sqrt{{d_m}^2 - \frac{2E_m}{\tan\alpha_{0.6Fm}}}$ E_m is the area under the capacity curve up to "d_m"	

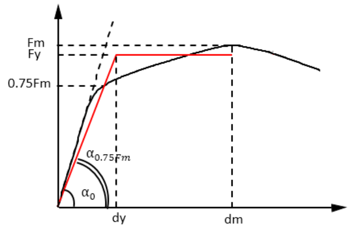
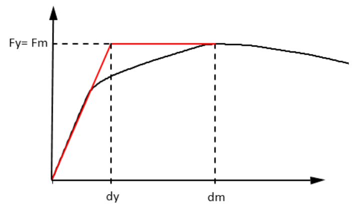
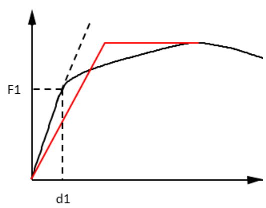
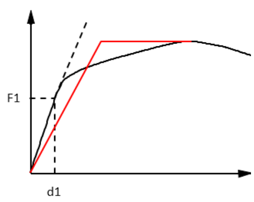
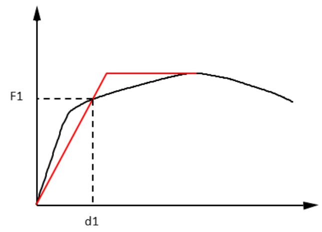
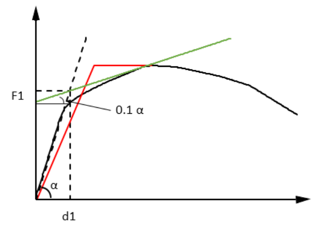
F_y-d_y-4	Knee point of the idealized bilinear elastic-perfectly plastic curve obtained by the equivalence of the area under curves up to d_m, where the initial stiffness of the idealized system (F_y/d_y) is equal to the secant stiffness of the capacity curve of the structure at $0.75F_m$ ($\tan\alpha_{0.75Fm}$).	1) $F_y/d_y = \tan\alpha_{0.75Fm}$ 2) $d_y = d_m - \sqrt{{d_m}^2 - \frac{2E_m}{\tan\alpha_{0.75Fm}}}$ E_m is the area under the capacity curve up to "d_m"	
F_y-d_y-5	Knee point of the idealized bilinear elastic-perfectly plastic curve with $F_y = F_m$ and the initial stiffness of the idealized system (F_y/d_y) equals to the initial stiffness of the capacity curve ($\tan\alpha_0$).	1) $F_y/d_y = \tan\alpha_0$ 2) $F_y = F_m$	

Table 3-3 Possible Definitions for the First Significant Yielding (F_1) and d_1

Identifier	Definition	Schematic View
F_1-d_1-1	A point on the capacity curve corresponding to the first global plasticization of the structure.	
F_1-d_1-2	A point on the capacity curve corresponding to the first yielding of any elements of the structure (Local Plasticization).	

F_1-d_1-3*	An intersection point between the capacity curve and the initial stiffness of the idealized elastic-perfectly plastic system.	
F_1-d_1-4	An intersection point between the straight line with a slope equal to the initial stiffness of the capacity curve ($\tan\alpha_0$) and the tangent to the same curve with slope equal to 10% of $\tan\alpha_0$.	

* F_1-$D_1$3 cannot be generated with F_y-d_y-2 and F_y-d_y-5 as there is no intersection point in these methods

Table 3-4 All Possible q Factor Evaluation Methods

	Fy-dy-1				**Fy-dy-2**				**Fy-dy-3**				**Fy-dy-4**				**Fy-dy-5**			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	F1-d1-1	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**	**F1-d1-1**	**F1-d1-2**	**F1-d1-3**	**F1-d1-4**

These 90 values of the behaviour factor estimated by re-analysis of the static non-linear curve ($q_{stat} = q_{\Omega}.q_{\mu}.$) will be compared with the behaviour factor originally assumed for the design of each of the case study buildings. If the estimated q_{stat} factor (=q obtained from re-analysis of the static nonlinear curve) is found to differ for more than 20% from the one originally assumed for design for any of the buildings, the method will be excluded from the data base. Eventually, the optimal methods/combinations of parameters will be identified. Figure 3-3 shows the behaviour factor evaluation procedure based on the nonlinear static analysis.

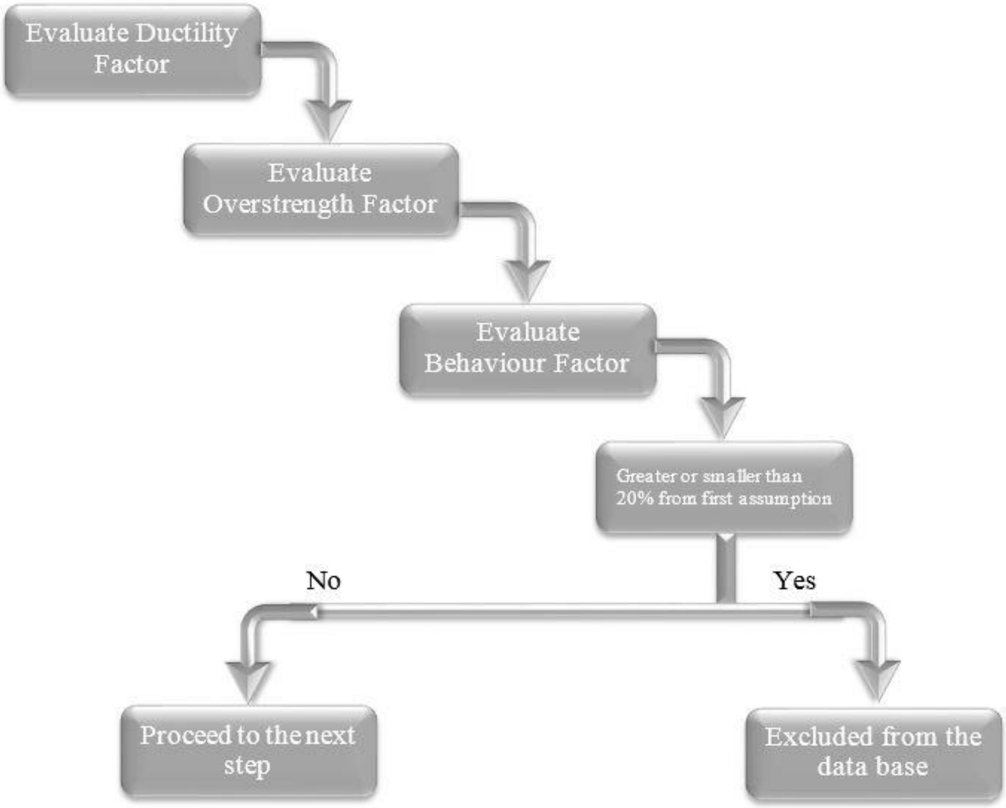


Figure 3-3 Behaviour Factor Evaluation Procedure based on the Nonlinear Static Analysis

3.2 Evaluation of the Behaviour Factor via Incremental Dynamic Analysis

In order to assess the reliability of the behaviour factor evaluated for the various case study buildings by re-analysis of the pushover curve (i.e. by means of a simplified procedure well suitable for adoption in every day's engineering practice) those behaviour factor values will be cross-checked on those obtainable by means of a more sophisticated, and hence more precise, procedure based on non-linear Incremental Dynamic Analysis (*IDA*) according to the Ballio-Setti method [10]. Despite such a method, for its complicity is not suitable for application in every day's engineering practice, it will be adopted within this research work, with the sake of comparison.

3.2.1 Site Hazard and Records Selection

After the nonlinear static analysis, incremental dynamic analysis (IDA) [39], [40] was performed in order to validate the optimal behaviour factor obtained by the above definitions (see the research flowchart, Figure 1-4). To apply incremental dynamic analysis, 30 appropriate sets of ground motion records [61], [62], for 3 high seismicity sites in Europe (i.e. Athens, Greece – Perugia, Italy – Focsani, Romania) with a peak ground acceleration equal to $a_g=0.3g$

were selected. The sites were selected based on EU-SHARE seismicity model [63] and a novel intensity measure (*IM*) [64]–[67]. Figure 3-4 implies Europe hazard sites map with 10% probability of exceedance in 50 years. The records for each site were also selected based on conditional spectrum selection [68], [69] of an average spectral acceleration *AvgSa* [67], [70], [71] as expressed in Eq. 3-2. The thirty ground motion record sets are shown in Table 9-2 in APPENDIX-A.

$$AvgSa\ (T_{Ri}) = \left(\prod_{i=1}^{n} S_a(T_{Ri})\right)^{1/n} \qquad \text{Eq. 3-2}$$

Where T_{Ri} can be selected as linearly spaced within the range of [T_L, T_H]. T_L denotes a low period near the minimum second mode of the structure and T_H is a high period which is close to 1.5 times of the maximum of the first mode period. For having a better accuracy two *AvgSa*, one for short period structures (i.e., 2 and 4 storey-building) and one for long period structures (i.e., 8 and 12 storey-building) were considered.

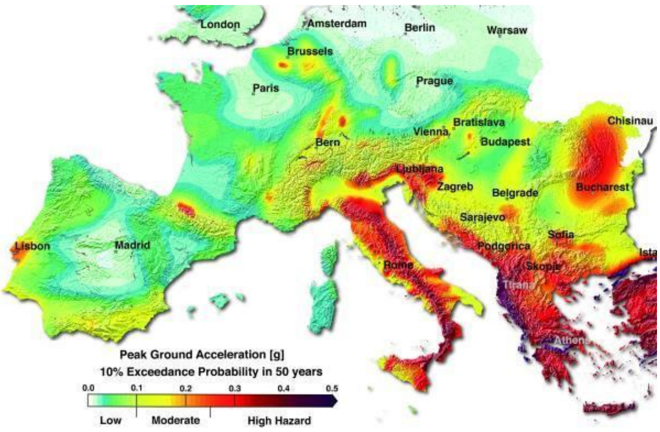


Figure 3-4 Europe Seismic Hazard Map

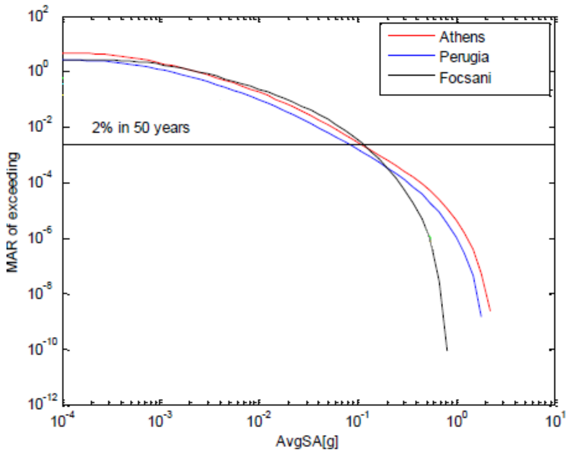


Figure 3-5 Hazard Curves for the Three European Sites for *AvgSa* with a Period Range of [0.3s, 3.0s] and an Increment of 0.2s

3.2.2 Records Scaling

In order to cover the entire range of structural response, each record should be scaled. The scaling should cover the elastic response of the structure, yield point, elasto-plastic part and eventually the dynamic instability. This can be achieved by an advanced *hunt & fill* algorithm to minimize the number of runs per record using the intensity measure (*IM*) of 5%-damped of the first mode spectral acceleration $S_a(T_1, 5\%)$ [39]. According to Vamvatsikos and Cornell [39] the hunt & fill algorithm increases the intensity measure (*IM*) until the dynamic instability occurs. Then some more runs are required for the intermediate intensity measure-level to increase the accuracy of the lower IM-level. Hence, each record will be scaled 12 times/runs/analysis in terms of IM i.e., $S_a(T_1, 5\%)$ as follows:

1st run is identifying the elastic regain and the initial stiffness of the structure.

2nd to 6th runs are trying to hunt for the first numerical non-convergence where the dynamic instability might occur.

7th and 8th runs are trying to find the better bracket of dynamic instability of the structure, in such way that the gap between the highest dynamic stability and the dynamic instability should not be greater than 10% of the previous intensity measure level. Hunt & fill algorithm does not, however, place the new scale in the middle of the previous levels, but instead the algorithm scales the next run close to the $^2/_3$ of the non-convergence one. In this way, the gap between the highest convergent run and non-convergent is always less than 10%.

9th to 12th runs are used to fill in the large gaps left during the 2nd until the 6th run.

When the runs/analysis are completed for each record, the results will be plotted as distinct IDA curves using superior spline interpolation by connecting all the points in terms of intensity measure (*IM*) and structural response (Damage Measure), represented by an engineering demand parameter "*EDP*" (e.g., maximum inter-storey drift ratio). Such curves can capture collapse due to simulated modes of failure by the characteristic flattening at high levels of intensity. Non-simulated modes of failure can also be incorporated by earlier termination of each curve, for instance, when a non-simulated shear failure is deemed to have occurred.

3.2.3 Limit-State Definition on IDA Curve

In order to ascertain the level of seismic intensity where the desired performance is violated, two limit-states need to be selected. Design codes refer to a life safety limit state (i.e., 10% in

50 years probability of exceedance). They also guarantee safety against collapse by restricting the probability of exceedance to 1-2% in 50 years (GC limit state). FEMA P695 [28] uses collapse as the point of behaviour factor determination, but it is difficult to capture collapse accurately. Therefore, choosing between the collapse and life safety limit-state is a difficult issue. Hence, potentially, we must check for both, making sure that both limit-states are satisfied and the most critical one governs the q-factor value.

Performance assessment is generally built upon two different approaches. The intensity based approach is typical of the design itself and its uses a single level of seismic intensity where one checks whether the response limitations are satisfied or not.

An improved risk-basis for the determination of q-factors would directly adjust them to ensure compliance with the target of a uniform collapse risk. This carries significant advantages as it tailors performance to each specific system's characteristics, delivering higher accuracy and fidelity of results. Still, this becomes a more difficult method needing significant hazard information for providing appropriate q-factors.

Hence, the performance of each structure will be verified against two limit-states, called Life Safety (*LS*) and Global Collapse (*GC*). *LS* will be checked against the mean annual frequency of 10% in 50 years, while *GC* for the 1 or 2% in 50 years limit. Each limit-state will be checked against brittle and ductile modes of failure. Strength checks will be employed to verify that no potential structural element enters a brittle mode of failure (e.g., exceedance of shear or axial strength) which will be satisfied automatically using force-based design approach. Ductile modes of failure will be checked against deformation to verify that no members of the structure exceed its plastic deformation capacity.

LS checking will be verified based on provisions of EN 1998-1 [2], ASCE 41-13 [72], FEMA P795 [73] and the method developed by Vulcu et al. [74], [75]. *LS* checking will be assessed through identifying the deformation against two performance levels called Significant Damage (*SD*) and Near Collapse (*NC*). According to FEMA 356 [76] *SD* and *NC* can be defined as follows:

SD is the significant damage with some margin against the total collapse of the element. *SD* limit-State is the point in which the strength of the element decreases 20% of the maximum strength, however, the displacement/rotation should not exceed 75% of *NC*.

NC is the heavy damage with low residual strength and stiffness of the elements. *NC* limit-state is the point where the element loses 80% of its strength.

GC checking is a numerically more challenging task than *LS*. It requires a very robust model that is capable of following the behavior of the structure to the global collapse. In this research case studies, this will be performed by checking only for simulated modes of failure which are explicitly incorporated in the model. Non-simulated modes of failure may also be introduced in post-processing of the results. In both cases, a single global collapse point will be established in each individual IDA curve, using the flat-line for simulated modes and the earliest occurring non-simulated mode, whichever comes first, to assess the collapse fragility. If the model is not capable of displaying global collapse, a more conservative check may be performed for ductile modes of failure, whereby the global collapse shall be assumed to occur when the first ductile element reaches its ultimate (fracturing) deformation. The guidelines of FEMA P-58-1 [77] and FEMA P-695 [28] may be utilized to assign appropriate lognormal dispersions to strength, deformation and global instability checks if needed.

3.2.4 Acceptance or Rejection of the Initial Assumption of the Design-Basis Behaviour Factor

The acceptance or rejection of the design-basis (initial assumption) of the behaviour factor will be assessed according to Cornell et al. [41] using the fragility-hazard convolution method to determine the mean annual frequency (*MAF*) of the damage exceedance. The framework is based on realizing a performance objective expressed as the probability of exceeding a specified performance level. Performance levels are quantified as expressions relating generic structural variables “demand” and “capacity” that are described by nonlinear, dynamic displacements of the structure (*LS* or *GC*). Common probabilistic analysis tools are used to convolve both the randomness and uncertainty characteristics of ground motion intensity, structural “demand”, and structural system “capacity” in order to express the probability of achieving the specified performance level.

Using the following equations, the mean annual frequency (*MAF*) of exceeding the damage state of interest can be determined.

$$\lambda_{DS} = \int_{IM} P[D > C|IM]|d\lambda(IM)| \qquad \text{Eq. 3-3}$$

Where

DS can be either *LS* or *GC*

IM (Intensity Measure) is the average spectral *AvgSa* of geometric mean of 5 to 10 $S_a(T_i)$ ordinates linearly spaced within the range of [T_2, $1.5T_1$], where T_1 and T_2 are the first and the second natural period of vibration of the system, respectively.

$\lambda(IM)$ is the *MAF* of exceedance of the hazard curve

D and *C* are the engineering demand parameter (*EDP*) that can be used to obtain the exceedance of *LS* or *GC*. *LS* is usually the response parameter that at best expresses the exceedance of *SD*. *GC* always can be adopted as the maximum inter-storey drift.

Using Eq. 3-4 and by giving the definition above, if the design of each building frame has been found to be compatible with *LS* and *CP* checking, the q-factor may be deemed to be accepted. If the verification fails, the design q-factor will consider as not acceptable.

$$\frac{\lambda_{DS}}{\lambda_{DSlim}} > \exp(K_x \, . \, k \, . \, \beta_u) \qquad \text{Eq. 3-4}$$

Where

λ_{DSlim} is the maximum allowable *MAF* limit whose exceedance signals violation of the damage state.

K_x is the standard normal variants associated with a confidence level of *x*%, $K_x = \Phi^{-1}(x)$, e.g. $K_x \approx 1$ for $x = 85\%$.

k is the local slope of the hazard curve in log-log space.

β_u is the total dispersion due to uncertainty, assuming log-normality which can be achieved by the following equation:

$$\beta_u = \sqrt{\beta_{TD}^2 + \beta_{DR}^2 + \beta_{AS}^2 + \beta_C^2} \qquad \text{Eq. 3-5}$$

Where

β_{TD} is due to test data quality rating

β_{DR} is due to design rules quality rating

β_{AS} is archetype sample size

β_C is due to element capacity test dispersion

To calculate the above parameters, the guidelines of FEMA P-695 [78] and FEMA P-58 [77] are used.

Figure 3-6 displays the concept of performance assessment for a given damage state [79] by extracting the fragility curve from nonlinear dynamic analysis. Then, using fragility-hazard convolution method, convolving the fragility curve with the hazard curve over all values of intensity measure (*IM*).

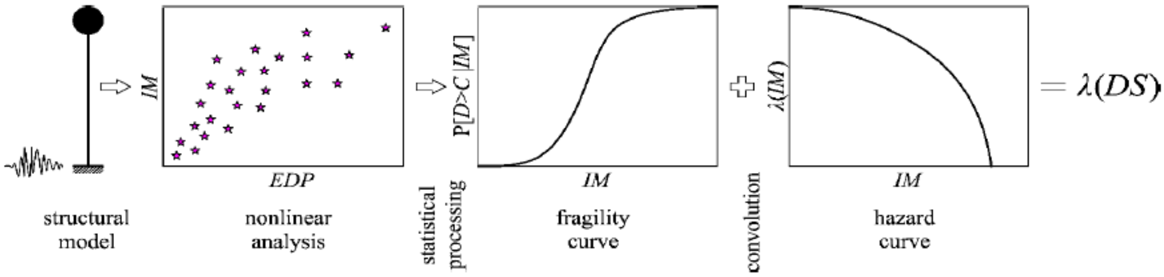


Figure 3-6 Design-Basis Behaviour Factor Acceptance Procedure [79]

3.2.5 A Method for the Calculative Determination of the Behaviour Factor via IDA

A rational procedure for the determination of the behaviour factor has been proposed by P. Setti [10] and G. Ballio *et al.* [80], [81]. By comparing the elastic with the inelastic behaviour, and identifying the point at which the linear response results “unsafely” estimating the structural behaviour. This can be achieved by plotting in a non-dimensional plane $^{\alpha}/_{\alpha_1}$ or $^{\delta}/_{\delta_1}$ the ductility demand curve (obtained by incremental inelastic dynamic analysis) with the straight line, in the same plane, representing the linear elastic behaviour.

By this definition the behaviour factor corresponds to the value beyond which a linear elastic analysis is no longer a safe solution, because the global ductility demand estimated by means of a non-linear analysis is larger than that estimated with a linear analysis. The value of the behaviour factor can be identified as the point in which the response by nonlinear analysis is initially lower than that expected with the elastic analysis, crosses the strength line and become higher than the linear response. Figure 3-7 displays the relationship between the response of the structure in terms of displacements, the behaviour factor and the intensity of the acceleration.

The behaviour factor, hence, can be obtained indirectly corresponding to the ratio between the maximum ground acceleration in which the structure may resist α and the acceleration in which the first plastic mechanism occur α_1.

$$q = {}^{\alpha}/_{\alpha_1} \qquad \text{Eq. 3-6}$$

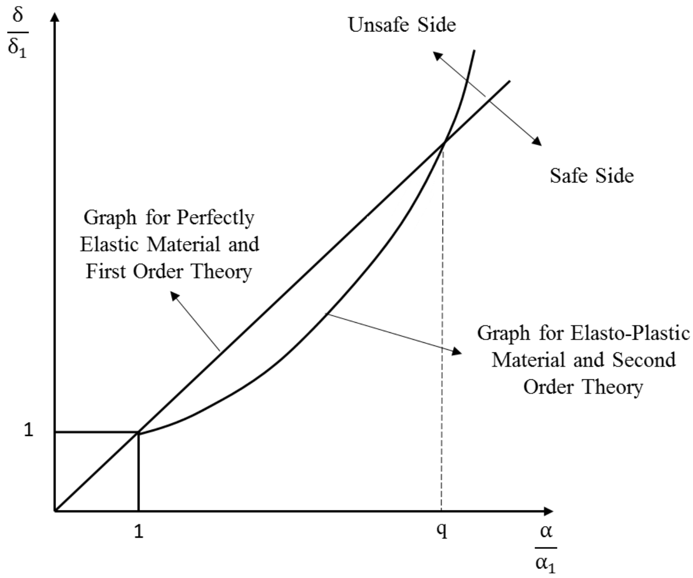


Figure 3-7 The Relationship between the Response of the Structure, the Behaviour Factor and the Intensity of the Acceleration

Now, if F_y be the seismic force corresponding to the elastic yield point of the structure, the assumed validity of the ductility factor theory ensures that the elastic-plastic oscillator will be able to resist an acceleration α which is q times greater than the acceleration α_1 needed to take the structure into the first plastic phase.

$$F_y = \alpha_1\, R(T)\, m = \left({}^{\alpha}/_{q}\right) R(T)\, m \qquad \text{Eq. 3-7}$$

Where

m is the mass of structure

$R(T)$ is the normalized spectrum

α is the ground acceleration

α_1 is the ground acceleration in which the first yield or plastic mechanism occurs (local plasticization)

$$R(T) = R_0 \qquad T \leq T_0 \qquad \text{Eq. 3-8}$$

$$R(T) = \frac{R_0}{(T/T_0)^k} \qquad T \geq T_0 \qquad \text{Eq. 3-9}$$

R_0, T_0 and k are the parameters that define the design spectrum.

If the structural response in terms of displacements from the elastic-plastic and the elastic oscillators of the same period do not perfectly correspond, and if it is still assumed that the material is ductile enough to permit the structure to deform plastically, the design based on Eq. 3-7 to Eq. 3-9 will be on the safe side. Hence, for an assigned ground acceleration α it is still possible to reduce the acceleration to α_d (design acceleration) or α_1 (acceleration in which the first plastic mechanism occur) with the assurance that the structural ductility demand for accelerations from α_d or α_1 to α will not be greater than the value of the behaviour factor q chosen to define the design spectrum.

Thus, each ratio $\alpha/_{\alpha_1} = q$ between the intensity of the maximum ground acceleration and the acceleration in which the first plastic mechanism occurs corresponds to a response in terms of displacement $\delta/_{\delta_1} = q$ (the maximum displacement of the oscillator subjected respectively to the maximum ground accelerations α and to α_1). The bisecant of the plane $\alpha/_{\alpha_1} = \delta/_{\delta_1} = q$ locates the points which verify the linearity of the response.

If, however, the elastic-plastic response of the structure differs from what would be expected through the ductility factor theory, the design criterion based on the value of acceleration α is still on the side of safety, but only as long as the displacement response $\delta/_{\delta_1}$ is less than q.

The intersection of the bisecant line with curve $\alpha/_{\alpha_1} - \delta/_{d_1}$, obtained as the nonlinear structural response by an incremental dynamic analysis represents the maximum value of the behaviour factor to be assumed for the structure.

Chapter 4

CASE STUDIES

4 CASE STUDIES

4.1 General Assumptions

In order to evaluate the behaviour factor for steel moment resisting frame structures based on re-analysis of the pushover curve using the definitions given in chapter three, 102 composite steel-concrete Moment Resisting Frame systems (*MRFs*) with different height/storey, bay length and number of bays are examined. The case studies are subdivided into two parts. a) 96 typical/ordinary steel moment resisting frame structures as conventional buildings. b) 6 innovative dissipative bolted and welded beam splices [34], [36], [38], [82]–[86] as non-conventional buildings which make the structure more complex than the conventional one as part of the *EU-RFCS* research project *INNOSEIS* [87] are investigated. *INNOSEIS* project focuses on valorisation of the innovative dissipative (anti-seismic) devices suitable for steel structures [32]–[37].

All building configurations are vertically and horizontally regular. The height of each story is considered as equal to 4m. The buildings consist of steel-concrete composite moment resisting frame in the Y-direction and concentrically braced steel frame in the X-direction. The concentric bracing system is located to accommodate the columns around their weak axis bending and the moment resisting frame system is located in the direction along which the columns are placed with strong axes bending. The buildings are considered as general offices (class-B). The vertical seismic load and snow load are neglected for all case studies. The assumed design peak ground acceleration is 0.3g with the spectrum type 1 and soil type C. The buildings are designed according to EN 1993-1 [30], EN 1994-1 [31], EN 1998-1 [2] and to the specific design guidelines of the dissipative systems (FUSEIS bolted and welded beam splices) [88] where needed. The buildings are designed with different values of the behaviour factor equal to 2, 3, 4, 5, 6 and 7 for conventional structures and q_{des}=4 assumed for non-conventional structures. Considering "non-conventional structures" as case study, the proposed value of the design behaviour factor is not present in EN 1998-1: 2004 [2]. The proposed design behaviour factor of such structures are studding during this research. This, perhaps, is one of the typical applications of the proposed assessment procedure, allowing to overcome the codes.

4.2 Description of the Buildings

4.2.1 Material

4.2.1.1 Non-dissipative zones

The materials adopted for non-dissipative zones are as follows:

- Structural Steel: S355
- Concrete: C25/30
- Steel Sheeting: Fe320
- Reinforcing Steel: B500C

4.2.1.2 Dissipative zones

During the earthquake, it is expected that the dissipative zones yield before other zones i.e., non-dissipative zones, hence, according to EN 1998-1 [2], the yield strength $f_{y,max}$ of the dissipative zones is satisfied by Eq. 4-1.

$$f_{y,max} \leq 1.1 \gamma_{ov} f_y \qquad \text{Eq. 4-1}$$

Where

γ_{ov} is the overstrength factor, the recommended value is 1.25

f_y is the nominal yield strength of the steel

4.2.2 Loads and load combinations

A summary of the applied loads according to EN 1991-1-1: 2002 [89] is given in the following;

- Dead Loads:

  2.75 kN/m² composite slab + steel sheeting

- Superimposed Loads:

  Services, ceiling, raised floor: 0.70 kN/m² for intermediate floors

  1.00 kN/m² for top floor

  Perimeter walls 4.00 kN/m

- Live Loads:

  Offices (Class B): 3.00 kN/m²

  Movable partitions 0.80 kN/m²

  Total live load: 3.80 kN/m²

  Snow load to be ignored

- Seismic Load:

Importance factor: $\gamma_I = 1.0$

Peak ground acceleration: $\alpha_{gR} = 0.30 \cdot g$

Ground Type C – Type 1 spectrum:

S =1.15 T_B = 0.20 Sec. T_C = 0.60 Sec. T_D = 2.00 Sec.

Lower bound factor: $\beta = 0.2$

Vertical ground acceleration to be ignored.

Behaviour factor q= 2, 3, 4, 5, 6 and 7 for conventional structures and q=4 for non-conventional structures

The seismic masses are calculated according to Eq. 4-2 and presented in Table 4-1 which represents the coefficients which are used for the various load combinations.

$$\sum_{j>1} \boldsymbol{G_{k,j}} + \sum_{i>1} \boldsymbol{\Psi_{2,i} \cdot \varphi_i \cdot Q_{k,i}} \quad \text{Eq. 4-2}$$

Table 4-1 Coefficients for Various Load Combinations

Coefficient	Value
γ_G	1.35
γ_Q	1.50
Ψ_2 Office (Class B)	0.30
Ψ_2 Roof	0.00
φ Correlated floors	0.80
φ Roof	1.00

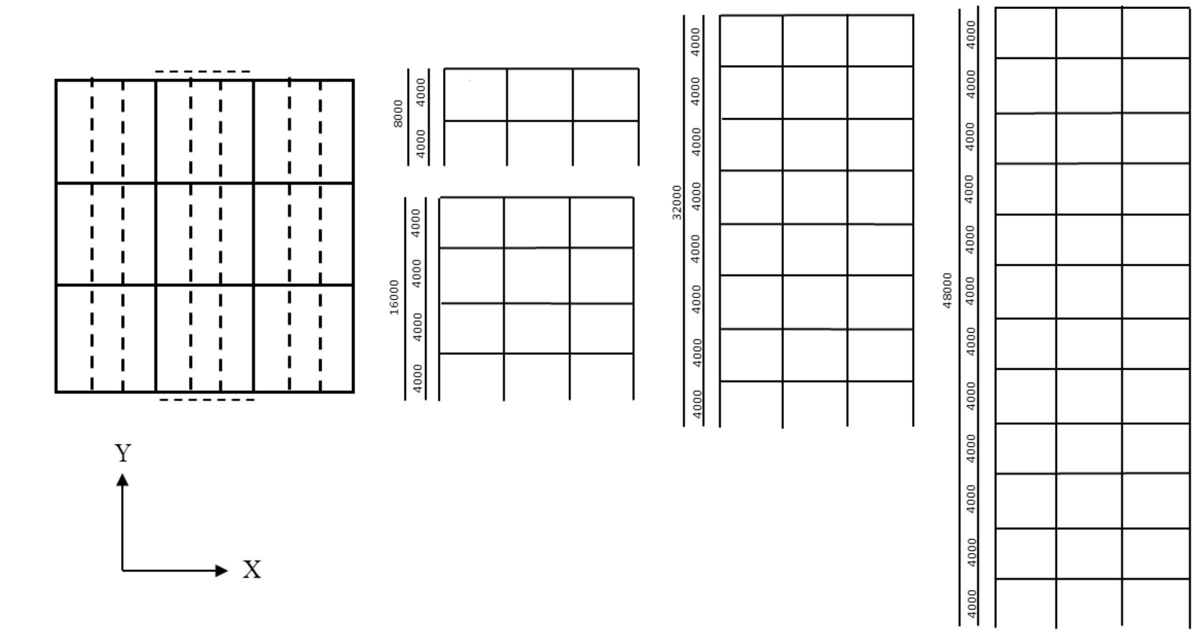
4.2.3 Typical MRF Buildings (*Conventional Structures*)

4.2.3.1 Structural Geometry

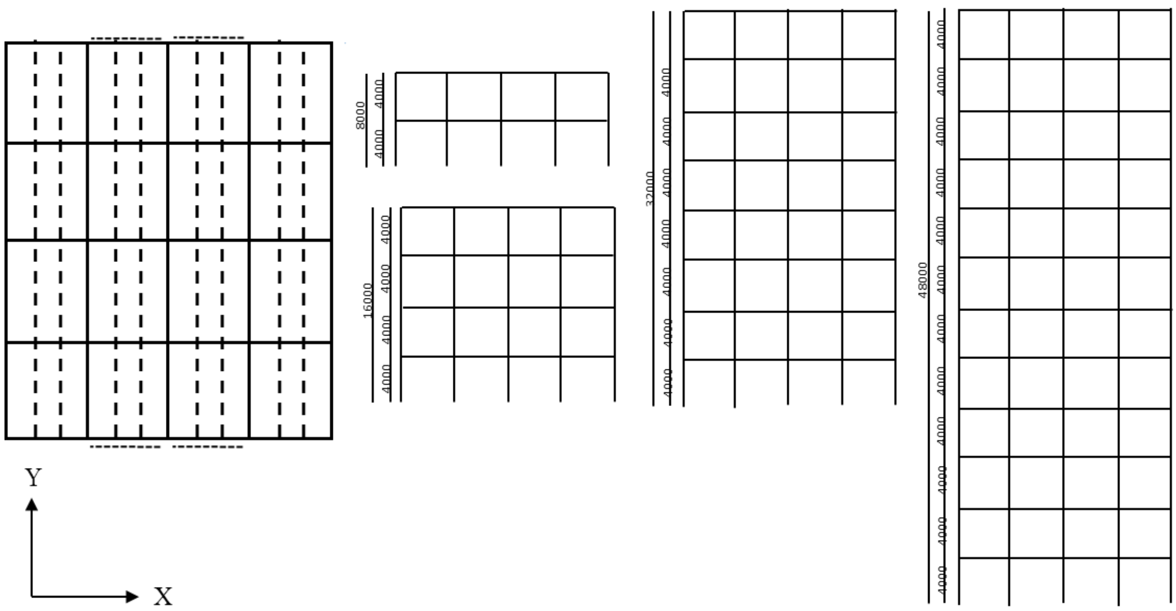
Conventional structures comprise four building configurations as follows:

- 2 storey as low-rise buildings
- 4 storey as mid-rise buildings
- 8 storey as high-rise buildings
- 12 storey as tall buildings

The conventional structures are also different in number of bays in the Y-direction (3 and 4 bays) and the length of the bays (6m and 8m). Figure 4-1 shows the schematic view of plan and elevation of the 2/4/8 and 12-story of the typical *MRF* structures under consideration a) 3-bay configuration b) 4-bay configuration. The number of bays are fixed to 3 with the length equal to 8m in X-direction where the load resisting frame system is considered as concentrically X-braced frames (*X-CBFs*).



a) 3-Bays Configuration



b) 4-Bays Configuration

Figure 4-1 Schematic View of Plan and Elevation of 2/4/8 and 12-story of the typical *MRF* structures a) 3-Bays Configuration b) 4-Bays Configuration (diameter in mm)

For simplicity, from now on the case studies may be introduced by an identifier. For instance, *12S-3B-6m* is represented the 12-Storey 3-Bays and 6m length of the bay and *4S-4B-8m* is represented the 4-Storey 4-Bays and 8m length of the bay.

Figure 4-2 displays the schematic view of an example of 8S-3B-6m. Table 4-2 and Table 4-3 show the structural height (*H*) and width (*B*) for 3-bay and 4-bay buildings.

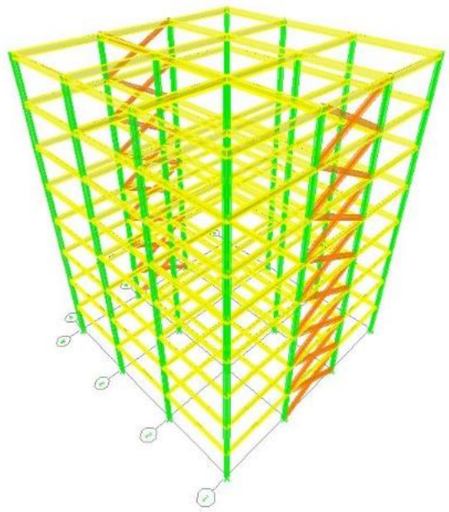


Figure 4-2 Schematic View of an Example of 8-Storey 3-Bays having the Length of the Bay Equal to 8m

Table 4-2 Structural Height (*H*) and Width (*B*) of the Typical 3-Bay *MRF* Systems

3-Bays				
No. of Storey / Bay length	2-Storey	4-Storey	8-Storey	12-Storey
8 Meter	H= 8m B= 24m	H= 16m B= 24m	H= 32m B= 24m	H= 48m B= 24m
6 Meter	H= 8m B= 18m	H= 16m B= 18m	H= 32m B= 18m	H= 48m B= 18m

Table 4-3 Structural Height and Width of the Typical 4-Bay *MRF* Systems

4-Bays				
No. of Storey / Bay length	2-Storey	4-Storey	8-Storey	12-Storey
8 Meter	H= 8m B= 32m	H= 16m B= 32m	H= 32m B= 32m	H= 48m B= 32m
6 Meter	H= 8m B= 24m	H= 16m B= 24m	H= 32m B= 24m	H= 48m B= 24m

4.2.4 Innovative Dissipative FUSEIS Bolted & welded Beam Splices For Non-Conventional Structures

4.2.4.1 Introduction

The bolted/welded beam splices are a kind of seismic fuses for steel and composite steel-concrete moment resisting frames that provide good seismic performance and easiness of repair work. These systems are designed to act as fuses, forcing the plastic hinges to develop within the fuse devices, preventing the spreading of damage into the main structural members (beams and columns). Hence, concentrating all the damage into the pre-defined location, repair work is limited only to replacing the splice plates with the new ones, thus ensuring low-cost and very quick repair work [38], [90].

Dissipative beam splices (FUSEIS 2) [38], [88] consist in a cross-sectional weakening part located at the beam ends at a certain distance from beam-to-column connections, obtained introducing a discontinuity on the composite beams and then splicing the two parts using additional steel plates (see Figure 4-3 and Figure 4-4) a) bolted [91], [92] or b) welded [82], [84] to the web and flange of the main beam. Figure 4-4 shows the schematic representation of the bolted beam splice as an example. The behaviour of these beam splices was studied numerically and experimentally during the European research project *FUSEIS* funded by "Research Fund for Coal and Steel" [93], and the results have been published in many articles [38], [82], [84], [91]–[94].

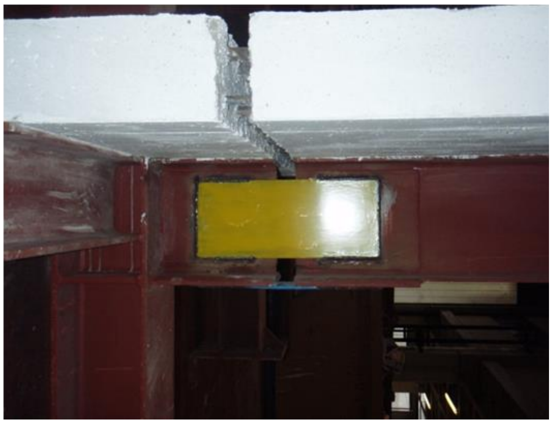
The part of the beam near to the connection is reinforced with additional plates in order to obtain an adequate over-strength, hence concentrating all the damage to the beam splice. The gap in the reinforced concrete slab prevents damage due to concrete crushing and due to bending during earthquake motions. The longitudinal rebars are continuous over the gap, thus ensuring the transmission of stresses.

In case of *bolted* beam splices, the connections between the steel plates and the beams are obtained by means of high strength friction grip (*HSFG*) bolts. These bolts are tightened according to the provisions given in EC 14399-2 [95].

In case of *welded* beam splices, the welding process used is shielded metal arc welding, also known as "*stick*" welding.

Considering non-conventional case studies, proposed values of the q factors are not present in EN1998-1 [2]. This is one of the typical applications of the proposed assessment procedure, allowing to overcome the codes. The assumed design behaviour factor is set to be equal to 4.0.

a) Bolted



b) Welded

Figure 4-3 Dissipative Beam Splices a) Bolted b) Welded

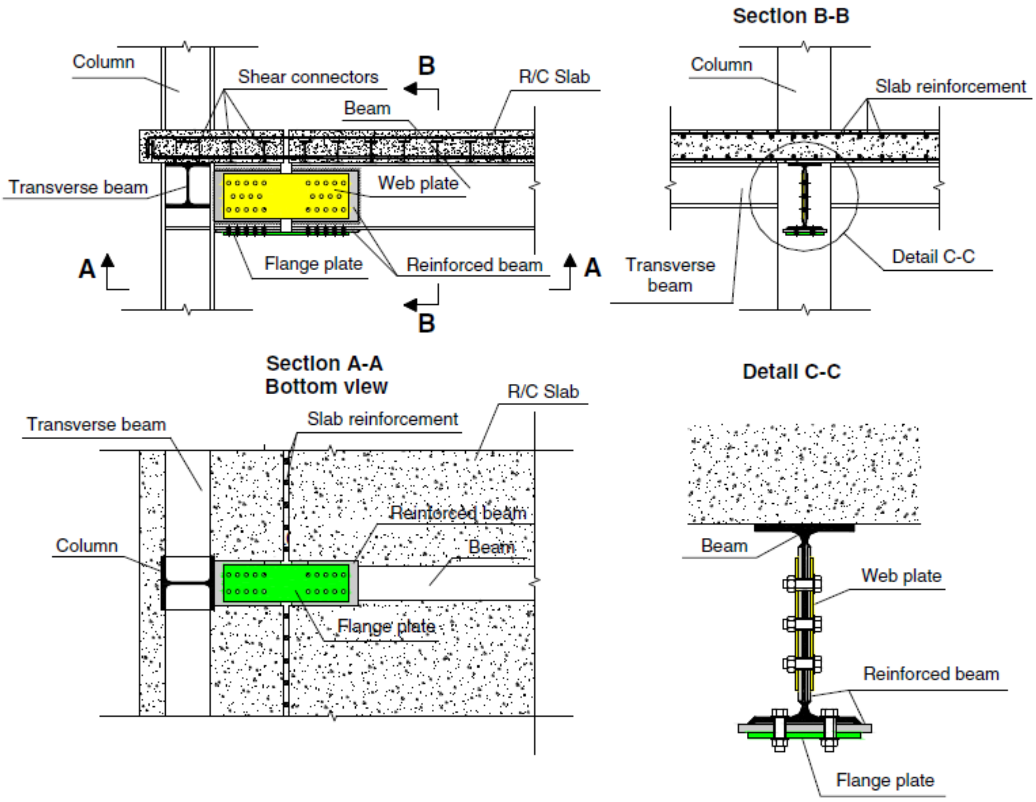


Figure 4-4 Bolted Beam Splice Main Section Elements

4.2.4.2 Structural Geometry

The non-conventional structures comprise three building configurations as follows:

- 2 storey, low-rise buildings
- 4 storey, mid-rise buildings
- 8 storey, high-rise buildings

The number of bays are fixed to 3 with the length equal to 8m for both Y-direction where the bolted/welded beam splices are employed and X-direction where X-braced frame systems (*X-CBFs*) are applied. Figure 4-5 shows the schematic view of plan and elevation of the 2/4/8-story of non-conventional buildings. Figure 4-6 displays the schematic view of an example of non-conventional 2-storey building. Table 4-4 shows the structural height (*H*) and width (*B*) for non-conventional structure having bolted and welded beam splices.

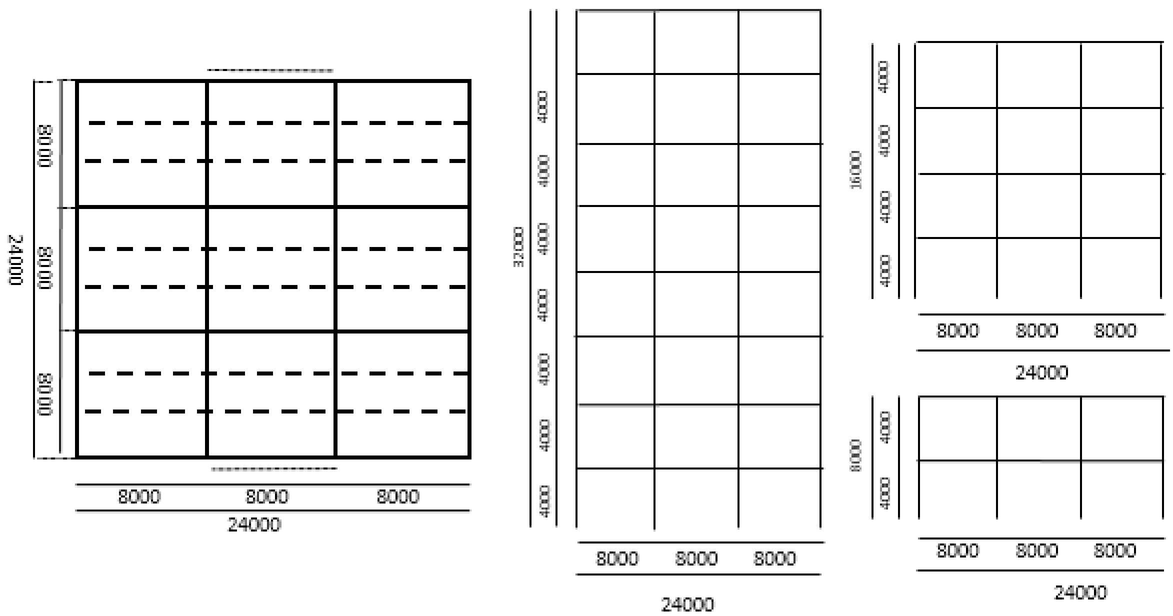


Figure 4-5 Schematic View of Plan and Elevation of the 2/4/8-story of non-conventional building (diameter in mm)

Table 4-4 Structural Height and Width for Non-Conventional Structure Having Bolted/Welded Beam Splices

3-Bays			
No. of Storey / Bay length	2-Storey	4-Storey	8-Storey
8 Meter	H= 8m B= 24m	H= 16m B= 24m	H= 32m B= 24m

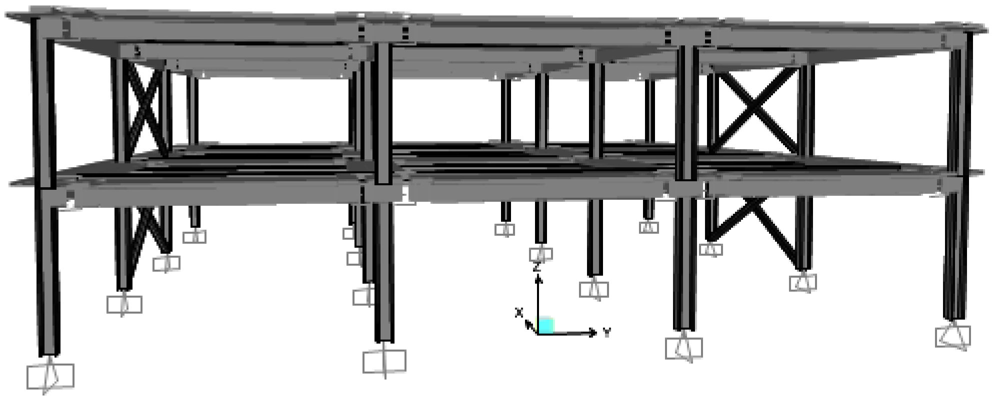


Figure 4-6 Schematic View of An Example of Non-Conventional 2-Storey Building

4.3 Analysis and Design

4.3.1 Current Steel Design and Seismic Design Provision Requirements

The structures must be designed according to all requirements specified in the code. In this research, the current steel code EN 1993-1-1: 2005 [96] which is based on capacity design and seismic provision EN1998-1: 2004 [2] are used. Some of the code requirements can be listed as follows:

4.3.1.1 Strength

The resistance of steel members in a conservative approximation for all cross section classes can be achieved by a linear summation of the utilization ratios for each stress resultant. The resistance of steel members subjected to the combination of N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ can be calculated by the following equation:

$$\frac{N_{Ed}}{N_{Rd}}+\frac{M_{y,Ed}}{M_{y,Rd}}+\frac{M_{z,Ed}}{M_{z,Rd}}\leq 1. \qquad \text{Eq. 4-3}$$

Where

N_{Ed} is the normal design force

N_{Rd} is the design values of the resistance to normal forces

$M_{y,Ed}$ is the design bending moment in Y-axis

$M_{y,Rd}$ is the design values of the resistance to bending moment in Y-axis

$M_{z,Ed}$ is the design bending moment in Z-axis

$M_{z,Rd}$ is the design values of the resistance to bending moment in Z-axis.

4.3.1.2 Buckling Resistance

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \qquad \text{Eq. 4-4}$$

Where

N_{Ed} is the design value of the compression force

$N_{b,Rd}$ is the design buckling resistance of the compression member $N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$

Where χ is the reduction factor for the relevant buckling mode and γ_{M1} can be taken as 1.0.

A laterally unrestrained member subject to major axis bending should also satisfy the lateral torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \qquad \text{Eq. 4-5}$$

Where

M_{Ed} is the design value of the moment and

$M_{b,Rd}$ is the design buckling resistance moment

4.3.1.3 Drift Constraints

For buildings having ductile, non-structural elements, the codified allowable drift can be calculated as follows:

$$d_r \, \nu \leq 0.0075h \qquad \text{Eq. 4-6}$$

Where

h is the storey height

ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. The recommended values of ν are 0,4 for importance classes III and IV and 0,5 for importance classes I and II.

d_r is the design inter-storey drift which can be evaluated as the difference of the average lateral displacements at the top and bottom of the storey under the consideration multiplied by the relevance design behaviour factor.

$$d_r = d_S . q \qquad \text{Eq. 4-7}$$

4.3.1.4 Second-Order Effects (P-Δ Effects)

$$\theta = \frac{P_{tot} . d_r}{V_{tot} . h} \leq 0.1 \qquad \text{Eq. 4-8}$$

Where

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation.

V_{tot} is the total seismic storey shear

If $0.1 < \theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$. However, according to EN 1998-1 [2], the value of the coefficient θ shall not exceed 0.3.

4.3.1.5 Weak-Beam Strong-Column Situation

The following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns

$$\Sigma\, M_{Rc} \geq 1.3\ \Sigma\, M_{Rb} \qquad \text{Eq. 4-9}$$

Where

ΣM_{Rc} is the sum of the design values of the bending resistance of the columns framing the joint. The minimum value of column bending resistance within the range of column axial forces produced by the seismic design situation should be used in Eq. 4-9.

ΣM_{Rb} is the sum of the design values of the bending resistance of the beams framing the joint. When partial strength connections are used, the bending resistance of these connections is taken into account in the calculation of ΣM_{Rb}.

4.3.2 Design Principal of the Beam Splices as Non-Conventional Structures

In the building design process of beam splices, the cross-sections of the relevant structural elements should be first pre-designed for the same building, but without any dissipative elements i.e. Bolted/welded Beam Splices, considering the relevant limit states. The bolted/welded beam splices, then should be included at all beam ends that belong to the *MRF* system. To design a building equipped with *FUSEIS* bolted/welded beam splices, different steps as follows should be carried out.

First, design the conventional building without dissipative elements and verify all codified requirements according to EN 1993-1-1: 2005 [96] and EN 1998-1: 2004 [2]. At the end of this step, the cross sections of the steel columns and the composite steel-concrete beams are selected. Then, seismic response spectrum analysis (*RSA*) on the building should be performed. Thus the bending moment M_{Ed} at the ends of all beams will be identified. These values are taken as reference for the performance required to the dissipative beam splices in terms of moment resistance ($M_{Ed} \approx M_{y,\ fuse}$). In fact, in the building subjected to the design seismic actions (*ULS*), the exploitation of the post-elastic resources of the dissipative and reparable joints is to be guaranteed. It is worth noting that the distribution of the bending moment associated with seismic actions is not uniform along the different floors, with the result that the beams at lower stories are more stressed than the ones at upper levels. This observation leads to assume several reference resistance thresholds of beam splices for multi-storey buildings. Therefore, the final layout of the structure should be characterized by increasing beam splice dimensions for lower beam levels in order to activate a uniform global collapse mechanism and hence to avoid the onset of brittle soft-storey mechanisms.

Since the fuse plates may buckle at hogging rotations, the bending behaviour of the fuses is asymmetric in most of the cases. During earthquake both cases can occur and the global behaviour is governed by the lower resisting once. Therefore, there is a need for computing both sagging and hogging resistant moments of the fuse, $M^{+}_{Rd,fuse}$ and $M^{-}_{Rd,fuse}$, respectively. The buckling behaviour of the fuse plates may be controlled by the geometric slenderness, given in Eq. 4-11. By assuming a plastic distribution of forces for bending-shear interactions, the contribution of the web plates of the fuse to the bending resistance should be neglected. The bending resistance of the beam splices should be obtained through an elastic-plastic analysis considering an adequate value for α.

$$\alpha = \frac{M_{Rd,fuse}}{M_{pl,Rd,beam}} \qquad \text{Eq. 4-10}$$

$$\lambda = \frac{L_0}{t_f} \qquad \text{Eq. 4-11}$$

Where

t_f is the flange thickness of the beam splices

L_0 is the free buckling length which is based on buckling mechanism of the FUSEIS (see Fig. 4-7) can be calculated by the Eq. 4-12.

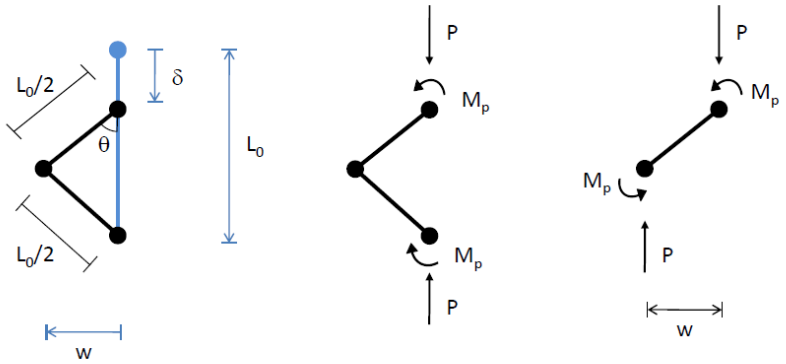


Fig. 4-7: Buckling mechanism of the FUSEIS

$$L_0 = \frac{2\sqrt{2}\, M_p}{A f_y \sqrt{\varepsilon}} \quad \text{Eq. 4-12}$$

Where

A is the area of flange plate

F_y is the yield strength of the flange plate

M_p is plastic moment of the rectangular cross-section of the plate which can be calculated by Eq. 4-13.

$$M_p = \frac{b_f * t_f{}^2}{4} * f_y \quad \text{Eq. 4-13}$$

Where

t_f is the thickness of the flange plate

b_f is the width of the flange plate

Generally, two main parameters govern the verification results: the bending resistance and the initial elastic stiffness of the FUSEIS beam splices. Once the bending resistance and the stiffness level required to verify the structure are identified, the geometrical properties of beam splices have been finalized. The area of the flange plate can be calculated referring to the hogging bending resistance required by the following equation:

$$A_{f,fuse} = \frac{M^{-}_{Rd,fuse}}{f_{yd}\, z} \qquad \text{Eq. 4-14}$$

Where

$M^{+}_{Rd,fuse}$ is the sagging resistant moment of the bolted beam splice

f_{yd} is design yield strength of the structural steel according to EN1993-1-1

z is the distance between the flange plate and the center of gravity of the rebar layers which can be calculated as follows:

$$z = h_a + h_p + \frac{h_c}{2} \qquad \text{Eq. 4-15}$$

Where

h_a is the height of the steel beam

h_p is the height composite beam

h_c is the height of concrete slab

The web plates of the bolted beam splice are designed to resist shear forces only. According to the capacity design principles, the maximum shear forces that could possibly be developed on the beam ends depends on the resistant capacities of the beams. The dimension of the web plates can be obtained by the following equation:

$$A_w = \frac{V_{Ed}\sqrt{3}}{f_{yd}} \qquad \text{Eq. 4-16}$$

Where

V_{Ed} is the total shear force

$$V_{Ed} = V_{Ed,M} + V_{Ed,G} \qquad \text{Eq. 4-17}$$

Where

$V_{Ed,M}$ is the shear force due to moment resistance of the fuse:

$$V_{Ed,M} = \frac{M^{+}_{fuse,Rd} - M^{-}_{fuse,Rd}}{d} \qquad \text{Eq. 4-18}$$

Where

$V_{Ed,G}$ is the shear force due to gravity loads

d is the distance between the fuses.

The shear buckling can be verified by the following equation:

$$\frac{h_w}{t_w} < \frac{72}{\eta}\sqrt{\frac{235}{f_{yd}}} \qquad \text{Eq. 4-19}$$

Where

η is a parameter that may be assumed to be equal to 1.2 steel grades up to and including S460. For higher steel grades η = 1.00 is recommended

4.3.3 Modeling and Simulation

The modelling of the buildings was performed by the commercial finite element software *SAP2000.Ve.19*. Since 2D model can offer faster and more reliable convergence, especially when investigating conditions close to of global collapse, offering accuracy without a heavy computational burden, only one of the mid identical frames was considered as two-dimensional rectangular frame fixed at the base. All beams and columns were simulated as beam elements, while no-section shell elements were used for the distribution of the load's area. The loads are added manually as a line loads and pin loads. The buildings are designed according to EN1993-1 [30], EN1998-1 [2], EN1994-1 [31] and to the specific design guideline of the bolted/welded beam splice as a dissipative system where needed [87], [88] .

4.3.4 Seismic Design Situation

According to EN 1998-1:2004 [2], to account for uncertainties in the location of masses and thus for the rotational component of the seismic motion, additional accidental mass eccentricity of 5% in both directions are considered. To account for the torsional effects, the story seismic forces in both main directions were calculated based on the lateral force method of EN 1998-1: 2004 [2]. The final seismic design situation accounting for accidental torsional effects was derived by Eq. 4-20 and Eq. 4-21.

$$E = E_x + 0.3E_y \pm T \qquad \text{Eq. 4-20}$$

$$E = 0.3E_x + E_y \pm T \qquad \text{Eq. 4-21}$$

Where:

T is considered as $T_x + T_y$;

T_x and T_y are accidental torsional effects of applied story seismic force with eccentricity of

5% in X and Y direction, respectively;

E_x and E_y are results of analysis without accidental torsion by applying RSA in X and Y direction, respectively.

The seismic combination is calculated according to Eq. 4-22.

$$\sum_{j>1} G_{k,j} + \sum_{i>1} \psi_2 \times Q_{k,i} + E \qquad \text{Eq. 4-22}$$

Where:

$G_{k,j}$ is the gravity load effects in seismic design situation;

$Q_{k,i}$ is the movable load effects in seismic design situation;

ψ_2 is given for the general office as for class B

E is the effect of the seismic action including accidental torsional effects

See Table 4-1 for the coefficients used for the load combinations.

4.3.5 Response Spectrum Analysis

The response spectrum analysis which permits the multiple modes of response of a building to be taken into account in the frequency domain is considered in the design scenario. In this kind of analysis, the response of a structure is defined as a combination of many modes that in a vibrating string correspond to the harmonics. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass, then they are combined to provide an estimate of the total response of the structure by calculating the magnitude of forces in all directions.

The commonly used methods for combining the peak response quantity of interest for a MDOF system are as follows:

• Absolute Sum (*ABSSUM*) method,

• Square root of sum of squares (*SRSS*) method, and

• Complete quadratic combination (*CQC*) method

In *ABSSUM* method, the peak responses of all the modes are added algebraically, assuming that all modal peaks occur at the same time. The maximum response can be achieved as follows:

$$r_{max} = \sum_{i=1}^{n} |r_i| \qquad \text{Eq. 4-23}$$

According to Chopra 2007, the *ABSSUM* method provides a much conservative estimate of resulting response quantity and thus provides an upper bound to peak value of total response [97].

In the *SRSS* method, the maximum response is obtained by square root of sum of square of response in each mode of vibration and can be addressed by:

$$r_{max} = \sqrt{\sum_{i=1}^{n} r_i^2} \qquad \text{Eq. 4-24}$$

The *SRSS* method of combining maximum modal responses is fundamentally sound where the modal frequencies are well separated. *SRSS* is also suitable when periods differ by more than 10%.

In *CQC*, the maximum response from all the modes can be obtained by:

$$r_{max} = \sqrt{\sum_{i=1}^{n} . \sum_{j=1}^{n} r_i . \alpha_{ij} . r_j} \qquad \text{Eq. 4-25}$$

Where

r_i and r_j are maximum responses in the i^{th} and j^{th} modes, respectively

α_{ij} is correlation coefficient

CQC is suitable when periods are closely spaced, with cross-correlation between mode shapes.

The combination method used in this research is the square root of the sum of the squares (*SRSS*).

4.3.5.1 Design Spectrum

For the horizontal components of the seismic action the design spectrum, $S_d(T)$, can be defined by the following expressions:

$$0 \leq T \leq T_B : \; S_d(T) = a_g . S . \left[\frac{2}{3} + \frac{T}{T_B} . \left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \qquad \text{Eq. 4-26}$$

$$T_B \leq T \leq T_C : \; S_d(T) = a_g . S . \frac{2.5}{q}$$ Eq. 4-27

$$T_C \leq T \leq T_D : \; S_d(T) = \begin{cases} = a_g . S . \frac{2.5}{q} . \left[\frac{T_C}{T}\right] \\ \geq \beta . a_g \end{cases}$$ Eq. 4-28

$$T_D \leq T : \; S_d(T) = \begin{cases} = a_g . S . \frac{2.5}{q} . \left[\frac{T_C T_D}{T^2}\right] \\ \geq \beta . a_g \end{cases}$$ Eq. 4-29

Where

$S_d(T)$ is the design response spectrum

T is the vibration period of a linear single-degree-of-freedom system

a_g is the design ground acceleration

T_B is the lower limit of the period of the constant spectral acceleration branch

T_C is the upper limit of the period of the constant spectral acceleration branch

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor

q is the behaviour factor

β is the lower bound factor for the horizontal design spectrum. The recommended value for β is 0,2.

Figure 4-8 shows the shape of the design response spectrum. η denotes the damping correction factor with a reference value of η = 1 for 5% viscous damping. Figure 4-9 displays the design response spectrum for different values of the behaviour factor used in the design scenario.

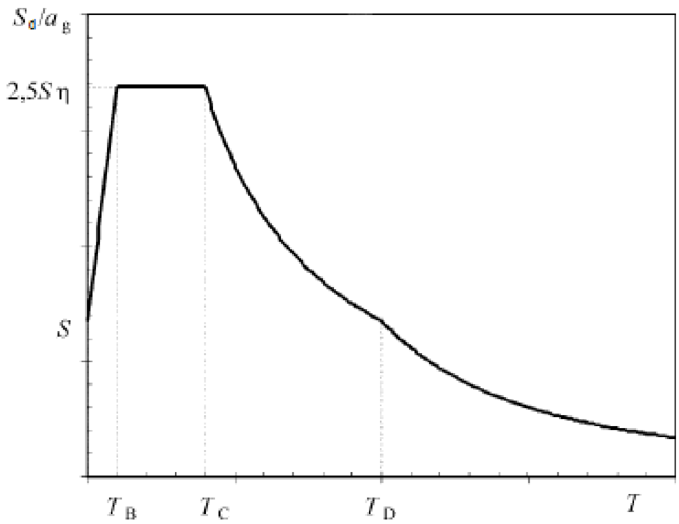


Figure 4-8 Shape of Design Response Spectrum

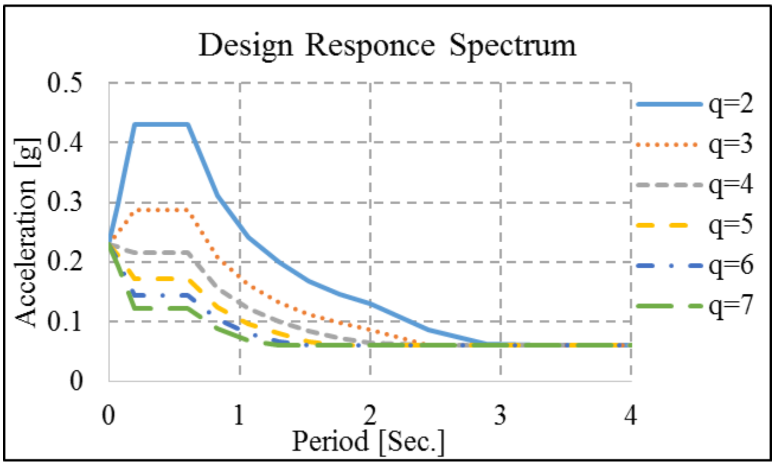


Figure 4-9 Design Response Spectrum for Different Values of Behaviour Factor

4.4 Design Summary

The section sizes of all structural elements are determined using relevant Eurocode at Ultimate Limit State (*ULS*). The deflections are checked at Serviceability Limit State (*SLS*), using persistent situation load combinations. The masses of each floor are computed as per 1.0 Dead+0.3 Live load combination.

Slabs are designed as composite for all floors. They have been designed and checked according to the requirements of EN 1994-1 [31] for all possible load combinations/situations. The thickness of the steel sheet is designed as 0.80 mm and the longitudinal reinforcement as Ø8/100. The steel beam is assumed to be connected to the concrete slab with the full shear transfer. Figure 4-10 displays the designed composite slab section.

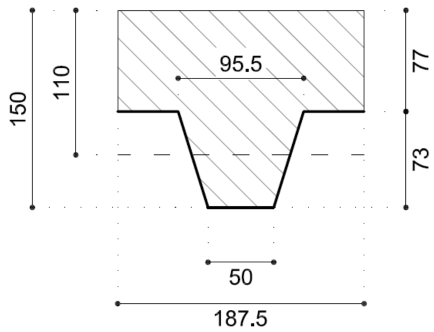


Figure 4-10 Designed Composite Slab Section

For all floors and buildings having 8 meter length of the bay, IPE450 and buildings with 6 meter length of the bay, IPE 400 have been chosen for beams. Table 4-5 shows the beam section for all types of building configurations. Secondary beams are composite and simply supported. Construction phases were critical for the design of these beams, so temporary supports for these beams will be placed in order to reduce both bending deformation and section size. HEA 200 has been chosen for secondary beams for all floors and buildings.

Table 4-5 Beam Section for all Types of Building Configurations

Length of the Bay	Beam Section
8m	IPE450 + concrete slab
6m	IPE 400 + concrete slab

4.4.1 Typical MRF Buildings (Conventional Buildings)

All the structures are designed according to EN1993-1 [30], EN1998-1 [2] and EN1994-1 [31]. Therefore, all the design criteria and performance requirements are fulfilled. Columns are designed as steel members, with their section varying depending on the floor and the building. The assigned sections for different design *q* factors are given in detail in APPENDIX-B from Table 9-3 to Table 4-6.

4.4.2 Innovative Dissipative FUSEIS Bolted & Welded Beam Splices (*Non-Conventional Structures*)

4.4.2.1 Design of Columns

Figure 4-11 displays plan view of the typical non-conventional structure. Blue lines illustrate the X-bracing system, whereas the red lines imply the beam splices. Table 4-6 shows the columns section for the 8, 4 and 2 storey buildings having bolted and welded beam splices.

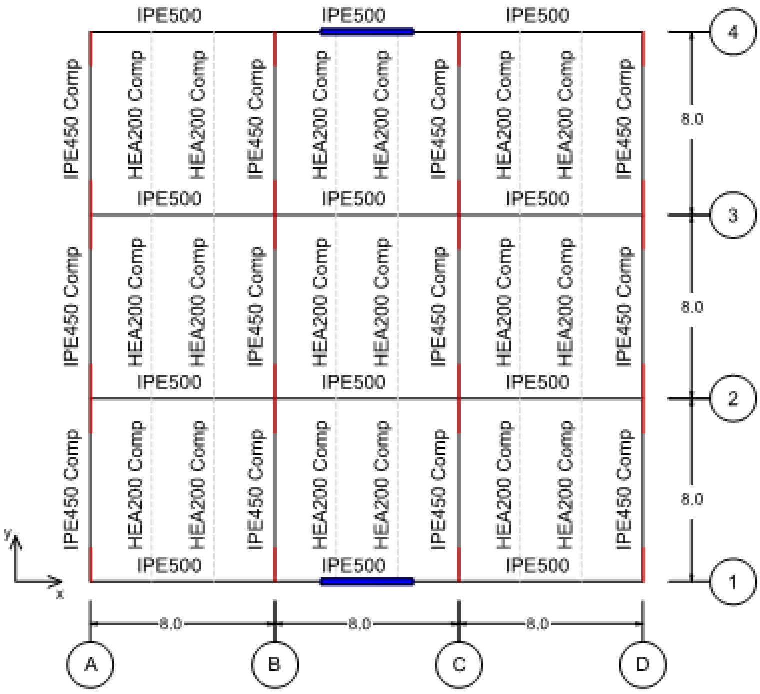


Figure 4-11 Plan View of the Typical Non-Conventional Structure. Blue Lines Illustrate the X-Bracing System, Whereas the Red Lines Imply the Beam Splices

Table 4-6 Columns Section for the 8/4 and 2 Storey Buildings

	8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM550	HEB550	HEM450	HEB450	HEM360	HEB360
3-4	HEM500	HEB500	HEM360	HEB360	----	----
5-6	HEM450	HEB450	----	----	----	----
7-8	HEM360	HEB360	----	----	----	----

4.4.2.2 Design of Beam Splices

The design ensures that yielding will take place in the beam splice prior to any yielding or failure elsewhere. Therefore, the design of buildings with beam splices is based on the assumption that the fuses are able to dissipate energy by the formation of plastic bending mechanisms.

Free buckling length is calculated as 200mm for all beam splices. Figure 4-12 shows the resistance capacity ratio and elastic stiffness for beam splices 1, 2 and 3. The beam splices 1, 2 and 3 denote 170x12mm, 170x10mm and 170x8mm, respectively. The resistance capacity ratio α can be evaluated by the ratio of the maximum moment capacity of the beam splice and plastic moment of the main beam. Figure 4-13 displays the designed beam splices constitutive law in terms of moment-rotation. The assigned bolted and welded fuse’s dimension and their distribution in height are given in detail in Figure 4-14, Table 4-7 and Table 4-8 for all floors and buildings.

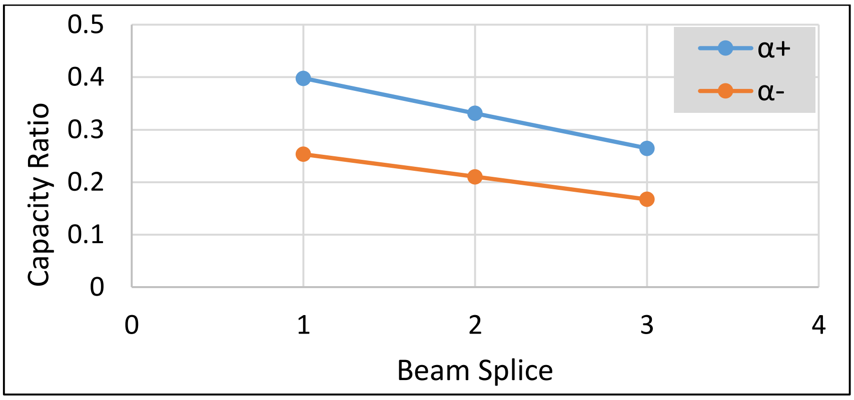
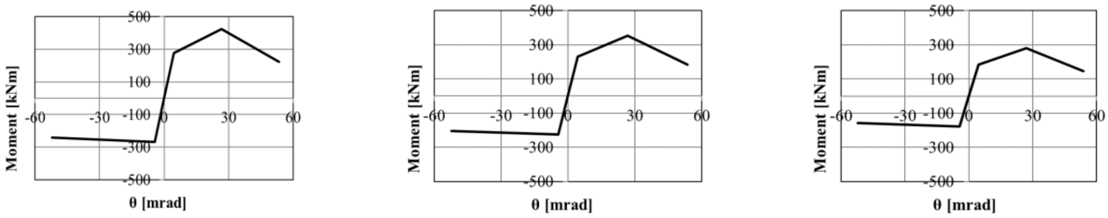


Figure 4-12 Resistance Capacity Ratio for Beam Splices 1, 2 and 3



1) 2) 3)

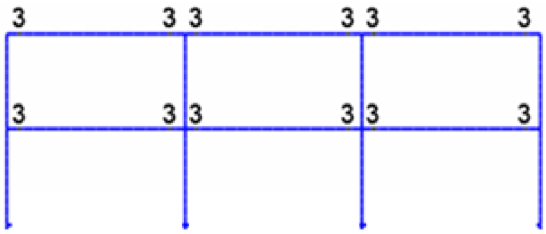
Figure 4-13 Beam Splices constitutive law in Terms of Moment-Rotation 1) 170x12mm 2) 170x10mm 3) 170x8mm

Table 4-7 Dimension of the Flange Plate of Beam Splices and Their Distribution in Height

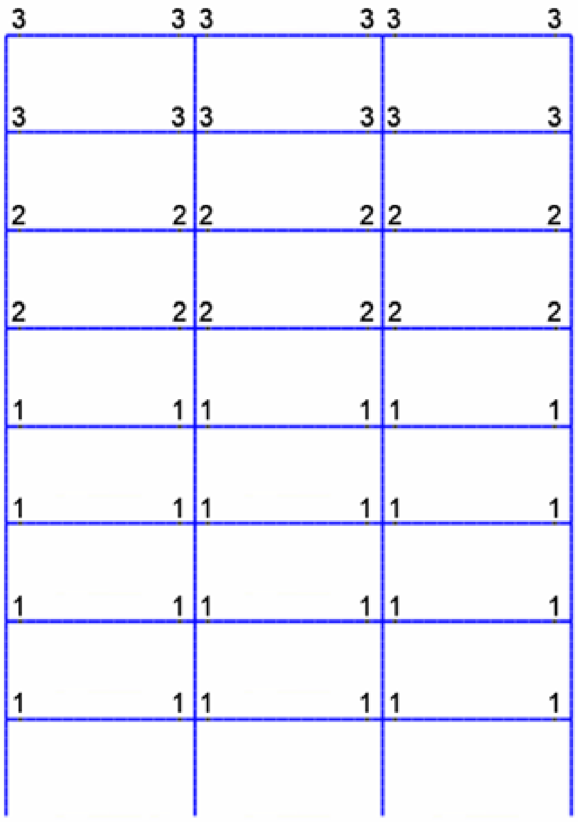
Storey	8-Storey	4-Storey	2-Storey
1-2	170x12 (mm)	170x10 (mm)	170x8 (mm)
3-4	170x12 (mm)	170x8 (mm)	----
5-6	170x10 (mm)	----	----
7-8	170x8 (mm)	----	----

Table 4-8 Dimension of the Web Plate of Beam Splices

8-Storey	4-Storey	2-Storey
170x6 (mm)	170x6 (mm)	170x6 (mm)



a) 2-Storey



c) 8-Storey

b) 4-Storey

Figure 4-14 Distribution of Assigned Bolted and Welded Beam Splices

4.5 Response Spectrum and Modal Analysis Results

Figure 4-15 to Figure 4-18 display the inter-storey drift of all structures with respect to their design behaviour factors. The 3cm allowable drift in EC8 considering the height of each storey equal to 4m (0.0075h where h denotes the height of storey) refers to the buildings having ductile non-structural elements.

For 2 and 4-storey buildings higher inter-storey drift develops in the first storey, but smaller inter-storey drifts in the upper stories. The maximum inter-storey drifts for 8 and 12-storey buildings exhibits in 3rd floor, with the exception for 8S-3B-6m (max. inter-storey drift is on the 2nd floor) and for 12S-3/4B-8m (max. inter-storey drift is on the 4th floor), after which the inter-storey drift decreases. The 2 and 4-storey buildings display a constant inter-storey drift throughout the stories of the frame than the taller structures i.e. 8 and 12-storey. The inter-storey drift for 8-storey buildings display less consistent behaviour, and for 12-storey buildings it shows a more inconsistent than the other type of buildings. Comparatively, the 12-storey buildings present inconsistencies between the 3rd and 9th storey for buildings having 6m length of the bay and between 4th and 9th storey for buildings having 8m length of the bay. The consistency of the inter-storey drift is desirable to reduce local damage, often to non-structural components caused by large inter-storey drifts.

The buildings designed having q=2 experience smaller inter-storey drifts than the frames designed by q>2. The maximum inter-storey drift for all buildings is obtained for buildings designed by q=7. The buildings designed by q=7, if compared with other buildings designed by q<7, produce the smaller inter-storey drifts in the lower stories, combined with the highest drifts in the upper stories.

The results of multi-modal analysis are summarized in APPENDIX-C from Table 9-27 to Table 9-48. The results show that more than 90% of the mass participation in the first three modes of vibration.

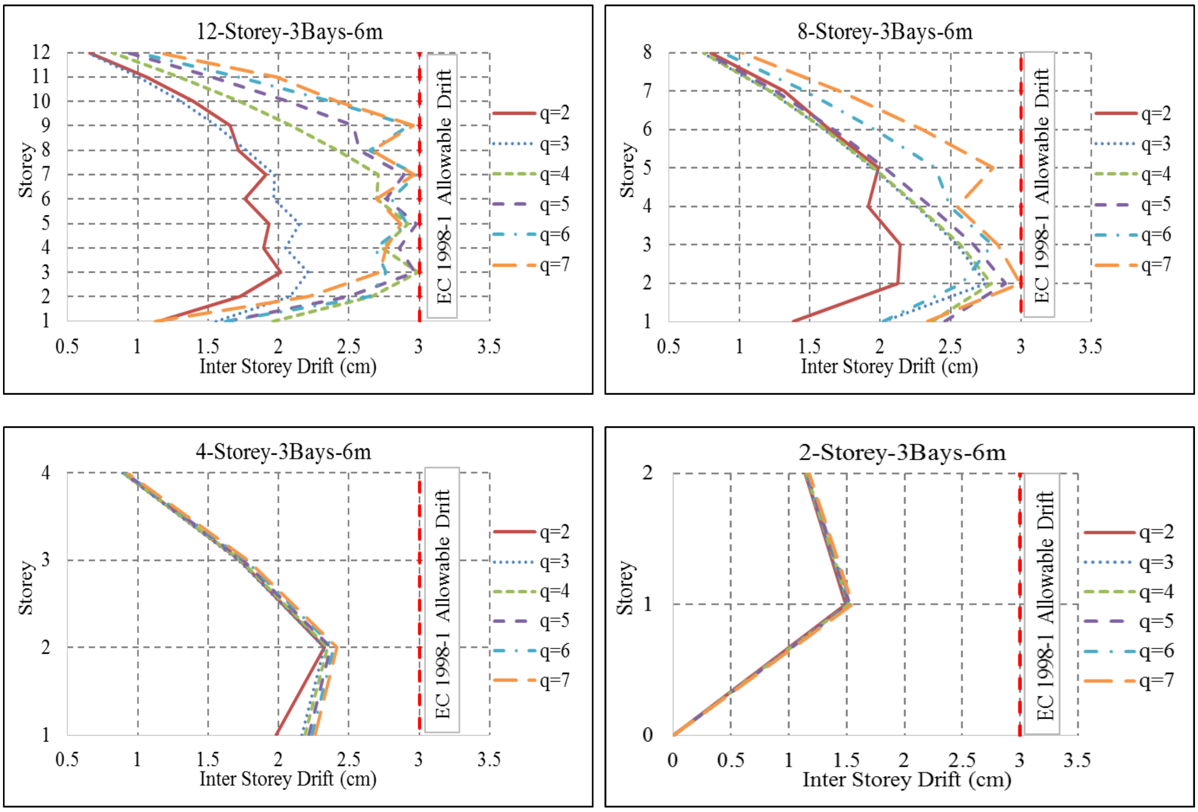


Figure 4-15 Inter-Storey Drift for Buildings Having 3B-6m

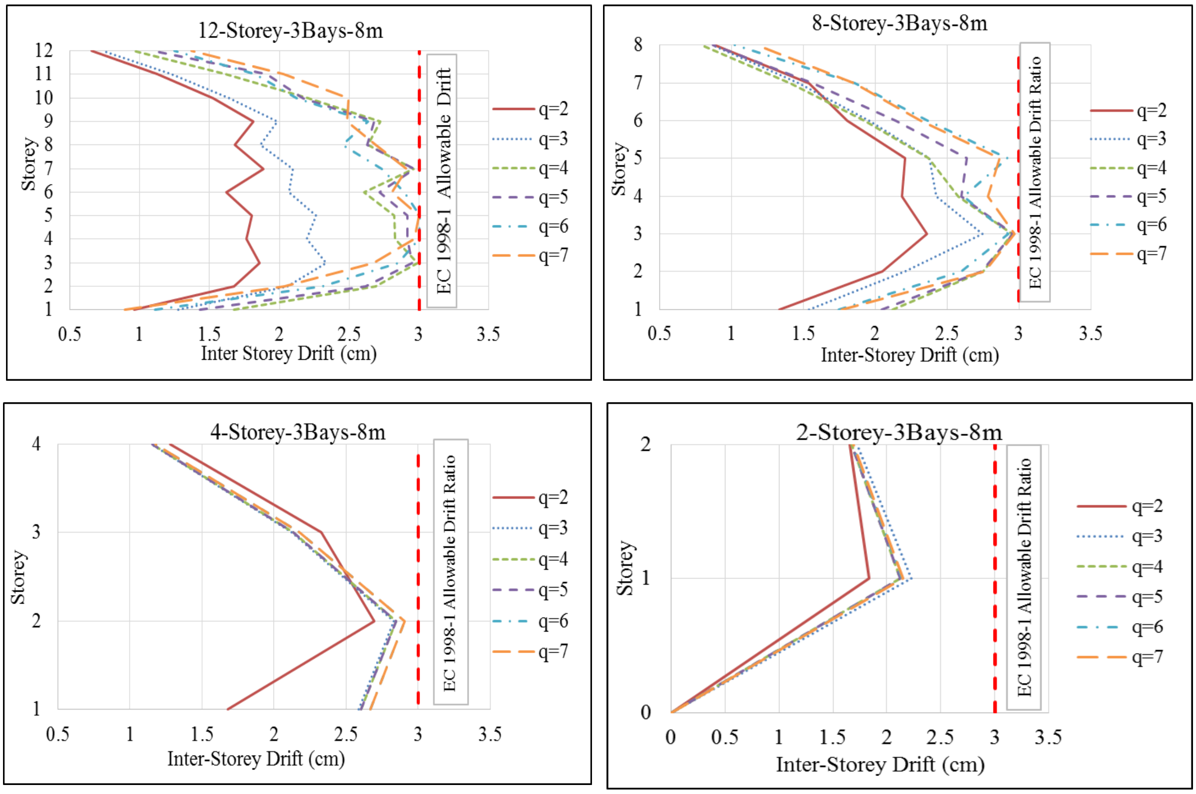


Figure 4-16 Inter-Storey Drift for Buildings Having 3B-8m

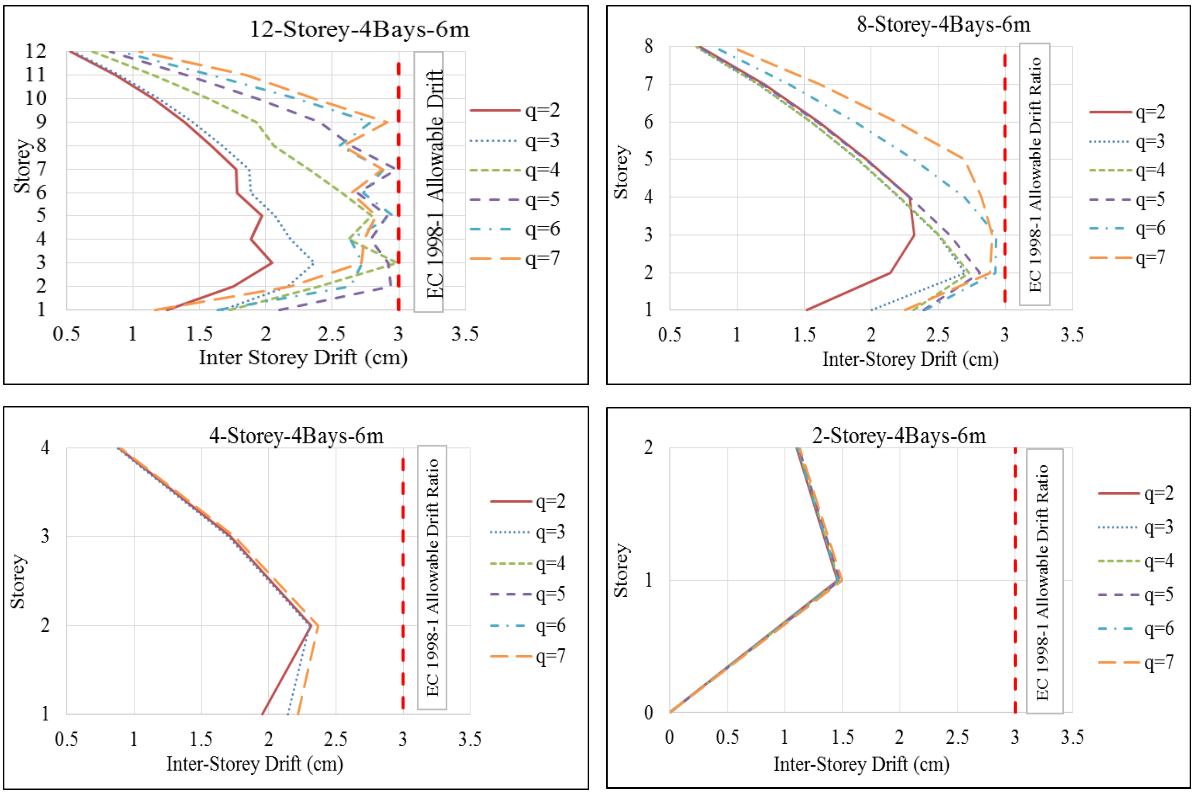


Figure 4-17 Inter-Storey Drift for Buildings Having 4B-6m

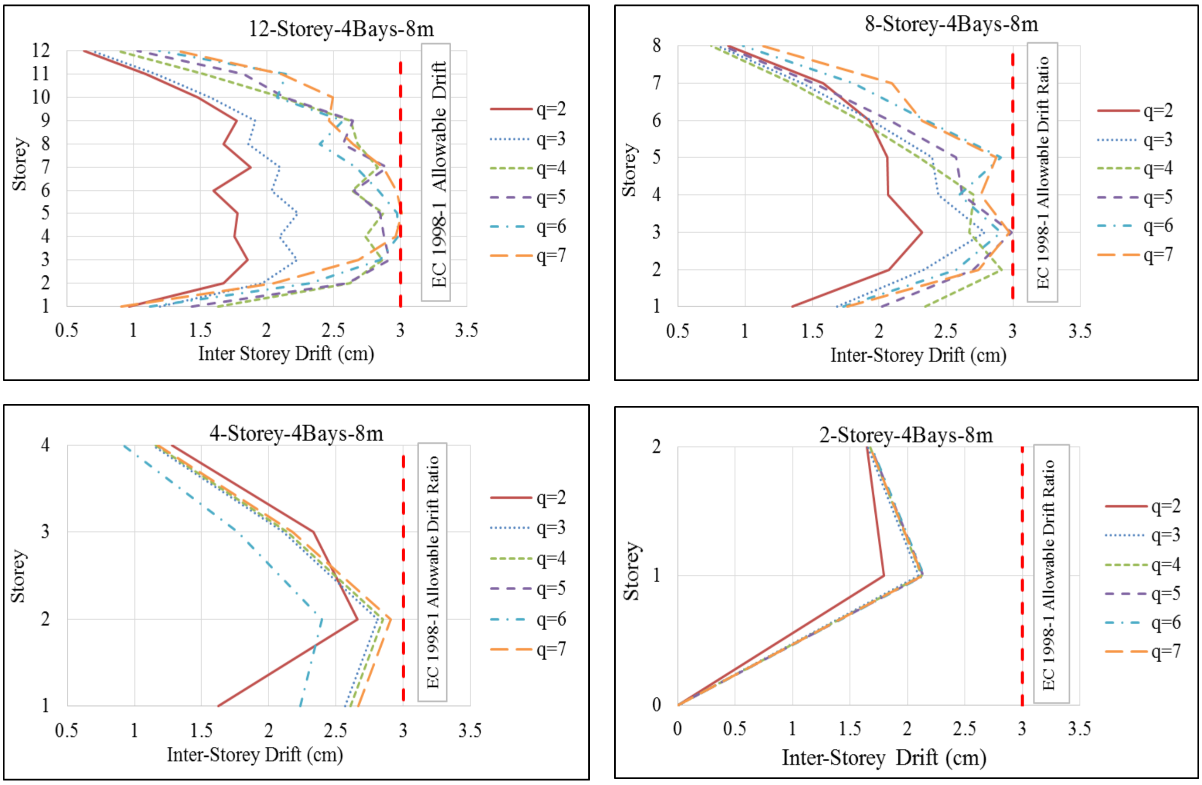


Figure 4-18 Inter-Storey Drift for Buildings Having 4B-8m

Chapter 5

SEISMIC ANALYSIS RESULTS AND DISCUSSION

5 SEISMIC ANALYSIS RESULTS AND DISCUSSION

This chapter will present an insight into the seismic performance of steel moment resisting frame structures, described in the previous chapter. As previously mentioned, two analysis strategies are followed in order to estimate the seismic performance in terms of force-displacement relationships, i.e. pushover analysis and incremental nonlinear dynamic analysis (*IDA*). Detailed information will be given related to the influence of structural behavior factor demand into the design process in regards of the performance objectives.

5.1 Nonlinear Static Analysis (Push-Over)

The current section of the chapter provides the results of nonlinear static analysis together with the results of re-analysis of the pushover curves based on methodology addressed in chapter 3. The chapter also introduces the optimal methods to define a consistent behaviour factor for steel moment resisting frame systems based on pushover curves. In parallel, a novel strategy for an optimal design of steel structures in high seismicity zones through balancing the initial cost and the lifetime seismic damage is introduced. The optimization of the design procedure is achieved by minimizing the initial structural material cost based upon the behaviour factor consideration.

The results of nonlinear static analysis are obtained considering that the vertical loads are applied in seismic combination, in plane global imperfection, out-of-plane local imperfection and the incremental horizontal load pattern as the first mode shape.

5.1.1 Introduction

Linear procedures, in general, are applicable when the structure is expected to remain nearly elastic for the level of ground motion or when the design results are in the nearly uniform distribution of nonlinear response throughout the structure. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and acceptability criteria to avoid unintended performance. Therefore, procedures incorporating inelastic analysis can reduce the uncertainty and conservatism.

The nonlinear static analysis, also known as "*pushover*" analysis is a method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behavior until an ultimate condition is reached. The response of the structure can be obtained as the response of an equivalent *SDOF* system in which the response is controlled by a single mode and the deflected shape of the *MDOF* system that remain constant during the analysis. The differential equation of an *MDOF* system can be written as:

$$M\{\Phi\}\ddot{x}_t + C\{\Phi\}\dot{x}_t + Q = -M\{1\}\ddot{x}_g \qquad \text{Eq. 5-1}$$

Where

M is the mass matrix

{Φ} is the shape vector

C is the damping matrix

Q is the storey force vector

$\ddot{x}_g$ is the ground acceleration

If defining the reference *SDOF* displacement x^* as

$$x^* = \frac{\{\Phi\}^T M\{\Phi\}}{\{\Phi\}^T M\{1\}} x_t \qquad \text{Eq. 5-2}$$

Then multiplying the Eq. 5-1 by $\{\Phi\}^T$, and substituting for x_t into Eq. 5-2 the following differential equation for the response of *SDOF* system can be obtained.

$$M^*\ddot{x}^* + C^*\dot{x}^* + Q^* = -M^*\ddot{x} \qquad \text{Eq. 5-3}$$

Where

$$M^* = \{\Phi\}^T M\{1\} \qquad \text{Eq. 5-4}$$

$$Q^* = \{\Phi\}^T Q \qquad \text{Eq. 5-5}$$

$$C^* = \{\Phi\}^T C\{\Phi\} \frac{\{\Phi\}^T M\{1\}}{\{\Phi\}^T M\{\Phi\}} \qquad \text{Eq. 5-6}$$

Output generates a static-pushover curve which plots a strength-based parameter *V* against deflection/displacement δ_t. The results provide insight into the ductile capacity of the structural system, and indicate the mechanism, load level, and deflection at which failure occurs. When analyzing frame objects, material nonlinearity is assigned to discrete hinge locations where plastic rotation occurs according to FEMA-356 [76].

5.1.2 Lateral Load Pattern Selection

Lateral load may represent the range of base shear induced by earthquake loading, and its configuration may be proportional to the distribution of mass along building height, mode shapes, or other practical means. Since the behaviour factor of the structure directly depends on the choices of the lateral load pattern, it could be attractive to use the load pattern that follows more or less the same path of the time variant distribution of inertia forces. The distribution of inertia forces varies with the severity of the ground motion and the time within the ground motion. The invariant load pattern proportional to the deflected first mode shape of the structure is used in this research. The basic assumptions are that the distribution of inertia forces is constant during the earthquake and the maximum deformations obtained from the invariant load pattern is comparable to those expected in the design earthquake if [98]:

1. The response of the structure is not severely affected by higher mode effects
2. The structure has only a single load yielding mechanism that can be detected by an invariant load pattern.

5.1.3 Modeling and Simulations

5.1.3.1 General Assumptions

The modelling of all case studies was performed by means of the finite element program SAP2000. All case studies simulated with a linear-elastic model by appropriate beam elements while the plasticity is concentrated at the all end and mid span of the beams and in the columns at the base. Hence, the simulation is done based on the design rules [2], [88], [96] which are intended to ensure that yielding, will take place at the ends of all beams and at the base of the columns. The potential non-linear behavior of the beams is simulated with a lumped plasticity approach by defining a non-linear plastic hinge at the beam end/mid and at the base of the columns. To characterize the non-linear behavior of a plastic hinge, the generalized force–deformation properties suggested in FEMA 356 [76] are implemented. For the beams flexural moment hinges are considered, while plastic hinges accounting for the interaction between axial forces and bending moment are defined for the columns. Beam-to-column joints are considered as rigid connections

5.1.3.2 Specific Assumptions for Innovative Dissipative Structures (*Non-Conventional Structures*)

The modelling of the innovative dissipative structures (non-conventional structures) is done based on the specific design rules of bolted/welded beam splices [88] which are intended to ensure that yielding will take place in the beam splices prior to any other yielding or failure elsewhere.

The bolted/welded beam splices are modeled as non-linear link elements with a length equal to the free buckling length of the beam splices and they are placed at a distance equal to the beam depth from the beam-to-column connections. The link element is a non-linear spring with six independent internal deformation components for which a non-linear generalized force–deformation relationship is defined. The multi-linear plastic pivot model was used as hysteresis rule including asymmetrical cross-section behavior with pinching and strength degradation. The link behavior is defined by a moment-rotation curve characterized by different positive and negative moment capacities and initial stiffness of the beam splices. The non-linear behavior is assigned only to the rotational degree of freedom of the link with respect to the major axis of inertia. The constitutive law adopted for the non-linear link is able to represent the dissipated energy, the stiffness and the maximum moment of the beam splices during the cyclic loading history. The initial input parameters of the monotonic moment-rotation diagram of the bolted/welded beam splices are obtained based on the specific rules of designing the welded/bolted beam splices [87], [88] and based on FUSEIS experimental results [38], [86].

The length of the beams was subdivided into different elements in order to take into account both the presence of the beam splices and the part of the beam reinforced with additional welded plates. The part of the beam reinforced with additional welded plates, aimed at avoiding spreading of plasticity to the connection, is reproduced in the numerical models by using different cross-sections and plastic hinge properties around the beam splices. The length of these regions is assumed in the model in accordance with the specific design rules of the bolted/welded beam splices [88]. Figure 5-1 displays the summary of lumped plasticity modelling-approach for non-conventional structures.

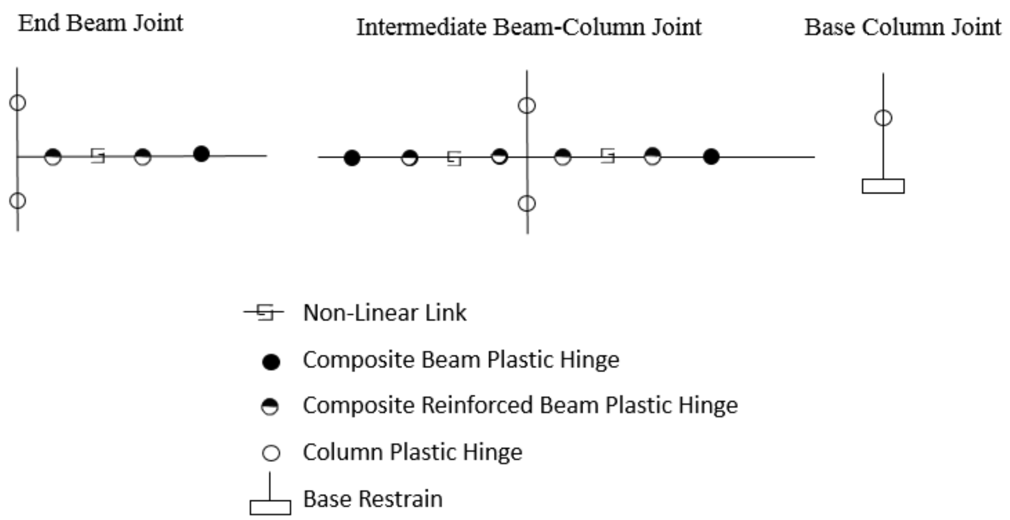


Figure 5-1 Summary of Lumped Plasticity Modelling-Approach for Non-Conventional Structures

5.1.4 Numerical Results

5.1.4.1 Typical MRF Buildings (*Conventional Structures*)

The pushover analysis is applied to all case studies, consisting of 16 series of structures (2, 4, 8, 12 storey each one having 4 different geometries) each structure was designed having the behaviour factor equal to 2, 3, 4, 5, 6 and 7. The structural geometries are as follows:

1. 3-Bays-6m length of the bays
2. 3-Bays-8m length of the bays
3. 4-Bays-6m length of the bays
4. 4-Bays-8m length of the bays

Figure 5-2 to Figure 5-5 display the pushover curves for all structures (in total of 16x6=96 structures) with respect to their design behaviour factors.

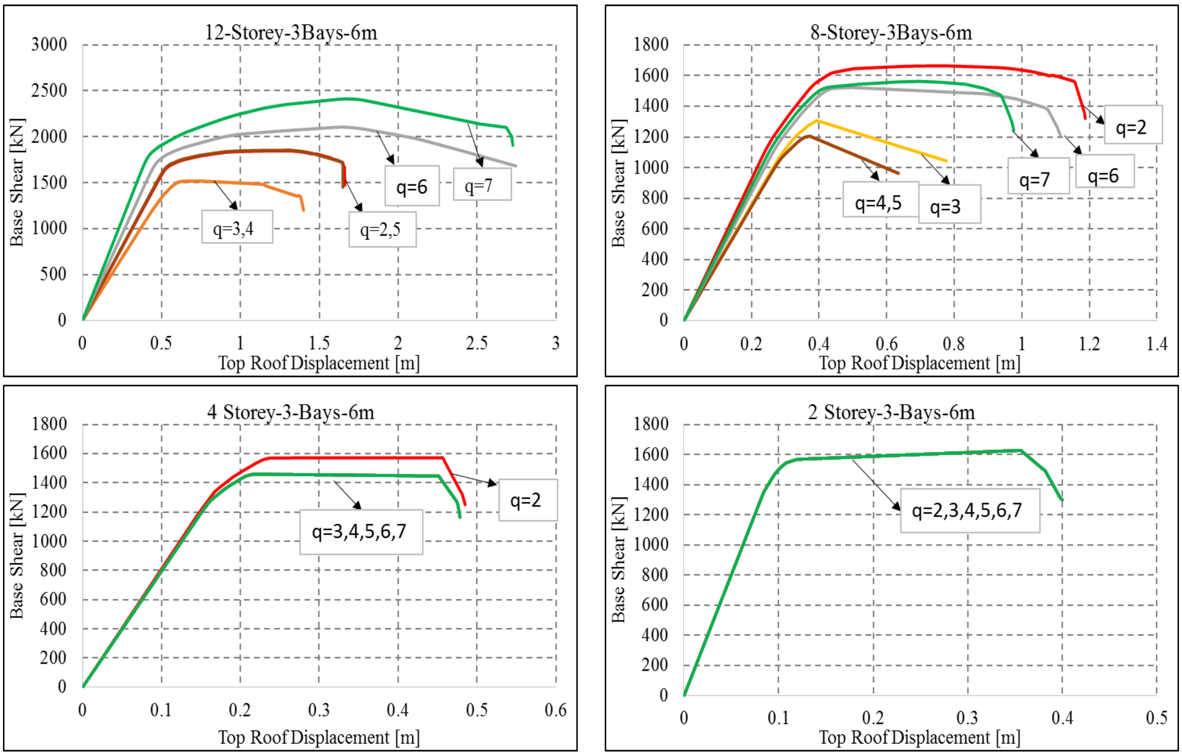


Figure 5-2 Pushover Curves for Buildings Having 3B-6m

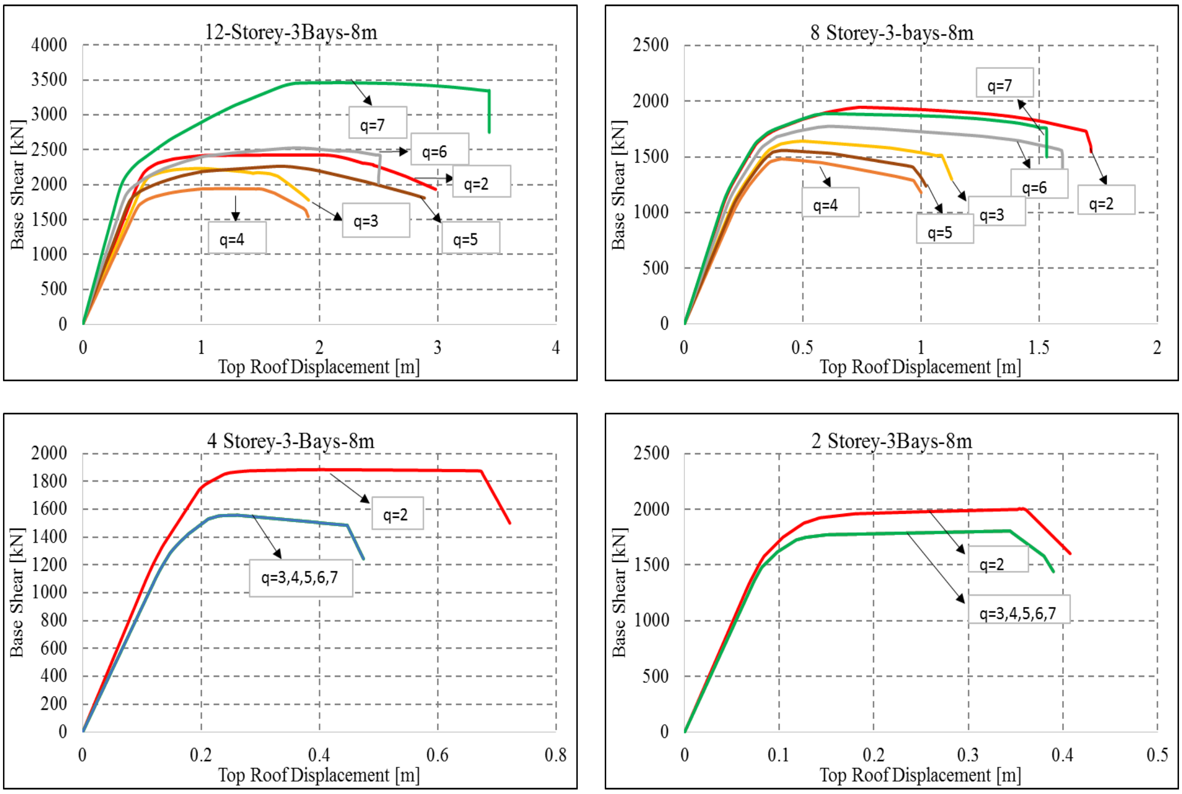


Figure 5-3 Pushover Curves for Buildings Having 3B-8m

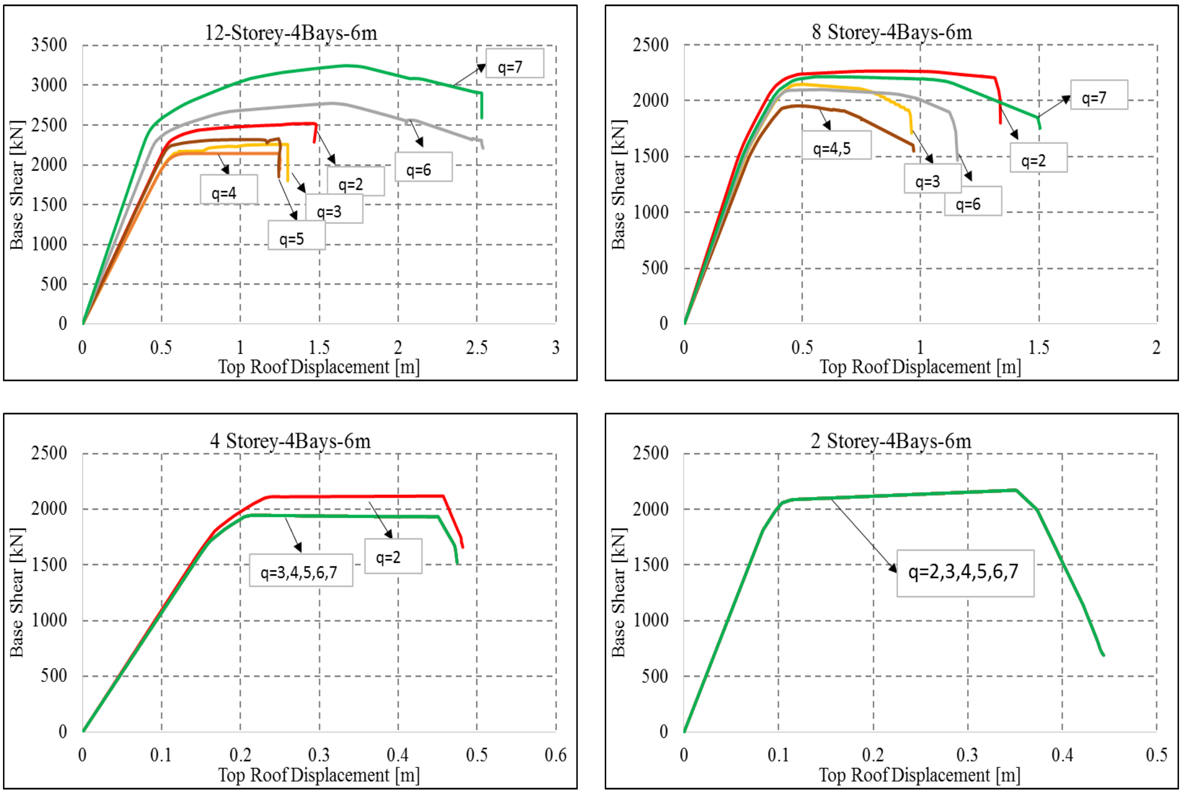


Figure 5-4 Pushover Curves for Buildings Having 4B-6m

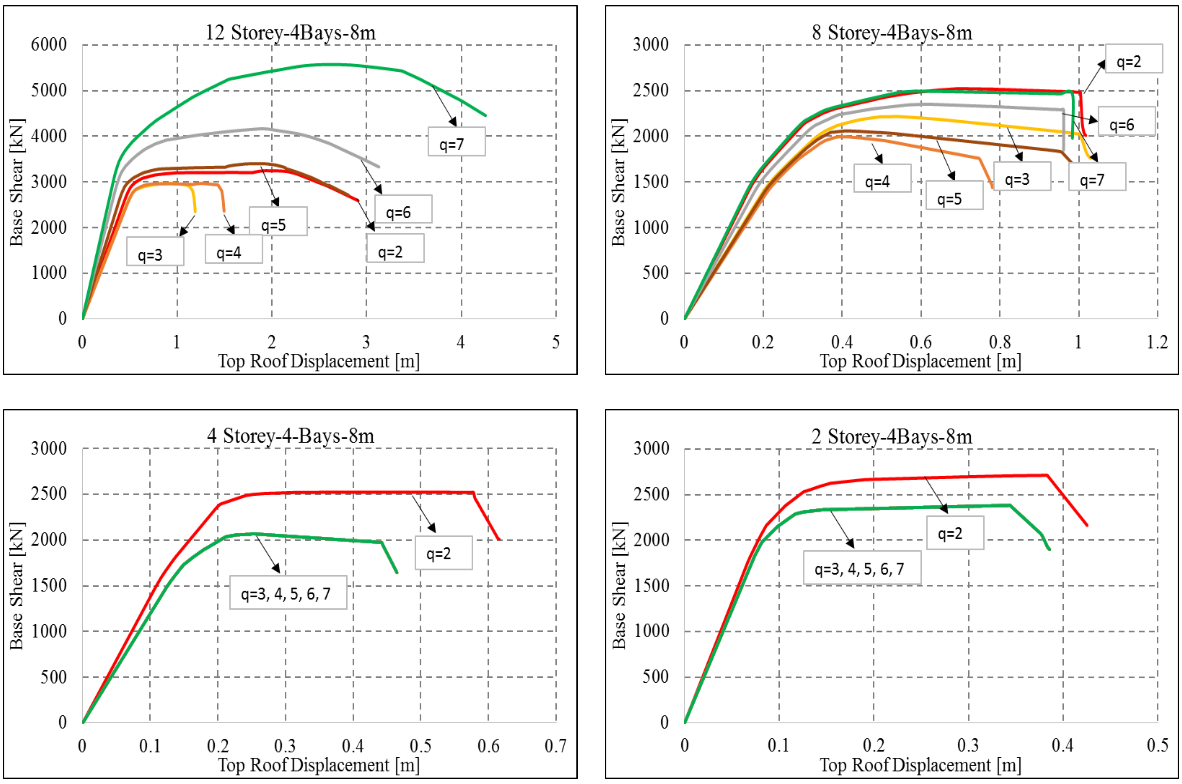


Figure 5-5 Pushover Curves for Buildings Having 4B-8m

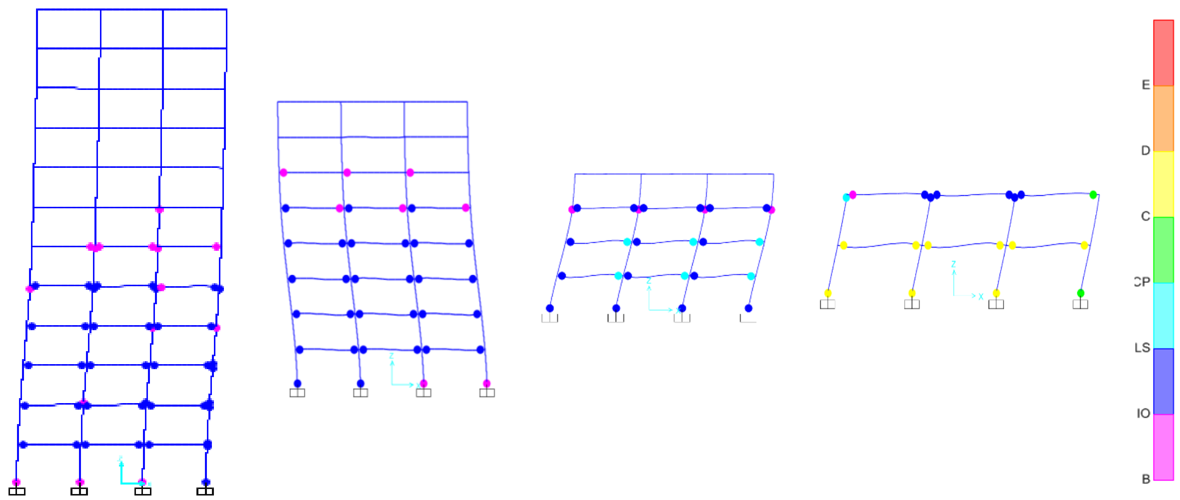


Figure 5-6 Non Linear Pushover Analysis-the Formation of Plastic Hinges at the Displacement Equal to 60cm (δ= 60cm) for Conventional Structures

Figure 5-6 displays the formation of plastic hinges for conventional structure at the displacement equal to 60cm (δ= 60cm). The color scale denotes the damage experiences at the hinges with the yellow dot representing collapse, as well as previous stages of damage including immediate occupancy, life safety and collapse prevention. As the frames are designed in accordance with requirements to develop the "weak beam–strong column" mechanism, the plasticity is generally limited to the beams, with some plasticity forming in columns. Plasticity within the columns promotes "soft storey" collapse, a mechanism that is particularly problematic in taller frames as the p-delta effect becomes more important.

5.1.4.2 Innovative Dissipative FUSEIS Bolted & welded Beam Splices (*Non-Conventional Buildings*)

The static nonlinear analysis (*Pushover*) is applied into the following non-conventional structures:

1. 2-Storey 3-Bays-6m length of the bays, Bolted and Welded beam splices
2. 4-Storey 3-Bays-6m length of the bays, Bolted and Welded beam splices
3. 8-Storey 3-Bays-6m length of the bays, Bolted and Welded beam splices

Figure 5-7 displays the pushover curves for 2/4 and 8 storey having bolted and welded beam splices (*non-conventional structures*). As expected, the non-conventional frames having bolted and welded beam splices are experiencing a lower capacity than the conventional frame. This is due to the loss of strength and stiffness as a result of using the beam splices. It is evident that the incorporation of bolted and welded beam splices reduce the strength of the structure, as the frames reach a higher displacement with a smaller base shear (see Figure 5-2 to Figure 5-7).

Figure 5-8 shows the formation of plastic hinges at the displacement equal to 60cm (δ= *60cm*) for bolted and welded beam splices. As the figure implies, the plastic formation for bolted and welded beam splices concentrates mainly in the beam splices which exhibit increasing the ability to distribute plasticity throughout all stories of the frame and hence leaving other elements in the elastic region. In contrast to conventional frames designed according to EC8, all the formation of plastic hinges occurred at the beam ends and at the base columns (see Figure 5-6 and Figure 5-8).

Figure 5-9 highlights additional plastic hinges forming in the upper stories of the frame in the beam splices than the conventional ones. It also shows that the formation of a plastic mechanism occurs more or less simultaneously in all floors ensuring that no soft storey mechanism occurs in the structures. This larger distribution of plasticity aids the reduction of inter-storey drifts in the mid-stories and allows the consistent inter-storey drifts.

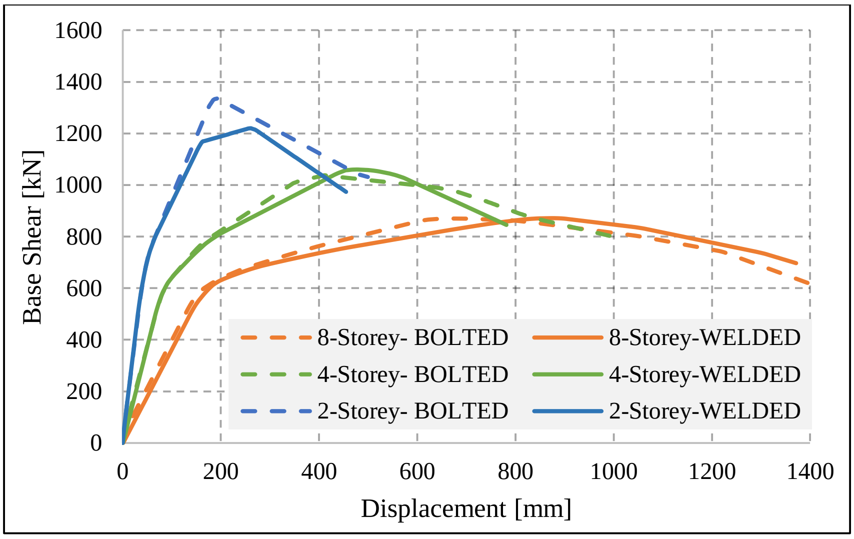
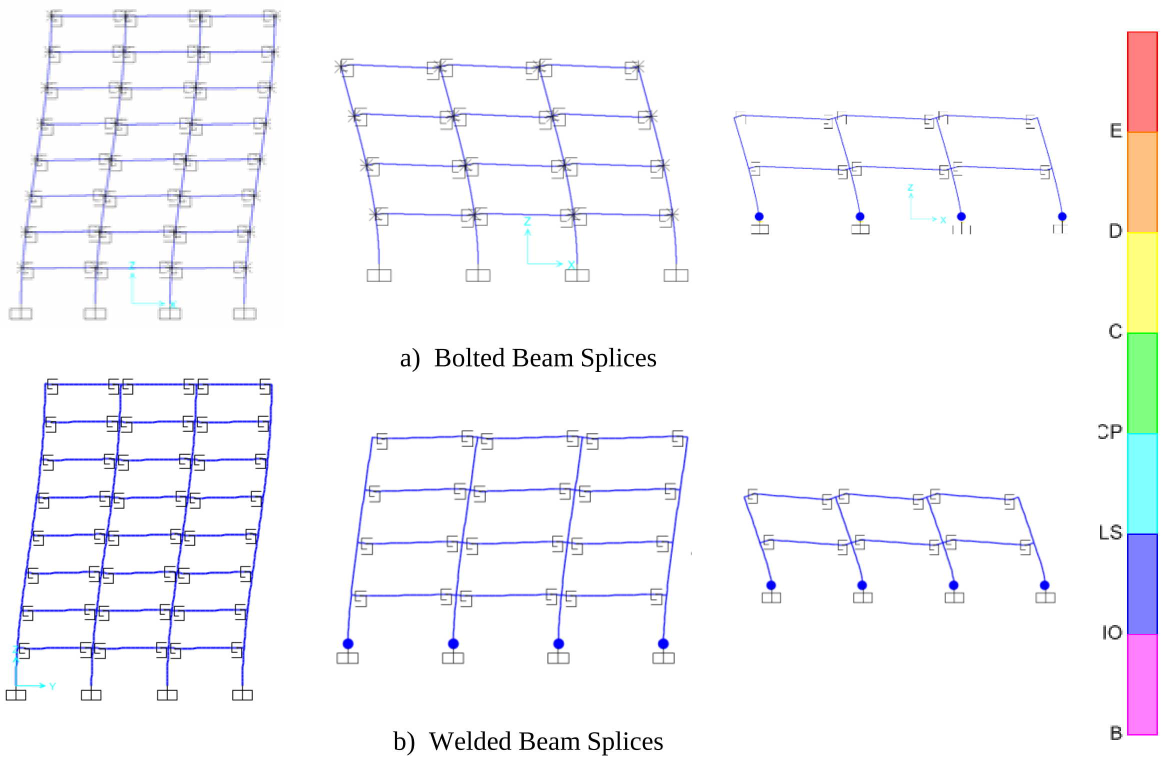


Figure 5-7 Push-Over Curve for 2, 4 and 8 Storey-Building Having Bolted and Welded Beam Splices (*Non-Conventional Structures*)



a) Bolted Beam Splices

b) Welded Beam Splices

Figure 5-8 Non Linear Pushover Analysis-the Formation of Plastic Hinges at the Displacement Equal to 60cm (δ= *60cm*), a) Bolted Beam Splices, b) Welded Beam Splices

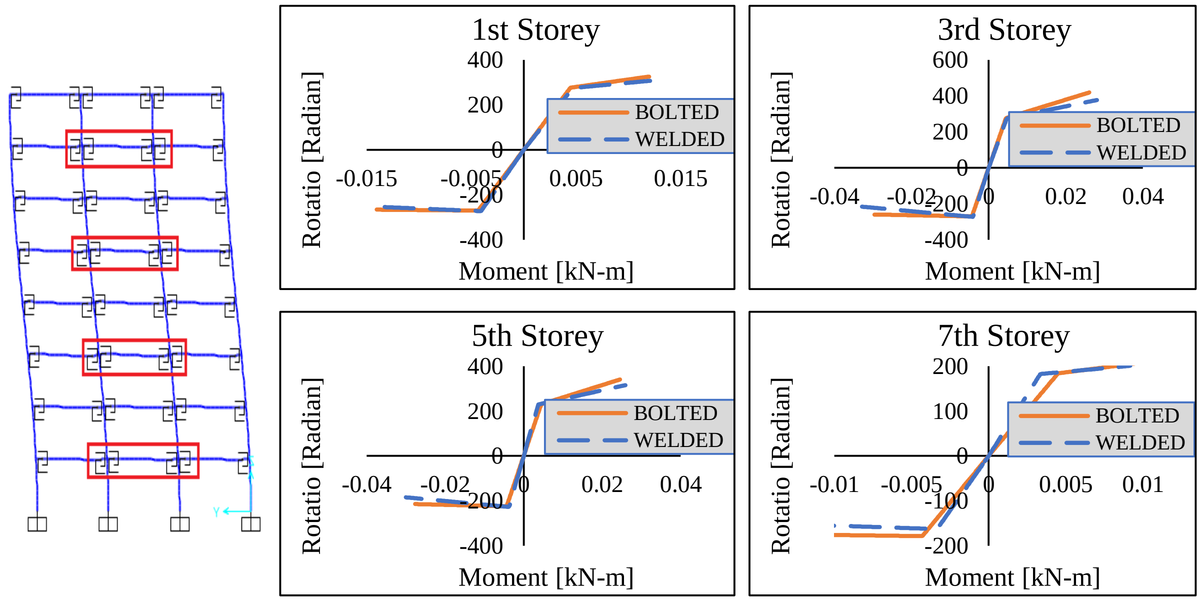


Figure 5-9 Global Plastic Mechanism of 8 Storey Bolted/Welded Beam Splices at δ= 60cm

5.1.5 Discussion of the Results of Pushover Analysis

MRFs are known to be flexible structures and hence, their design is often governed by the need to satisfy deformation criteria under earthquake loading, or limitation of *P-Δ* effects under design earthquake loading. When considering the lower behaviour factor e.g. q=2.0, results in a high lateral stiffness due to having high lateral loads. Hence, the designed members' sizes have enough stiffness and strength to overcome all other limitations such as inter-storey drift or damage limitations. While designing by a higher behaviour factor i.e. q=7, results in a low lateral stiffness due to having low lateral loads. In this case the damage limitations could not be satisfied as the stiffness of the designed members' sizes are too low to overcome the limitation of the lateral movements (see Eq. 4-6 and Eq. 4-8). This is more critical when the number of stories and, hence, the p-Δ effects increase. However, as Figure 5-2 to Figure 5-5 and Figure 5-10 to Figure 5-13 imply, for lower structures (2 and 4-Storey buildings), the allowable inter-storey drift which is well acceptable in practice, does not govern the design procedure based on the current steel design criteria of EN 1993-1-1: 2005 [96] and on the seismic design provision EC1998-1: 2004 [2]. Whereas another factor such as the column-beam moment ratio which leads to a strong column-weak beam mechanism, is more numerous. On the other hand, when considering the high-rise or tall buildings (8 and 12-Storey buildings), it is demonstrated that the codified allowable drift is controlling the seismic frame design procedure rather than other factors.

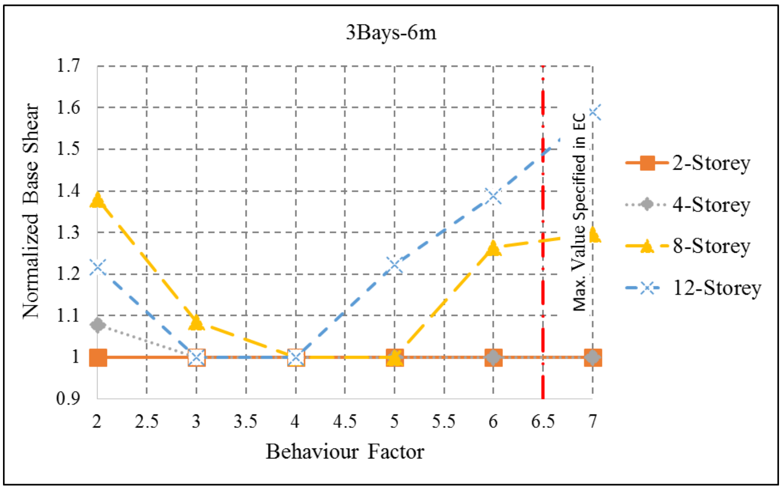


Figure 5-10 Behaviour Factor vs. Normalized Base Shear for Buildings Having 3B-6m

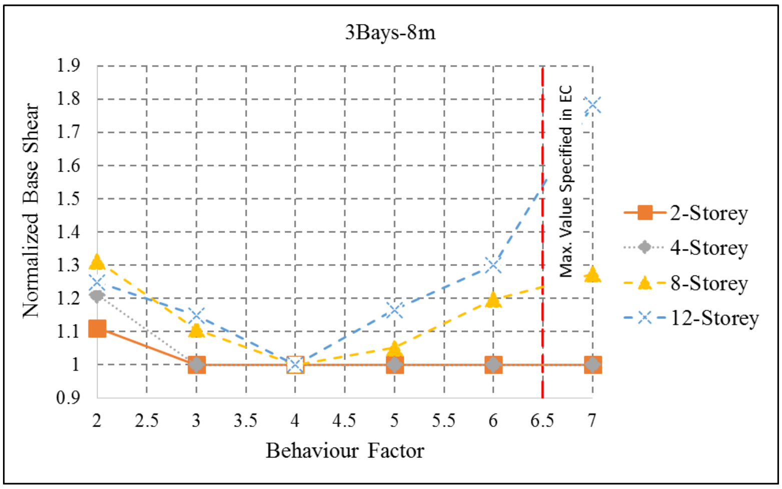


Figure 5-11 Behaviour Factor vs. Normalized Base Shear for Buildings Having 3B-8m

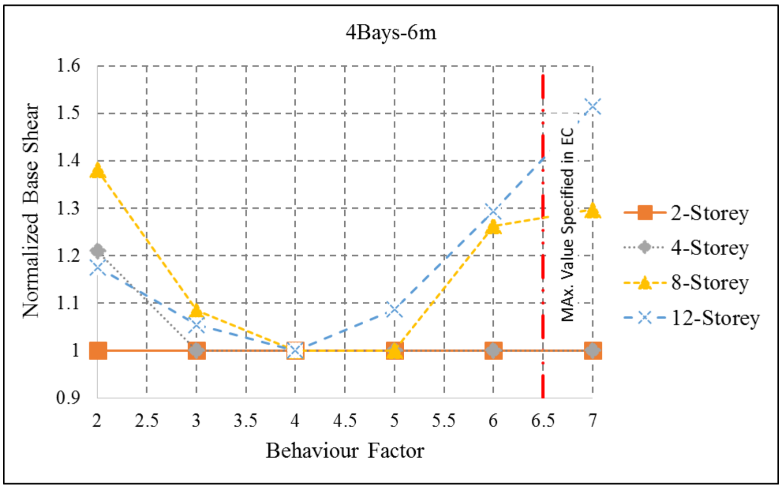


Figure 5-12 Behaviour Factor vs. Normalized Base Shear for Buildings Having 4B-6m

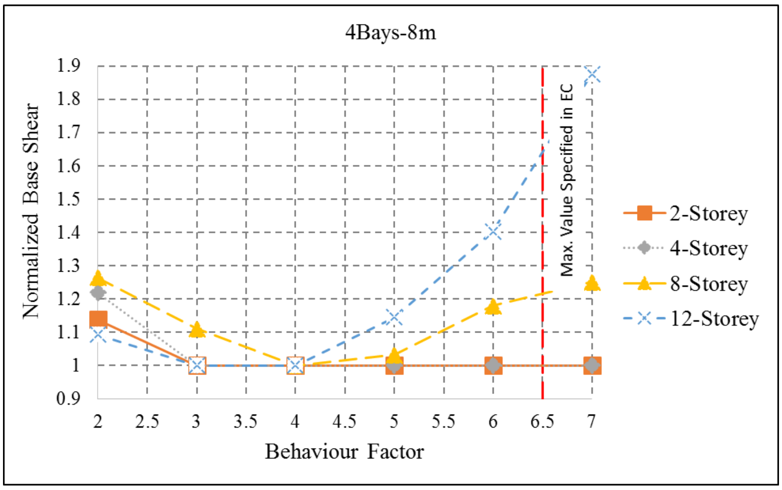


Figure 5-13 Behaviour Factor vs. Normalized Base Shear for Buildings Having 4B-8m

Figure 5-10 to Figure 5-13 show the behaviour factor versus normalized base shear. The normalized base shear is calculated by the following equation:

$$V_N^{q=2,3,4,\dots} = \frac{V_{q=2,3,4,\dots}}{\min\left(V_{q=2}, V_{q=3}, V_{q=4}, \dots\right)} \qquad \text{Eq. 5-7}$$

Where

V_N is the normalized base shear for each design behaviour factor

V_q is the maximum base shear for each design behaviour factor

For example, for 4-storey 3bays and 6 m length of the bay designed by q=2, the maximum capacity of the structure obtained as 1571 kN over the minimum capacity obtained by the different design q factor as 1459kN which results in 1.08 (see Figure 5-2 and Figure 5-10).

For 8S-3B-6m (Figure 5-2 and Figure 5-10) and 8S-3B-8m (Figure 5-3 and Figure 5-11) buildings, increasing the behaviour factor from 2.0 to 3.0 results in decreasing the capacity of the structure by approximately 30% and 20%, respectively. On the other hand, increasing the behaviour factor to a value greater than 5.0 for 8S-3B-6m and greater than 4.0 for 8S-3B-8m results in increasing the capacity of the structures by approximately 30% and 20%, respectively. In the same way, for 12S-3B-6m (Figure 5-2 and Figure 5-10) and 12S-3B-8m (Figure 5-3 and Figure 5-11) buildings, increasing the behaviour factor from 2.0 to 3.0 decreases the capacity of the structures by only 20% and 10%, respectively. But for a value greater than 4.0, if satisfying all the design constraints, the capacity of the structure dramatically increases by increasing the behaviour factor. The results also show that the maximum allowable value of the behaviour factor (q_{des}=6.5) specified in EC8 may increase the capacity of the structure by 50% for tall structures (12-storey) and 30% for high-rise structures (8-storey) compared to a situation when designing by q_{des}=4.0.

This is show that, by using current design codes (EC8, EC3), increasing the behaviour factor results in changing the limit state governing the design from "*strength*" to "*equilibrium*" or "*stability*" for tall structures. Therefore, from the results once can be concluded, the design of structures for tall and mid-rise buildings is governing by the damage limitations.

Increasing the behaviour factor for lower structures such as 2 and 4-storey, irregardless of the number of bays and length of the bay, has not much influence on the capacity of the structure. The exception is for the buildings designed with behaviour factor equal to 2.0 which shows increasing the capacity of the structure by the maximum of 20% for 4-storey and 10%

for 2-storey. This proves that the p-Δ effects and the effects of lateral movements are not much influencing the design of low (2-storey) and mid-rise (4-storey) structures and hence the design limit stat for this kind of structures is predominated by *"strength"* criteria.

On the other hand, increasing the number of bays (3 bays to 4 bays) for the same structures, do not affect the trend of the results, but only slightly modifies the overall performance of the structures quantitatively (see Figure 5-11 and Figure 5-13).

From an economical point of view the initial material cost of the steel columns may be calculated based on the available standard steel sections in the current European market at the cost of 3.32 Euro per kg. It is reminded that only the cost of columns is provided in this section. Other costs for foundations, beams, connections of the joints, concrete slab, (assumed to be fixed during the design procedure of all structures), heating/cooling/electrical installments etc. are not considered. Table 5-1 to Table 5-4 show the initial material costs (cost of columns) in thousand euro. As from Table 5-1 to Table 5-4 imply, having different design behaviour factor for 2 and 4-storey building has no direct effect on the initial cost of the structures as increasing the design behaviour factor from 3.0 to 7.0 results the same structure. Whereas, for taller buildings such as 8 and 12- storey, the design behaviour factor has an important influence on the cost of the structures. This shows that as the height of the building increases, the influence of the q factor on the initial cost of the structures is more relevant. For example, the initial material cost of the structure for 12S-3B-6m building designed by q=4.0 is evaluated as 237.0 thousand Euro while, if the same structure is designed by q=7.0, the material cost increases by more than 50% to 358.6 thousand Euro. Considering the same behaviour factor for an 8-storey building, the material cost increases by 9% from 144.8 thousand Euro to 159.2 thousand Euro. It is more evident when the number of bays and the length of the bay increases. For example, for 12S-4B-8m building, if the design behaviour factor chosen as 3.0 than the design behaviour factor equal to 7.0 results in increasing the initial material costs by 169% from 564.6 thousand Euro to 1520.1 thousand Euro. On the other hand, for 12S-3B-6m and 12S-4B-8m building, reducing the behaviour factor from 3.0 to 2.0 results in increasing the cost by more than 15.5% and 12%, respectively. Hence, from an economical point of view, it can be concluded that the optimal design behaviour factor with respect to the initial cost of tall buildings should fall between 2.5 to 4.5.

Table 5-1 The Initial Material Costs for 3B-6m (in Thousand Euro)

		2-Storey	4-Storey	8-Storey	12-Storey	
Behaviour Factor	2	36.2	73.0	162.8	274.1	Thousand Euro
	3	36.2	72.4	146.2	237.0	
	4	36.2	72.4	144.8	237.0	
	5	36.2	72.4	144.8	276.4	
	6	36.2	72.4	154.8	342.0	
	7	36.2	72.4	159.2	358.6	

Table 5-2 The Initial Material Costs for 3B-8m (in Thousand Euro)

		2-Storey	4-Storey	8-Storey	12-Storey	
Behaviour Factor	2	39.0	83.9	224.1	373.1	Thousand Euro
	3	39.0	78.1	182.5	317.1	
	4	39.0	78.1	168.2	342.9	
	5	39.0	78.1	172.9	400.8	
	6	39.0	78.1	199.3	500.3	
	7	39.0	78.1	223.7	905.6	

Table 5-3 The Initial Material Costs for 4B-6m (in Thousand Euro)

		2-Storey	4-Storey	8-Storey	12-Storey	
Behaviour Factor	2	60.3	121.7	283.9	452.8	Thousand Euro
	3	60.3	120.5	240.1	406.6	
	4	60.3	120.5	251.6	398.8	
	5	60.3	120.5	251.6	421.6	
	6	60.3	120.5	264.8	548.2	
	7	60.3	120.5	291.5	580.2	

Table 5-4 The Initial Material Costs for 4B-8m (in Thousand Euro)

		2-Storey	4-Storey	8-Storey	12-Storey	
Behaviour Factor	2	67.1	145.1	379.8	632.8	Thousand Euro
	3	67.1	134.2	295.7	564.6	
	4	67.1	134.2	285.4	590.0	
	5	67.1	134.2	295.3	679.3	
	6	67.1	134.2	347.0	840.1	
	7	67.1	134.2	380.1	1520.1	

5.2 Incremental Dynamic Analysis (IDA)

Incremental Dynamic Analysis (*IDA*) for each case study was carried out to derive a refined representation of the relationship among engineering demand parameters (*EDPs*) of interest and the ground motion intensity measure (*IM*) that will eventually be exploited for the robust assessment of the behaviour factor. For the purpose of this study, the maximum inter-storey drift ratio (i.e. θ_{max}) is adopted as the EDP. The average spectral acceleration (*AvgSa*) is shown in Eq. 3-2. It will be shown that the value of the behaviour factor chosen for design is acceptable according to the methodology given in chapter 3 which is based on the explicit performance assessment of structures using multiple performance targets on a mean annual frequency of exceedance as proposed by Vamvatsikos et al. (2017) [99]. Furthermore, the value of the behaviour factor obtained based on the re-analysis of the pushover curves will be compared with the results of *IDA* proposed by P. Setti [10]. To do so, 3 high seismicity sites across Europe are first selected (see chapter 3). Then, 30 actual records which represent the above-mentioned sites are applied and scaled 12 times according to the procedure presented by Vamvatsikos (2002) [39] in order to capture the entire range of response of the structure. Hence, for each type of structure (102 types in total), 360 nonlinear dynamic analysis is carried out.

5.2.1 Modeling and Simulations

Nonlinear models are developed in *OpenSees* to facilitate Incremental Dynamic Analysis for each case study (conventional and non-conventional structures). Only one mid identical frames as 2D model according to section 4.3.3 and 5.1.3 was considered. The models incorporate accurate hysteresis, including both in-cycle and cyclic degradation, of all system components that may enter the nonlinear range. Component modelling is able to accurately reproduce both the monotonic (with in-cycle degradation) and the hysteretic (with cyclic degradation) performance of these elements. Each nonlinear element is able to display a clearly defined fracturing deformation (drift, rotation, strain or displacement) whereby it loses all strength and stiffness and ceases to function.

The models consist of lumped plasticity elements for those members/parts that are expected to undergo excessive deformations in the nonlinear range of the system; that primarily includes the *FUSEIS* bolted and welded beam splices where required, the beams as well as the columns. The plastic-hinge properties for the conventional structures including the non-dissipative elements are calculated according to the provisions of relevant codes (e.g. FEMA-356 [76]). On the contrary, for the non-conventional structures, Moment-rotation plastic hinges are considered at the ends of the FUSEIS bolted and welded links, with their properties being determined from the analytical investigations (see section 4.4.2). Uncertainty dispersions of βLSU = 0.2 and βGCU = 0.3 are assumed, together with a moderate confidence level of x = 80%.

The *OpenSees* models are first compared against existing *SAP2000* models that were used for the design of the structures in chapter 4 and the results of pushover curve (see section 5.1). A full comparison can be found in the APPENDIX-D-1 and APPENDIX-D-2 from Figure 9-2 to Figure 9-20 for both conventional and non-conventional structures, respectively.

As an example, Figure 5-14 displays the 30 incremental dynamic records for 12-3B-6m having the behaviour factor equal to 2, 3, 4, 5, 6 and 7 with respect to $AvgS_a\,(T, \%)$ along with the associated *SD* and *GC* capacities in terms of spectral acceleration and the maximum roof drift ratio (θ_{max}). The full results can be found in APPENDIX-E-1 and APPENDIX-E-2 from Figure 9-21 to Figure 9-39.

5.2.2 Acceptance or Rejection of the Initial Assumption of the Design Behaviour Factor

The seismic fragility outputs (i.e. Figure 5-15) are conveniently convolved with the seismic hazard curves as shown in Figure 3-5 for the high-seismicity sites of Athens, Perugia and Focsani. The result is the mean annual frequency of exceedance for the limit states of interest. The mean annual frequency of exceedance ($\lambda x(DS)$) for *GC* objectives of 1% and 2% in 50yrs, the associated limiting values ($\lambda DSlim$) and the margin ratio ($\lambda DSlim/\lambda x(DS)$), is summarized in Table 5-5 for 12S-3B-6m as an example. The full results are shown in APPENDIX-F-1 and APPENDIX-F-2 from Table 9-49 to Table 9-66 for both conventional and non-conventional structures, respectively. By comparing the margin ratio against its allowable value of *1.0* determines the acceptance or rejection of the initial assumption of the design-basis behaviour factor. For all case studies and all sites, the *LS* and *GC* objectives are easily satisfied which shows that the initial (*design*) assumption of the behaviour factors are acceptable (see Table 9-49 to Table 9-64 in APPENDIX-F-1 and APPENDIX-F-2). The exception is for 8S-3B-6m bolted beam splices for the site of Athens where the global collapse (*GC*) cannot be satisfied, hence rejecting the initial design behaviour factor (see Table 9-65). This does not necessarily mean that the design behaviour factor tested was erroneous. The proposed approach is not only a test for design behaviour factor, but also of the design methodology itself, the availability of adequate experimental results (for non-conventional structures) to accurately determine the behaviour of the elements and the nonlinear modelling approach adopted.

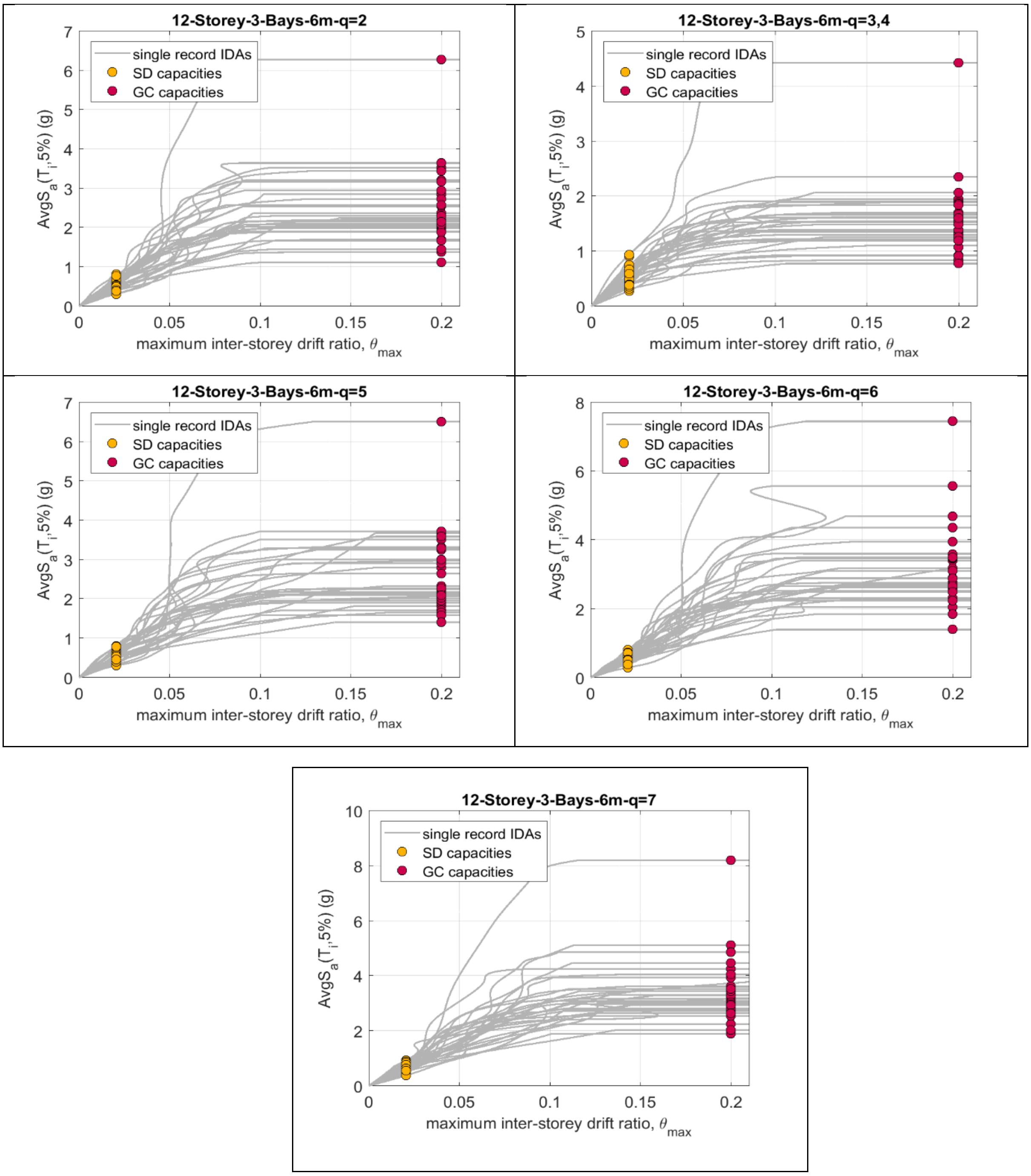


Figure 5-14 30 Incremental Dynamic Records for 12S-3B-6m

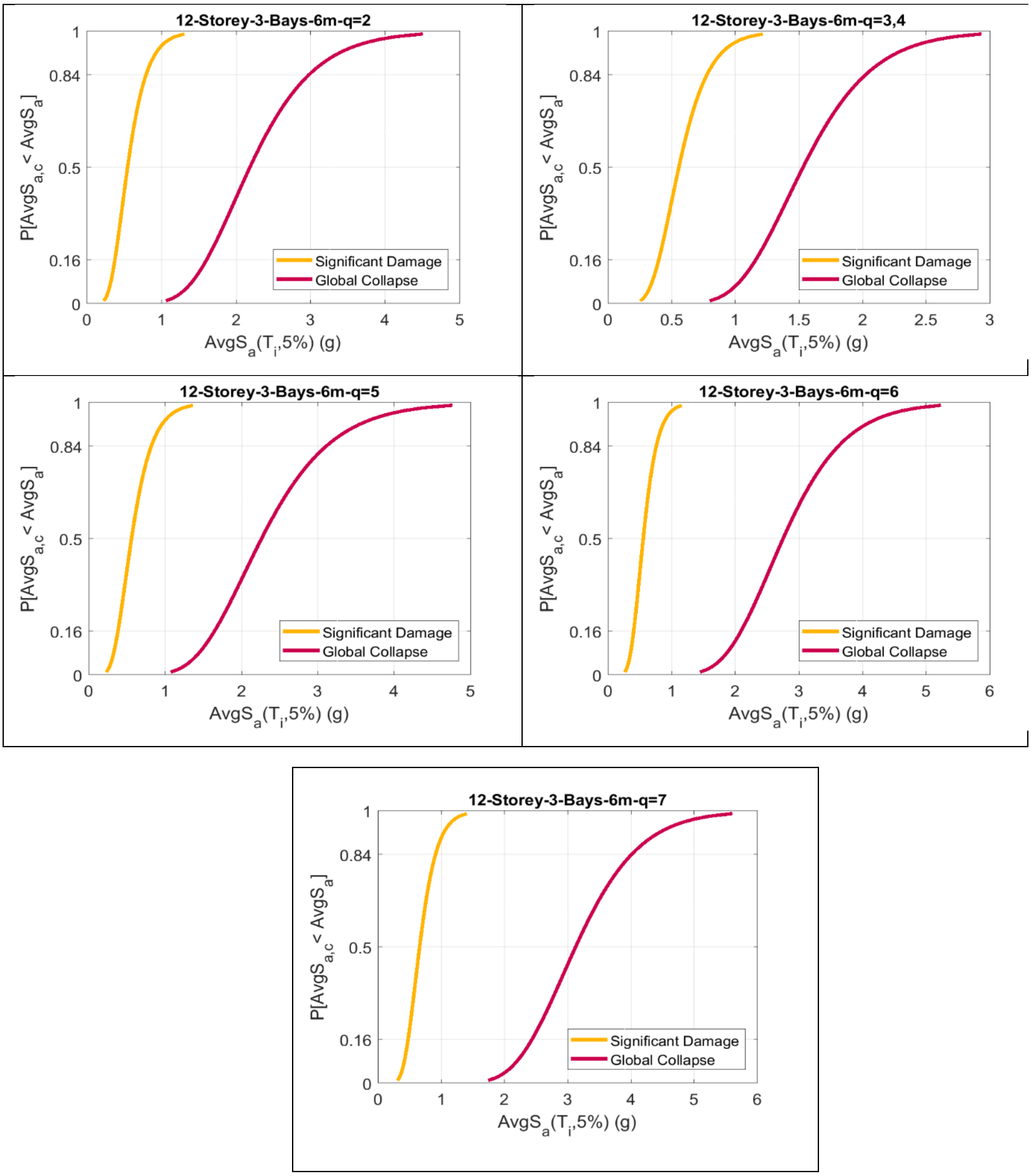


Figure 5-15 Fragility Curves for 12S-3B-6m

Table 5-5 Behaviour Factor Verification via The Limit State and the Mean Annual Frequency Estimation for 12S-3B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2,5	SD	0.674	2.107	3.126	1.000	✓
		GC	0.004	0.201	44.812		
	q=3,4	SD	0.463	2.107	4.553	1.000	✓
		GC	0.017	0.201	11.499		
	q=6	SD	0.706	2.107	2.983	1.000	✓
		GC	0.001	0.201	208.681		
	q=7	SD	0.518	2.107	4.069	1.000	✓
		GC	0.000	0.201	677.099		
Perugia	q=2,5	SD	0.705	2.107	2.988	1.000	✓
		GC	0.002	0.201	88.238		
	q=3,4	SD	0.486	2.107	4.334	1.000	✓
		GC	0.011	0.201	17.949		
	q=6	SD	0.684	2.107	3.080	1.000	✓
		GC	0.000	0.201	593.910		
	q=7	SD	0.457	2.107	4.615	1.000	✓
		GC	0.000	0.201	2443.854		
Focsani	q=2,5	SD	0.365	2.107	5.768	1.000	✓
		GC	0.000	0.201	30807.222		
	q=3,4	SD	0.168	2.107	12.518	1.000	✓
		GC	0.000	0.201	2145.596		
	q=6	SD	0.304	2.107	6.923	1.000	✓
		GC	0.000	0.201	4258012.278		
	q=7	SD	0.164	2.107	12.855	1.000	✓
		GC	0.000	0.201	290296052.156		

Chapter 6

BEHAVIOUR FACTOR ESTIMATION

6 BEHAVIOUR FACTOR ESTIMATION

The current chapter provides the behaviour factor estimations/calculations based on both 1) re-analysis of the pushover curve (the combinations of different approaches/physical procedures for the reference parameters, i.e., F_1, F_y, d_y, d_m) and 2) Incremental Dynamic Analysis (*IDA*) using P.Setti methodology [10]. The methodology given in chapter 3 and the results of chapter 5 are used to estimate and calculate the behaviour factor. Further, a comparison is made between the *"average"* values obtained by IDA and the *"single"* value obtained by re-analysis of the pushover curve for the *"optimal methods"*. The *"optimal methods"* are the combinations of the reference parameters (i.e., F_1, F_y, d_y, d_m) to estimate the behaviour factor that at best return the initial design behaviour factor (q_{des}).

6.1 Behaviour Factor Calculations Based on Re-Analysis of the Pushover Curve

To calculate the behaviour factor, the above-mentioned pushover response curves given in chapter 5 (Figure 5-2 and Figure 5-5) are re-analysed based on the combinations of the possible definitions of the reference parameters. The reference parameters are previously presented in Table 3-4 and 90 different values of the q-factor are derived for each case study.

In order to identify which method gives a q-factor consistent with the assumed design behaviour factor (q_{des}), hereafter attention is focused on those methods giving the results ranging within ±20% with respect to the assumed design q-factor value. Any method that gives the value within the above-mentioned range are deemed to be accepted and colored with green presented in a table (see as an example Table 6-1 for 12S-3B-6m). The results show that a large scatter of the obtained results in terms of q-factor is evident. The behaviour factor acceptance (q_{acc}) percentile for each method is then identified and shown in the relevant table for each series of structures (for the full result see APPENDIX-G-).

6.1.1 Behaviour Factor Acceptance (q_{acc})

In some cases, as shown in the previous section, changing the design behaviour factor will not essentially change the response of the structure. This is more evident in the lower storey structures, but can also happen in tall buildings (see Figure 5-2 to Figure 5-5). For instance, as shown in Figure 5-2, if the 12-storey 3-bays and 6m length of the bay (*12S-3B-6m*) designed with the behaviour factor equal to 3.0 and 4.0 results in more or less the same structure (Figure 5-10) and hence the same pushover curve will be obtained (Figure 5-2). In this case, the accepted behaviour factor is considered as the median of the design values. For example, for the above mentioned case the accepted behaviour factor will be considered as 3.5 ($q_{acc}=3.5$).

In some other cases, for example, for 8-storey 3-bays and 6m length of the bay (*8S-3B-6m*) designed with q=2, the response of the structure is more or less the same as q=6 (Figure 5-2). The accepted behaviour factor in this case is considered as 6.0 ($q_{acc}=6.0$). If the building designed with q=2 shows the response between the two upper designed behaviour factor (e.g. between 5.0 and 6.0), the accepted behaviour factor is the lowest value ($q_{acc}=5.0$).

The q_{acc} for the rest of the cases is identified as the same value of the design behaviour factor (*qdes*).

6.1.2 Typical MRF Buildings (*Conventional Structures*)

As an example, Table 6-1 shows the behaviour factor calculated for 12S-3B-6m if designed by q=3, 4. The full behaviour factor calculations for all case studies can be found in APPENDIX-G-1 from Table 9-67 to Table 9-125. According to the above definition of acceptance behaviour factor, the median of the two design behaviour factor is considered as the acceptance q factor for this type of building ($q_{acc}=3.5$). As explained earlier, the green color shows that the calculated behavior factor is in the acceptance range of ±20% with respect to the assumed design q-factor.

Table 6-2 displays the behaviour factor acceptance percentile for each method of combination for all conventional case studies. The full results can be found in APPENDIX-G-3 from Table 9-132 to Table 9-149. As the table implies, the definition of F_1-d_1-2 (see Table 3-3) is estimating the behaviour factor better than other definitions for the first significant yielding of the structure (F_1). On the other hand, the definition of d_m-1 (maximum horizontal roof displacement (d_m) corresponding to the maximum strength of the structure) is poorly able to estimate the behaviour factor and in some cases is failed. The results also show that the combination of F_y-d_y-1, F_1-d_1-2 and d_m-$3/4/5$ (*"method 38, 56 and 74"*) is the best combination to define and estimate the behaviour factor by the acceptance percentile of 85%, 88% and 88%, respectively. Table 6-3 shows the dispersion of the calculated behaviour factor for conventional structures. The dispersion from the target behaviour factor acceptance (q_{acc}) for the above mentioned combinations are 25%, 24.4% and 24.1%.

Table 6-1 Behaviour Factor Calculated for 12S-3B-6m, q_{des}=3, 4 q_{acc}=3.5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.6	**2.0**	**1.5**	**1.3**	**1.6**	**2.0**	---	**1.3**	**1.6**	**2.0**	**2.2**	**1.3**	**1.6**	**2.0**	**1.8**	**1.3**	**1.6**	**2.0**	---	**1.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.5	**3.2**	**2.2**	**2.1**	**2.6**	**3.3**	---	**2.2**	**2.6**	**3.3**	**3.6**	**2.2**	**2.6**	**3.3**	**2.9**	**2.2**	**2.6**	**3.3**	---	**2.2**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.7	**3.4**	**2.3**	**2.3**	**2.9**	**3.6**	---	**2.4**	**2.9**	**3.6**	**4.0**	**2.4**	**2.9**	**3.6**	**3.2**	**2.4**	**2.9**	**3.6**	---	**2.4**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.8	**3.4**	**2.3**	**2.3**	**3.0**	**3.8**	---	**2.5**	**3.0**	**3.8**	**4.1**	**2.5**	**3.0**	**3.7**	**3.3**	**2.5**	**3.0**	**3.7**	---	**2.5**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.8	**3.4**	**2.3**	**2.3**	**3.0**	**3.8**	---	**2.5**	**3.0**	**3.8**	**4.1**	**2.5**	**3.0**	**3.7**	**3.3**	**2.5**	**3.0**	**3.7**	---	**2.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 6-2 Behaviour Factor Acceptance Percentile for All Conventional Structures

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	20	**29**	**2**	**4**	**33**	**36**	---	**18**	**33**	**36**	**25**	**18**	**30**	**35**	**30**	**18**	**33**	**36**	---	**14**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	42	**69**	**9**	**7**	**42**	**76**	---	**28**	**42**	**76**	**47**	**28**	**43**	**81**	**46**	**25**	**42**	**74**	---	**25**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	43	**85**	**10**	**10**	**45**	**70**	---	**38**	**45**	**70**	**46**	**38**	**49**	**71**	**49**	**31**	**45**	**70**	---	**38**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	46	**88**	**10**	**10**	**46**	**70**	---	**45**	**46**	**70**	**43**	**46**	**49**	**71**	**50**	**41**	**46**	**70**	---	**45**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	46	**88**	**8**	**11**	**46**	**68**	---	**45**	**44**	**68**	**42**	**45**	**47**	**70**	**51**	**41**	**44**	**68**	---	**45**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 6-3 The Dispersion of the Calculated Behaviour Factor for All Conventional Structures

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0.35	**0.31**	**0.34**	**0.38**	**0.41**	**0.39**	---	**0.43**	**0.41**	**0.38**	**0.40**	**0.43**	**0.39**	**0.35**	**0.40**	**0.42**	**0.41**	**0.39**	---	**0.43**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0.24	**0.24**	**0.22**	**0.23**	**0.32**	**0.33**	---	**0.30**	**0.32**	**0.33**	**0.26**	**0.30**	**0.27**	**0.27**	**0.24**	**0.26**	**0.32**	**0.33**	---	**0.30**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0.25	**0.25**	**0.22**	**0.23**	**0.33**	**0.35**	---	**0.31**	**0.33**	**0.34**	**0.27**	**0.30**	**0.29**	**0.29**	**0.24**	**0.27**	**0.33**	**0.34**	---	**0.30**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0.24	**0.244**	**0.22**	**0.23**	**0.34**	**0.35**	---	**0.31**	**0.33**	**0.35**	**0.27**	**0.31**	**0.29**	**0.30**	**0.25**	**0.28**	**0.34**	**0.35**	---	**0.31**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0.24	**0.241**	**0.22**	**0.22**	**0.34**	**0.36**	---	**0.31**	**0.34**	**0.35**	**0.28**	**0.31**	**0.29**	**0.31**	**0.25**	**0.28**	**0.34**	**0.35**	---	**0.31**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

6.1.3 Innovative Dissipative FUSEIS Bolted & welded Beam Splices (*Non-Conventional Structures*)

The following tables refer to the results obtained for the non-conventional structures, here conceived with FUSEIS Bolted & welded Beam Splices. The same approach applied to conventional structures for the estimation of the behavior factor is applied here. Table 6-4 represents the acceptance percentile of innovative dissipative bolted and welded beam splices. The results highlighted that *"method 56 and 74"* are able to estimate the behaviour factor even for non-conventional buildings and only one structure (8-storey bolted beam splices) with a very small variation that is out of the acceptance range with a dispersion of about 6-7% (see Table 6-5). Extended results may be found in APPENDIX-G-2, in particular from Table 9-126 to Table 9-131 which are displayed the behaviour factor calculated for 8, 4 and 2 storey buildings occupied by bolted and welded beam splices.

Table 6-4 Behaviour Factor Acceptance Percentile for 8/4 and 2-Storey Bolted and Welded Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	50	67	0	0	50	50	---	83	50	50	17	67	67	67	17	17	50	50	---	83
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	100	100	67	50	0	0	---	50	17	17	17	33	83	67	83	50	0	0	---	50
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	100	83	83	50	0	0	---	17	0	0	0	33	33	50	50	83	0	0	---	17
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	67	83	83	67	0	0	---	0	0	0	0	17	0	0	50	50	0	0	---	17
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	67	83	67	67	0	0	---	0	0	0	0	17	0	0	50	50	0	0	---	0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 6-5 The Dispersion of the Calculated Behaviour Factor for 8/4 and 2-Storey Bolted and Welded Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0.06	0.08	0.09	0.14	0.08	0.06	---	0.13	0.11	0.13	0.17	0.19	0.11	0.14	0.18	0.20	0.09	0.07	---	0.13
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0.10	0.10	0.12	0.17	0.12	0.10	---	0.17	0.15	0.16	0.20	0.21	0.14	0.16	0.20	0.21	0.12	0.10	---	0.17
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0.08	0.08	0.11	0.15	0.11	0.08	---	0.14	0.13	0.14	0.18	0.20	0.12	0.14	0.18	0.20	0.11	0.08	---	0.14
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0.07	0.07	0.12	0.13	0.10	0.06	---	0.12	0.11	0.12	0.16	0.18	0.10	0.12	0.16	0.18	0.10	0.07	---	0.12
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0.07	0.06	0.14	0.13	0.12	0.09	---	0.12	0.11	0.10	0.14	0.16	0.08	0.10	0.14	0.16	0.12	0.09	---	0.12
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

6.1.4 The Optimal Methods

The results of the behaviour factor evaluations based on re-analysis of the pushover curve show that F_1-d_1-*2* allows a better estimate of the behaviour factor than other definitions of F_1. In the same way, d_m-*4* and d_m-*5* allow a better estimation of the behaviour factor than the other d_m-definitions based on the re-analysis of the pushover curves for all 102 case studies including conventional and non-conventional structures.

Among all possible combinations of definitions of the reference parameters given in chapter 3 for the assessment of the behaviour factor, the combinations of F_1-d_1-*2*, F_y-d_y-*1* and d_m-*4* or d_m-*5* represent the best correlation with respect to the assumed design value of the behaviour factor.

The definition of the reference parameters for method *56* and *74* are as follows:

F_m=F_y is the maximum actual strength of the structure

d_m is the displacement corresponding to 15% or 20% loss of strength in softening branch for methods *56* and *74*, respectively.

F_1 being the first yielding of any elements of the structure (*Local Plasticization*)

d_y is the displacement corresponding to the knee-point of the bilinear idealized elastic-perfectly plastic curve

$$d_y = 2(d_m - \frac{E_m}{F_y}) \quad \text{Eq. 6-1}$$

Where

E_m is the area under the capacity curve up to "d_m"

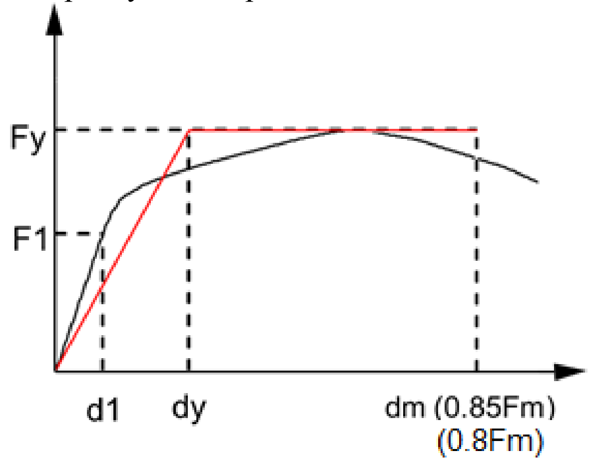


Figure 6-1 Optimal Methods to Define the Behaviour Factor

6.1.5 Optimal Methods and Discussion of the Results

In this section, the initial design behaviour factor (q_{des}) is compared with the behaviour factor estimated by re-analysis of the pushover curve using the *"optimal methods"* (i.e., *method 56 and 74*) and so called behaviour factor acceptance (q_{acc}) based on the methodology given in section 0. Figure 6-2 to Figure 6-22 summarize the reference parameters adopted for re-analysis of the pushover response curve, as well as the derived values of $q_{\Omega}.q_{\mu}.q_{\xi}$ leading to the assessment of the q-factor for each building for the *"optimal methods"* only. The results show that there is a little difference between "*method 56 and 74"* for parameters such as ductility and overstrength factor.

6.1.5.1 Typical MRF Buildings (*Conventional Structures*)

6.1.5.1.1 q_{acc}=7

Considering the acceptance q factor equal to 7, the ductility factor increases if the length of the bay increases which is more evident for 8-storey buildings. Increasing the number of the bays has not much influence on the value of the ductility factor for 12-storey buildings, but it results in an increment of q_{μ} for 8-storey buildings. The 8S-3B-8m-q_{des}=2, 8S-3B-6m-q_{des}=2 and 8S-4B-8m-q_{des}=2 show the same response as the buildings designed with the behaviour factor equal to 7.0, hence they are included in this group.

For what regards the overstrength factor the opposite is observed, which means the overstrength factor decreases by increasing the length of the bays. This is perhaps due to decreasing the stiffness of the system as the joint column-beam become "less resistant", in terms of a redundant structure. In such a case, once the first plasticity is reached, the structure tends to behave as a non-redundant and the progressive collapse of the structure is normally manifested without a significant increment of the overstrength. As the Figure 6-3 implies, the value specified in EC8 may underestimate the value of the overstrength if q_{acc} = 7.0.

Figure 6-4 shows that the behaviour factor increases by increasing the length of the bays with the exception for 12S-3B-8m-q_{des}=7. Increasing the number of the bays will also increase the value of the behaviour factor by a very low percentage. Considering the preceding result, while with a combination of the increment of the ductility factor and the overstrength factor, a resultant increase of the behavior factor is observed. The 8S-3B-8m-q_{des}=2, 8S-3B-6m-q_{des}=2 and 8S-4B-8m-q_{des}=2 show almost the same calculated behaviour factor as 7.0 (see Figure 6-4). The 12S-3B-8m-q_{des}=7 shows a very low value of the behaviour factor (q=4.5) compared to the other buildings. Such value is below the defined acceptance range of the behaviour factor. The 12S-3B-8m-q_{des}=7 highlights the role of the overstrength in the overall behavior of the structure. It is highly recommended that a particular care should be spent during the design process for ensuring a proper redundant behavior of the structure. The average calculated behaviour factor for this acceptance range is 6.8.

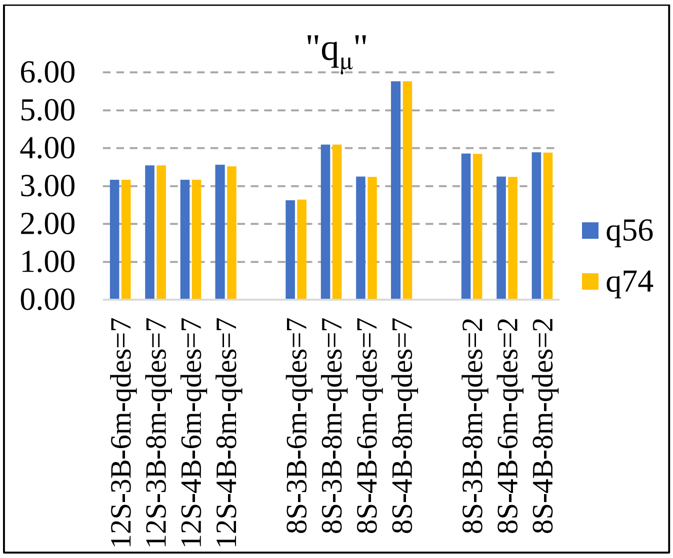


Figure 6-2 The Ductility Factor if Designed by q=7

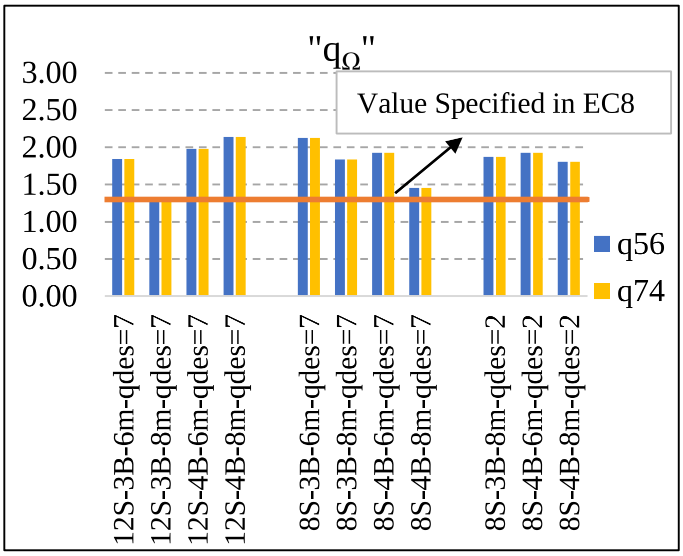


Figure 6-3 The Overstrength Factor if Designed by q=7

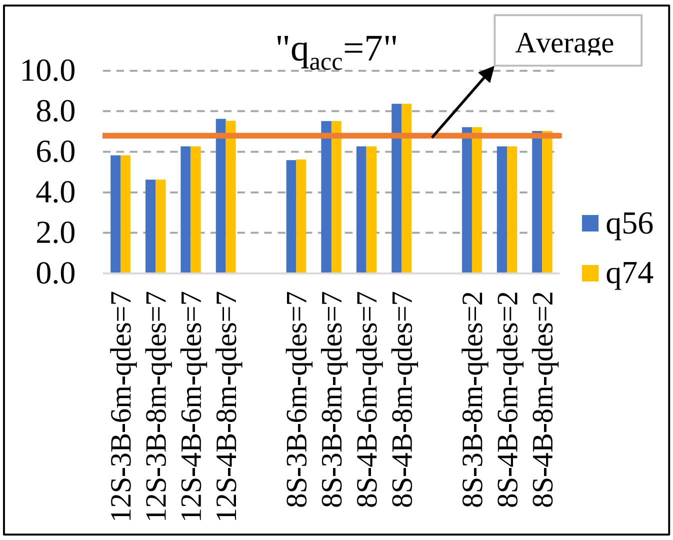


Figure 6-4 The Calculated Behaviour Factor if Designed by q=7

6.1.5.1.2 q_{acc}=6

The results for the same group of the structures designed with a q_{des}=6 are presented hereafter. The 12S-3B-8m-q_{des}=2 and 8S-3B-6m-q_{des}=2 show the same response as the buildings designed with the behaviour factor equal to 6.0, hence they are included in this group. The observed behaviour for the ductility factor and overstrength factor are as follows.

The ductility factor increases by increasing the length of the bay, while increasing the number of the bays will change the value of the ductility factor with a very low percentage (see Figure 6-5).

The overstrength factor increases by increasing the length of the bays for 3 bays buildings (3B-6m to 3B-8m), but 4 bays buildings, (4B-6m to 4B-8m) increasing the length of the bays results in reduction of the overstrength factor for 8-storey buildings while it has no effect for 12-storey buildings. As the Figure 6-6 implies the value specified in EC8 may underestimate the value of the overstrength if q_{acc} = 6.0.As discussed in the previous case/group, both the behavior factor and the overstrength factor are very sensitive to the flexibility and redundancy of the structure, where a combination of these aspects determines their values. In this case the observed increment of the overstrength factor by increasing the length of the bay can be explained by a more redundant behavior exhibited by a more flexible structure, which is resulted to be more redundant.

Figure 6-7 displays the results of the behaviour factor if q_{des}=6.0. As the figure implies by increasing the length of the bays for 12 and 8-storey buildings the behaviour factor increases. Increasing the number of the bays will also increase the value of the behaviour factor. 12S-4B-

8m-q_{des}=2 and 8S-3B-6m-q_{des}=2 show almost the same calculated behaviour factor as the buildings designed for q_{des} = 6.0 (see Figure 6-7). The average calculated behaviour factor in this acceptance range is 5.8.

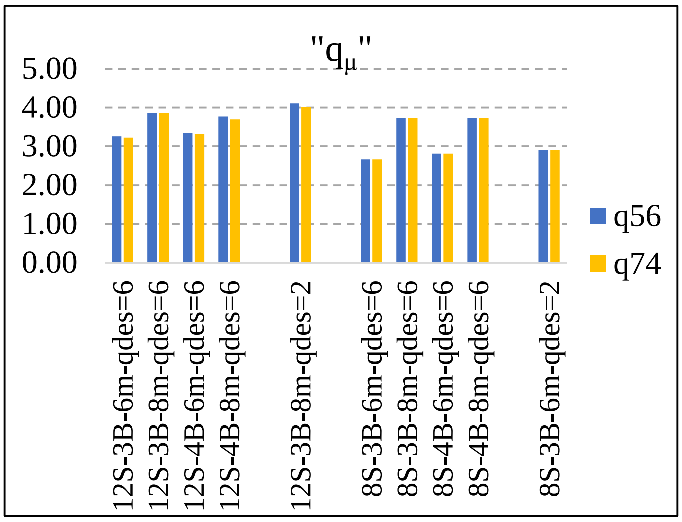


Figure 6-5 The Ductility Factor if Designed by q=6

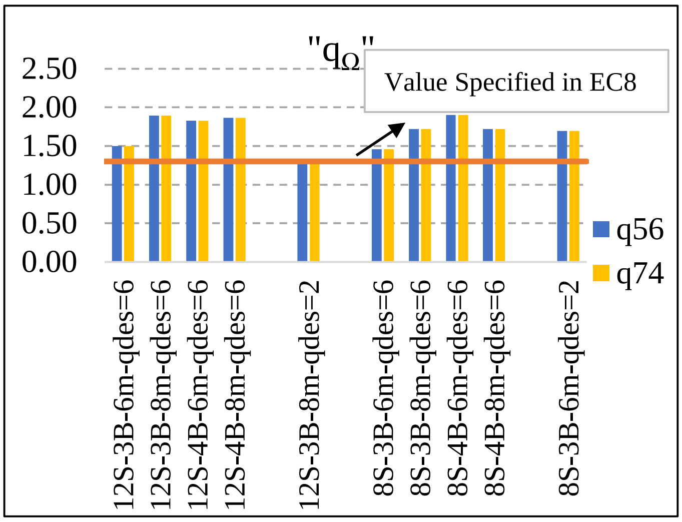


Figure 6-6 The Overstrength Factor if Designed by q=6

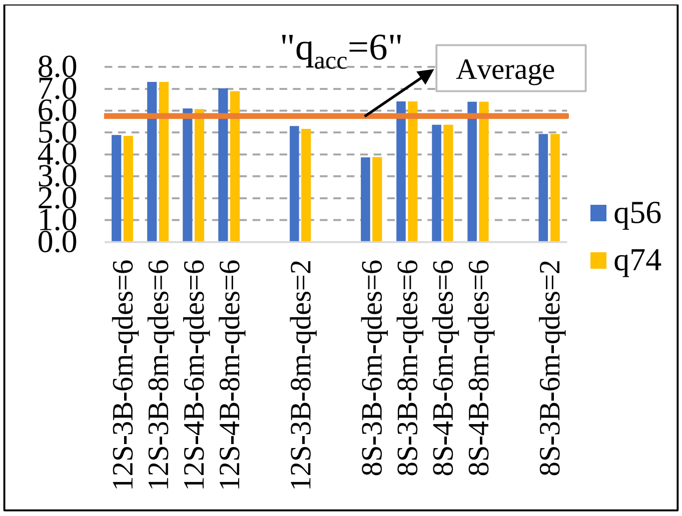


Figure 6-7 The Calculated Behaviour Factor if Designed by q=6

Worth mentioning that, some cases exhibit a behavior factor much lower compared to the design behaviour factor. This aspect is more important to be commented for these two first cases than the following ones. As a matter of fact, to pretend to design with a behavior factor being equal to 6.0 or 7.0, it is expected to have a very high ductility structure, and when the exhibited ductility has a behavior factor lower than 4 (normal behavior factor of the practice) all the procedure becomes questionable. It is highly recommended to have a deep insight into the response of the structures designed with a very high q_{des}, before certifying this values, in order to avoid such undesirable circumstances.

6.1.5.1.3 q_{acc}=5

The following structures show the same response of the structure the buildings designed with the behaviour factor equal to 5.0, hence they are included in this group.

- 12S-3B-6m-q_{des}=2, 5
- 12S-3B-8m-q_{des}=5
- 12S-4B-6m-q_{des}=2, 5
- 12S-4B-8m-q_{des}=2, 5

- 8S-3B-8m-q_{des}=5
- 8S-4B-8m-q_{des}=5

- 4S-3B-6m-q_{des}=3, 4, 5, 6, 7
- 4S-3B-8m-q_{des}=3, 4, 5, 6, 7
- 4S-4B-6m-q_{des}=3, 4, 5, 6, 7
- 4S-4B-8m-q_{des}=3, 4, 5, 6, 7

- 2S-3B-8m-q_{des}=3, 4, 5, 6, 7

- 2S-4B-8m-q_{des}=3, 4, 5, 6, 7

The ductility factor increases by increasing the length of the bay for 12 and 8-storey buildings which has no effect in the case of 4 and 2 storey buildings.

The overstrength factor increases by increasing the length of the bays for 8-storey and 4-storey buildings. the opposite is valid for 12-storey buildings. The value of the overstrength will increase by increasing the number of the bays for 4, 8 and 12-storey buildings. As the Figure 6-9 implies the value specified in EC8 in general underestimates the value of the overstrength factor for all types of considered buildings except for 2-storey buildings.

The obtained results can be explained by the design procedures, where starting from a $q_{des} \leq 5$, the role of seismic actions are suitably balanced by other design criteria. In fact for a range of behavior factor between 3.5 and 5 we find the best match between the observed behavior factor and the designed one, as it is demonstrated in the following paragraphs. Increasing the bay length highlights the role of proper seismic design by respecting the strong column-weak beam criterion. The increment of the storey number is reflected with an enhancement of the redundant behavior and the number of plastic hinges involved in the collapse mechanism.

Figure 6-10 displays that the behaviour factor increases by increasing the length of the bays for 4, 8 and 12-storey buildings. Increasing the number of the bays will also result in an increment in the value of the behaviour factor for 8 and 12-storey buildings, but leaves practically unchanged the behaviour factor for 4 and 2-storey buildings. The average calculated of the behaviour factor for this acceptance range is 4.3. Increasing the number of bays generally results in an increment of the ductility factor for buildings with 8 and 12-storeys (with the exception of 12S-6m buildings). For lower buildings (2 and 4-storeys) the ductility factor as well as the overstrength factor seem to be independent on the number of bays.

The last cases need a particular attention. The observed results are believed to be related to the structural configuration and the role of the behavior factor into the design configurations. Short structures result to be like a connection in parallel of different bays which result not increasing the overstrength factor of the structure. The redundancy or the ductility of such structures is highly attributed to the local element’s ductility, rather than to the structural compounds combination.

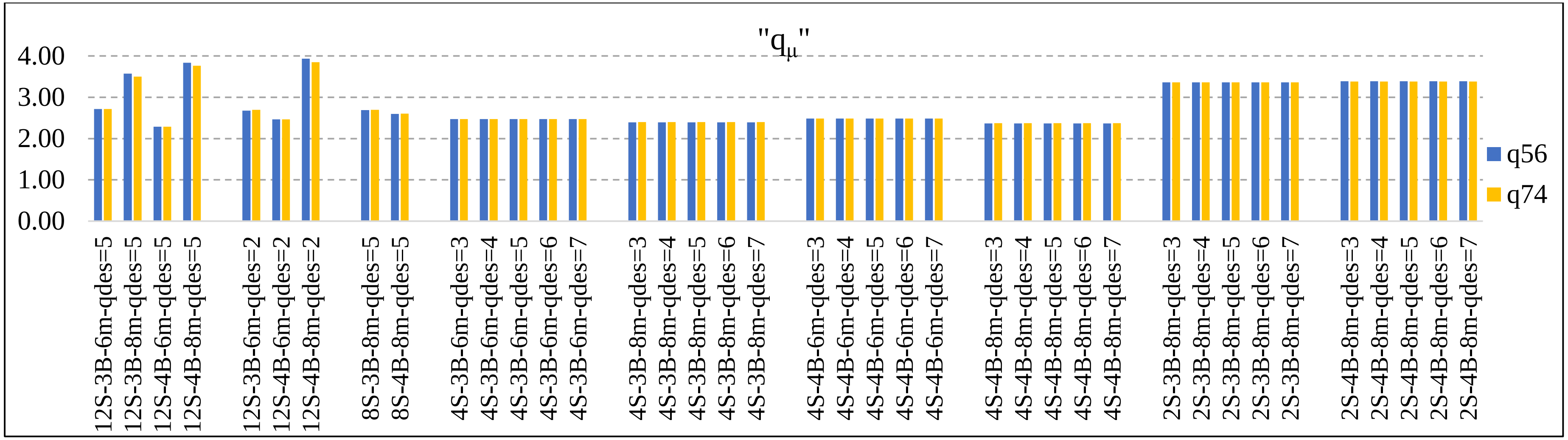


Figure 6-8 The Ductility Factor if Designed by q=5

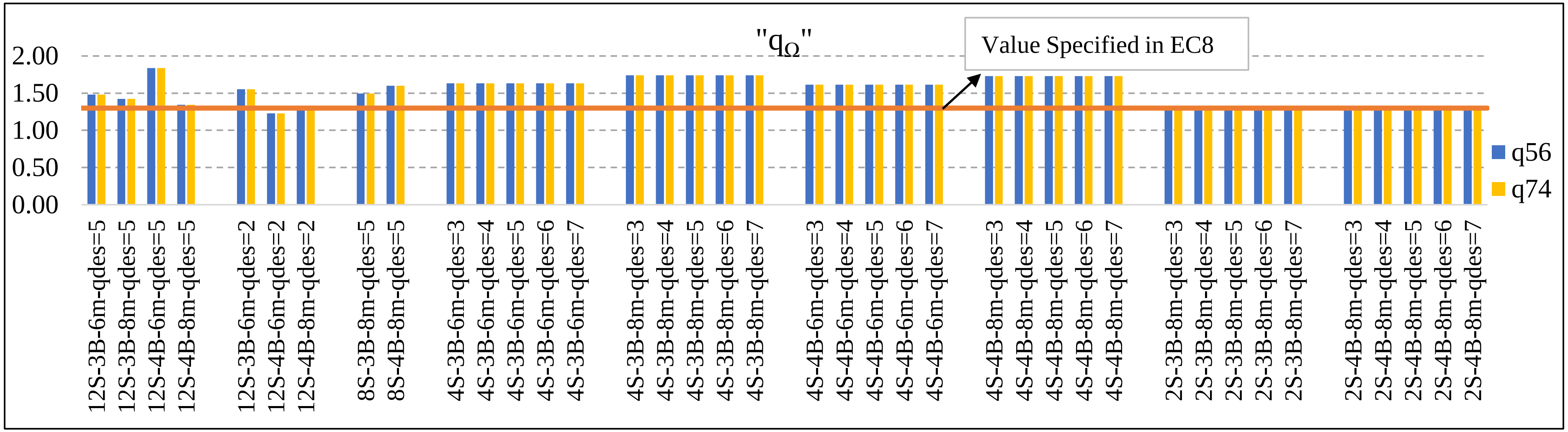


Figure 6-9 The Overstrength Factor if Designed by q=5

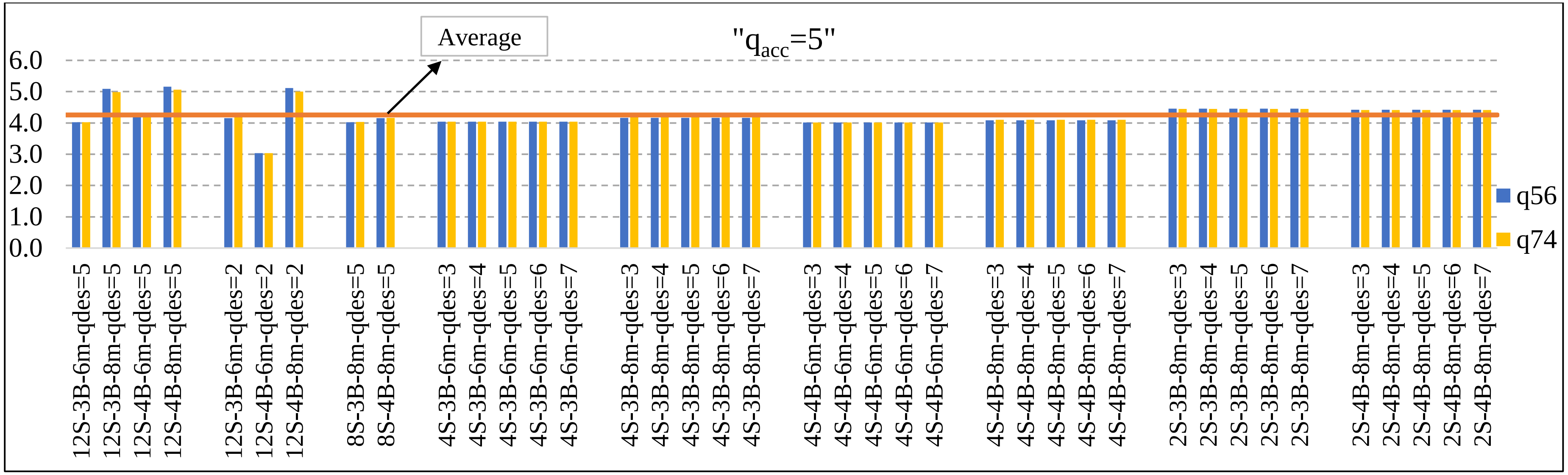


Figure 6-10 The Calculated Behaviour Factor if Designed by q=5

6.1.5.1.4 q_{acc}=4

This case is expected to be more similar to the previous case. As a matter of fact, considering the acceptance q factor as 4.0, increasing the length of the bay results in increasing the ductility of the structure. However, increasing the number of the bays, slightly increases the ductility of the structure (see Figure 6-11).

Increasing the length of the bays results in increasing the overstrength of the structure. The value of the overstrength will also increase by increasing the number of the bays. As the Figure 6-12 displays the value specified in EC8 in general underestimate the value of the overstrength factor if q_{acc} considered as 4.0.

Figure 6-13 displays that the behaviour factor increases by increasing the length of the bays as well as by increasing the number of the bays although, in this latter case, the influence is smaller. The average calculated of the behaviour factor for this acceptance range is 3.9.

The average behaviour factor being equal to 3.9 for the structures design with q_{des} = 4.0 is quite remarkable. For the previous case, q_{des} = 5.0, the average differed much more than this case but more important is to retention that the behavior factor was lower than 5.0. Form the practical point of view, it seems that the objective of having high value of behavior factor is not reliable. It is known that designing for a behavior factor higher than the expected ones could violate the safety considerations of the structure and to result in undesirable circumstances. In fact, one of the main aims of this research is to confront the results between the design and the performance expectations.

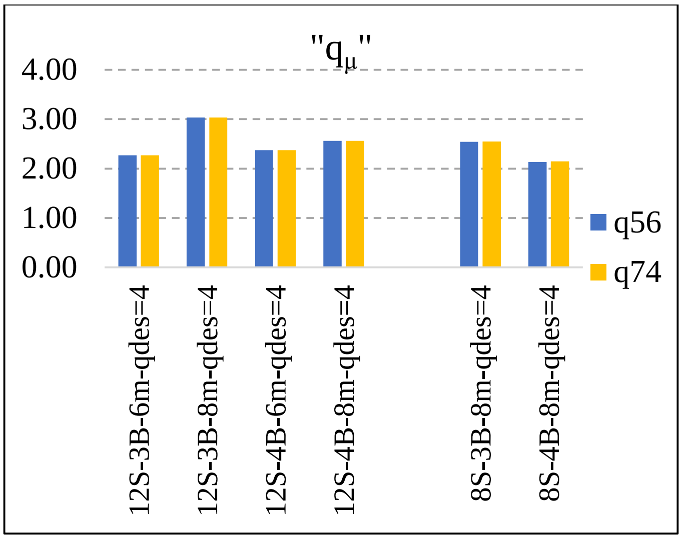


Figure 6-11 The Ductility Factor if Designed by q=4

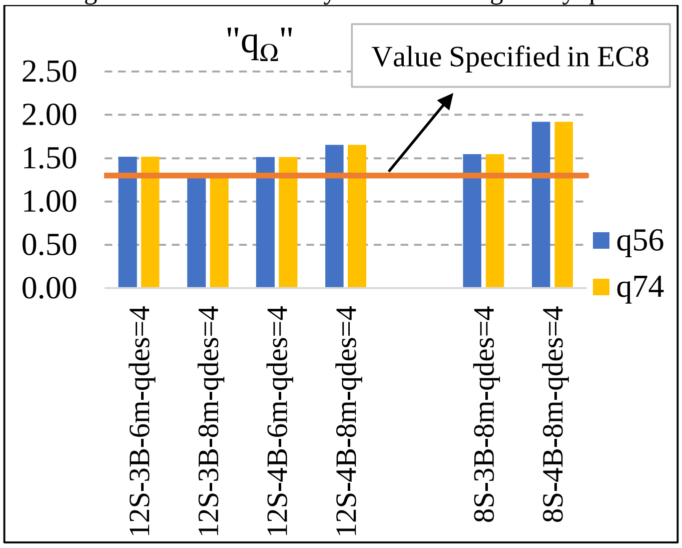


Figure 6-12 The Overstrength Factor if Designed by q=4

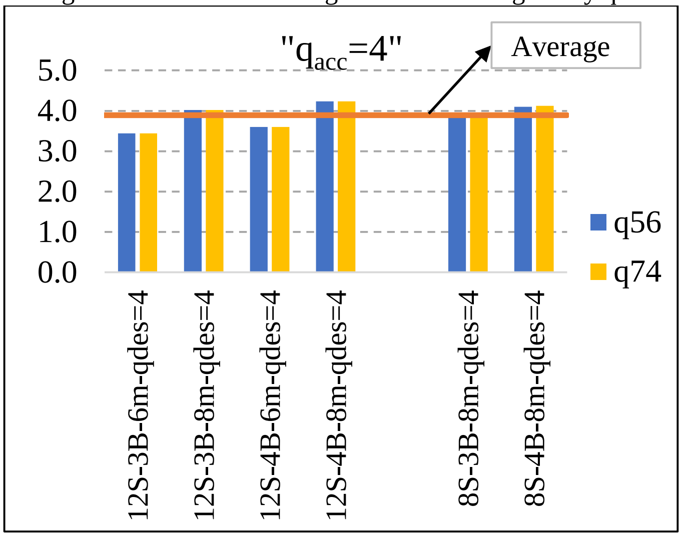


Figure 6-13 The Calculated Behaviour Factor if Designed by q=4

6.1.5.1.5 $q_{acc} = 3$

The results show that the ductility factor increases by increasing the length of the bays and the number of the bays for 8-storey buildings, while for 12-storey buildings, increasing the length of the bays does not results in an increase of the ductility of the structure (see Figure 6-14). This particular case may be explained by the “overdesign” of 8-storey buildings for seismic actions, compared to 12-storey buildings. In such circumstances, the bay length can become more flexible parameter for the structure.

As explained in section 0, in some cases, changing the design behaviour factor will not essentially change the response of the structure. For instance, as shown in Figure 5-2, if the 12-storey 3-bays and 6m length of the bay (12S-3B-6m) designed with the behaviour factor equal to 3.0 and 4.0 results in more or less the same structure (Figure 5-10) and hence the same pushover curve will be obtained (Figure 5-2). In this case, the accepted behaviour factor is considered as the median of the design values. For this case the accepted behaviour factor will be considered as 3.5 (q_{acc}=3.5). Hence, the 12S-3B-6m-q_{des}=3 is not included in this group, but in the group of q_{acc} = 3.5 together with 12S-3B-6m-q_{des}=4.

The overstrength factor increases only if the number of bays increases. Increasing the length of the bays practically does not influence on the overstrength factor of the structures. The reason is that, the elements (columns) are designed for considerable contribution of seismic loads, hence, the number of bays increases the redundancy of the structure, as a result of increasing the number of plastic hinges in the beams. Whereas increasing the bay length does not change the results. As the Figure 6-15 implies the value specified in EC8 in general approximate quite accurately the value of the overstrength factor accurately.

Figure 6-16 shows that increasing the length of the bays from 3B-6m to 3B-8m the behaviour factor increases for 8-storey buildings and remains unchanged for 4-bays. For the 12-storey buildings increasing the length of the bays from 4B-6m to 4B-8m results in a reduction of the value of the behaviour factor. Increasing the number of the bays will also increase the value of the behaviour factor for 8-storey buildings. The average calculated of the behaviour factor for this acceptance range is 3.1. Compared to the previous case (q_{acc}=4), the obtained behavior factor was slightly lower than 4 while now it is slightly higher than 3. In the later one, the plastic capacities of the structure are not being properly utilized, while in the first one, they are slightly being overestimated.

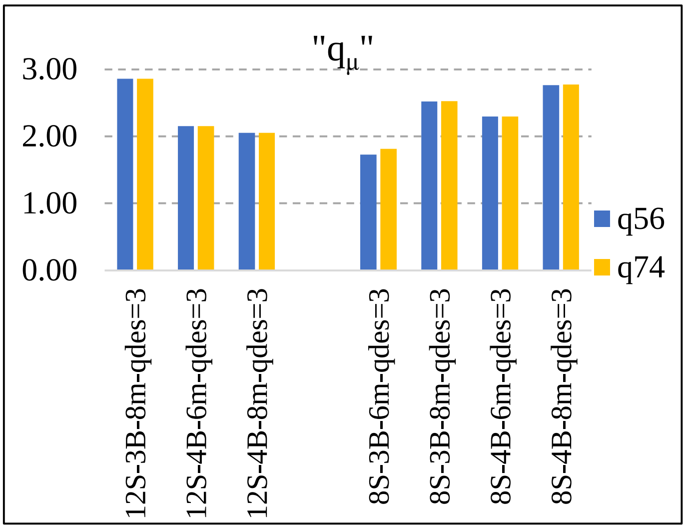


Figure 6-14 The Ductility Factor if Designed by q=3

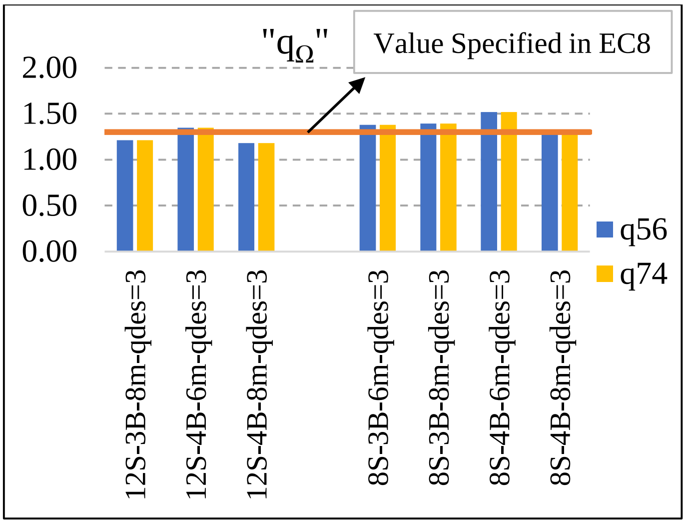


Figure 6-15 The Overstrength Factor if Designed by q=3

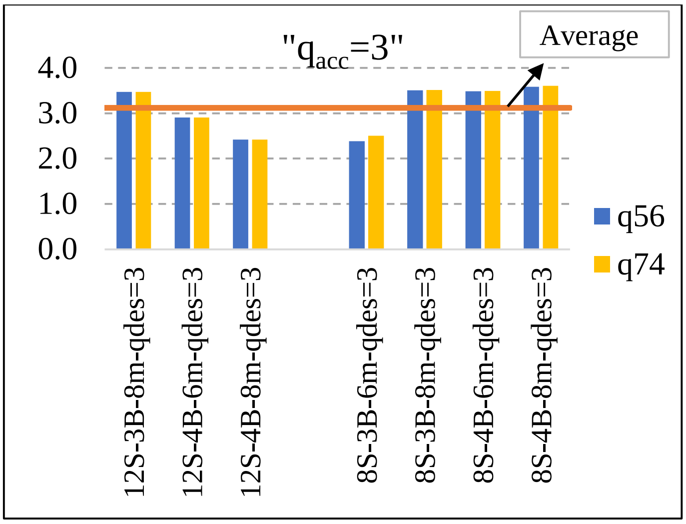


Figure 6-16 The Calculated Behaviour Factor if Designed by q=3

6.1.5.1.6 $q_{acc} = 2$

Practically, the opposite case of high ductility is to design without ductility. The design behavior factor being equal to 2, is the lowest design behaviour factor which is very uncommon for steel structures. In fact, many practices utilize a higher behavior factor to characterize materials which are even brittle. Such value is expected to be unrealistic for steel structures, as it results the following.

The results prove that the ductility factor increases by increasing the length of the bays and the number of the bays (see Figure 6-17). Based on the methodology given in section 0, the 2S-3B-6m-q_{des}=2 and 2S-4B-6m-q_{des}=2 are not included in this group, but in the group for q_{acc}=4.5 which is specified below.

The overstrength factor increases by increasing the length of the bays, while increasing the number of bays will not change the results. The value specified in EC8 can more or less estimate the value of the overstrength factor (see Figure 6-18).

Figure 6-19 shows that the behaviour factor increases by increasing the length of the bays, however, increasing the number of the bays does not affect the results. The average calculated of the behaviour factor for this acceptance range is 4.5 which shows that they should not be included in this group.

These results highlight the previous comment that a behavior factor being equal to 2.0 is not realistic for steel structures. Despite we design without ductile expectations, the steel material will provide a considerable ductility.

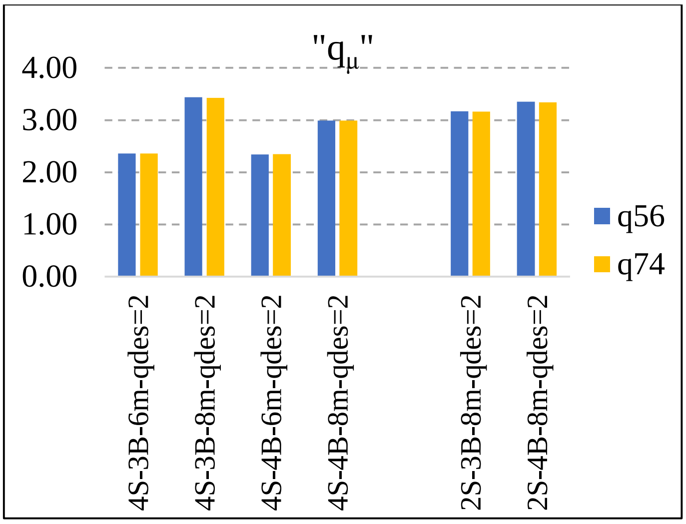


Figure 6-17 The Ductility Factor if Designed by q=2

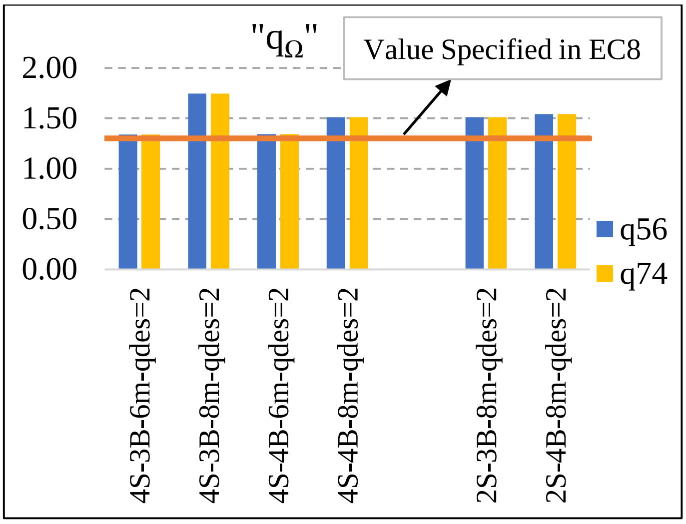


Figure 6-18 The Overstrength Factor if Designed by q=2

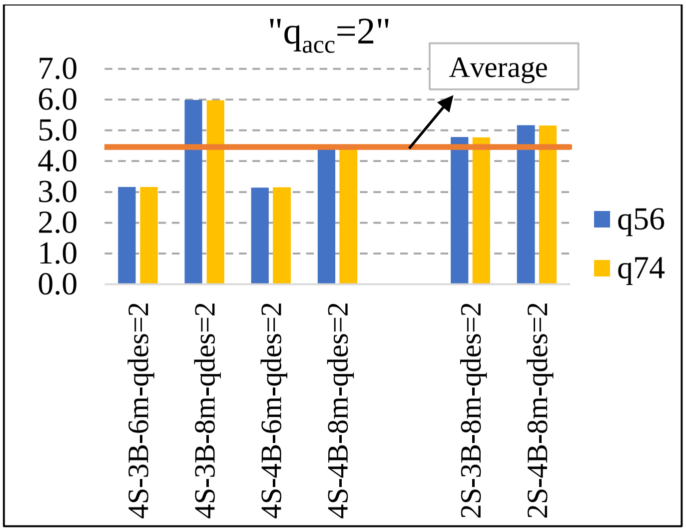


Figure 6-19 The Calculated Behaviour Factor if Designed by q=2

6.1.5.1.7 q_{acc} = 3.5 and 4.5

The last case is dedicated to the behavior factors being 3.5 and 4.5. Considering Figure 5-2 to Figure 5-5, changing the behaviour factor results the same structure and, hence, the same response curve will be obtained. Thus, the ductility factor, the over strength factor and the behaviour factor remain unchanged during the calculation of the behaviour factor (see Figure 6-20 to Figure 6-25). It proves our expectations for this behavior factor range, where the design process seem to be quite insensitive for small fluctuations of the behavior factor.

Table 6-6 The Acceptance Behaviour Factor

Type of Structure	Acceptance Behaviour Factor
12S-3B-6m-q_{des}=3 12S-3B-6m-q_{des}=4	q_{acc} = 3.5
8S-3B-6m-q_{des}=4 8S-3B-6m-q_{des}=5 8S-4B-6m-q_{des}=4 8S-4B-6m-q_{des}=5	q_{acc} = 4.5
2S-3B-6m-q_{des}=2, 3, 4, 5, 6, 7	q_{acc} = 4.5
2S-4B-6m-q_{des}=2, 3, 4, 5, 6, 7	q_{acc} = 4.5

The average calculated of the behaviour factor if considering q_{acc}=3.5 is 3.4 and if considering q_{acc}=4.5, is 3.9. The optimal and economical behavior factor here is proved to be fit between these values.

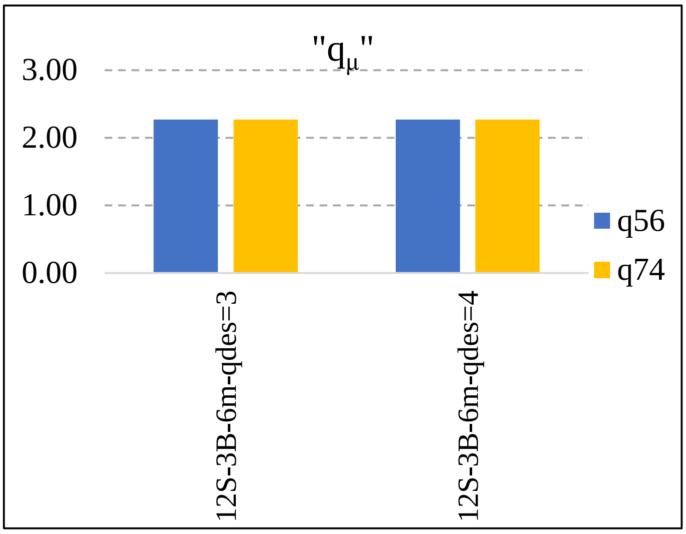


Figure 6-20 The Ductility Factor if q_{acc}=3.5

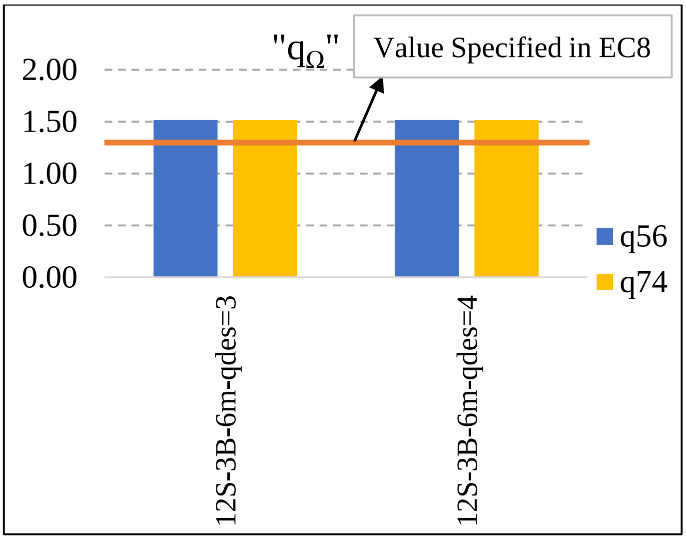


Figure 6-21 The Overstrength Factor if q_{acc}=3.5

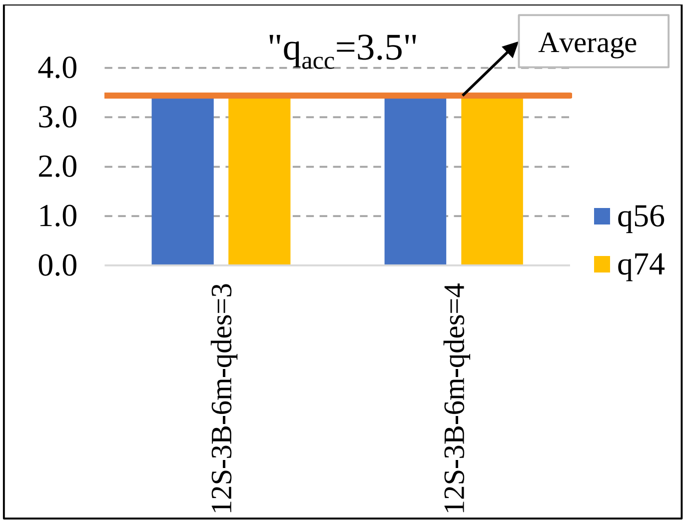


Figure 6-22 The Calculated Behaviour Factor if q_{acc}=3.5

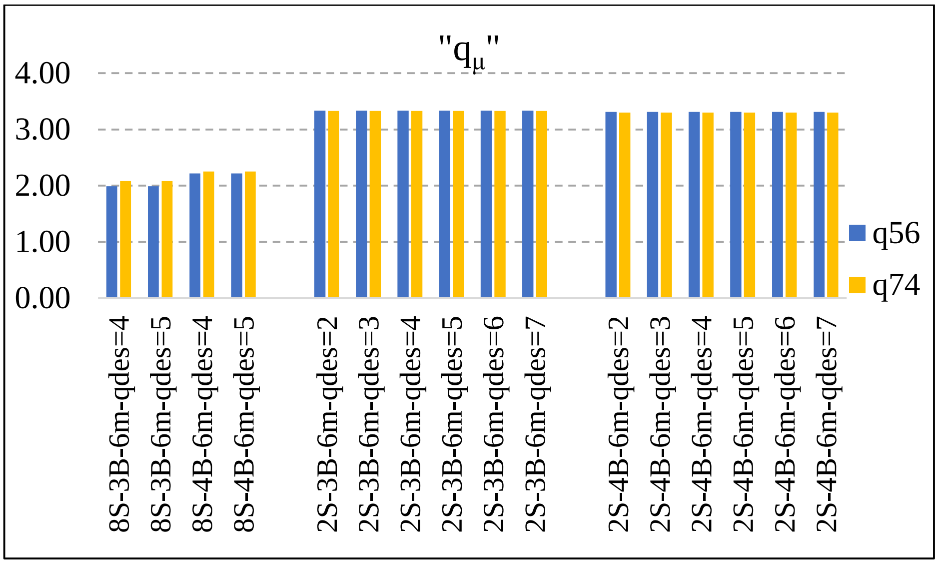


Figure 6-23 The Ductility Factor if q_{acc}=4.5

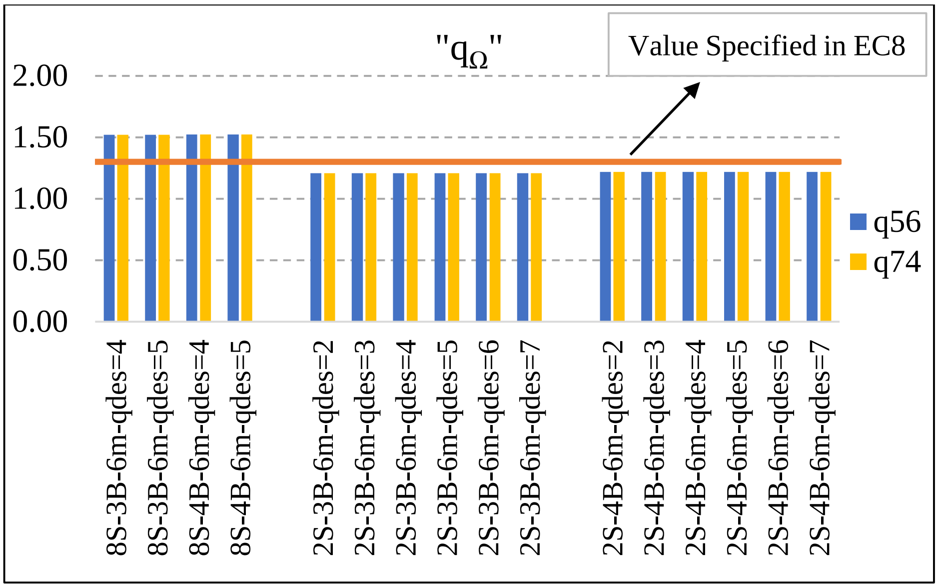


Figure 6-24 The Overstrength Factor if q_{acc}=4.5

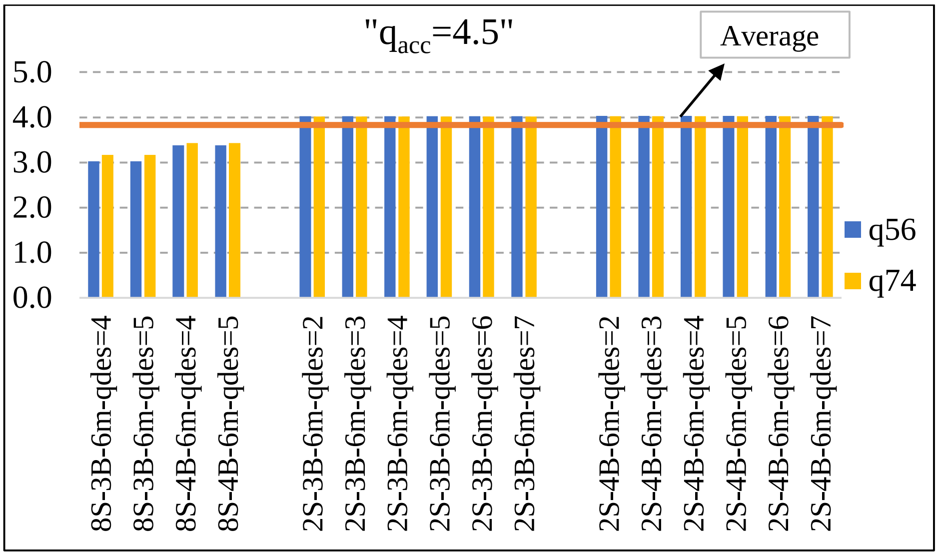


Figure 6-25 The Calculated Behaviour Factor if q_{acc}=4.5

6.1.5.2 Innovative Dissipative FUSEIS Bolted & welded Beam Splices (*Non-Conventional Structures*)

Figure 6-26 shows the value of the behaviour factor calculated for bolted and welded beam splices. As shown in chapter 4, the design value of the behaviour factor for non-conventional structures is considered to be 4.0. The selection of the design behavior factor does not follow the above rule for the non-conventional structures as it regards an innovative system still in experimental phase of investigation. As they are conceived to enhance the ductility of the structure, it is found as not reasonable to test some design criteria for low values of the behavior factors. Considering that the optimal behavior factor for steel frame structures is around 4.0, this target was aimed for the FUSEIS bolted and welded beam splices, as recommended by [38], [49], [87]. However, the value calculated based on the optimal methods (method 56 and 74), given the larger acceptance percentile corresponding to their design behaviour factor, lead to the assessment of a behaviour factor ranging from 4.6 to 5.0 with the average of 4.7. This shows that although the calculated behaviour factor is in the acceptance range of ±20% of q_{des}, and hence shows the accuracy of the given method, a re-design considering q_{des} =4.5 may be required to better estimate of the q-factor of such structures. The larger estimation of the behaviour factor is caused by having the larger ductility of the structure. The larger ductility arises due to permitting the structure to have higher deformability when incorporated by bolted or welded beam splices since by having a small base shear a large displacement may occurred.

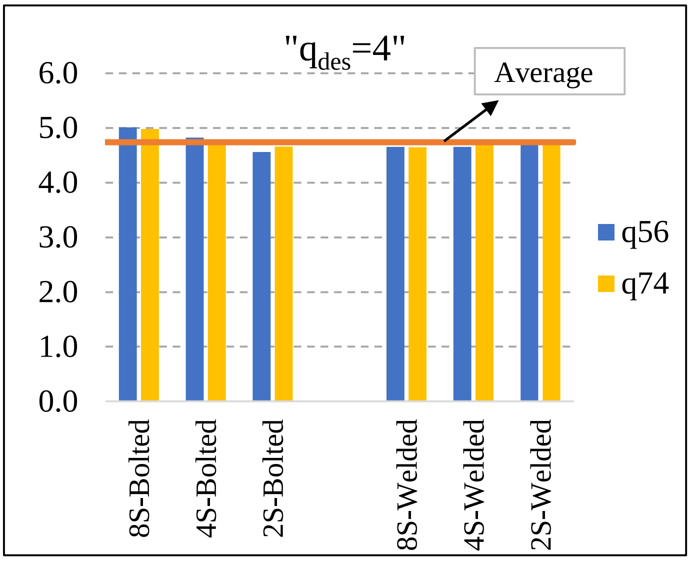


Figure 6-26 The Behaviour Factor Calculated for Bolted and Welded Beam Splices

6.1.6 Behaviour Factor Calculations via Incremental Dynamic Analysis (IDA)

The calculations of the behaviour factor based on P. Setti's method [10] for all case studies using 30 dynamic records explained in chapter 3 are presented in this section. According to [10], the behaviour factor can be estimated as the intersection between the ductility demand curve (obtained by incremental inelastic dynamic analysis) and a straight line (demonstrating the behaviour captured from an elastic dynamic analysis). By this definition the q-factor corresponds to the value beyond which a linear elastic analysis is no longer a safe solution, since the global ductility demand estimated by means of a non-linear analysis is larger than that estimated with a linear analysis (see section 2.4.1, section 3.2.5 and Figure 3-7).

As an example, Figure 6-27 shows the comparison of the 30 elastic-plastic responses obtained by increasing the intensity of the ground acceleration and the expected elastic response for 8S-3B-8m. Figure 6-28 displays the relationship between the response of the structure in terms of maximum roof drifts (θ_{max}), and the intensity of the acceleration for records 12 and 19.

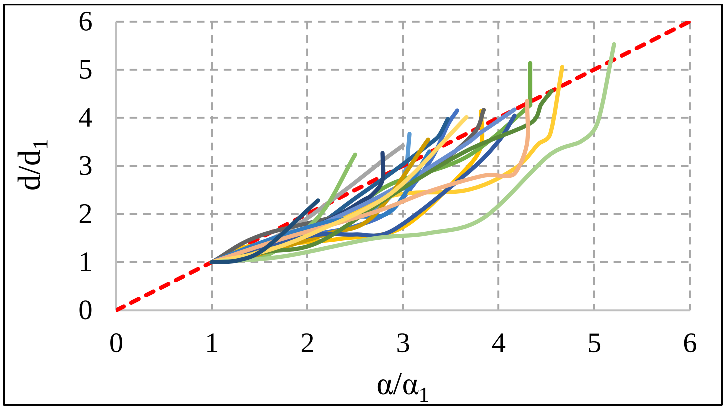


Figure 6-27 Comparison of the 30 Elastic-Plastic Responses Obtained by Increasing the Intensity of the Ground Acceleration and the Expected Elastic Response for 8S-3B-8m

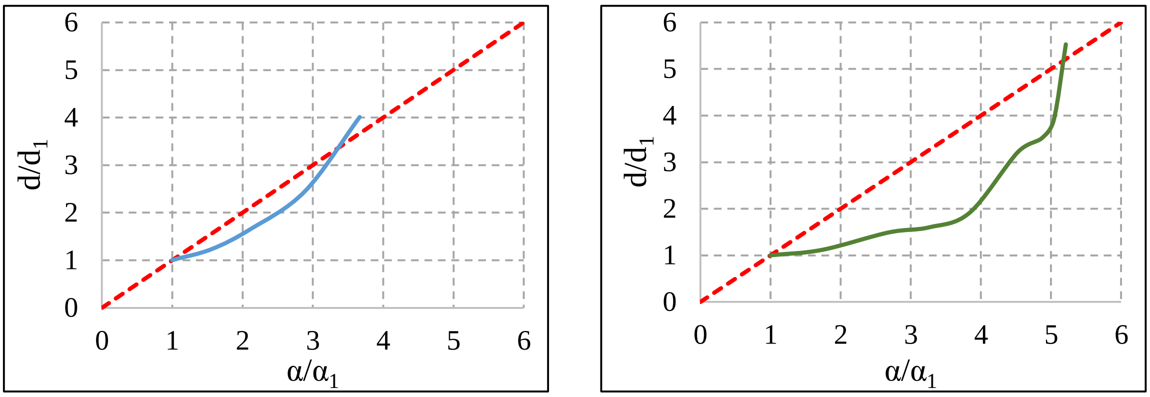
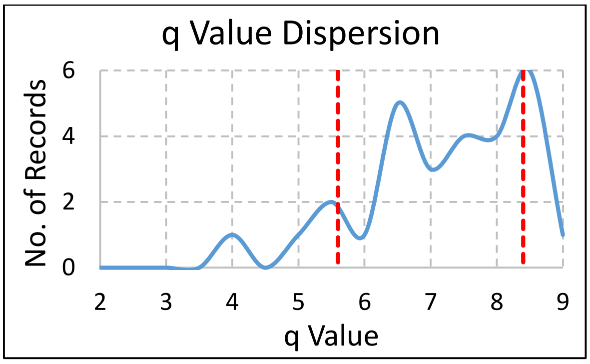


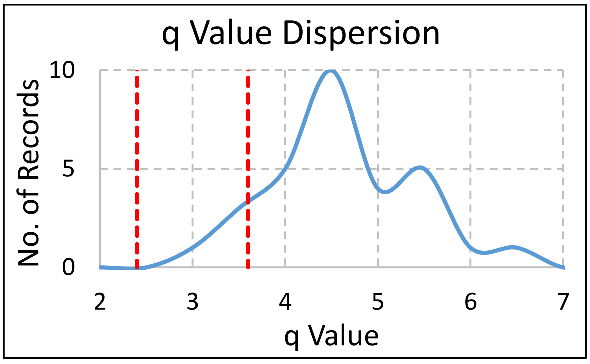
Figure 6-28 The Relationship between the Response of the Structure in Terms of Maximum Roof Drifts, and the Intensity of the Acceleration, (*left*) record 12 (*right*) record 19

6.1.6.1 Typical MRF Buildings (*Conventional Structures*)

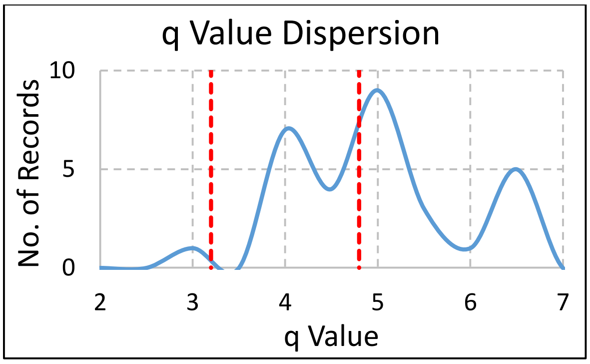
As an example, Figure 6-29 represents the q-value dispersion (obtained by 30 IDAs record) with respect to ±20% of their initial design behaviour factor (the red dashed lines.), for 8S-3B-8m. The full results may be found in APPENDIX-H from Figure 9-40 to Figure 9-55.



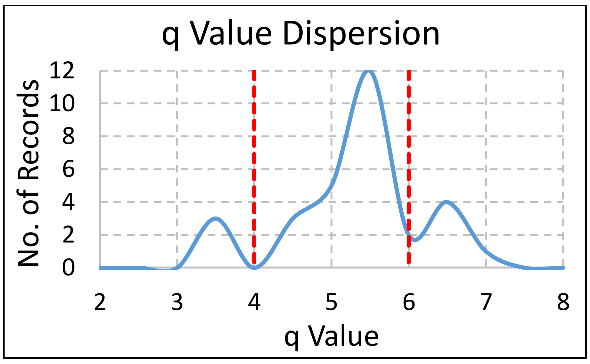
q_{des}=2, q_{acc}=7



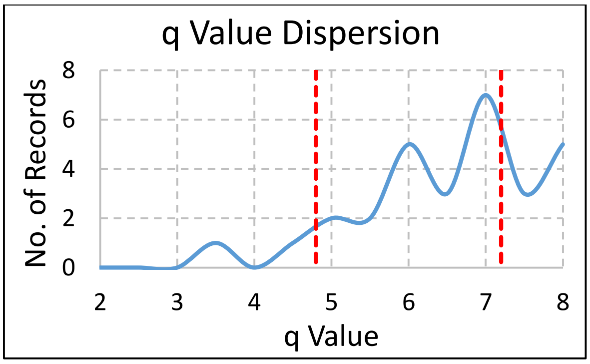
q_{des}=3 q_{acc}=3



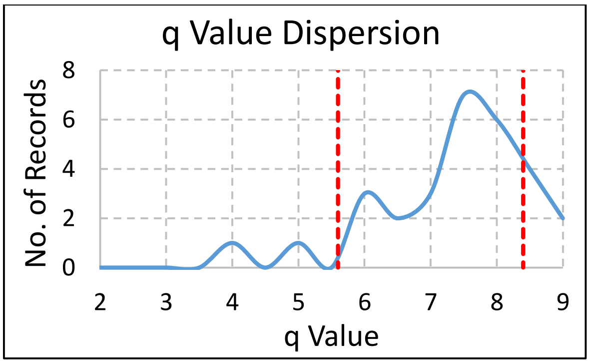
q_{des}=4 q_{acc}=4



q_{des}=5 q_{acc}=5



q_{des}=6 q_{acc}=6



q_{des}=7 q_{acc}=7

Figure 6-29 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% of Their Design Behaviour Factor for 8S-3B-8m

Figure 6-30 to Figure 6-33 display the calculated behaviour factor with respect to the acceptance range of ±20% from the assumed design behaviour factor ($q_{des.}$) and the total number of IDAs records for 12/8/4 and 2-storey buildings. The results show that the Setti's method is accurately able to capture the behaviour factor (77.1% of the total number of records) in which the design proceed (qdes). The exception is for the $q_{des} = 2$ when the design behaviour factor is differed from their acceptance behaviour factor ($q_{des} \neq q_{acc}$). For example, the average calculated behaviour factor for 4S-3/4B-6m and 4/2S-3/4B-8m buildings when designed with (q_{des}=2) are 3.5. This means that if a structure designs with the initial design behaviour factor of 2.0 ($q_{des.}$=2.0) results in a different behaviour factor.

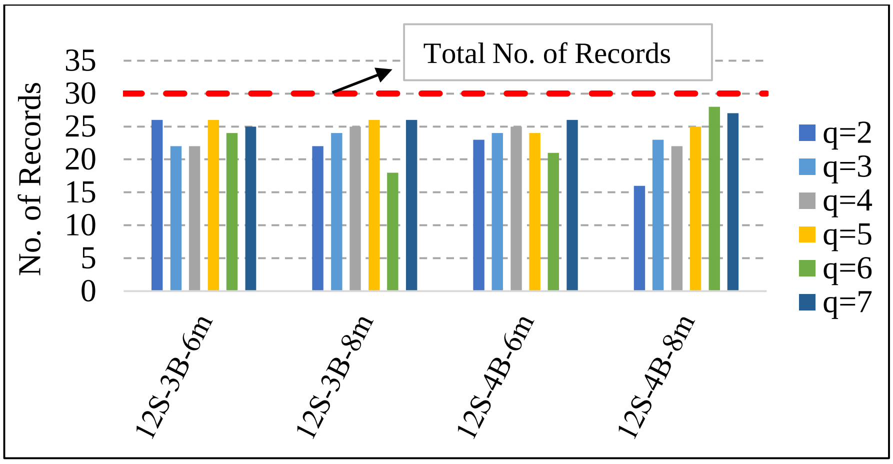


Figure 6-30 The Calculated Behaviour Factor with Respect to the Acceptance Range of ±20% from the Initial Design Behaviour Factor ($q_{des.}$) and the Total Number of IDAs Records for 12-Storey Buildings

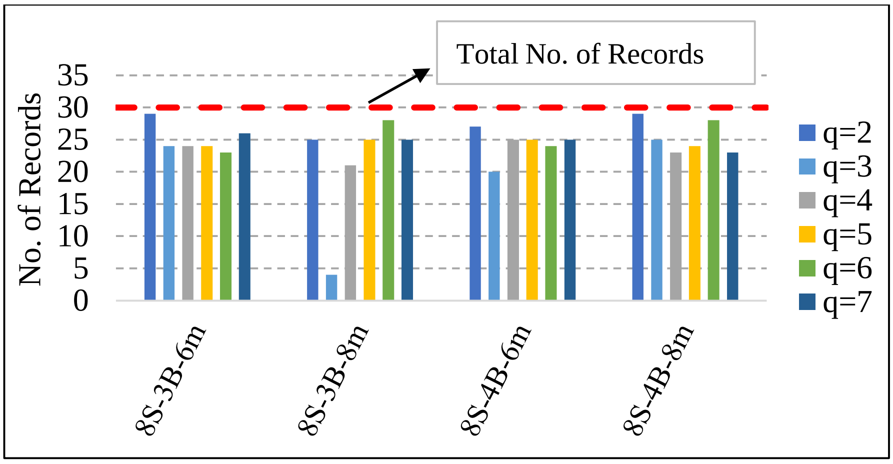


Figure 6-31 The Calculated Behaviour Factor with Respect to the Acceptance Range of ±20% from the Initial Design Behaviour Factor ($q_{des.}$) and the Total Number of IDAs Records for 8-Storey Buildings

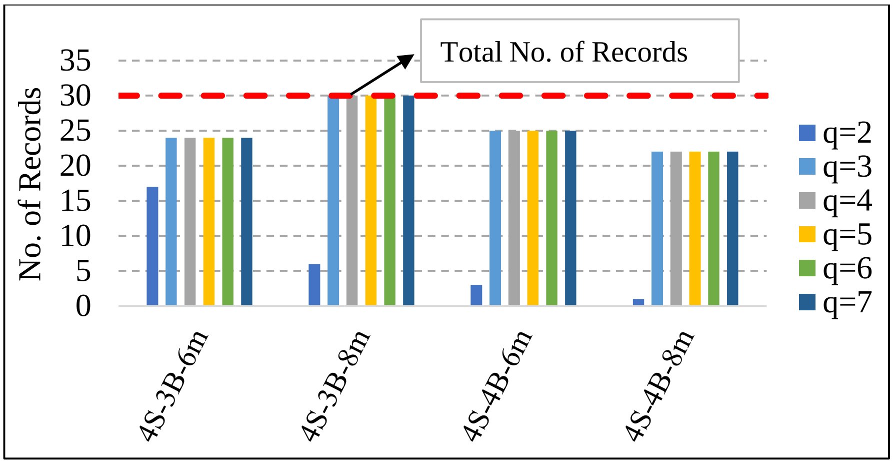


Figure 6-32 The Calculated Behaviour Factor with Respect to the Acceptance Range of ±20% from the Initial Design Behaviour Factor ($q_{des.}$) and the Total Number of IDAs Records for 4-Storey Buildings

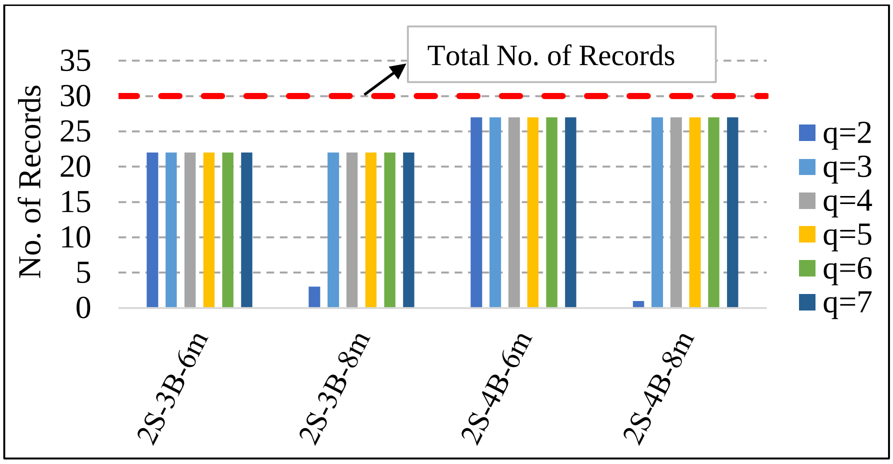
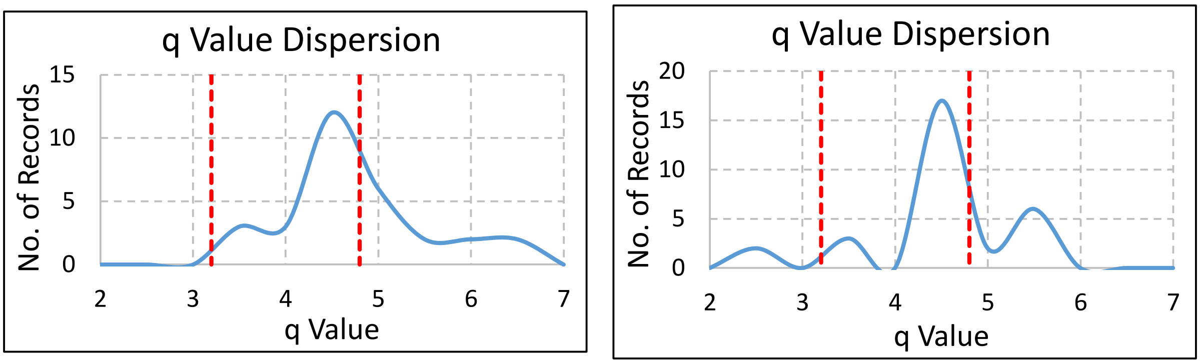


Figure 6-33 The Calculated Behaviour Factor with Respect to the Acceptance Range of ±20% from the Initial Design Behaviour Factor ($q_{des.}$) and the Total Number of IDAs Records for 2-Storey Buildings

6.1.6.2 Innovative Dissipative FUSEIS Bolted & welded Beam Splices (*Non-Conventional Structures*)

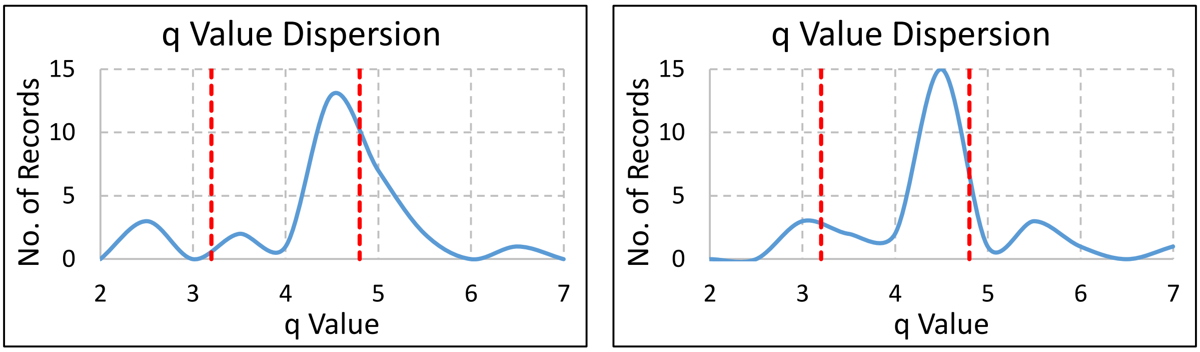
As explained in section 4.1 the design behaviour factor for non-conventional structures are not presented in EN 1998-1:2004. Hence, the design behaviour factor for this type of structure is assumed through the methodology given in [87], [88].

Figure 6-34 to Figure 6-36 display the dispersion of the value of the behaviour factor for bolted and welded beam splices (*non-conventional structures*) for 8/4 and 2S-3B-6m buildings occupied with bolted and welded beam splices. The figures imply that the behaviour factor calculated based on the nonlinear dynamic analysis for bolted and welded beam splices are quite different from what was assumed (design behaviour factor, q_{des}) at first.



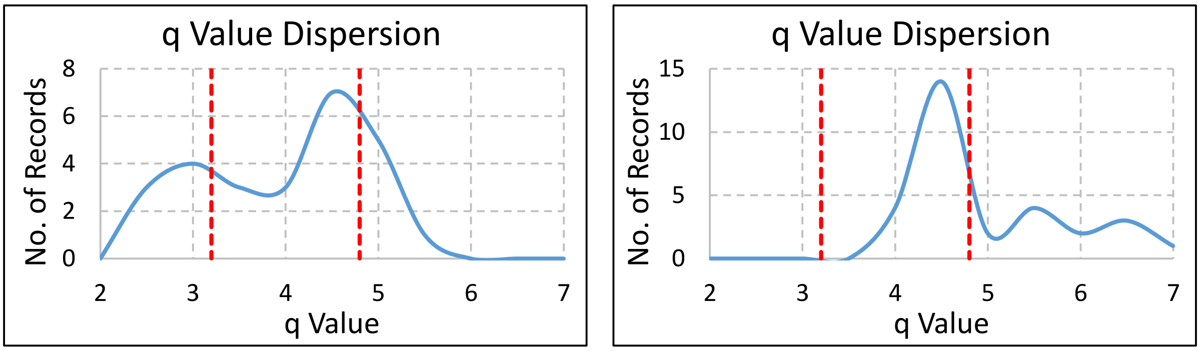
a) b)

Figure 6-34 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% of the Design Behaviour Factor (*q=4*) for 8S-3B-6m a) Bolted Beam Splices b) Welded Beam Splices



a) b)

Figure 6-35 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% of the Design Behaviour Factor (*q=4*) for 4S-3B-6m a) Bolted Beam Splices b) Welded Beam Splices



a) b)

Figure 6-36 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% of the Design Behaviour Factor (*q=4*) for 2S-3B-6m a) Bolted Beam Splices b) Welded Beam Splices

The initial design behaviour factor for the above mentioned structures is assumed to be 4.0 (qdes.=4.0) while the behaviour factor based on the nonlinear dynamic analysis is calculated as 4.50 (see Figure 6-34 to Figure 6-36). Although the initial assumption of the design behaviour factor is not far from the calculated one, a re-design of the structures for this type of buildings, considering qdes=4.5, is required.

Chapter 7

CONCLUDIG REMARKS

7 CONCLUDING REMARKS

The seismic design of structures is a paramount step in the design procedure of new structures in seismic prone areas. The structures are expected to resist seismic loads in terms of two features: (i) material strength of elements and (ii) dissipative properties of the elements, i.e. the ability to exceed the elastic properties through a ductile behavior without exhibiting collapse. As a matter of fact, neither of the above-mentioned features can stand alone against severe earthquakes, hence, a combination of their distribution through careful detailing of the structure is mandatory. This approach, in the structural design procedure, which is well accepted by all seismic design codes, can be satisfied through the so-called behavior factor. Still, the definition of the behavior factor for structures (especially for the newer structures) is a challenging task. On the other hand, to calculate the behaviour factor, many definitions for the reference parameters (i.e., F_1, F_y, d_y and d_m) can be introduced.

In this research, first, all the possible definitions of the reference parameters needed to define and calculate the behaviour factor by means of re-analysis of the pushover curve were presented and discussed. Then, the influence of different choices and combinations of the reference parameters in the assessment of the behaviour factor was shown for 102 case-study buildings (96 conventional and 6 non-conventional structures). The case-studies are assumed to be the steel moment resisting frame systems (*MRF*). The conventional structures are consisted of 2, 4, 8 and 12 storey buildings having 3/4 bay and 6/8m length of the bay. And they are designed with increasing value of the initial design behaviour factor from 2.0 to 7.0 with a gap of 1.0. The non-conventional structures occupied with FUSEIS bolted and welded beam splices are consisted of 2, 4 and 8 storey buildings having 3 bay and 8m length of the bay which are designed with the assumed initial design behaviour factor being equal to 4.0 (q_{des}=4.0).

In particular, the overstrength *"q_Ω"* and the ductility *"q_μ"* factor were identified and the behaviour factor *"q"* was calculated for all case-study buildings. Two main strategies were employed to assess the behavior factor, a) re-analysis of the pushover curves and b) Incremental Dynamic Analysis (*IDA*) using setti's method. The latter approach is generally considered as a more realistic representation of inelastic capacities for the structures, hence, it was used as a benchmark to find the pushover-based *"optimal methods"*.

The results were achieved by performing the nonlinear pushover analysis for each case study and re-analysing the results according to the 90 different possible combinations/methods of the different definitions of the reference parameters such as F_1 (significant yield strength), d_m (maximum horizontal roof displacement) and F_y (strength at the knee point of the idealized bilinear elastic-perfectly plastic curve based on equivalence of the area under the both capacity and bilinear curves). In order to identify which method gives a behaviour factor consistent with the assumed design behaviour factor, the attention was on those methods, giving the results ranging within ±20% with respect to the assumed initial design behaviour factor. Any method that gave the value within the above-mentioned range deemed to be accepted.

As a preliminary result for the conventional structures, it was found that the local plasticity (where the first sacrificial "dissipative" element of the structure enters to the nonlinearity or the occurrence of the first plastic hinge in the structure) as F_1 is more accurately and conveniently able to present the behaviour factor in closest agreement with the initial design behaviour factor rather than other reference parameters such as the global plasticity (where the entire of the structure enters into the nonlinearity). The results highlighted that the acceptance percentile of the obtained behaviour factor, if the local plasticity was used, is more than 88% of the total case studies compared to the combination where the global plasticity (46% of the total case studies) was used. It was also found that the maximum horizontal roof displacement (d_m) corresponding to 15% and 20% loss of structural load carrying capacity, in softening branch is able to lead to a definition of the behaviour factor more accurately than the other definitions for the horizontal roof displacement of the structure. In the same way, to calculate the behaviour factor based on re-analysis of the pushover curve, the best way to introduce F_y is found to be the maximum strength of the structure (F_m).

Hence, if the re-analysis of the response pushover curve is considered, the optimal combinations/methods of the definition of the reference parameters leading to an estimate of the behaviour factor in closest agreement with the design one can be considered as the method *56* and *74.* For these methods the maximum horizontal roof displacement (d_m) is corresponded to be 15% and 20% loss of strength in post hardening branch, respectively, and F_1 and F_y being the first significant yielding and the maximum strength of the structure. However, to calculate the behaviour factor, the results show that there is a very little difference between the method

56 and 74 which is ignorable. However, the *"method 74"* might be the ideal one (the "*Optimal Method*") due to having a less behaviour factor dispersion from the assumed design behaviour factor.

The same methodology is applied to 6 non-conventional structures and the same results has been obtained. In this case, the value of the behaviour factor calculated based on the optimal methods (method 56 and 74), given the larger acceptance percentile corresponding to their design behaviour factor ($q_{des.}$=4.0), lead to the behaviour factor values ranging from 4.6 to 5.0 with the average of 4.7.

Moreover, to identify the suitability of the "*method 74"* which is introduced earlier as the "*Optimal Method*", a comparison is made between the behaviour factor values obtained by means of re-analysis of the pushover response curve and the incremental non-linear dynamic analysis (*IDA*). The results highlighted that the behaviour factor obtained by the IDA method for conventional structures (*typical/ordinary steel MRF*) was able to capture the assumed initial design behaviour factor by more than 77% of the total number of records. With an exception when designing by an initial design behaviour factor being equal to 2.0 in which most of the time receiving the higher behaviour factor for both re-analysis of the pushover curves and IDA method. However, this is not found for the other initial design behaviour factor, i.e. $q_{des.}$=3.0 to 7.0.

The dispersion of the value of the behaviour factor for bolted and welded beam splices (*non-conventional structures*) show that the behaviour factor calculated based on the nonlinear dynamic analysis for bolted and welded beam splices are quite different from what assumed (initial design behaviour factor) at first. The design behaviour factor for the above mentioned structures is assumed as 4.0 ($q_{des.}=4.0$) while the behaviour factor, based on the nonlinear dynamic analysis, is calculated as 4.50. Hence, it seems to be that re-designing of the non-conventional structures are required considering the initial design behaviour factor to be 4.5 which may estimate the behaviour factor more accurately than the one obtained earlier as 83%. Accordingly, considering the above descriptions and to the section 5.1.5 the strength and the stiffness of the structures may be increased if designed by the behaviour factor greater than 4.0 due to involving other design criteria/control such as the damage limitations. Hence, it may result into a stiffer structure and therefore the criteria to accept or reject the design behaviour factor introduced in sections 3.2.4 and 5.2.2 will be automatically satisfied.

Eventually, by comparing the results of re-analysis of the pushover curve and incremental dynamic analysis (*IDA*) it can be concluded the suitability of the "*method 74"* (the "*Optimal Method*") to estimate the behaviour factor more accurately and more confidently than the other specified methods in this research. "*method 74"*, infact is the best method to estimate and determine consistent values of the behaviour factor for steel moment resisting frame (*MRF*) systems based on the pushover curve in closest agreement with the result of IDA.

Corresponding to the results of pushover analysis, the discrepancy between the design behavior factor (expected ductile behavior) and exhibited ductile capacity of the structure, for different structural configurations lead to the next issue of the thesis.

Hence, as a secondary result, this research present a way to make a balance between the initial material (columns, beams, etc.) weight/cost and lifetime seismic damage with acceptable seismic design criteria and construction complexity to identify the optimal seismic design of steel *MRF* systems based on the behaviour factor consideration. As a result, not only a least-weight structural design was achieved while satisfying a comprehensive set of design constraints, but it was also shown that there is the possibility for a considerable saving in designer's time and cost/weight of the building. The results show that as the height of the building increases, the influence of the behaviour factor on the material weight/cost of the structures is more relevant due to involving other factors such as codified design limitations. This is more critical when the number of stories and, consequently the p-Δ effects and the effects of the lateral movements increase. From the results can be concluded that considering the high-rise or tall buildings (8 and 12-Storey buildings), the codified allowable inter-storey drift is controlling the seismic frame design procedure rather than other factors. In other words, increasing the behaviour factor results in changing the limit state governing the design from "*strength*" to "*equilibrium*" or "*stability*" for tall structures. Therefore, from the results once can be concluded, the design of structures for tall and mid-rise buildings is governing by the damage limitations rather than "strength" criteria in capacity design. This means that although the capacity of structures might increase with increasing the behaviour factor, it leads to the overdesigning of the structure and hence increasing the initial material weight/cost.

Whereas, increasing the value of the behaviour factor for lower structures such as 2 and 4-storey, irregardless of the number of bays and length of the bay, has not much influence on the capacity of the structure. Hence, once can be concluded that for the lower structures the design limit stat is predominated by “strength” criteria only.

On the other hand, increasing the number of bays (3 bays to 4 bays) for the same structures having the same number of floors, do not affect the trend of the results, however, it slightly modifies the overall performance of the structures quantitatively.

For instance, the material weight of the structure for 12 storey building designs by q_{des}=7.0 is almost 50% more than the material weight of the same structure designs with q_{des}=4.0. Considering the same behaviour factor for an 8-storey building, the material weigh increases by 30%. It is more evident when the number of bays and the length of the bay increases. On the contrary, reducing the behaviour factor from 3.0 to 2.0 results in increasing the weight by a value between 15.5% and 12% for 12 and 8-storey buildings, respectively. However, decreasing the behaviour factor from 4.0 to 3.0 will not change the results considerably.

Therefore, from an economical point of view, it can be concluded that the optimal design behaviour factor with respect to the initial material weight/cost for steel MRF buildings should fall between 2.5 to 4.5, rather than the maximum value suggested by EN 1998-1: 2004 as 6.5.

The contribution of this thesis is believed to have an important impact in the design practice of steel resisting frame structures. Throughout a wide set of case studies, it was investigated in detail the sensitivity of the design behavior factor into the design process, for what regards the initial weight of the material, and of the exhibited post-elastic capacities of the structures. The prospective of this research is to extend these results to other structures typology and to provide valid recommendations for code amendments.

7.1 Future Work Developments

The research that has been undertaken for this thesis has highlighted a number of topics. However, several areas where further developments is lacking were highlighted in the research methodology and the literature on which further research would be beneficial.

Whilst some of these were addressed by the research in this thesis, others remain. In particular, further study can be done on observational studies of any changes in the frame system i.e. X-braced, eccentric braced system or any other structural type, irregularities of the structural systems (such as torsional irregularities, re-entrant corner, vertical irregularities) and soil interactions and so on.

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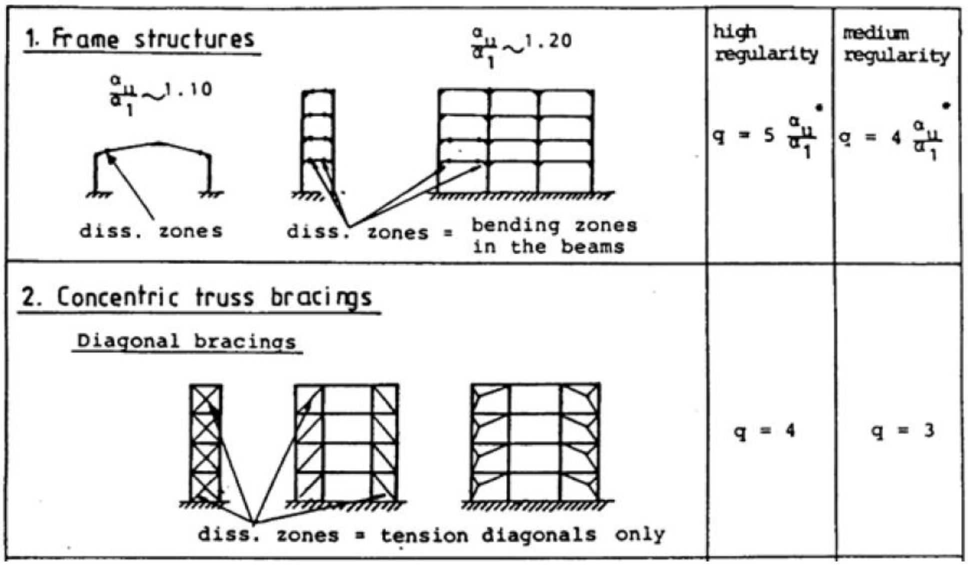
APPENDICES

9 APPENDICES

9.1 APPENDIX-A



a) EN 1988-1-1: 1998

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\,\alpha_u/\alpha_1$
b) Frame with concentric bracings Diagonal bracings V-bracings	 4 2	 4 2,5
c) Frame with eccentric bracings	4	$5\,\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\,\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	See section **5**	
f) Moment resisting frame with concentric bracing	4	$4\,\alpha_u/\alpha_1$
g) Moment resisting frames with infills Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section **7**	
Infills isolated from moment frame (see moment frames)	4	$5\,\alpha_u/\alpha_1$

b) EN1998-1-1: 2004

Figure 9-1 q Factors provided in the First and Last Version of EuroCode

Table 9-1 Behaviour Factor’s Values in Different Seismic Codes Provision for MRFS [6]

Country	Seismic Code	Year	Design Behaviour Factor
Albania	Earthquake Resistant Design Regulations*	1989	5
Algeria	Algerian Earthquake Resistant Regulations	2003	6
Argentina	Argentinean Standards for Earthquake Resistant Constructions (INPRES-CIRSOC 103)	2013	6
Australia	Structural Design Actions (AS1170.4)	2007	8
Bangladesh	Bangladesh National Building Code (BNBC 2006)	2006	12
Bulgaria	Design of Buildings and Facilities in Seismic Regions	2014	5
Canada	National Standard of Canada (CAN/CSA-S16-01)	2007	4
Chile	Chilean Standard of Seismic Design of Buildings (NCh433.Of 96)	2009	7
Colombia	Code for Earthquake Resistant Construction	2010	4.5
Cuba	New Proposal of Cuba Seismic Code*	1995	6
Ethiopia	Ethiopian Building Code Standards (EBCS 14)	2014	3.3
Europe	EuroCode 1998-1	2004	4-6.5
Greece	Greek code for Seismic Resistant Structures (EAK2000)	2000	4
India	Criteria For Earthquake Resistant Design Of Structures (IS 1893)	2002	5
Iran	Code for Seismic Resistance Design of Buildings (2800)	2014	3.5-7.5
Italy	Technical Standards for Construction (NTC 08)	2008	4-6.5
Japan	Seismic Design of Buildings (AIJ)	2000	4
Mexico	Construction Regulations for the Federal District	2003	4
New Zealand	Structural Design Actions (AS/NZS-1170)	2002	3
Philippines	National Structural Code of Philippines (NSCP-2010)	2010	2
Romania	Seismic Design Code P100-1	2013	5
Spain	Actions on Buildings (NBE-AE-88)	2007	4
South Korea	Korean Building Code	2005	3.5
Turkey	Specification for Buildings to be Built in Seismic Zones	2007	5, 8
USA	Uniform Building Code (UBC)	1997	8
	NERPH, FEMA P 750	2009	8
Venezuela	Regulation for Earthquake Resistant Building*	1989	6

*No available information on the revision of the code (IISEE)

Table 9-2 The Set of Thirty Ground Motion Records

No	Record seq.	Event	Year	Station	Mag.	Rrup (km)*
1	2459	Chi-Chi,Taiwan	1999	CHY026	6.2	38.88
2	2703	Chi-Chi, Taiwan-04	1999	CHY028	6.2	17.7
3	2509	Chi-Chi, Taiwan-03	1999	CHY104	6.2	35.05
4	3512	Chi-Chi, Taiwan-06	1999	TCU141	6.3	45.72
5	3320	Chi-Chi, Taiwan-06	1999	CHY111	6.3	68.97
6	322	Coalinga-01	1983	Cantua Creek School	6.36	24.02
7	3266	Chi-Chi, Taiwan-06	1999	CHY026	6.3	50.64
8	1039	Northridge-01	1994	Moorpark - Fire Sta	6.69	24.76
9	1074	Northridge-01	1994	Sandberg - Bald Mtn	6.69	41.56
10	2654	Chi-Chi, Taiwan-03	1999	TCU120	6.2	23.85
11	1159	Kocaeli, Turkey	1999	Eregli	7.51	142.29
12	1208	Chi-Chi, Taiwan	1999	CHY046	7.62	24.01
13	1141	Dinar, Turkey	1995	Dinar	6.4	3.36
14	126	Gazli, USSR	1976	Karakyr	6.8	5.46
15	3290	Chi-Chi, Taiwan-06	1999	CHY060	6.3	96.66
16	1791	Hector Mine	1999	Indio - Coachella Canal	7.13	73.55
17	807	Loma Prieta	1989	Sunol - Forest Fire Station	6.93	47.57
18	836	Landers	1992	Baker Fire Station	7.28	87.94
19	1203	Chi-Chi, Taiwan	1999	CHY036	7.62	16.04
20	179	Imperial Valley-06	1979	El Centro Array #4	6.53	7.05
21	1768	Hector Mine	1999	Barstow	7.13	61.2
22	354	Coalinga-01	1983	Parkfield - Gold Hill 5W	6.36	43.64
23	2111	Denali, Alaska	2002	R109 (temp)	7.9	43.0
24	147	Coyote Lake	1979	Gilroy Array #2	5.74	9.02
25	1259	Chi-Chi, Taiwan	1999	HWA006	7.62	47.86
26	150	Coyote Lake	1979	Gilroy Array #6	5.74	3.11
27	718	Superstition Hills-01	1987	Imperial Valley Wildlife Liquefaction Array	6.22	17.59
28	302	Irpinia, Italy-02	1980	Rionero In Vulture	6.2	22.69
29	921	Big Bear-01	1992	Palm Springs Airport	6.46	52.48
30	184	Imperial Valley-06	1979	El Centro Differential Array	6.53	5.09

9.2 APPENDIX-B

Assigned Columns Sections for Conventional Structures Having Different Design q Factors

Table 9-3 Columns Section for the 12/8/4/2S-3B-6m (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM450	HEB400	HEB450	HEB360	HEB360	HEB260	HEB340	HEB260
2	HEM450	HEB400	HEB400	HEB340	HEB340	HEB260	HEB340	HEB260
3-4	HEB450	HEB360	HEB400	HEB280	HEB340	HEB260	----	----
5-6	HEB400	HEB340	HEB360	HEB260	----	----	----	----
7-8	HEB360	HEB280	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-4 Columns Section for the 12/8/4/2S-3B-6m (q=3)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEB450	HEB300	HEB360	HEB280	HEB340	HEB260	HEB340	HEB260
2	HEB450	HEB300	HEB340	HEB260	HEB340	HEB260	HEB340	HEB260
3-4	HEB400	HEB300	HEB340	HEB260	HEB340	HEB260	----	----
5-6	HEB360	HEB280	HEB340	HEB260	----	----	----	----
7-8	HEB340	HEB260	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-5 Columns Section for the 12/8/4/2S-3B-6m (q=4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEB450	HEB300	HEB340	HEB260	HEB340	HEB260	HEB340	HEB260
3-4	HEB400	HEB300	HEB340	HEB260	HEB340	HEB260	----	----
5-6	HEB360	HEB280	HEB340	HEB260	----	----	----	----
7-8	HEB340	HEB260	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-6 Columns Section for the 12/8/4/2S-3B-6m (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM450	HEB400	HEB340	HEB260	HEB340	HEB260	HEB340	HEB260
3-4	HEB450	HEB360	HEB340	HEB260	HEB340	HEB260	----	----
5-6	HEB400	HEB340	HEB340	HEB260	----	----	----	----
7-8	HEB360	HEB300	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-7 Columns Section for the 12/8/4/2S-3B-6m (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM450	HEM450	HEB400	HEB300	HEB340	HEB260	HEB340	HEB260
3-4	HEM450	HEM450	HEB360	HEB280	HEB340	HEB260	----	----
5-6	HEB450	HEB450	HEB340	HEB260	----	----	----	----
7	HEB400	HEB400	HEB340	HEB260	----	----	----	----
8	HEB400	HEB320	HEB340	HEB260	---	---	---	---
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-8 Columns Section for the 12/8/4/2S-3B-6m (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM800	HEM650	HEB400	HEB300	HEB340	HEB260	HEB340	HEB260
2	HEM800	HEB650	HEB400	HEB300	HEB340	HEB260	HEB340	HEB260
3-4	HEB700	HEB600	HEB400	HEB300	HEB340	HEB260	----	----
5-6	HEB600	HEB500	HEB340	HEB260	----	----	----	----
7-8	HEB450	HEB400	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB340	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-9 Columns Section for the 12/8/4/2S-4B-6m (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM450	HEB450	HEB450	HEB400	HEB360	HEB260	HEB340	HEB260
2	HEM450	HEB450	HEB450	HEB400	HEB340	HEB260	HEB340	HEB260
3-4	HEB450	HEB400	HEB400	HEB360	HEB340	HEB260	----	----
5	HEB400	HEB300	HEB360	HEB280	----	----	----	----
6	HEB400	HEB300	HEB340	HEB260	----	----	----	----
7-8	HEB340	HEB260	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-10 Columns Section for the 12/8/4/2S-4B-6m (q=3)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEB500	HEB400	HEB450	HEB360	HEB340	HEB260	HEB340	HEB260
2	HEB450	HEB400	HEB400	HEB300	HEB340	HEB260	HEB340	HEB260
3-4	HEB400	HEB360	HEB360	HEB280	HEB340	HEB260	----	----
5-6	HEB360	HEB280	HEB340	HEB260	----	----	----	----
7-8	HEB340	HEB260	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-11 Columns Section for the 12/8/4/2S-4B-6m (q=4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEB450	HEB360	HEB360	HEB300	HEB340	HEB260	HEB340	HEB260
3	HEB400	HEB300	HEB360	HEB280	HEB340	HEB260	----	----
4	HEB400	HEB300	HEB340	HEB280	HEB340	HEB260	----	----
5-6	HEB360	HEB280	HEB340	HEB260	----	----	----	----
7-8	HEB340	HEB260	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	---	---	----	----	----	----
11-12	HEB340	HEB260	---	---	----	----	----	----

Table 9-12 Columns Section for the 12/8/4/2S-4B-6m (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEB450	HEB360	HEB360	HEB300	HEB340	HEB260	HEB340	HEB260
3	HEB450	HEB360	HEB360	HEB280	HEB340	HEB260	----	----
4	HEB450	HEB360	HEB340	HEB280	HEB340	HEB260	----	----
5-6	HEB400	HEB340	HEB340	HEB260	----	----	----	----
7-8	HEB360	HEB260	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	----	----	----	----	----	----
11-12	HEB340	HEB260	----	----	----	----	----	----

Table 9-13 Columns Section for the 12/8/4/2S-4B-6m (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM450	HEM450	HEB400	HEB360	HEB340	HEB260	HEB340	HEB260
3-4	HEM450	HEM450	HEB360	HEB300	HEB340	HEB260	----	----
5-6	HEB450	HEB400	HEB340	HEB260	----	----	----	----
7-8	HEB400	HEB320	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB260	----	----	----	----	----	----
11-12	HEB340	HEB260	----	----	----	----	----	----

Table 9-14 Columns Section for the 12/8/4/2S-4B-6m (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM800	HEM550	HEB450	HEB400	HEB340	HEB260	HEB340	HEB260
2	HEM800	HEM550	HEB400	HEB360	HEB340	HEB260	HEB340	HEB260
3	HEB700	HEB500	HEB400	HEB340	HEB340	HEB260	HEB340	HEB260
4	HEB700	HEB500	HEB360	HEB300	HEB340	HEB260	----	----
5-6	HEB600	HEB450	HEB340	HEB260	----	----	----	----
7-8	HEB450	HEB400	HEB340	HEB260	----	----	----	----
9-10	HEB340	HEB340	----	----	----	----	----	----
11-12	HEB340	HEB260	----	----	----	----	----	----

Table 9-15 Columns Section for the 12/8/4/2S-3B-8m (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM650	HEM550	HEM500	HEM450	HEB500	HEB300	HEB400	HEB280
2	HEM650	HEM550	HEM500	HEM450	HEB400	HEB280	HEB360	HEB280
3-4	HEM550	HEM500	HEB500	HEB400	HEB360	HEB280	----	----
5	HEM450	HEB450	HEB450	HEB320	----	----	----	----
6	HEM450	HEB450	HEB400	HEB300	----	----	----	----
7-8	HEB400	HEB360	HEB360	HEB280	----	----	----	----
9-10	HEB360	HEB280	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-16 Columns Section for the 12/8/4/2S-3B-8m (q=3)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM500	HEM450	HEB500	HEB400	HEB360	HEB280	HEB360	HEB280
2	HEM500	HEM450	HEB450	HEB400	HEB360	HEB280	HEB360	HEB280
3	HEM450	HEB400	HEB400	HEB340	HEB360	HEB280	----	----
4	HEB450	HEB400	HEB400	HEB340	HEB360	HEB280	----	----
5-6	HEB400	HEB400	HEB360	HEB340	----	----	----	----
7-8	HEB360	HEB360	HEB360	HEB280	----	----	----	----
9-10	HEB360	HEB280	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-17 Columns Section for the 12/8/4/2S-3B-8m (q=4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM550	HEM450	HEB400	HEB360	HEB360	HEB280	HEB360	HEB280
3-4	HEM500	HEB400	HEB360	HEB320	HEB360	HEB280	----	----
5-6	HEM450	HEB360	HEB360	HEB280	----	----	----	----
7-8	HEB400	HEB340	HEB360	HEB280	----	----	----	----
9-10	HEB360	HEB280	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-18 Columns Section for the 12/8/4/2S-3B-8m (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM700	HEM600	HEB450	HEB360	HEB360	HEB280	HEB360	HEB280
3-4	HEM650	HEM550	HEB400	HEB320	HEB360	HEB280	----	----
5-6	HEM600	HEB500	HEB360	HEB280	----	----	----	----
7-8	HEB500	HEB400	HEB360	HEB280	----	----	----	----
9-10	HEB400	HEB360	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-19 Columns Section for the 12/8/4/2S-3B-8m (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM1000	HEM1000	HEM450	HEB450	HEB360	HEB280	HEB360	HEB280
3-4	HEM900	HEM900	HEB450	HEB400	HEB360	HEB280	----	----
5	HEM800	HEM800	HEB360	HEB320	----	----	----	----
6	HEM800	HEM600	HEB360	HEB320	----	----	----	----
7-8	HEM700	HEM500	HEB360	HEB280	----	----	----	----
9-10	HEB500	HEB400	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-20 Columns Section for the 12/8/4/2S-3B-8m (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	2HEM1000	2HEM1000	HEM500	HEB500	HEB360	HEB280	HEB360	HEB280
3	2HEM900	2HEM900	HEB500	HEB500	HEB360	HEB280	----	----
4-5	2HEM900	2HEM900	HEB500	HEB450	HEB360	HEB280	----	----
6	2HEM800	2HEM700	HEB400	HEB400	----	----	----	----
7	2HEM600	2HEM550	HEB360	HEB280	----	----	----	----
8	2HEM550	HEM550	HEB360	HEB280	----	----	----	----
9	HEM550	HEM550	---	---	----	----	----	----
10	HEB450	HEB450	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-21 Columns Section for the 12/8/4/2S-4B-8m (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM650	HEM550	HEM500	HEM400	HEB500	HEB300	HEB400	HEB280
2	HEM650	HEM550	HEM500	HEM400	HEB400	HEB280	HEB360	HEB280
3-4	HEM550	HEM500	HEB500	HEB400	HEB360	HEB280	----	----
5	HEM450	HEB450	HEB450	HEB300	----	----	----	----
6	HEM450	HEB450	HEB400	HEB280	----	----	----	----
7-8	HEB400	HEB360	HEB360	HEB280	----	----	----	----
9-10	HEB360	HEB280	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-22 Columns Section for the 12/8/4/2S-4B-8m (q=3)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM550	HEM450	HEM400	HEB360	HEB360	HEB280	HEB360	HEB280
3-4	HEM450	HEB450	HEB400	HEB300	HEB360	HEB280	----	----
5-6	HEB450	HEB400	HEB360	HEB280	----	----	----	----
7-8	HEB400	HEB340	HEB360	HEB280	----	----	----	----
9-10	HEB360	HEB280	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-23 Columns Section for the 12/8/4/2S-4B-8m (q=4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1	HEM550	HEB450	HEB450	HEB300	HEB360	HEB280	HEB360	HEB280
2	HEM550	HEB450	HEB400	HEB300	HEB360	HEB280	HEB360	HEB280
3-4	HEM500	HEB400	HEB400	HEB300	HEB360	HEB280	----	----
5-6	HEM400	HEB360	HEB360	HEB280	----	----	----	----
7-8	HEB400	HEB300	HEB360	HEB280	----	----	----	----
9-10	HEB360	HEB280	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-24 Columns Section for the 12/8/4/2S-4B-8m (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM700	HEM600	HEB450	HEB360	HEB360	HEB280	HEB360	HEB280
3-4	HEM650	HEM550	HEB400	HEB300	HEB360	HEB280	----	----
5-6	HEM600	HEB500	HEB360	HEB280	----	----	----	----
7-8	HEB500	HEB400	HEB360	HEB280	----	----	----	----
9-10	HEB400	HEB360	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-25 Columns Section for the 12/8/4/2S-4B-8m (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	HEM1000	HEM1000	HEM450	HEB450	HEB360	HEB280	HEB360	HEB280
3-4	HEM900	HEM900	HEB450	HEB400	HEB360	HEB280	----	----
5	HEM800	HEM800	HEB360	HEB320	----	----	----	----
6	HEM800	HEM600	HEB360	HEB320	----	----	----	----
7-8	HEM700	HEM500	HEB360	HEB280	----	----	----	----
9-10	HEB500	HEB400	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

Table 9-26 Columns Section for the 12/8/4/2S-4B-8m (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Storey	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
1-2	2HEM1000	2HEM1000	HEM500	HEB500	HEB360	HEB280	HEB360	HEB280
3-4	2HEM900	2HEM900	HEB500	HEB500	HEB360	HEB280	----	----
5	2HEM900	2HEM900	HEB400	HEB400	----	----	----	----
6	2HEM800	2HEM700	HEB400	HEB400	----	----	----	----
7	2HEM600	2HEM550	HEB360	HEB280	----	----	----	----
8	2HEM550	HEM550	HEB360	HEB280	----	----	----	----
9	HEM550	HEM550	---	---	----	----	----	----
10	HEB450	HEB450	---	---	----	----	----	----
11-12	HEB360	HEB280	---	---	----	----	----	----

9.3 APPENDIX-C

Period of Vibration and Mass Participations for Conventional Structures

Table 9-27 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 6m Length of the Bay (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.282	75.8	1.523	79.1	0.829	85.8	0.448	92.1
II	0.796	12.4	0.524	11.3	0.278	9.9	0.161	7.4
III	0.464	4.2	0.307	3.8	0.172	3.1	0.043	0.0
SUM	----	92.4	----	94.2	----	98.8	----	99.5

Table 9-28 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 6m Length of the Bay (q=3, 4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.498	77.9	1.722	82.2	0.842	86.8	0.448	92.1
II	0.848	12.0	0.57	10.0	0.282	9.6	0.161	7.4
III	0.498	3.9	0.336	3.5	0.174	2.7	0.043	0.0
SUM	----	93.8	----	95.7	----	99.1	----	99.5

Table 9-29 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 6m Length of the Bay (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.273	76.004	1.745	83.2	0.842	86.8	0.448	92.1
II	0.793	12.159	0.578	10	0.282	9.6	0.161	7.4
III	0.462	4.241	0.34	3.4	0.174	2.7	0.043	0.0
SUM	----	92.404	----	96.6	----	99.1	----	99.5

Table 9-30 period of vibration and mass participation for Buildings Having 3-Bay and 6m Length of the Bay (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.091	75.1	1.616	80.3	0.842	86.8	0.448	92.1
II	0.759	12.7	0.547	11.2	0.282	9.6	0.161	7.4
III	0.440	4.4	0.322	3.9	0.174	2.7	0.043	0.0
SUM	----	92.2	----	95.4	----	99.1	----	99.5

Table 9-31 period of vibration and mass participation for Buildings Having 3-Bay and 6m Length of the Bay (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	1.909	73.1	1.576	80.5	0.842	86.8	0.448	92.1
II	0.691	12.5	0.544	11.5	0.282	9.6	0.161	7.4
III	0.399	4.4	0.316	3.4	0.174	2.7	0.043	0.0
SUM	----	90.0	----	95.4	----	99.1	----	99.5

Table 9-32 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 8m Length of the Bay (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.459	74.4	1.604	76.9	0.942	86.6	0.448	92.1
II	0.892	12.7	0.571	11.5	0.318	9.6	0.161	7.4
III	0.516	4.3	0.332	4.4	0.194	2.9	0.043	0.0
SUM	----	91.3	----	92.9	----	99.1	----	99.5

Table 9-33 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 8m Length of the Bay (q=3)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.612	76.2	1.801	78.4	1.041	81.6	0.448	92.1
II	0.928	11.8	0.620	11.5	0.346	11.2	0.161	7.4
III	0.541	4.3	0.365	4.0	0.211	3.9	0.043	0.0
SUM	----	92.3	----	93.8	----	96.7	----	99.5

Table 9-34 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 8m Length of the Bay (q=4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.596	76.2	1.911	80.7	1.041	81.6	0.448	92.1
II	0.929	12.2	0.646	10.7	0.346	11.2	0.161	7.4
III	0.537	4.0	0.382	3.9	0.211	3.9	0.043	0.0
SUM	----	92.5	----	95.3	----	96.7	----	99.5

Table 9-35 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 8m Length of the Bay (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.327	75.6	1.841	79.6	1.041	81.6	0.448	92.1
II	0.837	11.6	0.632	11.3	0.346	11.2	0.161	7.4
III	0.488	4.1	0.372	3.9	0.211	3.9	0.043	0.0
SUM	----	91.4	----	94.8	----	96.7	----	99.5

Table 9-36 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 8m Length of the Bay (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.099	74.6	1.693	77.5	1.041	81.6	0.448	92.1
II	0.752	10.6	0.593	12.0	0.346	11.2	0.161	7.4
III	0.436	4.5	0.349	4.1	0.211	3.9	0.043	0.0
SUM	----	89.8	----	93.5	----	96.7	----	99.5

Table 9-37 Period of Vibration and Mass Participation for Buildings Having 3-Bay and 8m Length of the Bay (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	1.945	72.4	1.593	77.6	1.041	81.6	0.448	92.1
II	0.707	10.4	0.563	11.4	0.346	11.2	0.161	7.4
III	0.402	5.2	0.330	4.3	0.211	3.9	0.043	0.0
SUM	----	88.1	----	93.4	----	96.7	----	99.5

Table 9-38 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 6m Length of the Bay (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.270	75.2	1.469	78.5	0.825	85.8	0.448	92.1
II	0.792	12.6	0.514	11.8	0.277	9.9	0.161	7.4
III	0.468	4.1	0.299	4.0	0.172	3.2	0.043	0.0
SUM	----	92.0	----	94.2	----	98.9	----	99.5

Table 9-39 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 6m Length of the Bay (q=3)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.400	77.2	1.573	79.0	0.840	86.8	0.448	92.1
II	0.822	11.9	0.532	10.9	0.282	9.5	0.161	7.4
III	0.488	4.1	0.314	4.0	0.174	2.7	0.043	0.0
SUM	----	93.2	----	93.9	----	99.1	----	99.5

Table 9-40 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 6m Length of the Bay (q=4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.413	78.1	1.648	81.8	0.840	86.8	0.448	92.1
II	0.822	11.2	0.555	10.6	0.282	9.5	0.161	7.4
III	0.487	4.2	0.328	3.6	0.174	2.7	0.043	0.0
SUM	----	93.5	----	96.0	----	99.1	----	99.5

Table 9-41 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 6m Length of the Bay (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.317	78.0	1.648	81.8	0.840	86.8	0.448	92.1
II	0.813	12.1	0.555	10.6	0.282	9.5	0.161	7.4
III	0.475	3.8	0.328	3.6	0.174	2.7	0.043	0.0
SUM	----	93.8	----	96.0	----	99.1	----	99.5

Table 9-42 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 6m Length of the Bay (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.088	75.6	1.568	79.7	0.840	86.8	0.448	92.1
II	0.757	12.4	0.534	11.2	0.282	9.5	0.161	7.4
III	0.441	4.4	0.315	4.0	0.174	2.7	0.043	0.0
SUM	----	92.4	----	94.9	----	99.1	----	99.5

Table 9-43 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 6m Length of the Bay (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	1.910	73.6	1.523	78.4	0.840	86.8	0.448	92.1
II	0.696	12.3	0.523	11.7	0.282	9.5	0.161	7.4
III	0.402	4.4	0.310	3.9	0.174	2.7	0.043	0.0
SUM	----	90.3	----	94.0	----	99.1	----	99.5

Table 9-44 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 8m Length of the Bay (q=2)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.457	74.6	1.616	77.4	0.934	81.3	0.448	92.1
II	0.892	12.6	0.575	11.5	0.316	11.3	0.161	7.4
III	0.518	4.2	0.334	4.3	0.193	3.9	0.043	0.0
SUM	----	91.4	----	93.2	----	96.5	----	99.5

Table 9-45 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 8m Length of the Bay (q=3, 4)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.588	75.8	1.817	78.6	1.041	86.6	0.448	92.1
II	0.920	11.8	0.621	11.3	0.346	9.6	0.161	7.4
III	0.540	4.4	0.368	4.3	0.211	2.9	0.043	0.0
SUM	----	92.0	----	94.2	----	99.1	----	99.5

Table 9-46 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 8m Length of the Bay (q=5)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.320	75.9	1.845	79.7	1.041	86.6	0.448	92.1
II	0.838	11.5	0.631	11.1	0.346	9.6	0.161	7.4
III	0.488	4.1	0.373	4.0	0.211	2.9	0.043	0.0
SUM	----	91.5	----	94.8	----	99.1	----	99.5

Table 9-47 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 8m Length of the Bay (q=6)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.320	75.9	1.845	79.7	1.041	86.6	0.448	92.1
II	0.838	11.5	0.631	11.1	0.346	9.6	0.161	7.4
III	0.488	4.1	0.373	4.0	0.211	2.9	0.043	0.0
SUM	----	91.5	----	94.8	----	99.1	----	99.5

Table 9-48 Period of Vibration and Mass Participation for Buildings Having 4-Bay and 8m Length of the Bay (q=7)

	12-Storey		8-Storey		4-Storey		2-Storey	
Mode	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)	T (Sec.)	Mass part. (%)
I	2.097	75.0	1.697	77.5	1.041	86.6	0.448	92.1
II	0.753	10.5	0.594	11.9	0.346	9.6	0.161	7.4
III	0.437	4.5	0.350	4.0	0.211	2.9	0.043	0.0
SUM	----	90.0	----	93.5	----	99.1	----	99.5

9.4 APPENDIX-D-1

Sap 2000 and *OpenSees* Comparison in Terms of Nonlinear Static Analysis for *Conventional* Structures

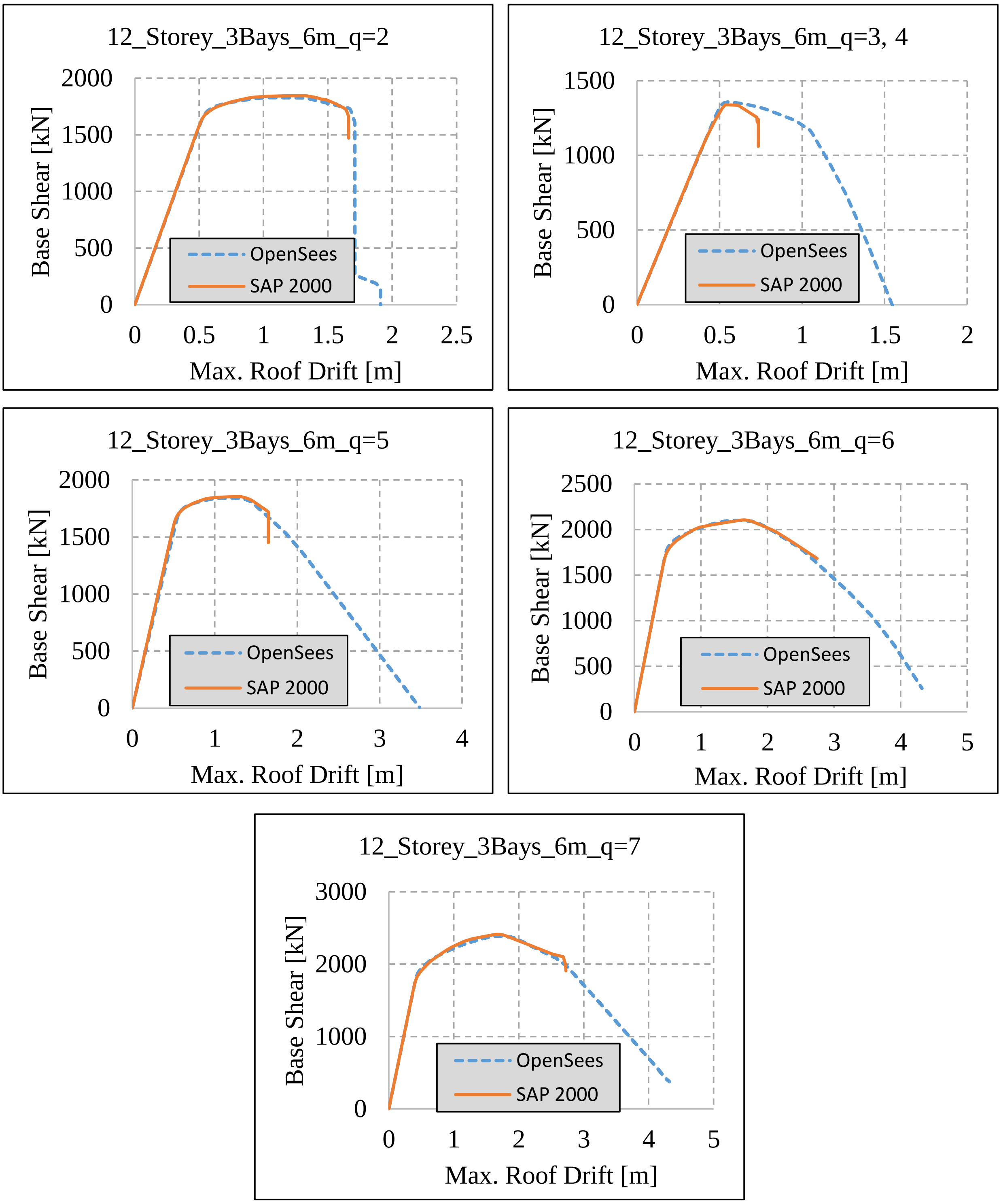


Figure 9-2 Sap 2000 and OpenSees Comparison for 12S-3B-6m

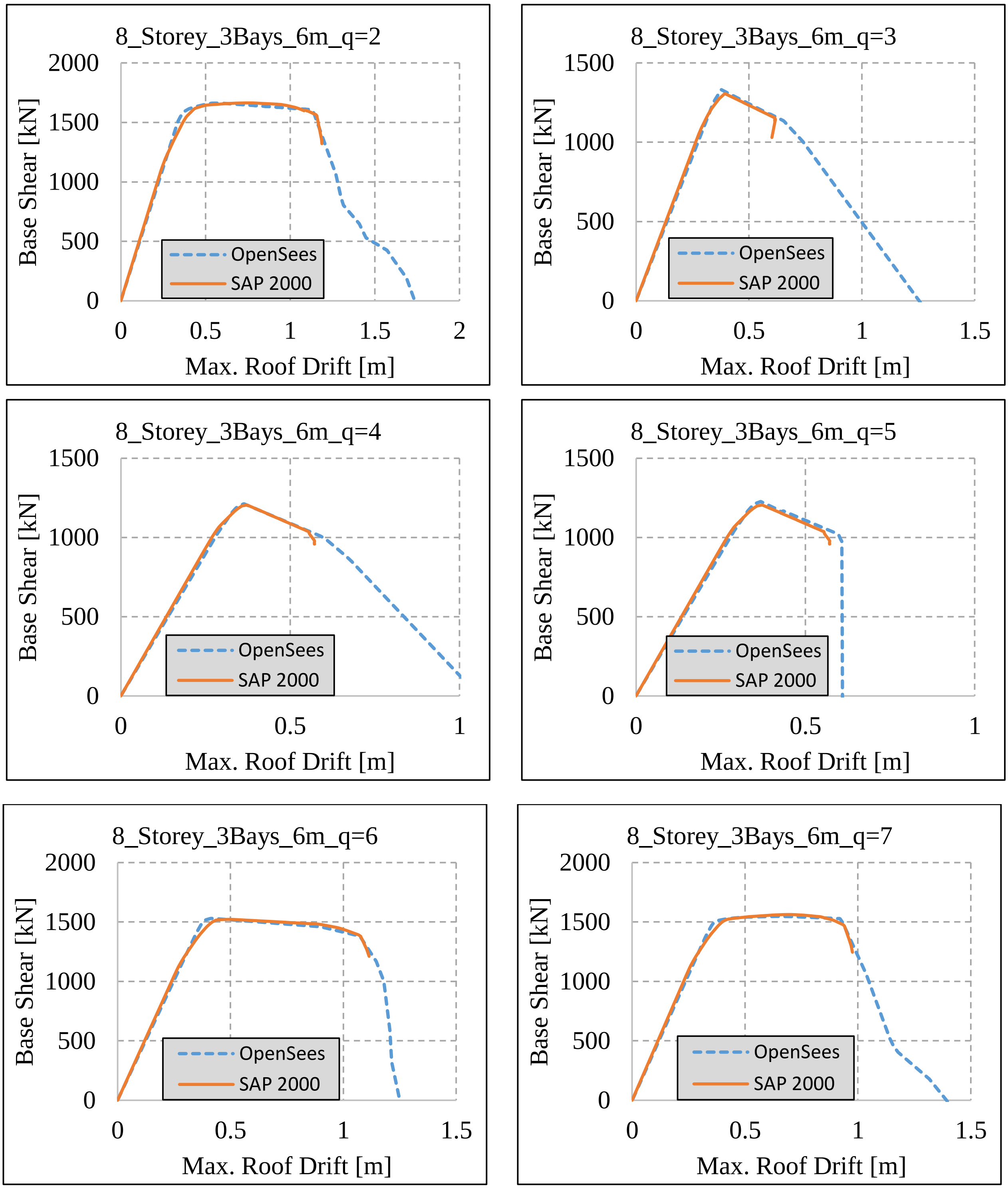


Figure 9-3 Sap 2000 and OpenSees Comparison for 8S-3B-6m

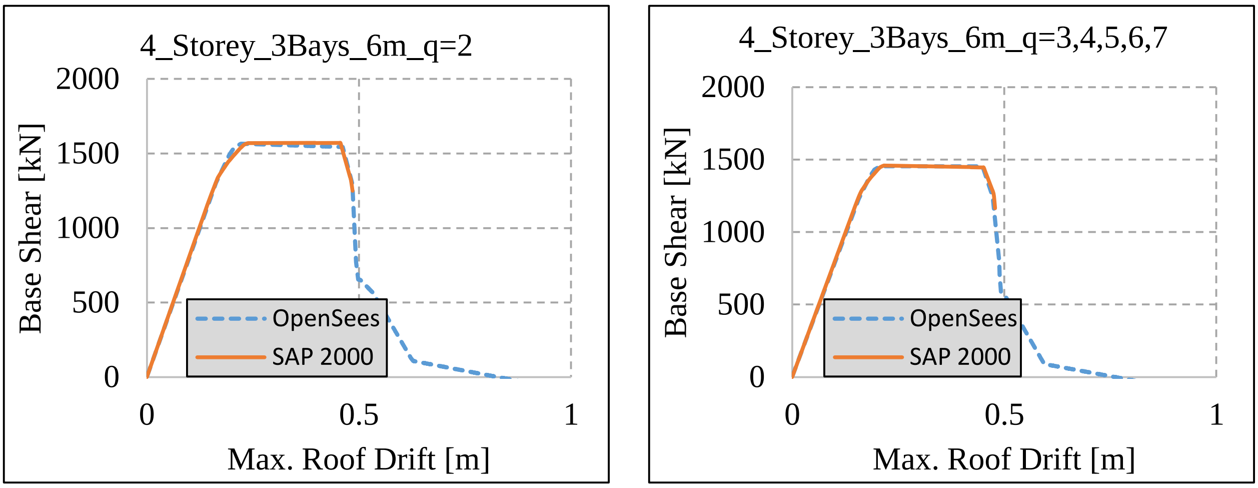


Figure 9-4 Sap 2000 and OpenSees Comparison for 4S-3B-6m

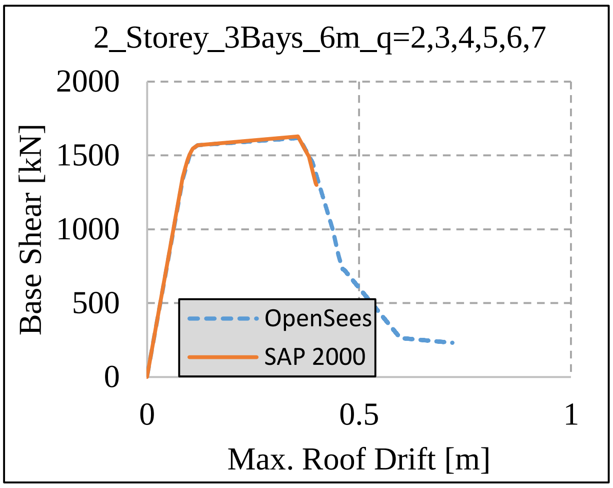


Figure 9-5 Sap 2000 and OpenSees Comparison for 2S-3B-6m

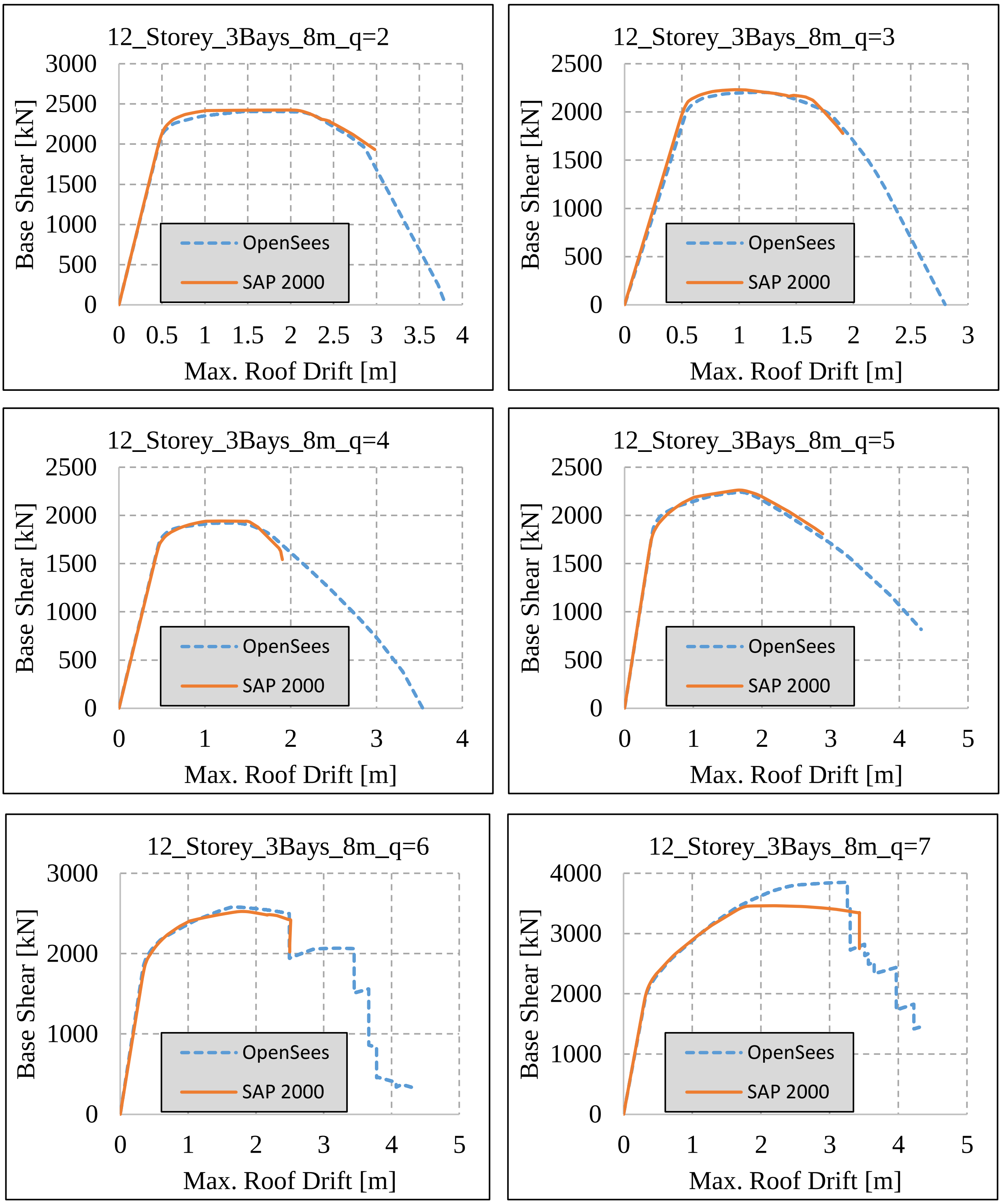


Figure 9-6 Sap 2000 and OpenSees Comparison for 12S-3B-8m

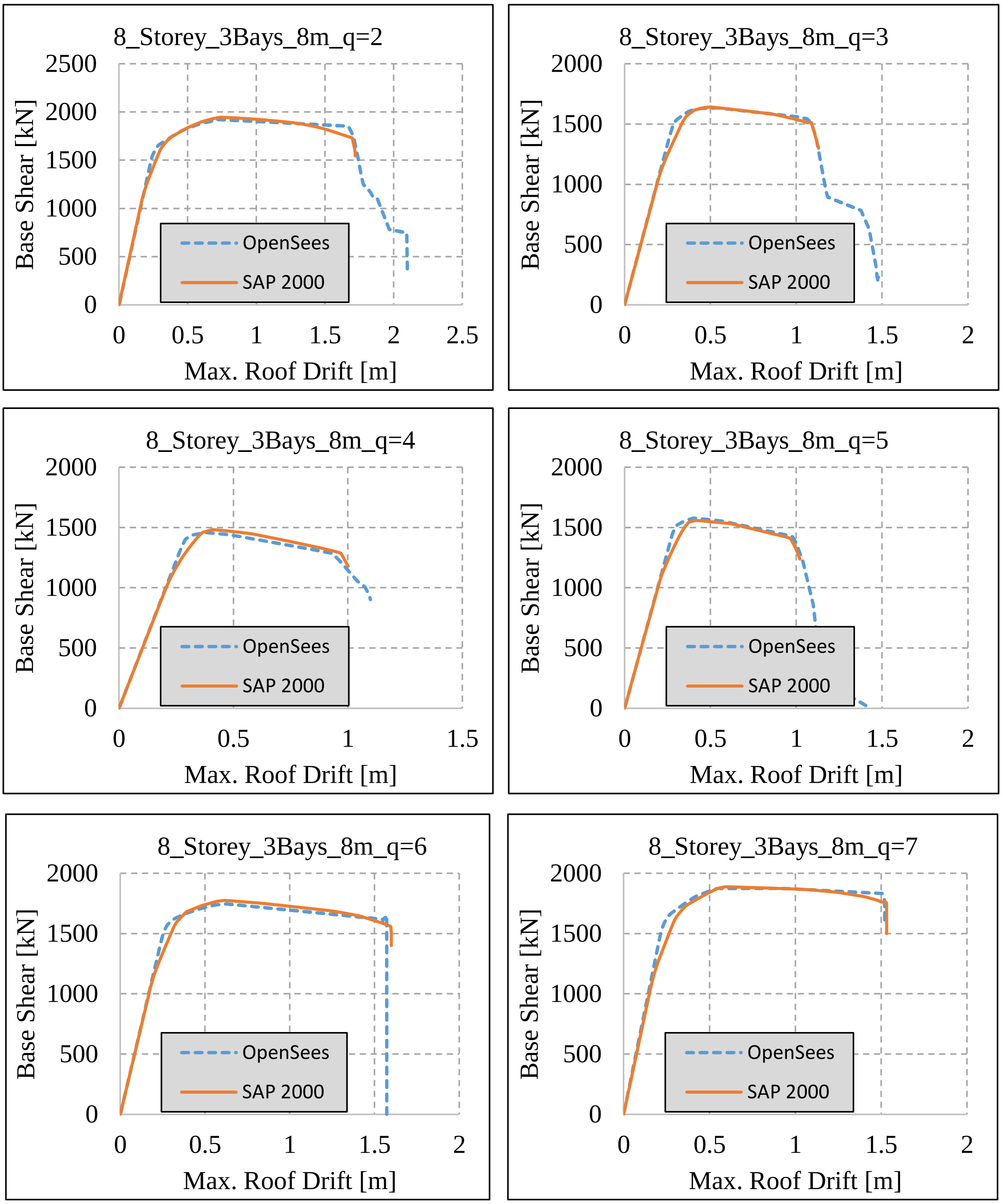


Figure 9-7 Sap 2000 and OpenSees Comparison for 8S-3B-8m

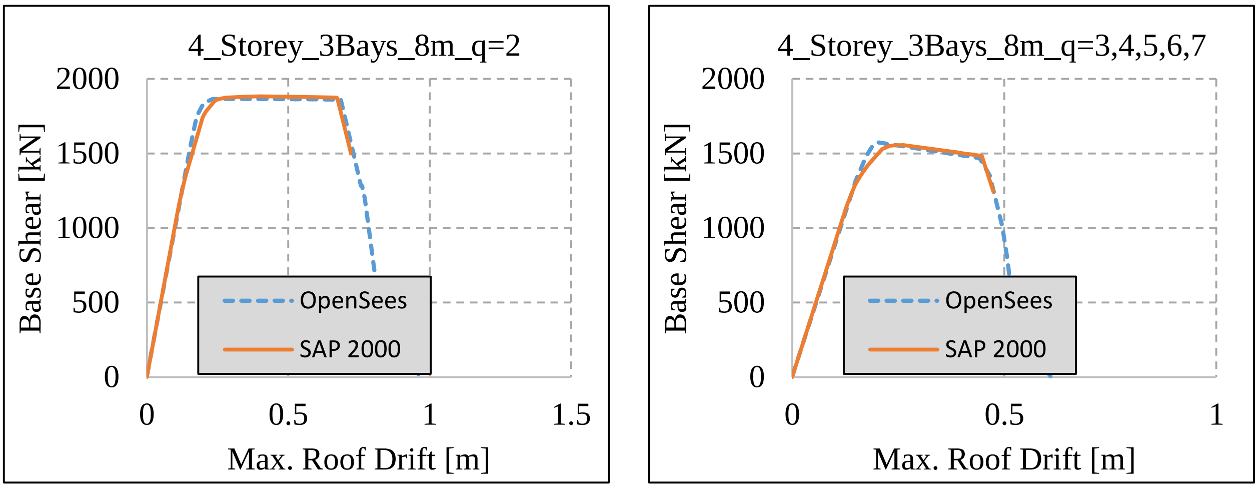


Figure 9-8 Sap 2000 and OpenSees Comparison for 4S-3B-8m

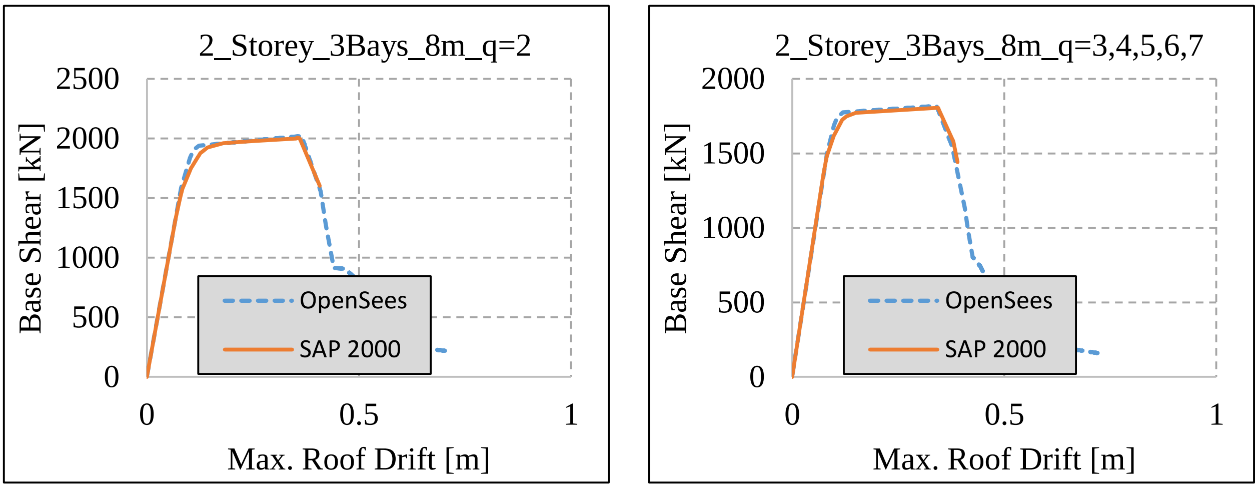


Figure 9-9 Sap 2000 and OpenSees Comparison for 2S-3B-8m

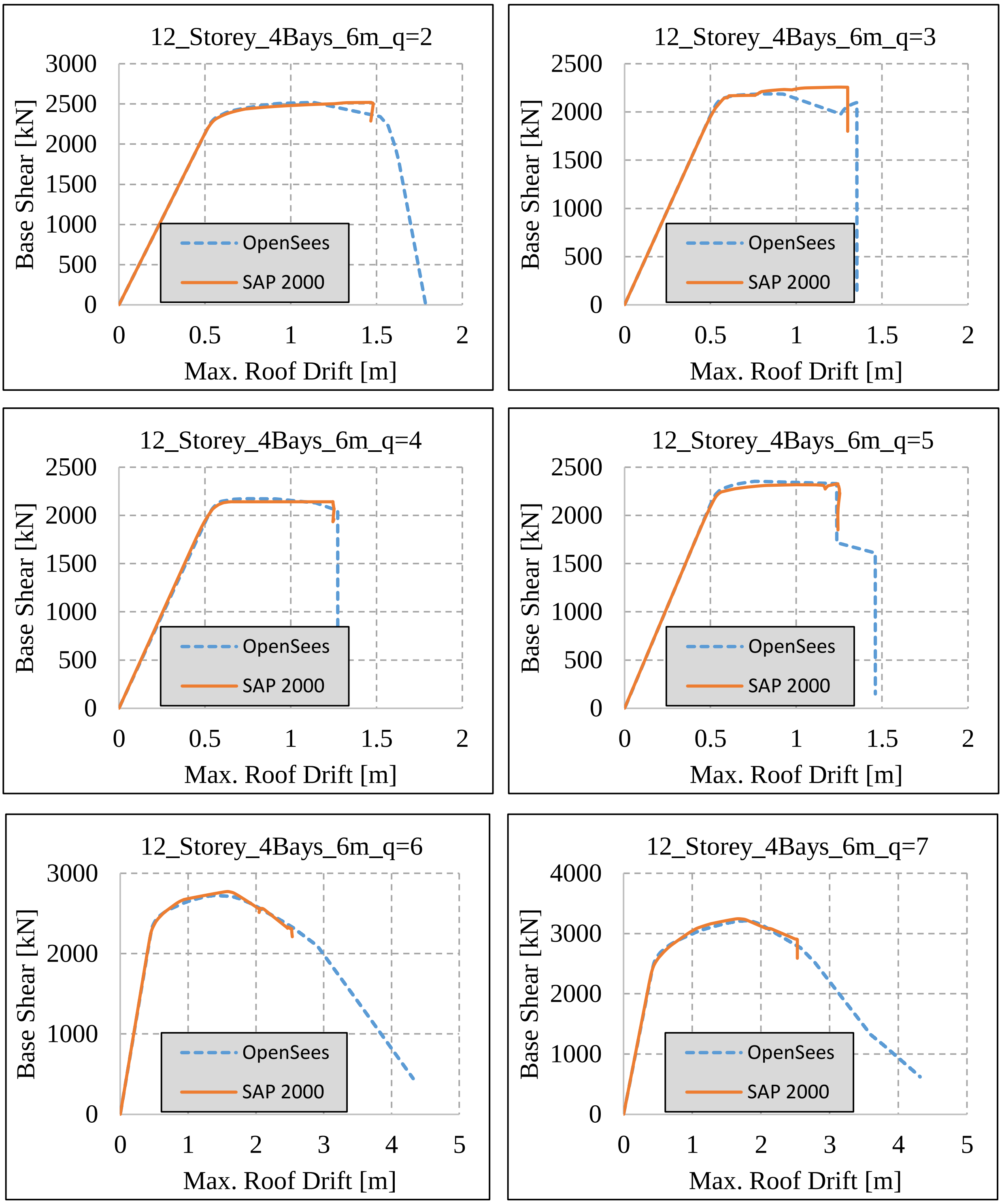


Figure 9-10 Sap 2000 and OpenSees Comparison for 12S-4B-6m

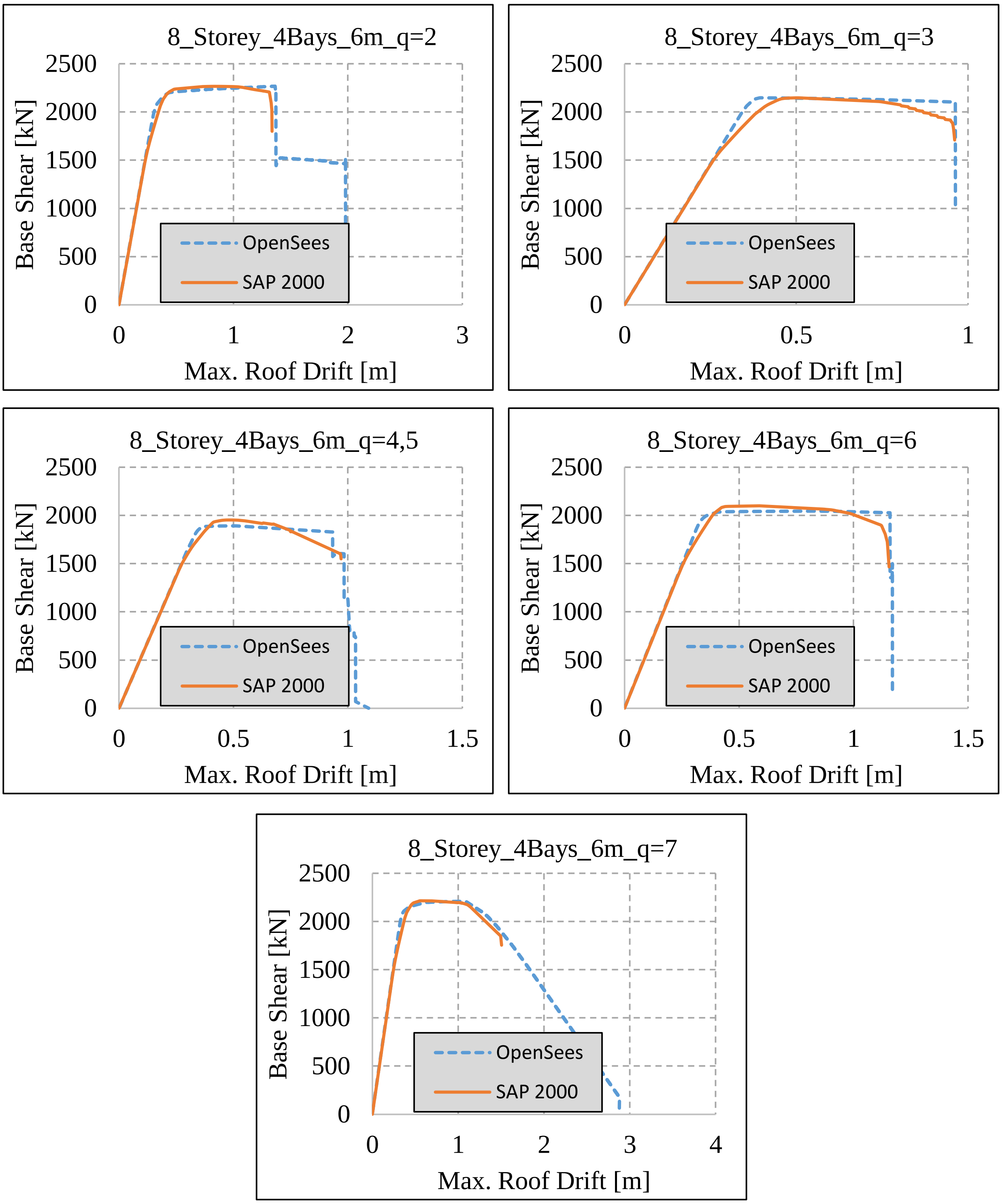
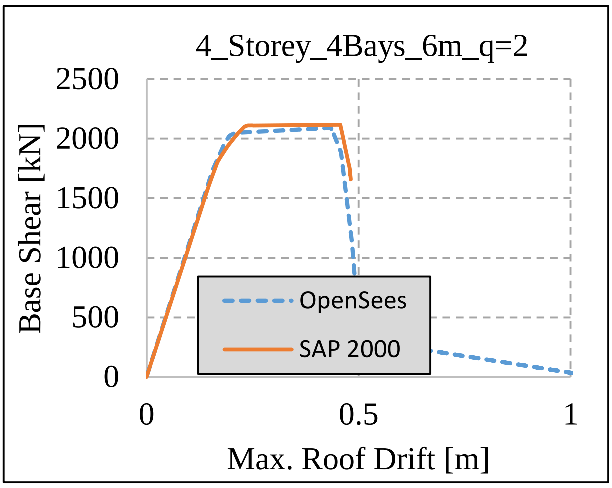


Figure 9-11 Sap 2000 and OpenSees Comparison for 8S-4B-6m



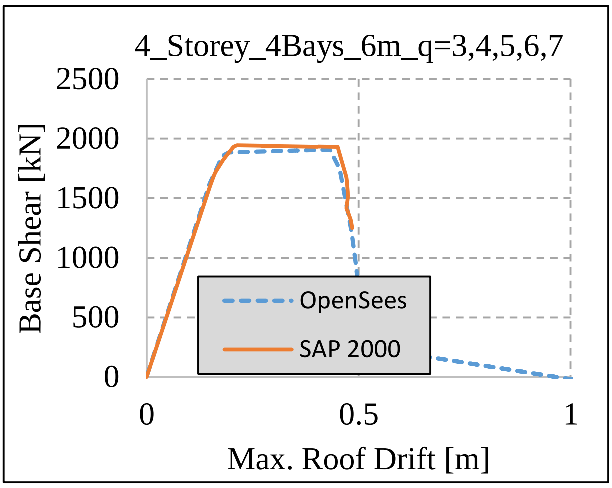


Figure 9-12 Sap 2000 and OpenSees Comparison for 4S-4B-6m

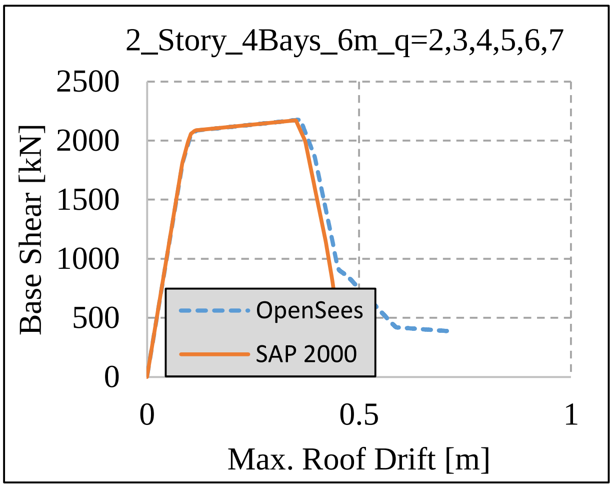


Figure 9-13 Sap 2000 and OpenSees Comparison for 2S-4B-6m

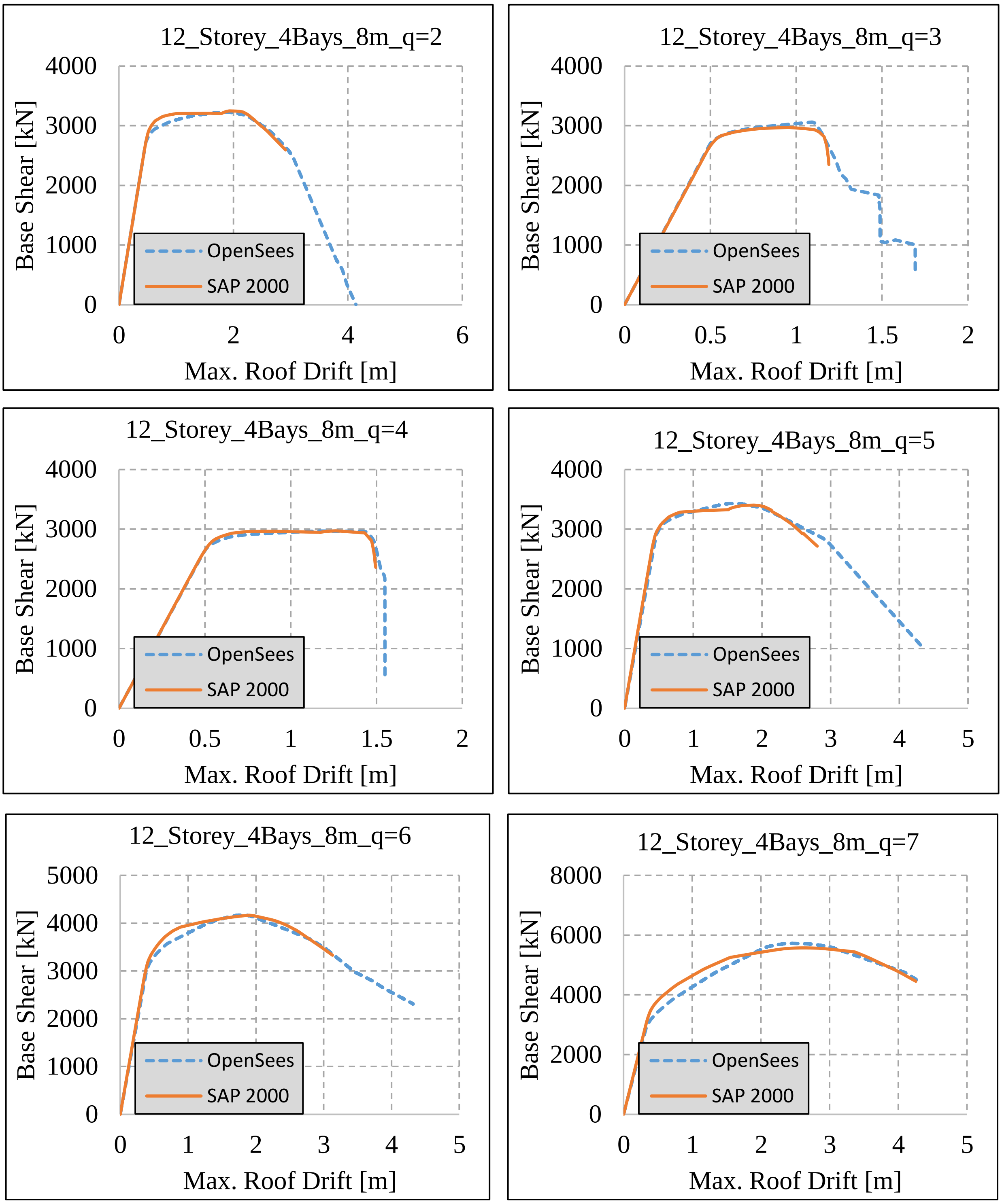


Figure 9-14 Sap 2000 and OpenSees Comparison for 12S-4B-8m

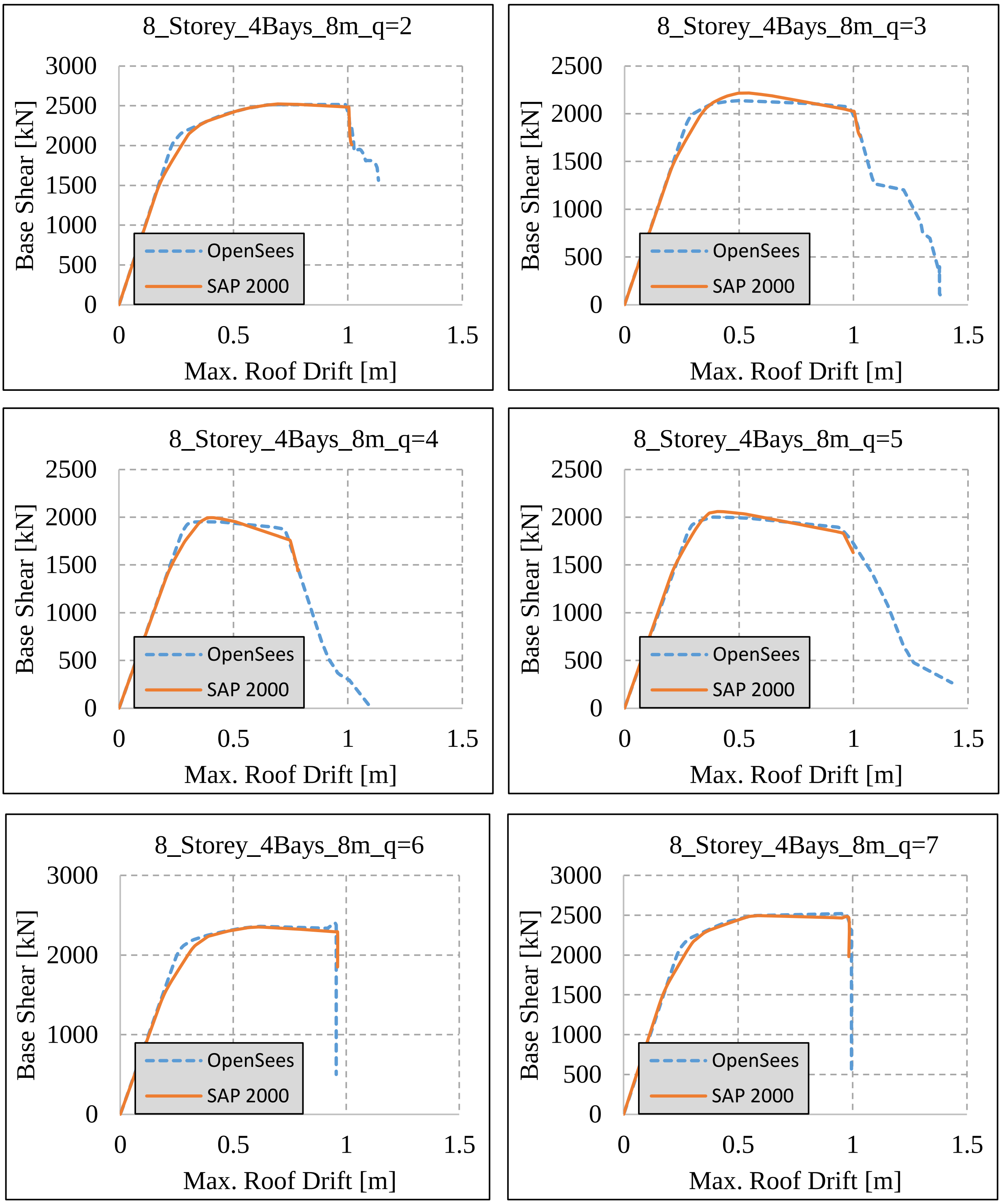


Figure 9-15 Sap 2000 and OpenSees Comparison for 8S-4B-8m

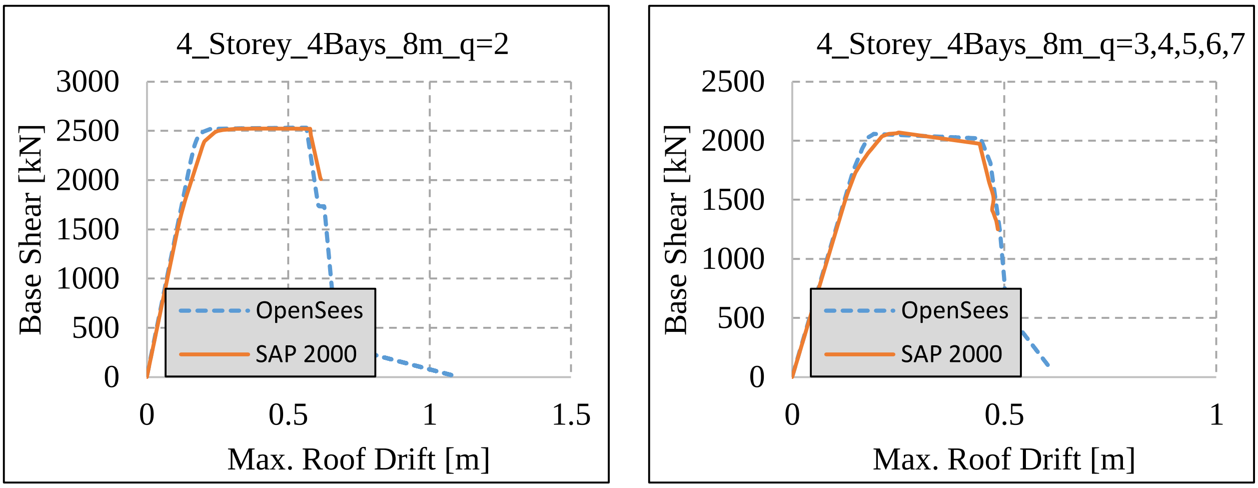


Figure 9-16 Sap 2000 and OpenSees Comparison for 4S-4B-8m

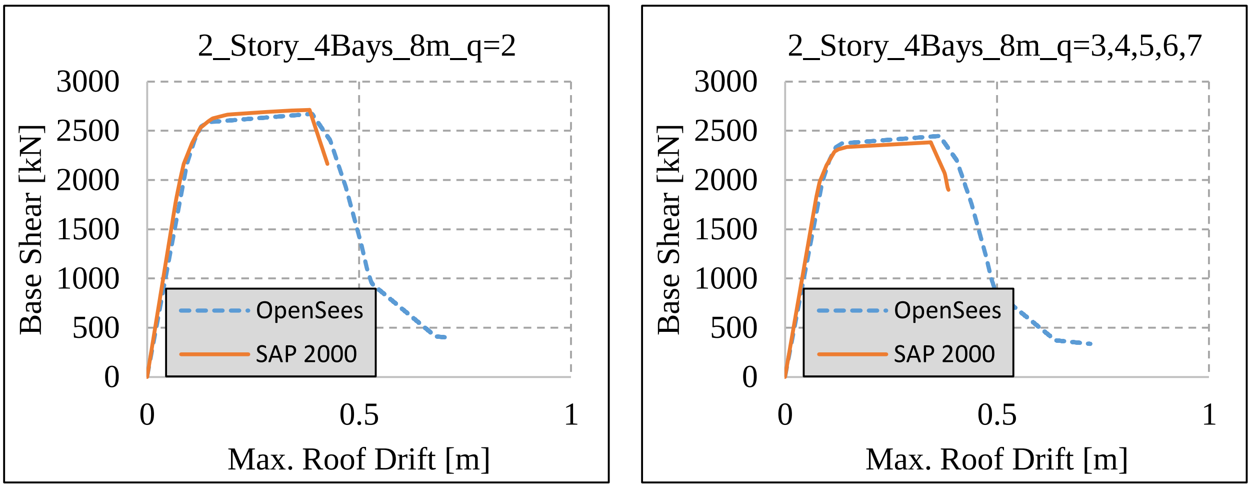


Figure 9-17 Sap 2000 and OpenSees Comparison for 2S-4B-8m

9.5 APPENDIX-D-2

Sap 2000 and *OpenSees* Comparison in Terms of Nonlinear Static Analysis for *Non-Conventional* Structures

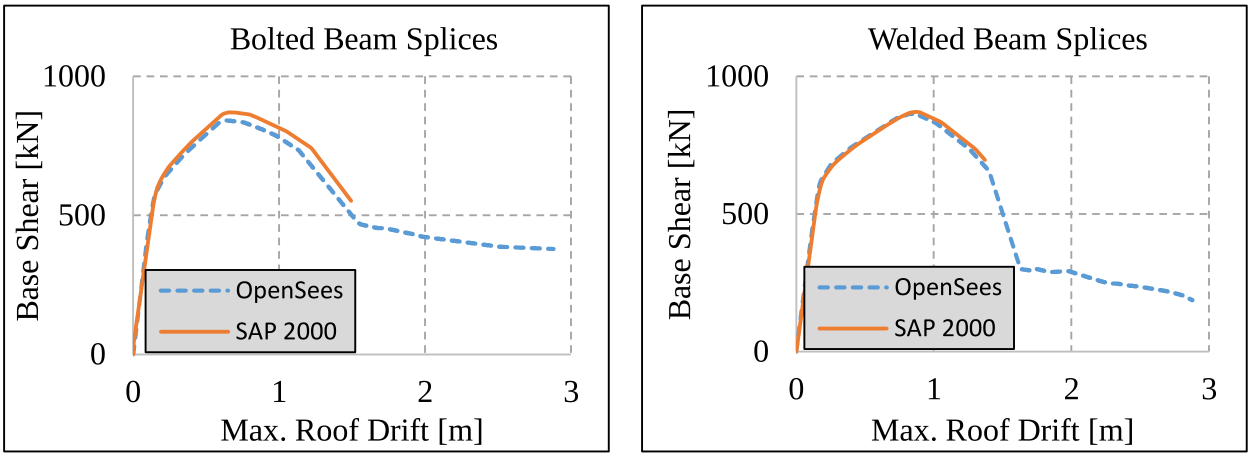


Figure 9-18 Sap 2000 and OpenSees Comparison for 8S-3B-6m

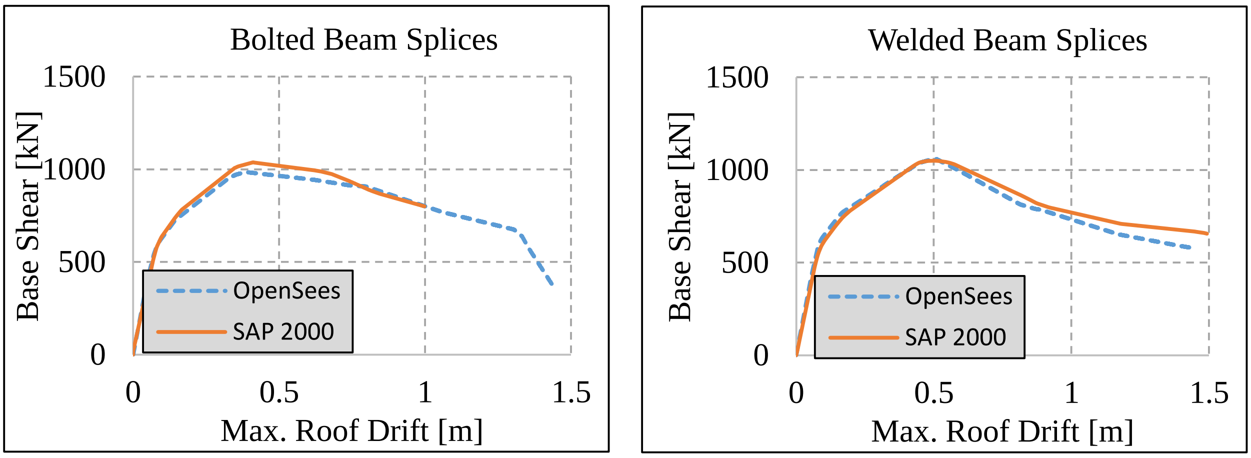


Figure 9-19 Sap 2000 and OpenSees Comparison for 4S-3B-6m

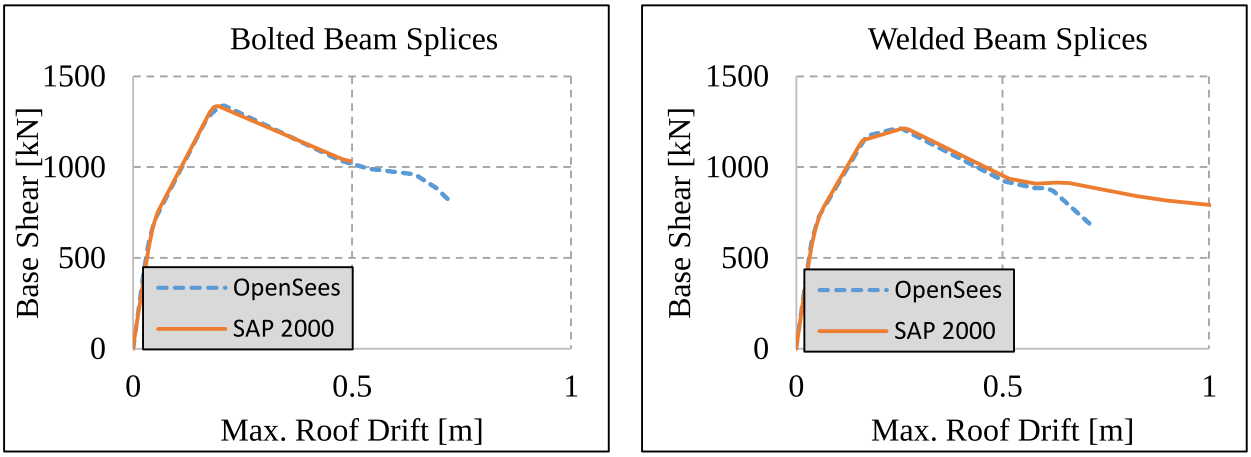


Figure 9-20 Sap 2000 and OpenSees Comparison for 2S-3B-6m

9.6 APPENDIX-E-1

Response of 30 IDAs Records for *Conventional* Structures

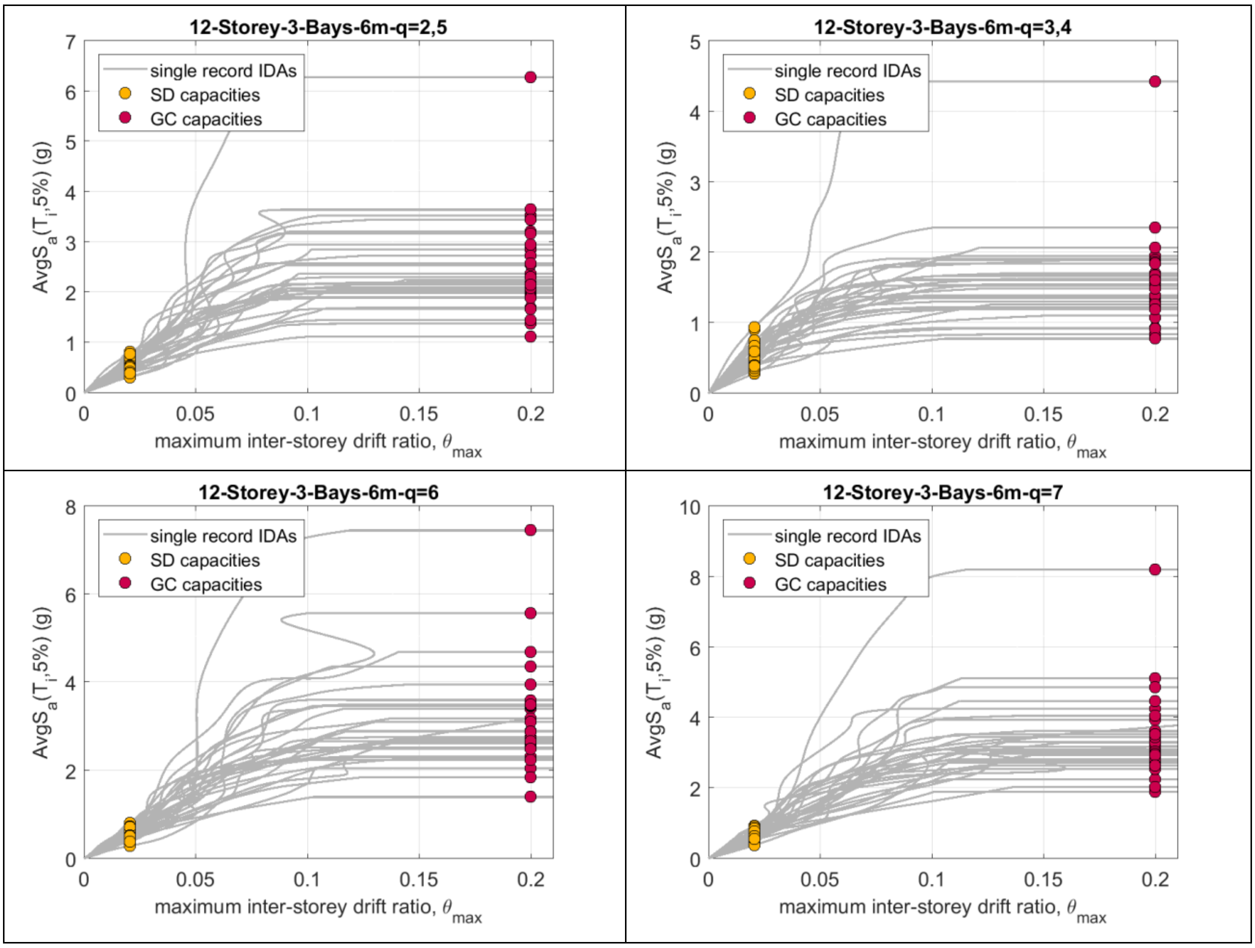


Figure 9-21 Incremental Dynamic Records for 12S-3B-6m

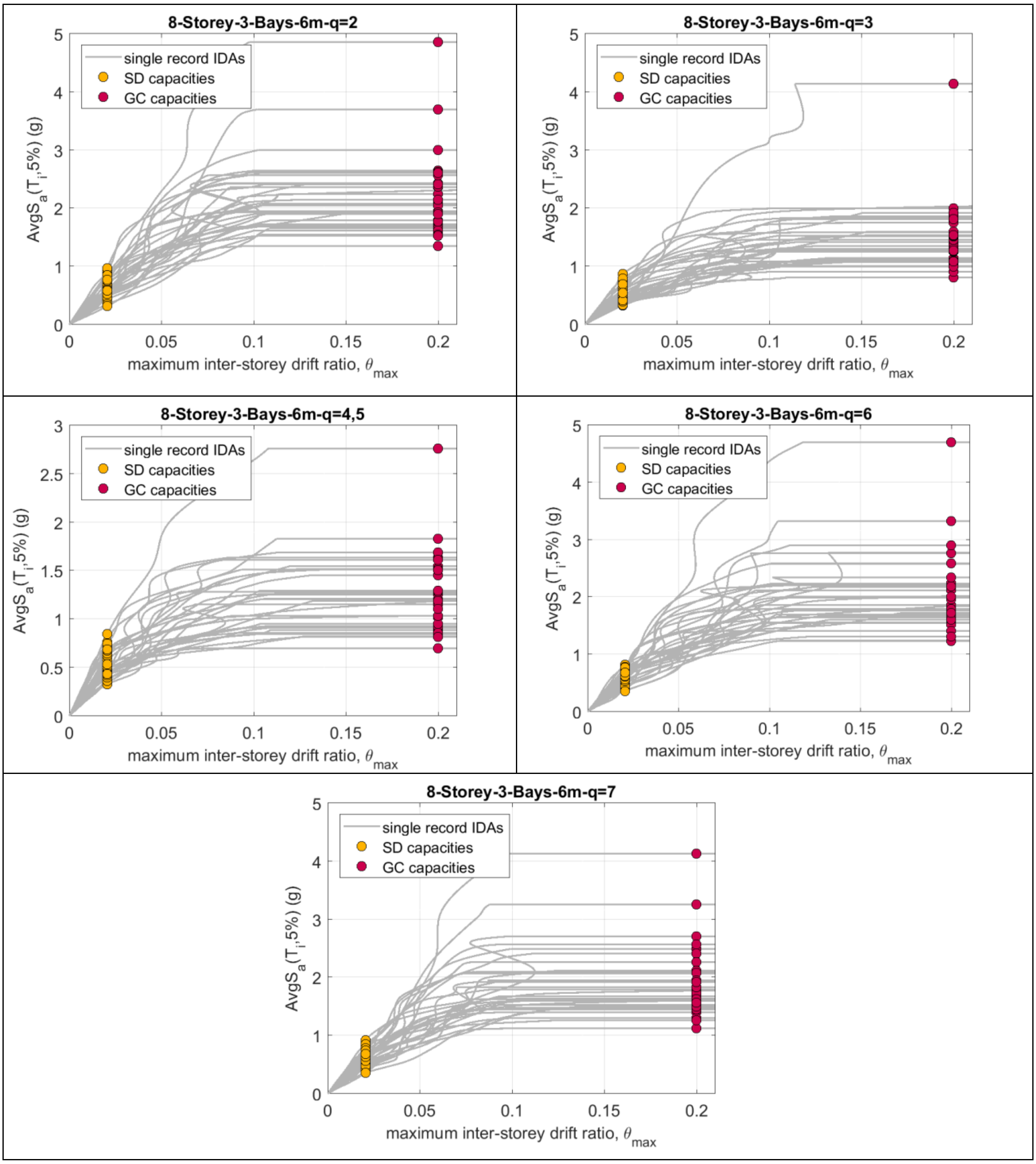


Figure 9-22 Incremental Dynamic Records for 8S-3B-6m

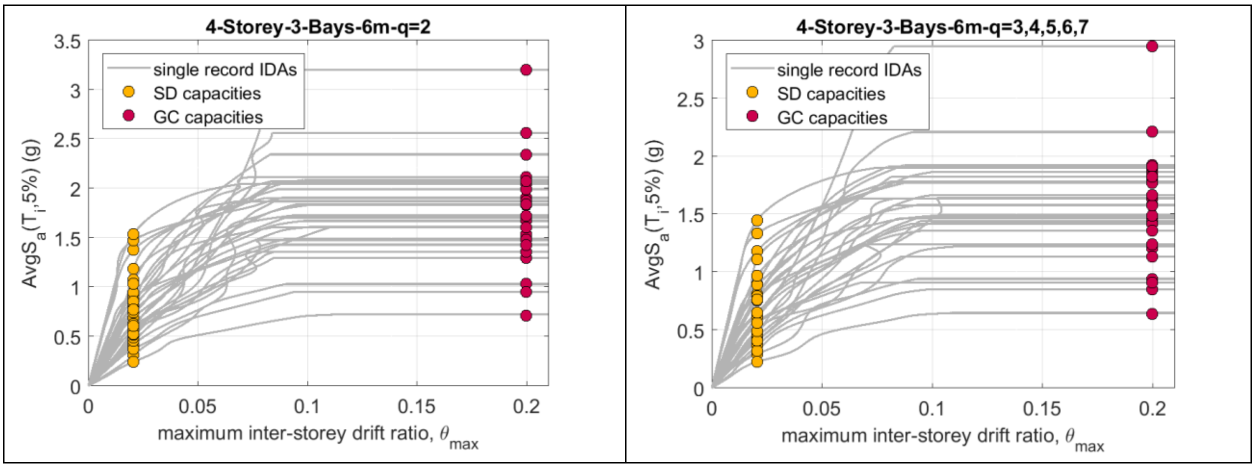


Figure 9-23 Incremental Dynamic Records for 4S-3B-6m

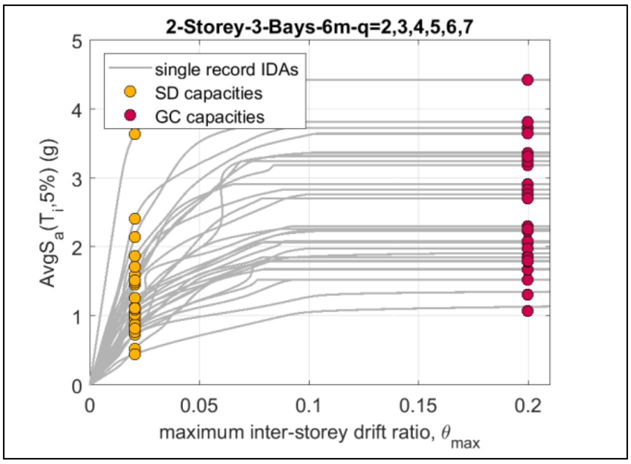


Figure 9-24 Incremental Dynamic Records for 2S-3B-6m

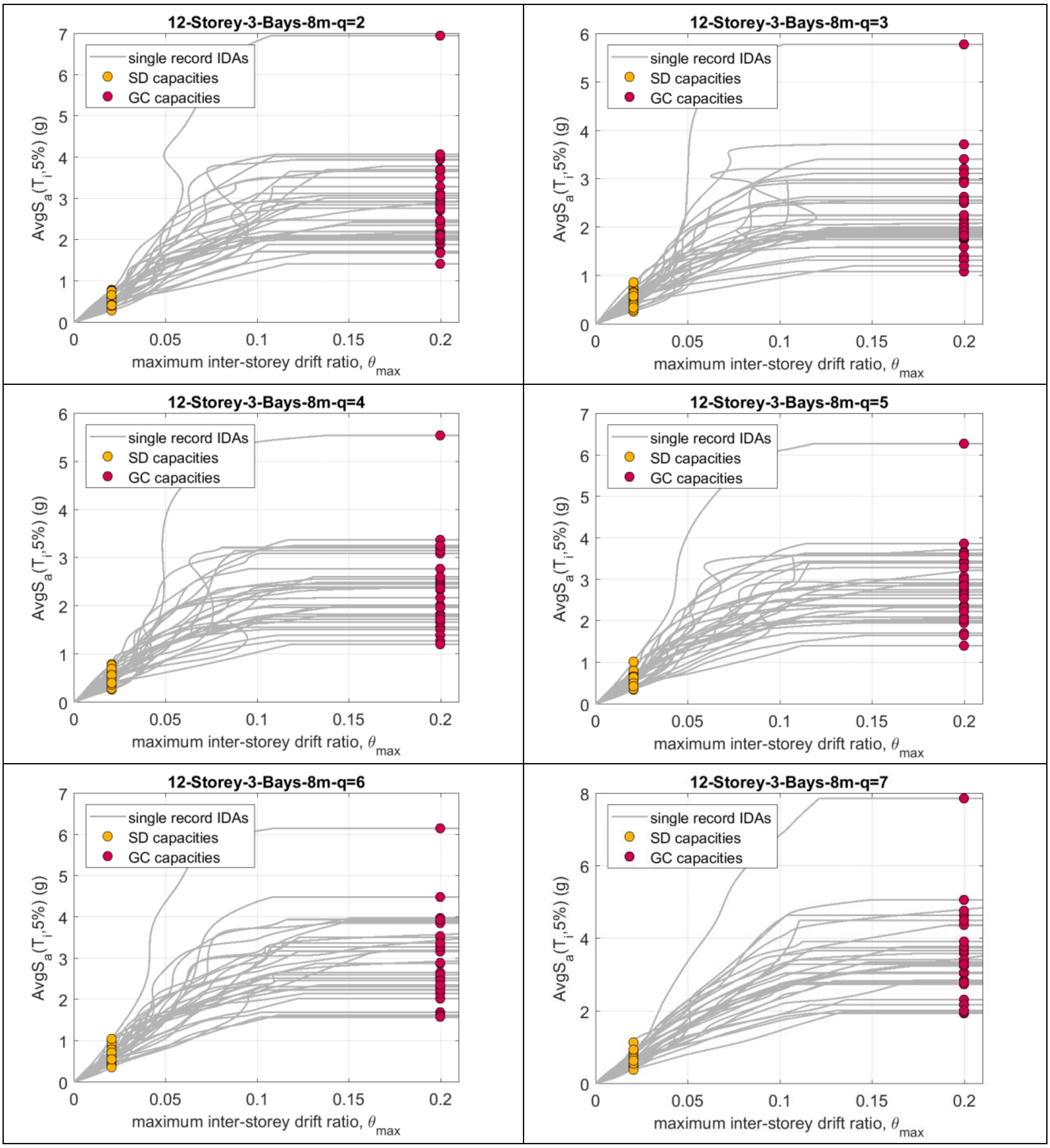


Figure 9-25 Incremental Dynamic Records for 12S-3B-8m

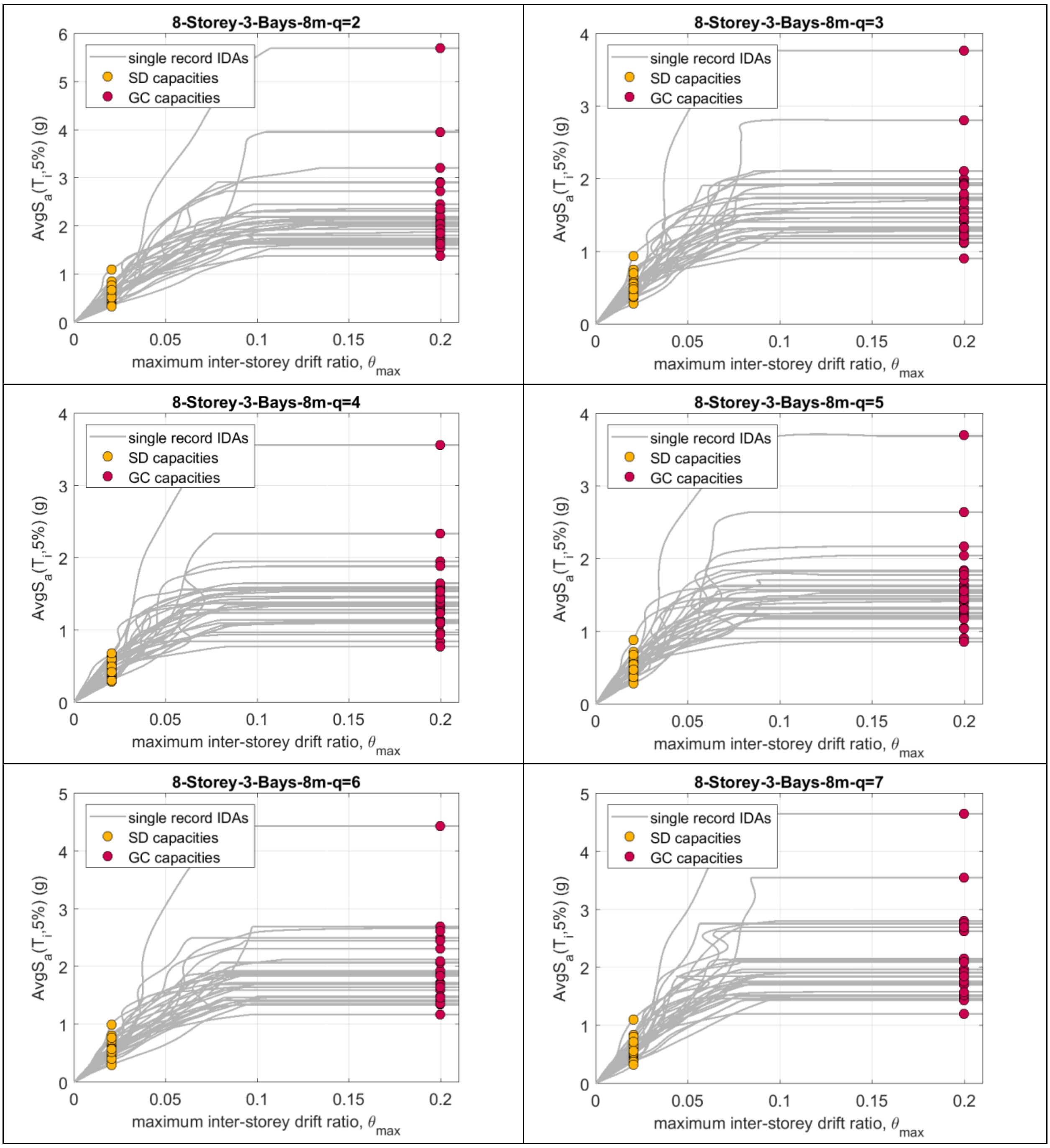


Figure 9-26 Incremental Dynamic Records for 8S-3B-8m

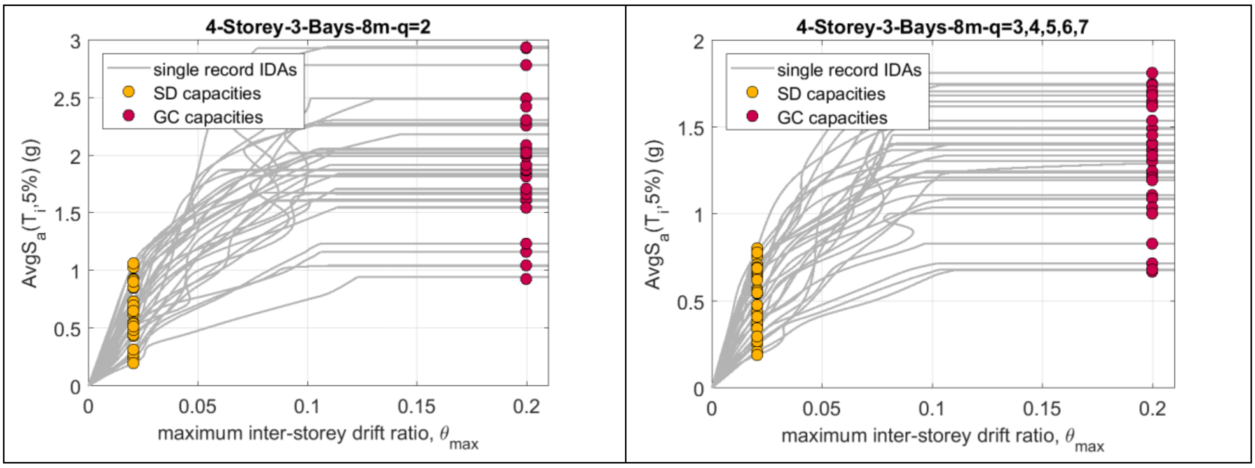


Figure 9-27 Incremental Dynamic Records for 4S-3B-8m

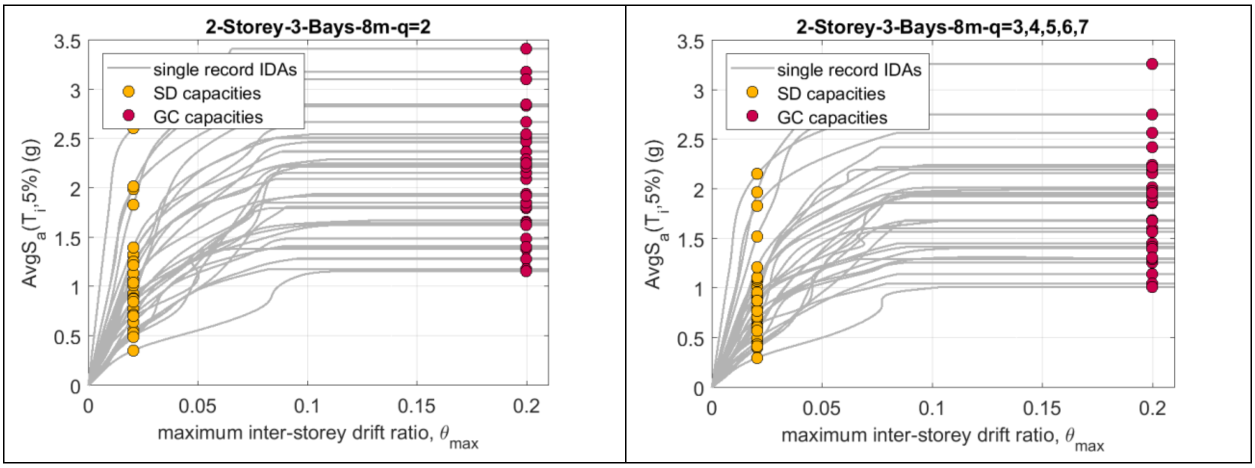


Figure 9-28 Incremental Dynamic Records for 2S-3B-8m

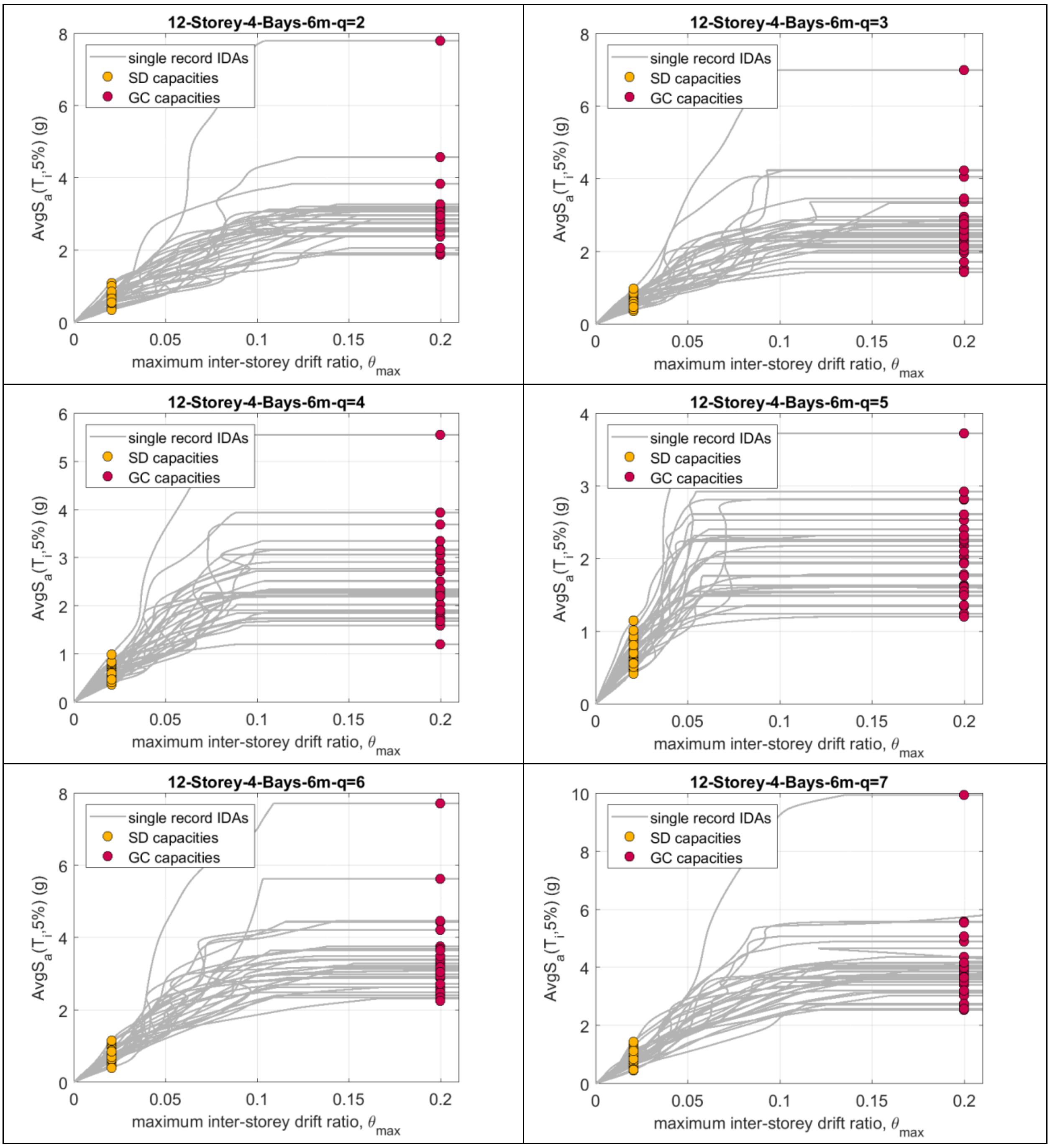


Figure 9-29 Incremental Dynamic Records for 12S-4B-6m

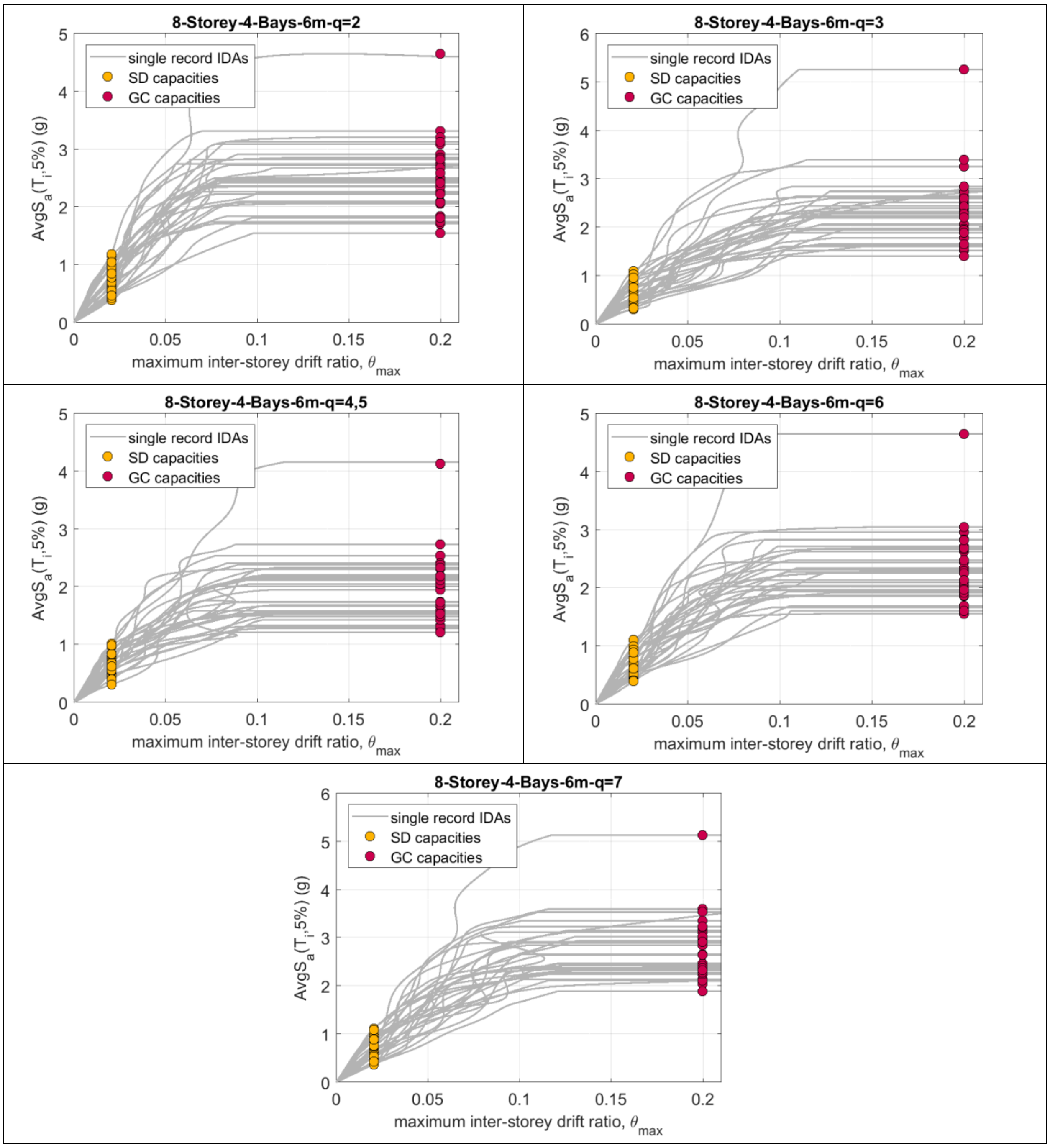


Figure 9-30 Incremental Dynamic Records for 8S-4B-6m

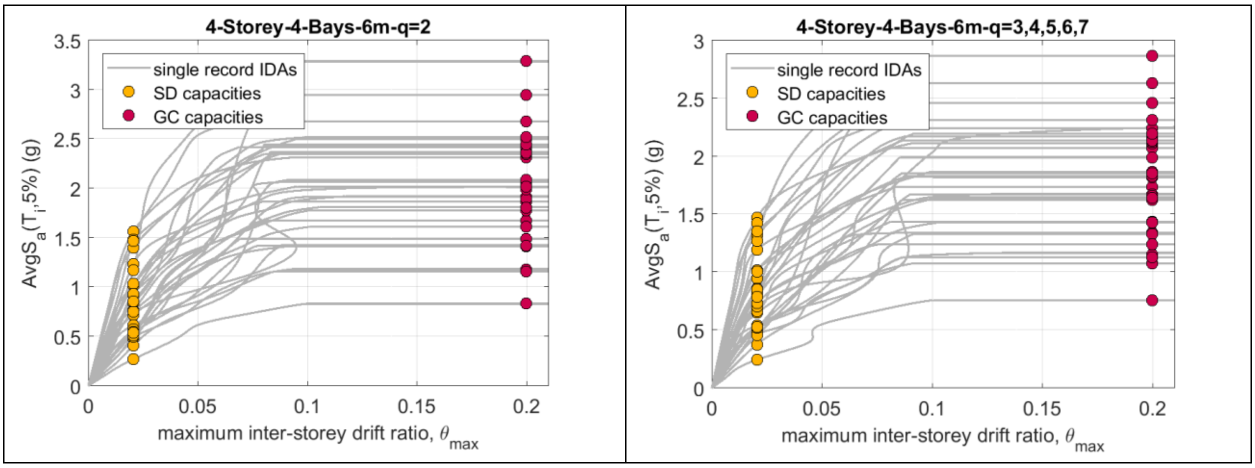


Figure 9-31 Incremental Dynamic Records for 4S-4B-6m

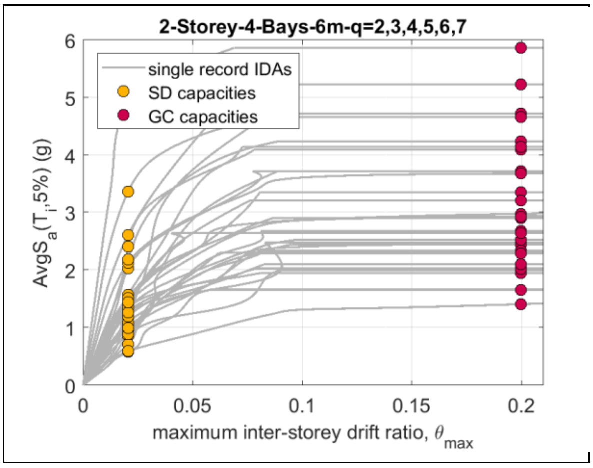


Figure 9-32 Incremental Dynamic Records for 2S-4B-6m

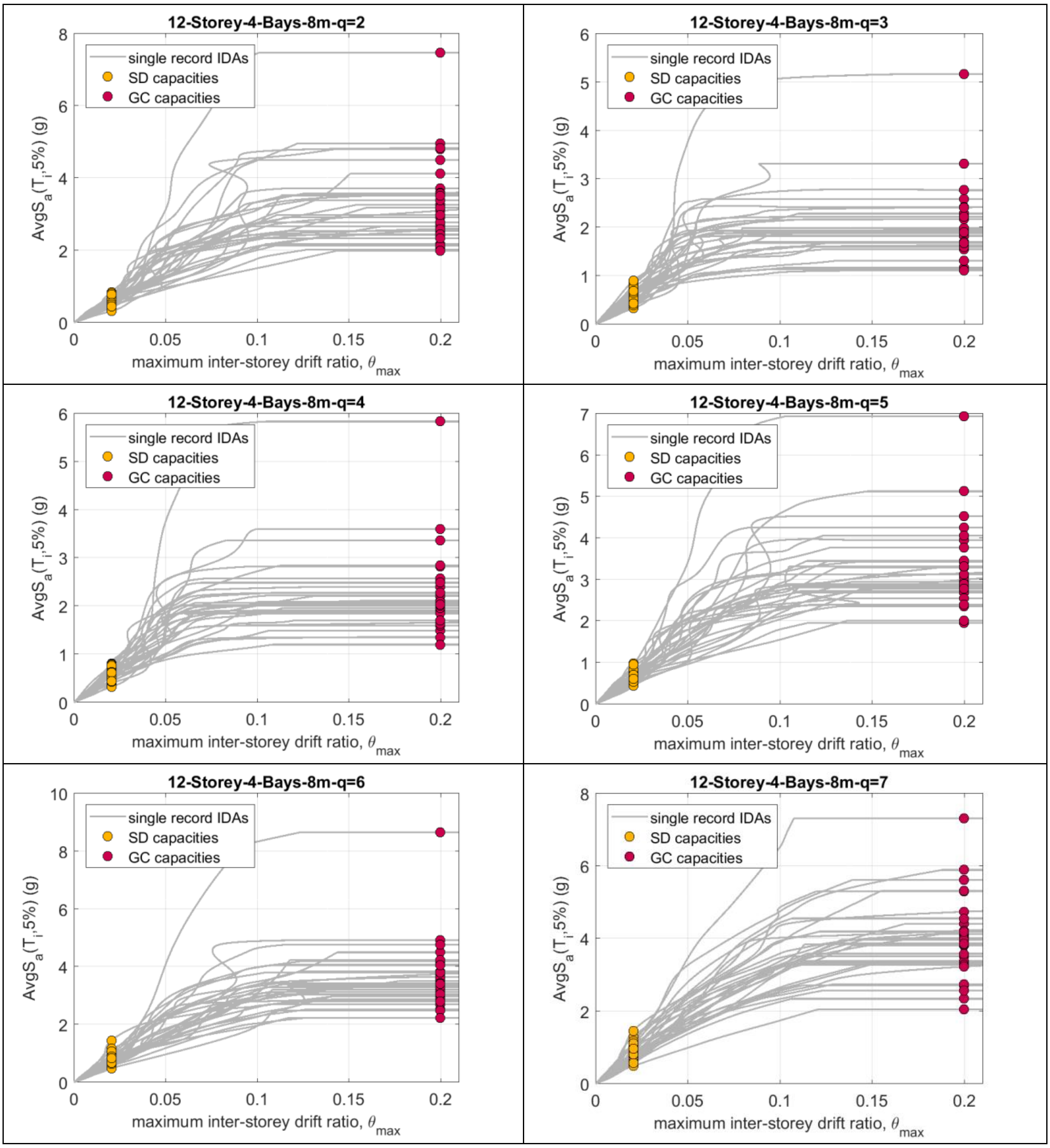


Figure 9-33 Incremental Dynamic Records for 12S-4B-8m

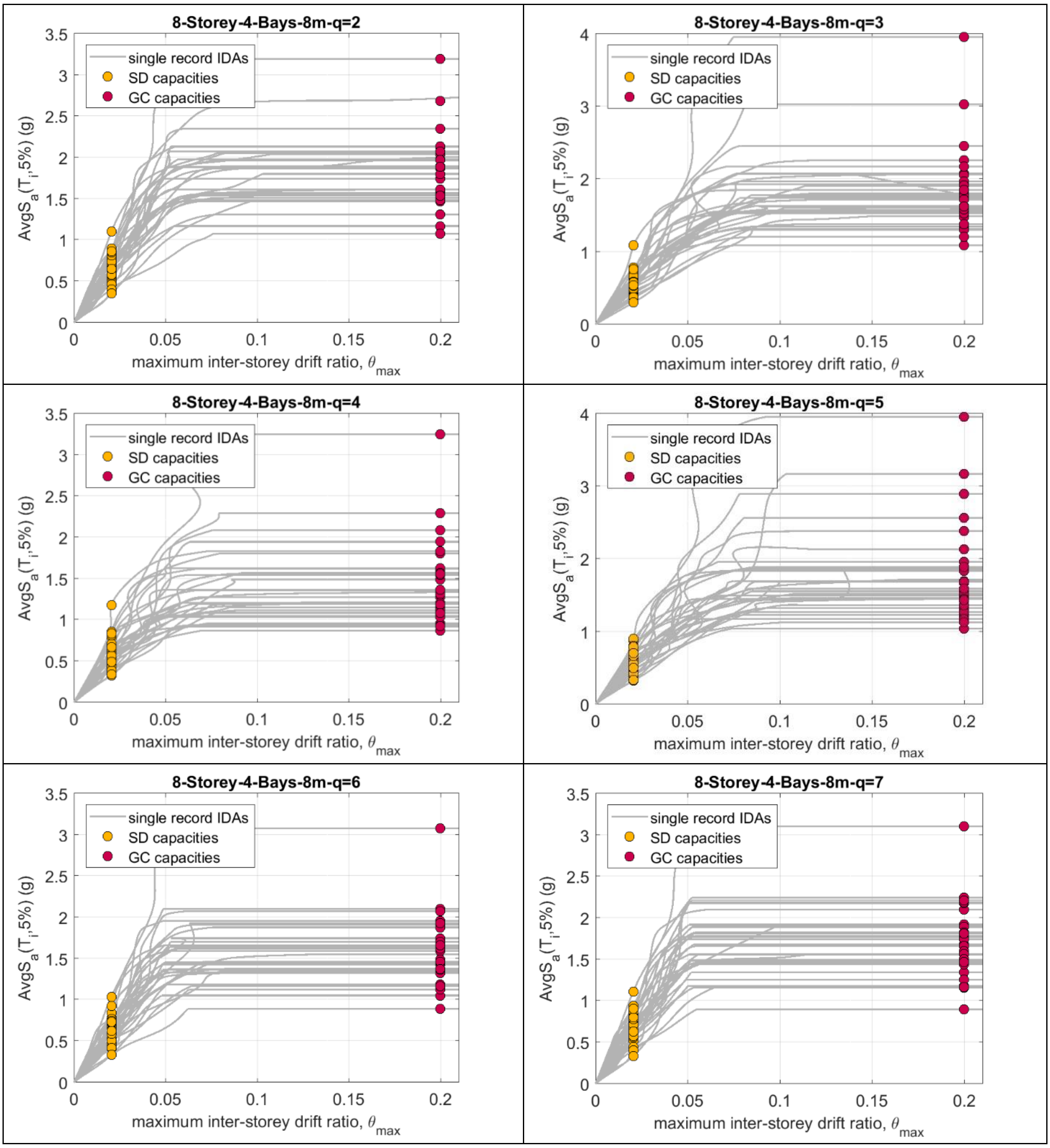


Figure 9-34 Incremental Dynamic Records for 8S-4B-8m

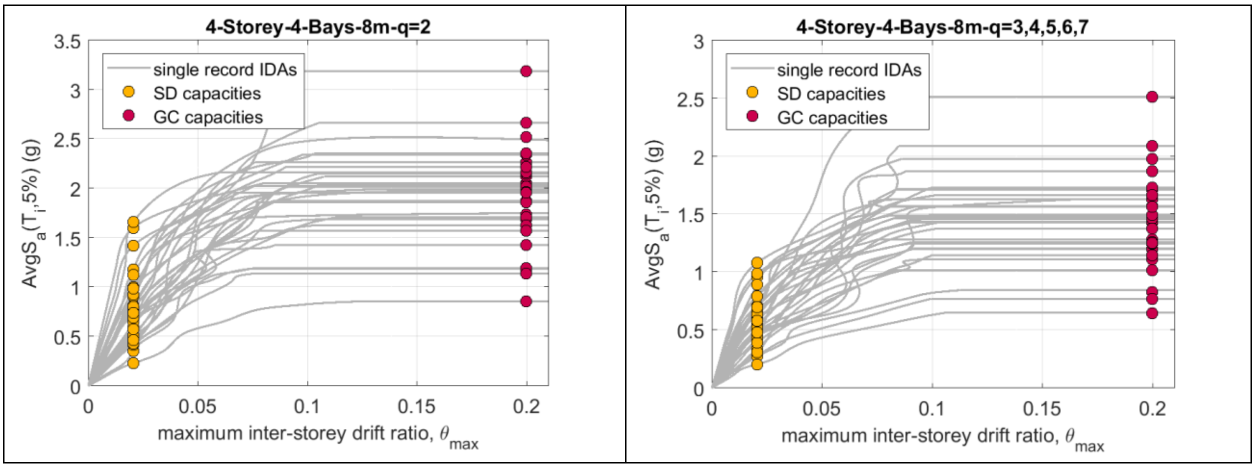


Figure 9-35 Incremental Dynamic Records for 4S-4B-8m

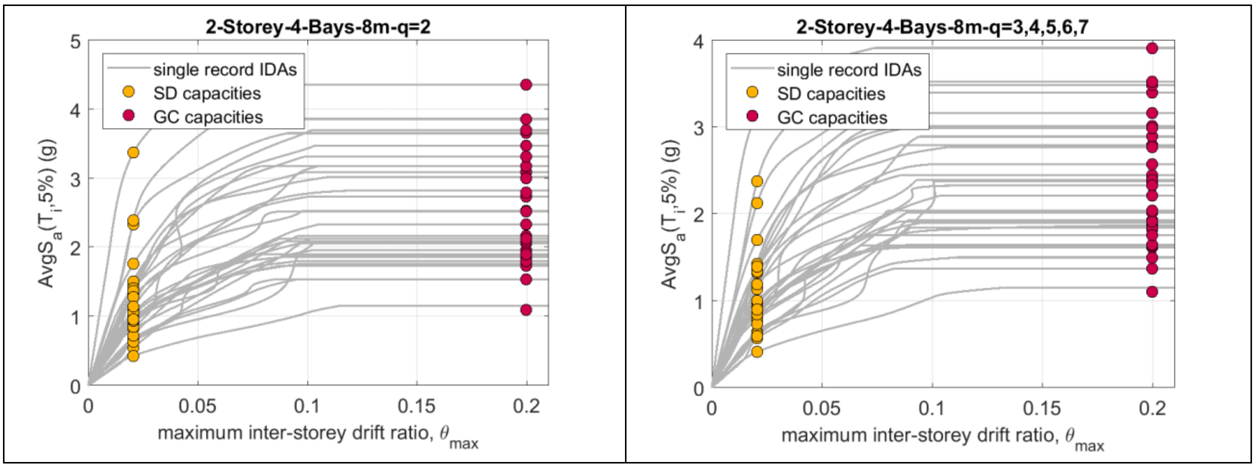
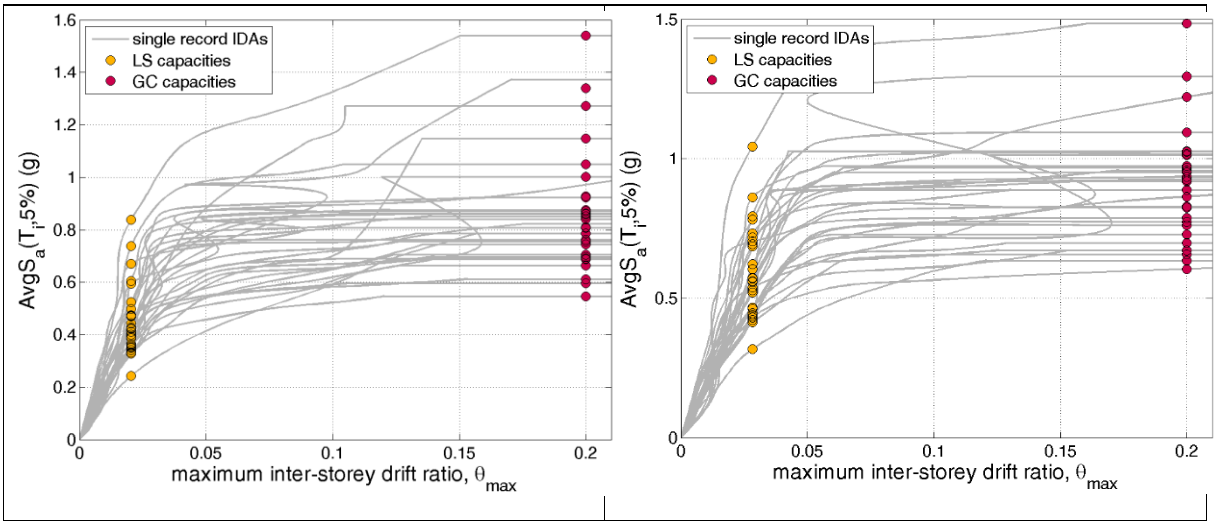


Figure 9-36 Incremental Dynamic Records for 2S-4B-8m

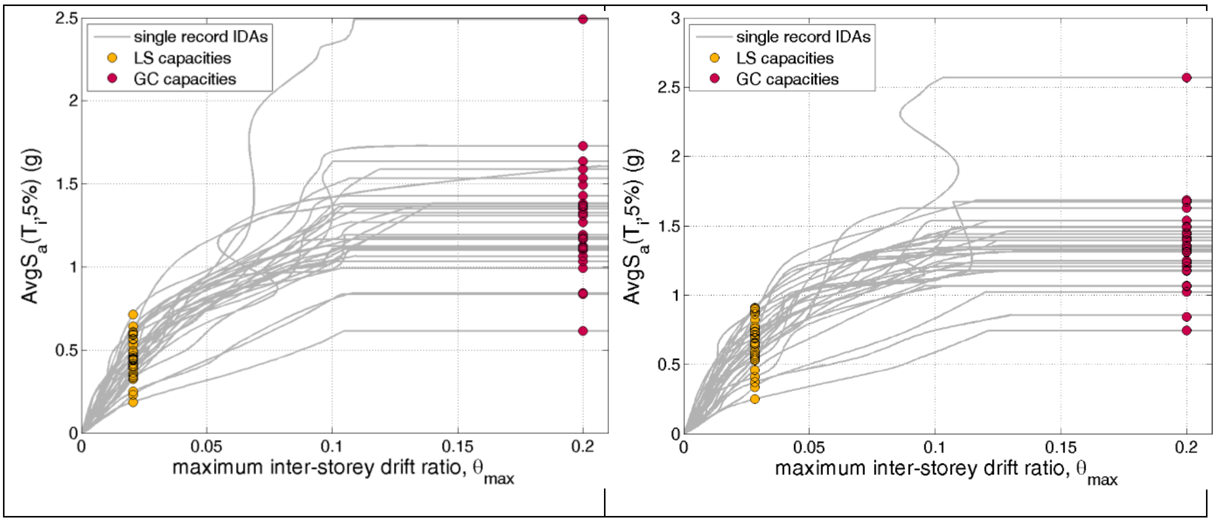
9.7 APPENDIX-E-2

Response of 30 IDAs Records for *Non-Conventional* Structures



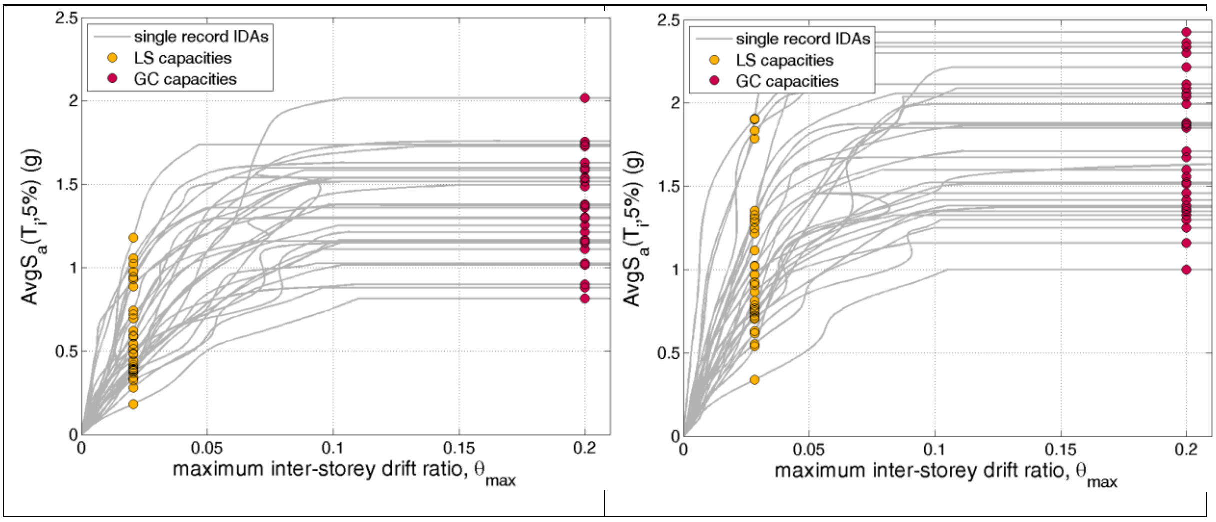
(a) (b)

Figure 9-37 Incremental Dynamic Records for 8S-3B-6m a) Bolted Beam Splices b) Welded Beam Splices



(a) (b)

Figure 9-38 Incremental Dynamic Records for 4S-3B-6m a) Bolted Beam Splices b) Welded Beam Splices



(a) (b)

Figure 9-39 Incremental Dynamic Records for 2S-3B-6m a) Bolted Beam Splices b) Welded Beam Splices

9.8 APPENDIX-F-1

Design Behaviour Factor Verifications (Initial Assumption) for *Conventional* Structures via IDA

Table 9-49 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 12S-3B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2,5	SD	0.674	2.107	3.126	1.000	✓
		GC	0.004	0.201	44.812		
	q=3,4	SD	0.463	2.107	4.553	1.000	✓
		GC	0.017	0.201	11.499		
	q=6	SD	0.706	2.107	2.983	1.000	✓
		GC	0.001	0.201	208.681		
	q=7	SD	0.518	2.107	4.069	1.000	✓
		GC	0.000	0.201	677.099		
Perugia	q=2,5	SD	0.705	2.107	2.988	1.000	✓
		GC	0.002	0.201	88.238		
	q=3,4	SD	0.486	2.107	4.334	1.000	✓
		GC	0.011	0.201	17.949		
	q=6	SD	0.684	2.107	3.080	1.000	✓
		GC	0.000	0.201	593.910		
	q=7	SD	0.457	2.107	4.615	1.000	✓
		GC	0.000	0.201	2443.854		
Focsani	q=2,5	SD	0.365	2.107	5.768	1.000	✓
		GC	0.000	0.201	30807.222		
	q=3,4	SD	0.168	2.107	12.518	1.000	✓
		GC	0.000	0.201	2145.596		
	q=6	SD	0.304	2.107	6.923	1.000	✓
		GC	0.000	0.201	4258012.278		
	q=7	SD	0.164	2.107	12.855	1.000	✓
		GC	0.000	0.201	290296052.156		

Table 9-50 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 8S-3B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.699	2.107	3.014	1.000	✓
		GC	0.004	0.201	48.879		
	q=3	SD	1.267	2.107	1.664	1.000	✓
		GC	0.048	0.201	4.168		
	q=4,5	SD	1.125	2.107	1.873	1.000	✓
		GC	0.094	0.201	2.143		
	q=6	SD	0.726	2.107	2.901	1.000	✓
		GC	0.009	0.201	21.468		
	q=7	SD	0.672	2.107	3.138	1.000	✓
		GC	0.017	0.201	11.935		
Perugia	q=2	SD	0.617	2.107	3.417	1.000	✓
		GC	0.002	0.201	133.274		
	q=3	SD	1.240	2.107	1.700	1.000	✓
		GC	0.029	0.201	6.963		
	q=4,5	SD	1.079	2.107	1.954	1.000	✓
		GC	0.064	0.201	3.148		
	q=6	SD	0.654	2.107	3.224	1.000	✓
		GC	0.004	0.201	48.035		
	q=7	SD	0.588	2.107	3.585	1.000	✓
		GC	0.008	0.201	23.670		
Focsani	q=2	SD	0.307	2.107	6.872	1.000	✓
		GC	0.000	0.201	1269382.403		
	q=3	SD	0.887	2.107	2.376	1.000	✓
		GC	0.001	0.201	343.683		
	q=4,5	SD	0.653	2.107	3.225	1.000	✓
		GC	0.005	0.201	43.582		
	q=6	SD	0.334	2.107	6.309	1.000	✓
		GC	0.000	0.201	41663.809		
	q=7	SD	0.255	2.107	8.263	1.000	✓
		GC	0.000	0.201	3994.732		

Table 9-51 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 4S-3B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.55978	2.107	3.76438	1.0000	✓
		GC	0.01218	0.201	16.5104		
	q=3,4,5,6,7	SD	0.8327	2.107	2.53058	1.0000	✓
		GC	0.02845	0.201	7.06456		
Perugia	q=2	SD	0.43193	2.107	4.87864	1.0000	✓
		GC	0.00463	0.201	43.421		
	q=3,4,5,6,7	SD	0.67993	2.107	3.09915	1.0000	✓
		GC	0.01305	0.201	15.4021		
Focsani	q=2	SD	0.22934	2.107	9.18804	1.0000	✓
		GC	2.00E-06	0.201	117201		
	q=3,4,5,6,7	SD	0.4881	2.107	4.3172	1.0000	✓
		GC	7.70E-05	0.201	2609.06		

Table 9-52 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 2S-3B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2,3,4,5,6,7	SD	0.13832	2.107	15.2346	1.0000	✓
		GC	0.00637	0.201	31.5428		
Perugia	q=2,3,4,5,6,7	SD	0.07664	2.107	27.4964	1.0000	✓
		GC	0.00222	0.201	90.4512		
Focsani	q=2,3,4,5,6,7	SD	0.02789	2.107	75.5579	1.0000	✓
		GC	2.60E-05	0.201	7681.98		

Table 9-53 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 12S-3B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.751	2.107	2.804	1.000	✓
		GC	0.001	0.201	225.034		
	q=3	SD	0.548	2.107	3.845	1.000	✓
		GC	0.007	0.201	29.292		
	q=4	SD	0.633	2.107	3.330	1.000	✓
		GC	0.005	0.201	38.662		
	q=5	SD	0.530	2.107	3.977	1.000	✓
		GC	0.001	0.201	218.964		
	q=6	SD	0.556	2.107	3.789	1.000	✓
		GC	0.001	0.201	277.355		
	q=7	SD	0.568	2.107	3.709	1.000	✓
		GC	0.000	0.201	413.839		
Perugia	q=2	SD	0.817	2.107	2.580	1.000	✓
		GC	0.000	0.201	552.565		
	q=3	SD	0.597	2.107	3.531	1.000	✓
		GC	0.004	0.201	48.797		
	q=4	SD	0.702	2.107	3.001	1.000	✓
		GC	0.003	0.201	65.554		
	q=5	SD	0.545	2.107	3.869	1.000	✓
		GC	0.000	0.201	565.405		
	q=6	SD	0.520	2.107	4.048	1.000	✓
		GC	0.000	0.201	798.077		
	q=7	SD	0.508	2.107	4.145	1.000	✓
		GC	0.000	0.201	1135.157		
Focsani	q=2	SD	0.473	2.107	4.454	1.000	✓
		GC	0.000	0.201	1254330.805		
	q=3	SD	0.249	2.107	8.464	1.000	✓
		GC	0.000	0.201	6034.138		
	q=4	SD	0.331	2.107	6.373	1.000	✓
		GC	0.000	0.201	7870.505		
	q=5	SD	0.250	2.107	8.442	1.000	✓
		GC	0.000	0.201	2500616.516		
	q=6	SD	0.189	2.107	11.143	1.000	✓
		GC	0.000	0.201	4532534.024		
	q=7	SD	0.191	2.107	11.056	1.000	✓
		GC	0.000	0.201	4222458.521		

Table 9-54 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 8S-3B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.783	2.107	2.690	1.000	✓
		GC	0.006	0.201	33.194		
	q=3	SD	1.114	2.107	1.891	1.000	✓
		GC	0.025	0.201	7.980		
	q=4	SD	1.479	2.107	1.424	1.000	✓
		GC	0.038	0.201	5.310		
	q=5	SD	1.018	2.107	2.071	1.000	✓
		GC	0.028	0.201	7.141		
	q=6	SD	0.951	2.107	2.216	1.000	✓
		GC	0.014	0.201	13.871		
	q=7	SD	0.673	2.107	3.131	1.000	✓
		GC	0.013	0.201	15.820		
Perugia	q=2	SD	0.705	2.107	2.989	1.000	✓
		GC	0.003	0.201	77.278		
	q=3	SD	1.078	2.107	1.954	1.000	✓
		GC	0.013	0.201	14.906		
	q=4	SD	1.517	2.107	1.389	1.000	✓
		GC	0.021	0.201	9.535		
	q=5	SD	0.977	2.107	2.158	1.000	✓
		GC	0.015	0.201	13.357		
	q=6	SD	0.890	2.107	2.367	1.000	✓
		GC	0.007	0.201	28.147		
	q=7	SD	0.593	2.107	3.551	1.000	✓
		GC	0.006	0.201	32.604		
Focsani	q=2	SD	0.360	2.107	5.853	1.000	✓
		GC	0.000	0.201	56845.020		
	q=3	SD	0.703	2.107	2.996	1.000	✓
		GC	0.000	0.201	2805.162		
	q=4	SD	1.294	2.107	1.629	1.000	✓
		GC	0.000	0.201	3077.606		
	q=5	SD	0.589	2.107	3.580	1.000	✓
		GC	0.000	0.201	4165.898		
	q=6	SD	0.535	2.107	3.939	1.000	✓
		GC	0.000	0.201	7644.268		
	q=7	SD	0.292	2.107	7.216	1.000	✓
		GC	0.000	0.201	6551.182		

Table 9-55 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 4S-3B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	1.039096	2.107	2.027927	1.0000	✓
		GC	0.007894	0.201	25.462569		
	q=3,4,5,6,7	SD	1.248778	2.107	1.687418	1.0000	✓
		GC	0.053095	0.201	3.785816		
Perugia	q=2	SD	0.900569	2.107	2.339865	1.0000	✓
		GC	0.002841	0.201	70.754732		
	q=3,4,5,6,7	SD	1.114819	2.107	1.890182	1.0000	✓
		GC	0.027943	0.201	7.193436		
Focsani	q=2	SD	0.872149	2.107	2.416112	1.0000	✓
		GC	1.00E-06	0.201	355956.8893		
	q=3,4,5,6,7	SD	1.09825	2.107	1.918699	1.0000	✓
		GC	4.68E-04	0.201	429.206874		

Table 9-56 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 2S-3B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.382349	2.107	5.511221	1.0000	✓
		GC	0.011069	0.201	18.158899		
	q=3,4,5,6,7	SD	0.44135	2.107	4.774469	1.0000	✓
		GC	0.015565	0.201	12.91368		
Perugia	q=2	SD	0.246885	2.107	8.535179	1.0000	✓
		GC	0.004182	0.201	48.062562		
	q=3,4,5,6,7	SD	0.294883	2.107	7.145928	1.0000	✓
		GC	0.006045	0.201	33.250974		
Focsani	q=2	SD	0.196778	2.107	10.708549	1.0000	✓
		GC	7.60E-05	0.201	2644.940728		
	q=3,4,5,6,7	SD	0.236327	2.107	8.916497	1.0000	✓
		GC	8.10E-05	0.201	2471.490201		

Table 9-57 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 12S-4B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.495	2.107	4.257	1.000	✓
		GC	0.000	0.201	1429.513		
	q=3	SD	0.606	2.107	3.477	1.000	✓
		GC	0.001	0.201	143.596		
	q=4	SD	0.621	2.107	3.392	1.000	✓
		GC	0.004	0.201	48.626		
	q=5	SD	0.453	2.107	4.649	1.000	✓
		GC	0.009	0.201	22.531		
	q=6	SD	0.328	2.107	6.424	1.000	✓
		GC	0.000	0.201	1220.625		
	q=7	SD	0.265	2.107	7.937	1.000	✓
		GC	0.000	0.201	5940.499		
Perugia	q=2	SD	0.429	2.107	4.912	1.000	✓
		GC	0.000	0.201	11341.429		
	q=3	SD	0.555	2.107	3.794	1.000	✓
		GC	0.000	0.201	412.732		
	q=4	SD	0.578	2.107	3.647	1.000	✓
		GC	0.002	0.201	109.041		
	q=5	SD	0.392	2.107	5.375	1.000	✓
		GC	0.004	0.201	47.641		
	q=6	SD	0.260	2.107	8.115	1.000	✓
		GC	0.000	0.201	5323.966		
	q=7	SD	0.206	2.107	10.250	1.000	✓
		GC	0.000	0.201	43280.608		
Focsani	q=2	SD	0.139	2.107	15.138	1.000	✓
		GC	0.000	0.201	94486083.000		
	q=3	SD	0.232	2.107	9.081	1.000	✓
		GC	0.000	0.201	3311108.693		
	q=4	SD	0.248	2.107	8.514	1.000	✓
		GC	0.000	0.201	68797.839		
	q=5	SD	0.126	2.107	16.706	1.000	✓
		GC	0.000	0.201	22054.539		
	q=6	SD	0.051	2.107	41.101	1.000	✓
		GC	0.000	0.201	26871629.813		
	q=7	SD	0.047	2.107	45.071	1.000	✓
		GC	0.000	0.201	45099476.930		

Table 9-58 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 8S-4B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.470	2.107	4.479	1.000	✓
		GC	0.002	0.201	114.814		
	q=3	SD	0.911	2.107	2.312	1.000	✓
		GC	0.003	0.201	73.127		
	q=4,5	SD	0.515	2.107	4.093	1.000	✓
		GC	0.012	0.201	16.912		
	q=6	SD	0.742	2.107	2.840	1.000	✓
		GC	0.002	0.201	106.547		
	q=7	SD	0.631	2.107	3.342	1.000	✓
		GC	0.000	0.201	470.006		
Perugia	q=2	SD	0.389	2.107	5.417	1.000	✓
		GC	0.001	0.201	367.571		
	q=3	SD	0.824	2.107	2.558	1.000	✓
		GC	0.001	0.201	209.231		
	q=4,5	SD	0.425	2.107	4.961	1.000	✓
		GC	0.005	0.201	38.190		
	q=6	SD	0.654	2.107	3.224	1.000	✓
		GC	0.001	0.201	366.986		
	q=7	SD	0.546	2.107	3.858	1.000	✓
		GC	0.000	0.201	2626.669		
Focsani	q=2	SD	0.197	2.107	10.671	1.000	✓
		GC	0.000	0.201	4506450.252		
	q=3	SD	0.589	2.107	3.578	1.000	✓
		GC	0.000	0.201	986566.946		
	q=4,5	SD	0.156	2.107	13.497	1.000	✓
		GC	0.000	0.201	34748.049		
	q=6	SD	0.412	2.107	5.115	1.000	✓
		GC	0.000	0.201	4775345.382		
	q=7	SD	0.339	2.107	6.216	1.000	✓
		GC	0.000	0.201	4536760.805		

Table 9-59 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 4S-4B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.487741	2.107	4.320349	1.0000	✓
		GC	0.008434	0.201	23.832293		
	q=3,4,5,6,7	SD	0.600547	2.107	3.50882	1.0000	✓
		GC	0.014054	0.201	14.302574		
Perugia	q=2	SD	0.364115	2.107	5.787206	1.0000	✓
		GC	0.003238	0.201	62.084565		
	q=3,4,5,6,7	SD	0.466903	2.107	4.51317	1.0000	✓
		GC	0.005885	0.201	34.154644		
Focsani	q=2	SD	0.195781	2.107	10.763095	1.0000	✓
		GC	8.00E-06	0.201	24862.22132		
	q=3,4,5,6,7	SD	0.299892	2.107	7.026574	1.0000	✓
		GC	2.90E-05	0.201	6836.149518		

Table 9-60 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 2S-4B-6m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2,3,4,5,6,7	SD	0.05887	2.107	35.794097	1.0000	✓
		GC	0.001968	0.201	102.118292		
Perugia	q=2,3,4,5,6,7	SD	0.030268	2.107	69.619054	1.0000	✓
		GC	0.000618	0.201	325.443087		
Focsani	q=2,3,4,5,6,7	SD	0.007549	2.107	279.143267	1.0000	✓
		GC	4.00E-06	0.201	45703.42171		

Table 9-61 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 12S-4B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.606	2.107	3.477	1.000	✓
		GC	0.001	0.201	202.425		
	q=3	SD	0.598	2.107	3.527	1.000	✓
		GC	0.006	0.201	36.458		
	q=4	SD	0.606	2.107	3.479	1.000	✓
		GC	0.006	0.201	33.941		
	q=5	SD	0.514	2.107	4.099	1.000	✓
		GC	0.000	0.201	658.126		
	q=6	SD	0.348	2.107	6.062	1.000	✓
		GC	0.000	0.201	2055.856		
	q=7	SD	0.265	2.107	7.964	1.000	✓
		GC	0.000	0.201	1184.298		
Perugia	q=2	SD	0.564	2.107	3.737	1.000	✓
		GC	0.000	0.201	524.623		
	q=3	SD	0.576	2.107	3.658	1.000	✓
		GC	0.002	0.201	84.784		
	q=4	SD	0.587	2.107	3.589	1.000	✓
		GC	0.003	0.201	69.375		
	q=5	SD	0.457	2.107	4.613	1.000	✓
		GC	0.000	0.201	2575.494		
	q=6	SD	0.282	2.107	7.460	1.000	✓
		GC	0.000	0.201	10448.621		
	q=7	SD	0.207	2.107	10.177	1.000	✓
		GC	0.000	0.201	4014.999		
Focsani	q=2	SD	0.227	2.107	9.271	1.000	✓
		GC	0.000	0.201	659455.529		
	q=3	SD	0.230	2.107	9.153	1.000	✓
		GC	0.000	0.201	775147.667		
	q=4	SD	0.231	2.107	9.127	1.000	✓
		GC	0.000	0.201	34797.881		
	q=5	SD	0.162	2.107	12.974	1.000	✓
		GC	0.000	0.201	14098590.162		
	q=6	SD	0.068	2.107	31.023	1.000	✓
		GC	0.000	0.201	212826836.097		
	q=7	SD	0.048	2.107	43.654	1.000	✓
		GC	0.000	0.201	28527481.365		

Table 9-62 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 8S-4B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.471	2.107	4.474	1.000	✓
		GC	0.007	0.201	28.784		
	q=3	SD	0.865	2.107	2.437	1.000	✓
		GC	0.013	0.201	15.198		
	q=4	SD	0.848	2.107	2.484	1.000	✓
		GC	0.058	0.201	3.476		
	q=5	SD	0.862	2.107	2.444	1.000	✓
		GC	0.027	0.201	7.315		
	q=6	SD	0.583	2.107	3.615	1.000	✓
		GC	0.030	0.201	6.746		
	q=7	SD	0.482	2.107	4.370	1.000	✓
		GC	0.021	0.201	9.666		
Perugia	q=2	SD	0.385	2.107	5.469	1.000	✓
		GC	0.003	0.201	78.369		
	q=3	SD	0.791	2.107	2.664	1.000	✓
		GC	0.006	0.201	33.058		
	q=4	SD	0.781	2.107	2.698	1.000	✓
		GC	0.035	0.201	5.723		
	q=5	SD	0.798	2.107	2.640	1.000	✓
		GC	0.015	0.201	13.053		
	q=6	SD	0.492	2.107	4.281	1.000	✓
		GC	0.015	0.201	12.972		
	q=7	SD	0.396	2.107	5.317	1.000	✓
		GC	0.010	0.201	19.416		
Focsani	q=2	SD	0.146	2.107	14.425	1.000	✓
		GC	0.000	0.201	3876816.758		
	q=3	SD	0.466	2.107	4.524	1.000	✓
		GC	0.000	0.201	28955.175		
	q=4	SD	0.460	2.107	4.579	1.000	✓
		GC	0.001	0.201	203.325		
	q=5	SD	0.492	2.107	4.285	1.000	✓
		GC	0.000	0.201	698.194		
	q=6	SD	0.202	2.107	10.439	1.000	✓
		GC	0.000	0.201	2211.348		
	q=7	SD	0.151	2.107	13.968	1.000	✓
		GC	0.000	0.201	3367.270		

Table 9-63 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 4S-4B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.679888	2.107	3.099349	1.0000	✓
		GC	0.006675	0.201	30.114237		
	q=3,4,5,6,7	SD	1.052059	2.107	2.002938	1.0000	✓
		GC	0.028687	0.201	7.006796		
Perugia	q=2	SD	0.546103	2.107	3.858635	1.0000	✓
		GC	0.002354	0.201	85.392365		
	q=3,4,5,6,7	SD	0.886388	2.107	2.377301	1.0000	✓
		GC	0.01292	0.201	15.558214		
Focsani	q=2	SD	0.394108	2.107	5.34678	1.0000	✓
		GC	1.00E-06	0.201	225017.0613		
	q=3,4,5,6,7	SD	0.695644	2.107	3.029152	1.0000	✓
		GC	3.30E-05	0.201	6020.560152		

Table 9-64 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for 2S-4B-8m

Site	Case study	Limit State	λ(‰)	λ_{lim}(‰)	Margin Ratio (λ_{lim}/ λ)	Allowable Margin Ratio	Check
Athens	q=2	SD	0.203747	2.107	10.342283	1.0000	✓
		GC	0.007682	0.201	26.165883		
	q=3,4,5,6,7	SD	0.26707	2.107	7.890093	1.0000	✓
		GC	0.006958	0.201	28.887046		
Perugia	q=2	SD	0.117722	2.107	17.899829	1.0000	✓
		GC	0.002686	0.201	74.822016		
	q=3,4,5,6,7	SD	0.159264	2.107	13.230959	1.0000	✓
		GC	0.00237	0.201	84.801203		
Focsani	q=2	SD	0.05416	2.107	38.907448	1.0000	✓
		GC	2.80E-05	0.201	7153.546861		
	q=3,4,5,6,7	SD	0.086866	2.107	24.258086	1.0000	✓
		GC	1.80E-05	0.201	10879.6016		

9.9 APPENDIX-F-2

Design Behaviour Factor Verifications (Initial Assumption) for *Non-Conventional* Structures via IDA

Table 9-65 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for Bolted Beam Splices

Site	Case study	Design behaviour factor	Limit State	$\lambda_x(DS)$ (‰)	λ_{DSlim} (‰)	Margin Ratio (λ_{lim} / λ_x)	Check
Athens	2-story	4	LS	1.639	2.107	1.286	✓
			GC	0.041	0.201	4.944	
	4-story	4	LS	1.415	2.107	1.489	✓
			GC	0.048	0.201	4.176	
	8-story	4	LS	1.536	2.107	1.372	✗
			GC	0.247	0.201	0.813	
Perugia	2-story	4	LS	1.340	2.107	1.573	✓
			GC	0.018	0.201	11.307	
	4-story	4	LS	1.259	2.107	1.673	✓
			GC	0.024	0.201	8.245	
	8-story	4	LS	1.605	2.107	1.313	✓
			GC	0.192	0.201	1.047	
Focsani	2-story	4	LS	1.906	2.107	1.106	✓
			GC	0.000	0.201	1677.86	
	4-story	4	LS	0.932	2.107	2.262	✓
			GC	0.000	0.201	2134.39	
	8-story	4	LS	1.210	2.107	1.741	✓
			GC	0.018	0.201	10.954	

Table 9-66 Design Behaviour Factor Verifications via The Limit State and the Mean Annual Frequency Estimation for Welded Beam Splices

Site	Case study	Design behaviour factor	Limit State	$\lambda_x(DS)$ (‰)	$\lambda_{DS\text{lim}}$ (‰)	Margin Ratio ($\lambda_{\text{lim}} / \lambda_x$)	Check
Athens	2-story	4	LS	0.277	2.107	7.614	✓
			GC	0.020	0.201	10.255	
	4-story	4	LS	0.542	2.107	3.887	✓
			GC	0.038	0.201	5.329	
	8-story	4	LS	0.813	2.107	2.591	✓
			GC	0.196	0.201	1.028	
Perugia	2-story	4	LS	0.164	2.107	12.813	✓
			GC	0.007	0.201	27.875	
	4-story	4	LS	0.410	2.107	5.144	✓
			GC	0.017	0.201	11.507	
	8-story	4	LS	0.776	2.107	2.714	✓
			GC	0.145	0.201	1.384	
Focsani	2-story	4	LS	0.080	2.107	26.471	✓
			GC	0.000	0.201	4930.03	
	4-story	4	LS	0.141	2.107	14.935	✓
			GC	0.000	0.201	10352.61	
	8-story	4	LS	0.375	2.107	5.621	✓
			GC	0.010	0.201	20.831	

9.10 APPENDIX-G-1

Behaviour Factor Calculations for *Conventional* Structures

9.10.1 12-Storey 3Bays and 6m Length of the Bay

Table 9-67 Behaviour Factor Calculated for 12-Storey 3Bays and 6m Length of the Bay, q=2, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.5	**3.4**	**2.4**	**2.3**	**2.6**	**3.5**	---	**2.4**	**2.6**	**3.5**	**3.8**	**2.4**	**2.6**	**3.5**	**3.0**	**2.4**	**2.6**	**3.5**	---	**2.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.9	**4.1**	**2.8**	**2.8**	**3.1**	**4.3**	---	**2.9**	**3.1**	**4.3**	**4.6**	**2.9**	**3.1**	**4.3**	**3.7**	**2.9**	**3.1**	**4.3**	---	**2.9**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.0	**4.2**	**2.9**	**2.9**	**3.2**	**4.4**	---	**3.0**	**3.2**	**4.4**	**4.8**	**3.0**	**3.2**	**4.4**	**3.8**	**3.0**	**3.2**	**4.4**	---	**3.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.0	**4.2**	**2.9**	**2.9**	**3.2**	**4.4**	---	**3.0**	**3.2**	**4.4**	**4.8**	**3.0**	**3.2**	**4.4**	**3.8**	**3.0**	**3.2**	**4.4**	---	**3.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.0	**4.2**	**2.9**	**2.9**	**3.2**	**4.4**	---	**3.0**	**3.2**	**4.4**	**4.8**	**3.0**	**3.2**	**4.4**	**3.8**	**3.0**	**3.2**	**4.4**	---	**3.0**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-68 Behaviour Factor Calculated for 12-Storey 3Bays and 6m Length of the Bay, q=3, 4, q_{acc}=3.5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.6	**2.0**	**1.5**	**1.3**	**1.6**	**2.0**	---	**1.3**	**1.6**	**2.0**	**2.2**	**1.3**	**1.6**	**2.0**	**1.8**	**1.3**	**1.6**	**2.0**	---	**1.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.5	**3.2**	**2.2**	**2.1**	**2.6**	**3.3**	---	**2.2**	**2.6**	**3.3**	**3.6**	**2.2**	**2.6**	**3.3**	**2.9**	**2.2**	**2.6**	**3.3**	---	**2.2**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.7	**3.4**	**2.3**	**2.3**	**2.9**	**3.6**	---	**2.4**	**2.9**	**3.6**	**4.0**	**2.4**	**2.9**	**3.6**	**3.2**	**2.4**	**2.9**	**3.6**	---	**2.4**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.8	**3.4**	**2.3**	**2.3**	**3.0**	**3.8**	---	**2.5**	**3.0**	**3.8**	**4.1**	**2.5**	**3.0**	**3.7**	**3.3**	**2.5**	**3.0**	**3.7**	---	**2.5**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.8	**3.4**	**2.3**	**2.3**	**3.0**	**3.8**	---	**2.5**	**3.0**	**3.8**	**4.1**	**2.5**	**3.0**	**3.7**	**3.3**	**2.5**	**3.0**	**3.7**	---	**2.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-69 Behaviour Factor Calculated for 12-Storey 3Bays and 6m Length of the Bay, q=5, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.5	**3.3**	**2.5**	**2.4**	**2.6**	**3.4**	---	**2.4**	**2.6**	**3.4**	**3.8**	**2.4**	**2.6**	**3.4**	**3.0**	**2.4**	**2.6**	**3.4**	---	**2.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.0	**3.9**	**2.9**	**2.8**	**3.1**	**4.0**	---	**2.9**	**3.1**	**4.0**	**4.5**	**2.9**	**3.1**	**4.0**	**3.6**	**2.9**	**3.1**	**4.0**	---	**2.9**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.1	**4.0**	**2.9**	**2.9**	**3.2**	**4.2**	---	**3.0**	**3.2**	**4.2**	**4.8**	**3.0**	**3.2**	**4.2**	**3.8**	**3.0**	**3.2**	**4.2**	---	**3.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.1	**4.0**	**2.9**	**2.9**	**3.2**	**4.2**	---	**3.0**	**3.2**	**4.2**	**4.8**	**3.0**	**3.2**	**4.2**	**3.8**	**3.0**	**3.2**	**4.2**	---	**3.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.1	**4.0**	**2.9**	**2.9**	**3.2**	**4.2**	---	**3.0**	**3.2**	**4.2**	**4.8**	**3.0**	**3.2**	**4.2**	**3.8**	**3.0**	**3.2**	**4.2**	---	**3.0**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-70 Behaviour Factor Calculated for 12-Storey 3Bays and 6m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.1	**3.7**	**2.8**	**2.8**	**3.7**	**4.4**	---	**3.3**	**3.7**	**4.4**	**4.9**	**3.3**	**3.7**	**4.4**	**3.9**	**3.3**	**3.7**	**4.4**	---	**3.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.8	**4.5**	**3.4**	**3.4**	**4.6**	**5.5**	---	**4.1**	**4.6**	**5.5**	**6.1**	**4.1**	**4.6**	**5.5**	**4.9**	**4.1**	**4.6**	**5.5**	---	**4.1**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.0	**4.8**	**3.6**	**3.6**	**5.2**	**6.1**	---	**4.6**	**5.2**	**6.1**	**6.8**	**4.6**	**5.2**	**6.1**	**5.4**	**4.6**	**5.2**	**6.1**	---	**4.6**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.1	**4.9**	**3.6**	**3.6**	**5.7**	**6.8**	---	**5.0**	**5.7**	**6.7**	**7.5**	**5.0**	**5.6**	**6.7**	**6.0**	**5.0**	**5.7**	**6.7**	---	**5.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.1	**4.8**	**3.5**	**3.6**	**6.3**	**7.5**	---	**5.6**	**6.1**	**7.3**	**8.1**	**5.4**	**6.1**	**7.3**	**6.5**	**5.4**	**6.1**	**7.3**	---	**5.4**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-71 Behaviour Factor Calculated for 12-Storey 3Bays and 6m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.2	4.3	2.8	2.7	4.2	5.6	---	3.5	4.2	5.6	5.0	3.5	4.1	5.4	3.9	3.4	4.2	5.6	---	3.5
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.9	5.2	3.4	3.3	5.2	7.0	---	4.4	5.2	7.0	6.3	4.4	5.1	6.7	4.9	4.2	5.3	7.0	---	4.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.3	5.7	3.6	3.6	6.1	8.1	---	5.1	6.1	8.1	7.3	5.1	5.9	7.9	5.7	4.9	6.1	8.1	---	5.1
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.4	5.8	3.6	3.7	6.8	9.1	---	5.7	6.8	9.0	8.1	5.7	6.6	8.7	6.3	5.5	6.8	9.0	---	5.7
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.4	5.8	3.6	3.7	6.9	9.1	---	5.7	6.8	9.1	8.2	5.7	6.6	8.8	6.4	5.5	6.9	9.1	---	5.7
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.2 8-Storey 3Bays and 6m Length of the Bay

Table 9-72 Behaviour Factor Calculated for 8-Storey 3Bays and 6m Length of the Bay, q=2, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.5	3.3	2.4	2.1	2.7	3.5	---	2.2	2.7	3.5	3.5	2.2	2.6	3.4	2.7	2.1	2.7	3.6	---	2.2
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.7	4.8	3.3	3.0	4.1	5.3	---	3.3	4.1	5.3	5.2	3.3	3.9	5.1	4.0	3.2	4.1	5.3	---	3.3
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.8	4.9	3.3	3.1	4.2	5.5	---	3.4	4.2	5.5	5.4	3.4	4.1	5.3	4.2	3.3	4.2	5.5	---	3.4
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.8	4.9	3.3	3.1	4.2	5.5	---	3.4	4.2	5.5	5.5	3.4	4.1	5.4	4.2	3.3	4.2	5.6	---	3.4
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.8	4.9	3.2	3.1	4.3	5.6	---	3.5	4.3	5.6	5.5	3.5	4.1	5.4	4.2	3.3	4.3	5.6	---	3.5
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-73 Behaviour Factor Calculated for 8-Storey 3Bays and 6m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.5	**1.6**	**1.3**	**1.1**	**1.6**	**1.6**	---	**1.2**	**1.6**	**1.6**	**1.9**	**1.2**	**1.6**	**1.6**	**1.5**	**1.2**	**1.6**	**1.6**	---	**1.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	1.9	**1.9**	**1.5**	**1.4**	**1.9**	**2.0**	---	**1.4**	**1.9**	**2.0**	**2.4**	**1.4**	**1.9**	**2.0**	**1.9**	**1.4**	**1.9**	**2.0**	---	**1.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.2	**2.2**	**1.7**	**1.6**	**2.3**	**2.3**	---	**1.7**	**2.3**	**2.3**	**2.8**	**1.7**	**2.3**	**2.3**	**2.3**	**1.7**	**2.3**	**2.3**	---	**1.7**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.4	**2.4**	**1.7**	**1.8**	**2.7**	**2.8**	---	**2.0**	**2.7**	**2.7**	**3.3**	**2.0**	**2.7**	**2.7**	**2.6**	**2.0**	**2.7**	**2.7**	---	**2.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.5	**2.5**	**1.8**	**1.8**	**3.2**	**3.2**	---	**2.4**	**3.1**	**3.1**	**3.7**	**2.3**	**3.1**	**3.1**	**3.0**	**2.3**	**3.1**	**3.1**	---	**2.3**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-74 Behaviour Factor Calculated for 8-Storey 3Bays and 6m Length of the Bay, q=4, 5 q_{acc}=4.5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.3	**1.5**	**1.3**	**1.2**	**1.3**	**1.6**	---	**1.2**	**1.3**	**1.6**	**1.9**	**1.2**	**1.3**	**1.6**	**1.5**	**1.2**	**1.3**	**1.6**	---	**1.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	1.5	**1.8**	**1.5**	**1.3**	**1.6**	**1.8**	---	**1.4**	**1.6**	**1.8**	**2.3**	**1.4**	**1.6**	**1.8**	**1.8**	**1.4**	**1.6**	**1.8**	---	**1.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	1.7	**2.0**	**1.6**	**1.5**	**1.8**	**2.1**	---	**1.6**	**1.8**	**2.1**	**2.6**	**1.6**	**1.8**	**2.1**	**2.1**	**1.6**	**1.8**	**2.1**	---	**1.6**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	1.8	**2.1**	**1.6**	**1.6**	**2.1**	**2.4**	---	**1.8**	**2.0**	**2.4**	**2.9**	**1.8**	**2.0**	**2.4**	**2.3**	**1.8**	**2.0**	**2.4**	---	**1.8**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	1.9	**2.2**	**1.7**	**1.7**	**2.4**	**2.8**	---	**2.1**	**2.3**	**2.6**	**3.3**	**2.0**	**2.3**	**2.6**	**2.6**	**2.0**	**2.3**	**2.6**	---	**2.0**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-75 Behaviour Factor Calculated for 8-Storey 3Bays and 6m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.8	**2.4**	**1.6**	**1.4**	**1.8**	**2.5**	---	**1.4**	**1.8**	**2.5**	**2.3**	**1.4**	**1.8**	**2.5**	**1.8**	**1.4**	**1.8**	**2.5**	---	**1.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.3	**4.6**	**2.8**	**2.6**	**3.6**	**4.9**	---	**2.8**	**3.6**	**4.9**	**4.5**	**2.8**	**3.6**	**4.9**	**3.6**	**2.8**	**3.6**	**4.9**	---	**2.8**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.5	**4.8**	**2.8**	**2.7**	**3.9**	**5.4**	---	**3.0**	**3.9**	**5.4**	**4.9**	**3.0**	**3.9**	**5.4**	**3.9**	**3.0**	**3.9**	**5.4**	---	**3.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.5	**4.8**	**2.8**	**2.7**	**4.0**	**5.5**	---	**3.1**	**4.0**	**5.5**	**5.0**	**3.1**	**4.0**	**5.5**	**4.0**	**3.1**	**4.0**	**5.5**	---	**3.1**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.5	**4.8**	**2.7**	**2.7**	**4.1**	**5.6**	---	**3.1**	**4.0**	**5.6**	**5.1**	**3.1**	**4.0**	**5.5**	**4.0**	**3.1**	**4.0**	**5.5**	---	**3.1**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-76 Behaviour Factor Calculated for 8-Storey 3Bays and 6m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.5	**3.9**	**2.2**	**1.9**	**2.7**	**4.2**	---	**2.0**	**2.7**	**4.2**	**3.3**	**2.0**	**2.6**	**4.1**	**2.6**	**2.0**	**2.7**	**4.2**	---	**2.0**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.3	**5.1**	**2.8**	**2.5**	**3.5**	**5.5**	---	**2.7**	**3.5**	**5.5**	**4.3**	**2.7**	**3.5**	**5.5**	**3.4**	**2.7**	**3.5**	**5.5**	---	**2.7**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.5	**5.4**	**2.8**	**2.7**	**3.9**	**6.0**	---	**3.0**	**3.9**	**6.0**	**4.7**	**3.0**	**3.8**	**6.0**	**3.7**	**2.9**	**3.9**	**6.0**	---	**3.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.6	**5.6**	**2.7**	**2.7**	**4.2**	**6.6**	---	**3.3**	**4.2**	**6.6**	**5.1**	**3.2**	**4.1**	**6.5**	**4.1**	**3.2**	**4.2**	**6.6**	---	**3.2**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.6	**5.6**	**2.7**	**2.8**	**4.6**	**7.3**	---	**3.6**	**4.5**	**7.1**	**5.6**	**3.5**	**4.5**	**7.0**	**4.4**	**3.4**	**4.5**	**7.1**	---	**3.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.3 4-Storey 3Bays and 6m Length of the Bay

Table 9-77 Behaviour Factor Calculated for 4-Storey 3Bays and 6m Length of the Bay, q=2, q_{acc}=2

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.4	2.7	2.3	2.0	2.5	2.7	---	2.1	2.5	2.7	3.4	2.1	2.5	2.7	2.7	2.1	2.5	2.7	---	2.1
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.8	3.1	2.6	2.4	2.9	3.2	---	2.4	2.9	3.2	4.0	2.4	2.9	3.2	3.2	2.4	2.9	3.2	---	2.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.8	3.1	2.7	2.4	2.9	3.3	---	2.5	2.9	3.3	4.0	2.5	2.9	3.3	3.2	2.5	2.9	3.2	---	2.5
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.8	3.2	2.6	2.4	3.0	3.3	---	2.5	3.0	3.3	4.1	2.5	3.0	3.3	3.3	2.5	3.0	3.3	---	2.5
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.8	3.2	2.6	2.4	3.0	3.3	---	2.6	3.0	3.3	4.1	2.5	3.0	3.3	3.3	2.5	3.0	3.3	---	2.5
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-78 Behaviour Factor Calculated for 4-Storey 3Bays and 6m Length of the Bay, q=3, 4, 5, 6, 7, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.3	1.9	1.3	1.2	1.3	1.9	---	1.2	1.3	1.9	2.0	1.2	1.3	1.9	1.6	1.2	1.3	1.9	---	1.2
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.8	4.0	2.7	2.5	2.8	4.1	---	2.5	2.8	4.1	4.2	2.5	2.8	4.1	3.3	2.5	2.8	4.0	---	2.5
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.8	4.0	2.7	2.5	2.9	4.2	---	2.6	2.9	4.2	4.3	2.6	2.9	4.2	3.4	2.6	2.9	4.1	---	2.6
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.8	4.0	2.7	2.5	3.0	4.2	---	2.6	2.9	4.2	4.3	2.6	2.9	4.2	3.5	2.6	2.9	4.2	---	2.6
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.8	4.0	2.7	2.5	3.0	4.2	---	2.6	2.9	4.2	4.3	2.6	2.9	4.2	3.5	2.6	2.9	4.2	---	2.6
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.4 2-Storey 3Bays and 6m Length of the Bay

Table 9-79 Behaviour Factor Calculated for 2-Storey 3Bays and 6m Length of the Bay, q=2, 3, 4, 5, 6, 7, q_{acc}=4.5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.7	**3.8**	**3.3**	**3.3**	**4.2**	**4.2**	**---**	**3.7**	**4.2**	**4.2**	**5.9**	**3.7**	**4.2**	**4.2**	**4.7**	**3.7**	**4.1**	**4.2**	**---**	**3.7**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.9	**3.9**	**3.4**	**3.4**	**4.3**	**4.4**	**---**	**3.8**	**4.3**	**4.4**	**6.1**	**3.8**	**4.3**	**4.4**	**4.9**	**3.8**	**4.3**	**4.4**	**---**	**3.8**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.9	**4.0**	**3.5**	**3.5**	**4.5**	**4.6**	**---**	**4.0**	**4.5**	**4.6**	**6.3**	**4.0**	**4.5**	**4.6**	**5.0**	**4.0**	**4.5**	**4.5**	**---**	**4.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.0	**4.0**	**3.5**	**3.5**	**4.6**	**4.7**	**---**	**4.0**	**4.6**	**4.6**	**6.4**	**4.0**	**4.6**	**4.6**	**5.1**	**4.0**	**4.5**	**4.6**	**---**	**4.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.0	**4.0**	**3.5**	**3.5**	**4.7**	**4.8**	**---**	**4.1**	**4.7**	**4.7**	**6.5**	**4.1**	**4.7**	**4.7**	**5.2**	**4.1**	**4.6**	**4.7**	**---**	**4.1**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.5 12-Storey 3Bays and 8m Length of the Bay

Table 9-80 Behaviour Factor Calculated for 12-Storey 3Bays and 8m Length of the Bay, q=2, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.9	4.4	3.7	3.6	4.2	4.6	---	3.8	4.2	4.6	6.0	3.8	4.2	4.6	4.8	3.8	4.1	4.6	---	3.8
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.6	5.1	4.3	4.2	5.0	5.5	---	4.6	5.0	5.5	7.2	4.6	5.0	5.5	5.7	4.6	5.0	5.5	---	4.5
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.8	5.3	4.4	4.4	5.5	6.1	---	5.0	5.5	6.1	7.8	5.0	5.5	6.1	6.3	5.0	5.4	6.0	---	5.0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.8	5.3	4.3	4.4	5.9	6.5	---	5.4	5.8	6.5	8.4	5.3	5.8	6.5	6.7	5.3	5.8	6.4	---	5.3
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.7	5.2	4.1	4.3	6.3	7.0	---	5.8	6.2	6.9	8.9	5.7	6.2	6.9	7.1	5.7	6.2	6.8	---	5.6
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-81 Behaviour Factor Calculated for 12-Storey 3Bays and 8m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.9	2.0	1.8	1.7	1.9	2.1	---	1.8	1.9	2.1	2.9	1.8	1.9	2.1	2.3	1.8	1.9	2.1	---	1.8
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.0	3.3	2.9	2.9	3.2	3.5	---	3.1	3.2	3.5	4.9	3.1	3.2	3.5	3.9	3.1	3.3	3.6	---	3.1
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.1	3.4	2.9	2.9	3.4	3.7	---	3.2	3.4	3.7	5.1	3.2	3.4	3.7	4.1	3.2	3.4	3.7	---	3.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.2	3.5	3.0	3.0	3.6	4.0	---	3.4	3.6	3.9	5.4	3.4	3.6	3.9	4.3	3.4	3.6	3.9	---	3.4
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.2	3.5	2.9	3.0	3.8	4.1	---	3.6	3.7	4.1	5.6	3.5	3.7	4.1	4.5	3.5	3.8	4.1	---	3.5
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-82 Behaviour Factor Calculated for 12-Storey 3Bays and 8m Length of the Bay, q=4, q_{acc}=4

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.4	2.7	2.3	2.2	2.6	2.9	---	2.4	2.6	2.9	3.7	2.4	2.6	2.9	2.9	2.4	2.6	2.9	---	2.4
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.4	3.8	3.2	3.1	3.6	4.1	---	3.4	3.6	4.1	5.2	3.4	3.6	4.1	4.2	3.4	3.6	4.1	---	3.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.5	4.0	3.2	3.2	3.9	4.4	---	3.6	3.9	4.4	5.5	3.6	3.9	4.4	4.4	3.6	3.9	4.4	---	3.6
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.5	4.0	3.3	3.3	4.1	4.7	---	3.8	4.1	4.7	5.9	3.8	4.1	4.7	4.7	3.8	4.1	4.7	---	3.8
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.5	4.0	3.2	3.3	4.2	4.8	---	3.9	4.2	4.7	6.0	3.9	4.2	4.7	4.8	3.9	4.2	4.8	---	3.9
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-83 Behaviour Factor Calculated for 12-Storey 3Bays and 8m Length of the Bay, q=5, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.6	3.8	3.1	3.1	4.4	4.8	---	3.8	4.4	4.8	5.6	3.8	4.4	4.7	4.4	3.8	4.4	4.8	---	3.8
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.4	4.7	3.8	3.8	5.6	6.0	---	4.8	5.6	6.0	7.1	4.8	5.6	6.0	5.6	4.8	5.6	6.1	---	4.8
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.7	5.0	4.0	4.0	6.4	6.9	---	5.5	6.4	6.9	8.0	5.5	6.4	6.8	6.4	5.4	6.4	6.9	---	5.5
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.7	5.1	4.0	4.1	7.1	7.6	---	6.1	7.0	7.6	8.9	6.0	7.0	7.5	7.1	6.0	7.1	7.6	---	6.1
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.6	5.0	3.8	4.0	7.8	8.4	---	6.7	7.7	8.2	9.7	6.6	7.6	8.2	7.7	6.5	7.7	8.3	---	6.6
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-84 Behaviour Factor Calculated for 12-Storey 3Bays and 8m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.0	**4.4**	**3.3**	**3.2**	**5.4**	**5.9**	---	**4.3**	**5.4**	**5.9**	**6.2**	**4.3**	**5.3**	**5.8**	**4.8**	**4.2**	**5.4**	**5.9**	---	**4.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.6	**6.2**	**4.6**	**4.5**	**7.5**	**8.3**	---	**6.0**	**7.5**	**8.3**	**8.6**	**6.0**	**7.4**	**8.1**	**6.8**	**5.9**	**7.6**	**8.3**	---	**6.1**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.6	**6.2**	**4.6**	**4.5**	**7.5**	**8.3**	---	**6.0**	**7.5**	**8.3**	**8.6**	**6.0**	**7.4**	**8.1**	**6.8**	**5.9**	**7.6**	**8.3**	---	**6.1**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.6	**6.2**	**4.6**	**4.5**	**7.5**	**8.3**	---	**6.0**	**7.5**	**8.3**	**8.6**	**6.0**	**7.4**	**8.1**	**6.8**	**5.9**	**7.6**	**8.3**	---	**6.1**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.6	**6.2**	**4.6**	**4.5**	**7.5**	**8.3**	---	**6.0**	**7.5**	**8.3**	**8.6**	**6.0**	**7.4**	**8.1**	**6.8**	**5.9**	**7.6**	**8.3**	---	**6.1**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-85 Behaviour Factor Calculated for 12-Storey 3Bays and 8m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.1	**4.7**	**3.1**	**2.8**	**7.0**	**8.0**	---	**4.8**	**6.8**	**7.8**	**6.4**	**4.7**	**4.3**	**4.9**	**3.2**	**2.9**	**7.0**	**8.0**	---	**4.8**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	6.3	**7.2**	**4.6**	**4.3**	**11.0**	**12.6**	---	**7.6**	**10.6**	**12.1**	**10.0**	**7.3**	**6.7**	**7.6**	**5.0**	**4.6**	**10.9**	**12.6**	---	**7.5**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	6.3	**7.2**	**4.6**	**4.3**	**11.0**	**12.6**	---	**7.6**	**10.6**	**12.1**	**10.0**	**7.3**	**6.7**	**7.6**	**5.0**	**4.6**	**10.9**	**12.6**	---	**7.5**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	6.3	**7.2**	**4.6**	**4.3**	**11.0**	**12.6**	---	**7.6**	**10.6**	**12.1**	**10.0**	**7.3**	**6.7**	**7.6**	**5.0**	**4.6**	**10.9**	**12.6**	---	**7.5**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	6.3	**7.2**	**4.6**	**4.3**	**11.0**	**12.6**	---	**7.6**	**10.6**	**12.1**	**10.0**	**7.3**	**6.7**	**7.6**	**5.0**	**4.6**	**10.9**	**12.6**	---	**7.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.6 8-Storey 3Bays and 8m Length of the Bay

Table 9-86 Behaviour Factor Calculated for 8-Storey 3Bays and 8m Length of the Bay, q=2, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.5	3.7	2.3	2.2	4.3	4.6	---	2.8	4.3	4.6	4.1	2.8	3.8	4.1	2.9	2.4	4.3	4.6	---	2.8
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	6.3	6.7	4.1	4.0	8.4	9.0	---	5.4	8.3	8.9	7.9	5.4	7.3	7.8	5.6	4.7	8.3	8.9	---	5.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	6.7	7.2	4.3	4.3	9.7	10.4	---	6.2	9.6	10.3	9.2	6.2	8.4	9.1	6.5	5.5	9.6	10.3	---	6.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	6.7	7.2	4.3	4.4	10.0	10.7	---	6.5	9.9	10.6	9.5	6.4	8.7	9.4	6.7	5.6	9.9	10.6	---	6.4
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	6.7	7.2	4.2	4.3	10.1	10.8	---	6.5	10.0	10.7	9.5	6.5	8.8	9.4	6.7	5.7	10.0	10.7	---	6.4
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-87 Behaviour Factor Calculated for 8-Storey 3Bays and 8m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.2	2.1	1.9	1.6	2.4	2.2	---	1.7	2.4	2.2	2.7	1.7	2.3	2.1	2.0	1.6	2.4	2.3	---	1.7
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.7	3.5	2.9	2.6	4.2	3.9	---	2.9	4.2	3.9	4.7	2.9	4.0	3.7	3.6	2.8	4.2	3.9	---	3.0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.7	3.5	3.0	2.6	4.2	3.9	---	2.9	4.2	3.9	4.7	2.9	4.0	3.8	3.6	2.8	4.2	4.0	---	3.0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.8	3.5	2.9	2.6	4.3	4.0	---	3.0	4.3	4.0	4.8	3.0	4.1	3.8	3.6	2.9	4.3	4.0	---	3.0
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.8	3.5	2.9	2.6	4.3	4.0	---	3.0	4.3	4.0	4.8	3.0	4.1	3.9	3.7	2.9	4.4	4.1	---	3.1
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-88 Behaviour Factor Calculated for 8-Storey 3Bays and 8m Length of the Bay, q=4, q_{acc}=4

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.8	**2.0**	**1.6**	**1.3**	**1.9**	**2.1**	---	**1.4**	**1.9**	**2.1**	**2.3**	**1.4**	**1.9**	**2.0**	**1.8**	**1.4**	**1.9**	**2.1**	---	**1.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.9	**3.2**	**2.4**	**2.1**	**3.2**	**3.5**	---	**2.3**	**3.2**	**3.5**	**3.8**	**2.3**	**3.1**	**3.4**	**2.9**	**2.3**	**3.2**	**3.5**	---	**2.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.4	**3.8**	**2.5**	**2.5**	**4.0**	**4.4**	---	**3.0**	**4.0**	**4.4**	**4.8**	**3.0**	**4.0**	**4.3**	**3.7**	**2.9**	**4.1**	**4.5**	---	**3.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.6	**3.9**	**2.6**	**2.6**	**4.6**	**5.0**	---	**3.3**	**4.5**	**5.0**	**5.3**	**3.3**	**4.4**	**4.9**	**4.2**	**3.2**	**4.6**	**5.0**	---	**3.3**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.6	**3.9**	**2.6**	**2.6**	**4.7**	**5.1**	---	**3.4**	**4.6**	**5.1**	**5.4**	**3.4**	**4.5**	**5.0**	**4.3**	**3.3**	**4.6**	**5.1**	---	**3.4**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-89 Behaviour Factor Calculated for 8-Storey 3Bays and 8m Length of the Bay, q=5, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.8	**1.9**	**1.6**	**1.3**	**1.9**	**2.0**	---	**1.4**	**1.9**	**2.0**	**2.3**	**1.4**	**1.9**	**2.0**	**1.8**	**1.4**	**2.0**	**2.1**	---	**1.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.3	**3.4**	**2.7**	**2.4**	**3.6**	**3.8**	---	**2.6**	**3.6**	**3.8**	**4.2**	**2.6**	**3.5**	**3.7**	**3.3**	**2.6**	**3.7**	**3.8**	---	**2.6**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.8	**4.0**	**2.8**	**2.8**	**4.6**	**4.8**	---	**3.3**	**4.6**	**4.8**	**5.3**	**3.3**	**4.4**	**4.6**	**4.1**	**3.2**	**4.6**	**4.8**	---	**3.3**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.8	**4.0**	**2.8**	**2.8**	**4.7**	**4.9**	---	**3.4**	**4.7**	**4.9**	**5.5**	**3.4**	**4.6**	**4.8**	**4.3**	**3.3**	**4.7**	**4.9**	---	**3.4**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.8	**4.0**	**2.7**	**2.8**	**4.8**	**5.0**	---	**3.5**	**4.8**	**5.0**	**5.6**	**3.5**	**4.7**	**4.9**	**4.3**	**3.4**	**4.8**	**5.0**	---	**3.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-90 Behaviour Factor Calculated for 8-Storey 3Bays and 8m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.9	**3.1**	**2.2**	**1.9**	**3.3**	**3.5**	---	**2.2**	**3.3**	**3.5**	**3.4**	**2.2**	**3.0**	**3.2**	**2.5**	**2.0**	**3.3**	**3.6**	---	**2.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.4	**5.8**	**3.7**	**3.7**	**6.7**	**7.2**	---	**4.5**	**6.7**	**7.2**	**7.0**	**4.5**	**6.1**	**6.6**	**5.1**	**4.1**	**6.8**	**7.3**	---	**4.6**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.9	**6.4**	**3.9**	**4.0**	**8.1**	**8.7**	---	**5.5**	**8.1**	**8.7**	**8.4**	**5.5**	**7.4**	**7.9**	**6.1**	**5.0**	**8.2**	**8.8**	---	**5.6**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	6.0	**6.4**	**3.9**	**4.1**	**8.6**	**9.2**	---	**5.8**	**8.5**	**9.1**	**8.9**	**5.8**	**7.8**	**8.3**	**6.5**	**5.2**	**8.7**	**9.3**	---	**5.8**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	6.0	**6.4**	**3.9**	**4.1**	**8.6**	**9.2**	---	**5.8**	**8.5**	**9.1**	**8.9**	**5.8**	**7.8**	**8.3**	**6.5**	**5.3**	**8.7**	**9.3**	---	**5.9**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-91 Behaviour Factor Calculated for 8-Storey 3Bays and 8m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.1	**3.2**	**2.1**	**1.9**	**3.6**	**3.9**	---	**2.3**	**3.6**	**3.8**	**3.5**	**2.3**	**3.2**	**3.4**	**2.5**	**2.0**	**3.6**	**3.8**	---	**2.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	6.9	**7.3**	**4.5**	**4.4**	**8.8**	**9.3**	---	**5.6**	**8.8**	**9.3**	**8.4**	**5.6**	**7.8**	**8.2**	**6.0**	**4.9**	**8.7**	**9.2**	---	**5.5**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	7.1	**7.5**	**4.6**	**4.5**	**9.4**	**9.9**	---	**6.0**	**9.4**	**9.9**	**9.0**	**5.9**	**8.3**	**8.8**	**6.4**	**5.3**	**9.3**	**9.9**	---	**5.9**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	7.1	**7.5**	**4.6**	**4.5**	**9.4**	**9.9**	---	**6.0**	**9.4**	**9.9**	**9.0**	**5.9**	**8.3**	**8.8**	**6.4**	**5.3**	**9.3**	**9.9**	---	**5.9**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	7.1	**7.5**	**4.6**	**4.5**	**9.4**	**9.9**	---	**6.0**	**9.4**	**9.9**	**9.0**	**5.9**	**8.3**	**8.8**	**6.4**	**5.3**	**9.3**	**9.9**	---	**5.9**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.7 4-Storey 3Bays and 8m Length of the Bay

Table 9-92 Behaviour Factor Calculated for 4-Storey 3Bays and 8m Length of the Bay, q=2, qacc=2

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.1	**3.5**	**2.6**	**2.1**	**3.4**	**3.8**	---	**2.3**	**3.4**	**3.8**	**3.6**	**2.3**	**3.2**	**3.6**	**2.7**	**2.2**	**3.4**	**3.8**	---	**2.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.3	**5.9**	**4.2**	**3.6**	**5.8**	**6.5**	---	**3.9**	**5.7**	**6.4**	**6.2**	**3.9**	**5.5**	**6.1**	**4.7**	**3.7**	**5.8**	**6.4**	---	**3.9**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.3	**6.0**	**4.2**	**3.6**	**5.9**	**6.6**	---	**3.9**	**5.9**	**6.6**	**6.3**	**3.9**	**5.6**	**6.2**	**4.8**	**3.7**	**5.9**	**6.6**	---	**3.9**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.4	**6.0**	**4.1**	**3.6**	**6.0**	**6.7**	---	**4.0**	**6.0**	**6.7**	**6.4**	**4.0**	**5.7**	**6.3**	**4.8**	**3.8**	**6.0**	**6.7**	---	**4.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.3	**6.0**	**3.8**	**3.6**	**6.1**	**6.8**	---	**4.1**	**6.0**	**6.8**	**6.5**	**4.1**	**5.7**	**6.4**	**4.9**	**3.9**	**6.1**	**6.8**	---	**4.1**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-93 Behaviour Factor Calculated for 4-Storey 3Bays and 8m Length of the Bay, q=3, 4, 5, 6, 7, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.9	**2.5**	**1.7**	**1.5**	**2.0**	**2.6**	---	**1.6**	**2.0**	**2.6**	**2.5**	**1.6**	**2.0**	**2.6**	**2.0**	**1.6**	**2.0**	**2.6**	---	**1.6**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.1	**4.1**	**2.6**	**2.4**	**3.4**	**4.5**	---	**2.7**	**3.4**	**4.5**	**4.3**	**2.7**	**3.4**	**4.5**	**3.4**	**2.7**	**3.4**	**4.5**	---	**2.7**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.2	**4.1**	**2.6**	**2.5**	**3.5**	**4.5**	---	**2.7**	**3.5**	**4.5**	**4.4**	**2.7**	**3.5**	**4.5**	**3.5**	**2.7**	**3.5**	**4.5**	---	**2.7**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.2	**4.2**	**2.6**	**2.5**	**3.6**	**4.7**	---	**2.8**	**3.6**	**4.6**	**4.4**	**2.8**	**3.5**	**4.6**	**3.5**	**2.8**	**3.5**	**4.6**	---	**2.8**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.2	**4.2**	**2.6**	**2.5**	**3.6**	**4.8**	---	**2.8**	**3.6**	**4.7**	**4.5**	**2.8**	**3.6**	**4.7**	**3.6**	**2.8**	**3.6**	**4.7**	---	**2.8**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.8 2-Storey 3Bays and 8m Length of the Bay

Table 9-94 Behaviour Factor Calculated for 2-Storey 3Bays and 8m Length of the Bay, q=2, q_{acc}=2

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.3	**4.5**	**3.4**	**3.2**	**5.0**	**5.2**	---	**3.7**	**5.0**	**5.2**	**5.8**	**3.7**	**4.9**	**5.1**	**4.5**	**3.6**	**5.0**	**5.2**	---	**3.7**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.5	**4.7**	**3.5**	**3.3**	**5.2**	**5.5**	---	**3.9**	**5.2**	**5.5**	**6.0**	**3.9**	**5.1**	**5.4**	**4.7**	**3.8**	**5.2**	**5.5**	---	**3.9**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.5	**4.8**	**3.6**	**3.4**	**5.4**	**5.7**	---	**4.0**	**5.4**	**5.7**	**6.2**	**4.0**	**5.3**	**5.5**	**4.9**	**4.0**	**5.4**	**5.7**	---	**4.1**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.6	**4.8**	**3.5**	**3.4**	**5.5**	**5.8**	---	**4.2**	**5.5**	**5.8**	**6.4**	**4.1**	**5.4**	**5.7**	**5.0**	**4.1**	**5.5**	**5.8**	---	**4.2**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.5	**4.8**	**3.5**	**3.4**	**5.7**	**6.0**	---	**4.3**	**5.7**	**6.0**	**6.6**	**4.3**	**5.6**	**5.8**	**5.2**	**4.2**	**5.7**	**6.0**	---	**4.3**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-95 Behaviour Factor Calculated for 2-Storey 3Bays and 8m Length of the Bay, q=3, 4, 5, 6, 7, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.0	**4.2**	**3.5**	**3.4**	**4.5**	**4.7**	---	**3.7**	**4.5**	**4.7**	**5.9**	**3.7**	**4.5**	**4.7**	**4.7**	**3.7**	**4.4**	**4.6**	---	**3.7**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.2	**4.3**	**3.7**	**3.5**	**4.6**	**4.8**	---	**3.9**	**4.6**	**4.8**	**6.1**	**3.9**	**4.6**	**4.8**	**4.9**	**3.9**	**4.6**	**4.8**	---	**3.9**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.2	**4.4**	**3.7**	**3.6**	**4.8**	**5.0**	---	**4.0**	**4.8**	**5.0**	**6.3**	**4.0**	**4.8**	**5.0**	**5.1**	**4.0**	**4.8**	**5.0**	---	**4.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.3	**4.5**	**3.7**	**3.6**	**5.0**	**5.2**	---	**4.2**	**4.9**	**5.2**	**6.5**	**4.1**	**4.9**	**5.2**	**5.2**	**4.1**	**4.9**	**5.2**	---	**4.1**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.3	**4.4**	**3.6**	**3.6**	**5.1**	**5.3**	---	**4.2**	**5.0**	**5.2**	**6.6**	**4.2**	**5.0**	**5.2**	**5.3**	**4.2**	**5.0**	**5.2**	---	**4.2**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.9 12-Storey 4Bays and 6m Length of the Bay

Table 9-96 Behaviour Factor Calculated for 12-Storey 4Bays and 6m Length of the Bay, q=2, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.6	**2.9**	**2.5**	**2.5**	**2.8**	**3.0**	---	**2.6**	**2.8**	**3.0**	**4.1**	**2.6**	**2.8**	**3.0**	**3.3**	**2.6**	**2.8**	**3.0**	---	**2.6**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.8	**3.0**	**2.7**	**2.6**	**2.8**	**3.1**	---	**2.7**	**2.8**	**3.1**	**4.2**	**2.7**	**2.8**	**3.1**	**3.3**	**2.7**	**2.9**	**3.1**	---	**2.7**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.8	**3.0**	**2.7**	**2.6**	**2.8**	**3.1**	---	**2.7**	**2.8**	**3.1**	**4.2**	**2.7**	**2.8**	**3.1**	**3.3**	**2.7**	**2.9**	**3.1**	---	**2.7**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.8	**3.0**	**2.7**	**2.6**	**2.8**	**3.1**	---	**2.7**	**2.8**	**3.1**	**4.2**	**2.7**	**2.8**	**3.1**	**3.3**	**2.7**	**2.9**	**3.1**	---	**2.7**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.8	**3.0**	**2.7**	**2.6**	**2.8**	**3.1**	---	**2.7**	**2.8**	**3.1**	**4.2**	**2.7**	**2.8**	**3.1**	**3.3**	**2.7**	**2.9**	**3.1**	---	**2.7**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-97 Behaviour Factor Calculated for 12-Storey 4Bays and 6m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.4	**2.8**	**2.2**	**2.2**	**2.5**	**3.0**	---	**2.3**	**2.5**	**3.0**	**3.6**	**2.3**	**2.5**	**3.0**	**2.9**	**2.3**	**2.5**	**3.0**	---	**2.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.5	**2.9**	**2.3**	**2.3**	**2.6**	**3.0**	---	**2.4**	**2.6**	**3.0**	**3.8**	**2.4**	**2.6**	**3.0**	**3.0**	**2.4**	**2.6**	**3.0**	---	**2.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.5	**2.9**	**2.3**	**2.3**	**2.6**	**3.0**	---	**2.4**	**2.6**	**3.0**	**3.8**	**2.4**	**2.6**	**3.0**	**3.0**	**2.4**	**2.6**	**3.0**	---	**2.4**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.5	**2.9**	**2.3**	**2.3**	**2.6**	**3.0**	---	**2.4**	**2.6**	**3.0**	**3.8**	**2.4**	**2.6**	**3.0**	**3.0**	**2.4**	**2.6**	**3.0**	---	**2.4**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.5	**2.9**	**2.3**	**2.3**	**2.6**	**3.0**	---	**2.4**	**2.6**	**3.0**	**3.8**	**2.4**	**2.6**	**3.0**	**3.0**	**2.4**	**2.6**	**3.0**	---	**2.4**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-98 Behaviour Factor Calculated for 12-Storey 4Bays and 6m Length of the Bay, q=4, q_{acc}=4

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.6	3.1	3.4	2.4	2.5	3.1	---	2.3	2.5	3.1	3.8	2.3	2.5	3.1	3.0	2.3	2.6	3.1	---	2.3
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.6	3.2	3.5	2.4	2.6	3.1	---	2.3	2.6	3.1	3.8	2.3	2.6	3.1	3.1	2.3	2.6	3.1	---	2.3
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.6	3.2	3.5	2.4	2.6	3.1	---	2.3	2.6	3.1	3.8	2.3	2.6	3.1	3.1	2.3	2.6	3.1	---	2.3
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.6	3.2	3.5	2.4	2.6	3.1	---	2.3	2.6	3.1	3.8	2.3	2.6	3.1	3.1	2.3	2.6	3.1	---	2.3
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.6	3.2	3.5	2.4	2.6	3.1	---	2.3	2.6	3.1	3.8	2.3	2.6	3.1	3.1	2.3	2.6	3.1	---	2.3
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-99 Behaviour Factor Calculated for 12-Storey 4Bays and 6m Length of the Bay, q=5, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.5	4.0	2.3	2.3	2.5	4.1	---	2.3	2.5	4.1	3.8	2.3	2.5	4.1	3.0	2.3	2.6	4.2	---	2.4
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.6	4.2	2.4	2.4	2.6	4.2	---	2.4	2.6	4.2	3.8	2.4	2.6	4.2	3.0	2.4	2.6	4.2	---	2.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.6	4.2	2.4	2.4	2.6	4.2	---	2.4	2.6	4.2	3.8	2.4	2.6	4.2	3.0	2.4	2.6	4.2	---	2.4
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.6	4.2	2.4	2.4	2.6	4.2	---	2.4	2.6	4.2	3.8	2.4	2.6	4.2	3.0	2.4	2.6	4.2	---	2.4
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.6	4.2	2.4	2.4	2.6	4.2	---	2.4	2.6	4.2	3.8	2.4	2.6	4.2	3.0	2.4	2.6	4.2	---	2.4
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-100 Behaviour Factor Calculated for 12-Storey 4Bays and 6m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.1	4.5	2.8	2.8	3.6	5.3	---	3.2	3.6	5.3	4.8	3.2	3.6	5.3	3.9	3.2	3.6	5.3	---	3.2
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.6	5.3	3.3	3.2	4.3	6.3	---	3.9	4.3	6.3	5.8	3.9	4.3	6.3	4.6	3.9	4.3	6.4	---	3.9
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.1	6.0	3.7	3.6	5.0	7.3	---	4.5	5.0	7.3	6.7	4.5	5.0	7.3	5.4	4.5	5.0	7.4	---	4.5
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.2	6.1	3.7	3.7	5.5	8.1	---	4.9	5.5	8.0	7.3	4.9	5.5	8.0	5.9	4.9	5.5	8.0	---	4.9
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.2	6.1	3.6	3.7	5.9	8.6	---	5.2	5.8	8.4	7.7	5.1	5.8	8.4	6.1	5.1	5.8	8.4	---	5.1
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-101 Behaviour Factor Calculated for 12-Storey 4Bays and 6m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.2	4.6	2.8	2.7	4.2	6.0	---	3.5	4.2	6.0	5.0	3.5	4.1	5.9	4.0	3.4	4.1	6.0	---	3.5
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.0	5.7	3.4	3.3	5.3	7.6	---	4.4	5.3	7.6	6.4	4.4	5.2	7.5	5.0	4.3	5.3	7.6	---	4.4
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.3	6.2	3.7	3.6	6.1	8.8	---	5.1	6.1	8.8	7.4	5.1	6.0	8.7	5.8	5.0	6.1	8.8	---	5.1
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.4	6.3	3.7	3.6	6.3	9.0	---	5.2	6.3	9.0	7.6	5.2	6.2	8.9	6.0	5.1	6.3	9.0	---	5.2
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.4	6.3	3.7	3.6	6.3	9.0	---	5.2	6.3	9.0	7.6	5.2	6.2	8.9	6.0	5.1	6.3	9.0	---	5.2
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.108-Storey 4Bays and 6m Length of the Bay

Table 9-102 Behaviour Factor Calculated for 8-Storey 4Bays and 6m Length of the Bay, q=2, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.2	**3.6**	**2.7**	**2.3**	**3.5**	**3.9**	---	**2.5**	**3.5**	**3.9**	**4.0**	**2.5**	**3.4**	**3.8**	**3.1**	**2.4**	**3.5**	**3.9**	---	**2.5**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.1	**5.7**	**4.1**	**3.6**	**5.6**	**6.3**	---	**4.0**	**5.6**	**6.3**	**6.4**	**4.0**	**5.4**	**6.0**	**4.9**	**3.9**	**5.6**	**6.3**	---	**4.0**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.1	**5.7**	**4.0**	**3.6**	**5.6**	**6.3**	---	**4.1**	**5.6**	**6.4**	**6.4**	**4.1**	**5.4**	**6.1**	**4.9**	**3.9**	**5.6**	**6.3**	---	**4.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.1	**5.7**	**4.0**	**3.7**	**5.6**	**6.4**	---	**4.1**	**5.6**	**6.4**	**6.4**	**4.1**	**5.4**	**6.1**	**5.0**	**3.9**	**5.6**	**6.3**	---	**4.1**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.1	**5.7**	**4.0**	**3.7**	**5.6**	**6.4**	---	**4.1**	**5.6**	**6.4**	**6.4**	**4.1**	**5.4**	**6.1**	**5.0**	**3.9**	**5.6**	**6.3**	---	**4.1**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-103 Behaviour Factor Calculated for 8-Storey 4Bays and 6m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.8	**2.0**	**1.6**	**1.3**	**1.9**	**2.1**	---	**1.4**	**1.9**	**2.1**	**2.3**	**1.4**	**1.9**	**2.1**	**1.8**	**1.4**	**1.9**	**2.1**	---	**1.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.9	**3.2**	**2.5**	**2.2**	**3.1**	**3.4**	---	**2.3**	**3.1**	**3.4**	**3.8**	**2.3**	**3.1**	**3.4**	**3.0**	**2.3**	**3.1**	**3.4**	---	**2.3**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.1	**3.4**	**2.5**	**2.3**	**3.5**	**3.8**	---	**2.6**	**3.5**	**3.8**	**4.2**	**2.6**	**3.4**	**3.8**	**3.3**	**2.5**	**3.5**	**3.8**	---	**2.6**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.2	**3.5**	**2.5**	**2.4**	**3.6**	**3.9**	---	**2.7**	**3.6**	**3.9**	**4.3**	**2.7**	**3.5**	**3.9**	**3.4**	**2.6**	**3.6**	**3.9**	---	**2.7**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.2	**3.5**	**2.5**	**2.4**	**3.6**	**4.0**	---	**2.7**	**3.6**	**3.9**	**4.3**	**2.7**	**3.5**	**3.9**	**3.4**	**2.6**	**3.6**	**3.9**	---	**2.7**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-104 Behaviour Factor Calculated for 8-Storey 4Bays and 6m Length of the Bay, q=4, 5, q_{acc}=4.5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.7	2.0	1.6	1.4	1.8	2.1	---	1.4	1.8	2.1	2.3	1.4	1.8	2.1	1.8	1.4	1.8	2.1	---	1.4
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.5	3.0	2.2	2.0	2.6	3.1	---	2.1	2.6	3.1	3.4	2.1	2.6	3.1	2.7	2.1	2.6	3.1	---	2.1
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.7	3.2	2.3	2.2	3.0	3.5	---	2.4	3.0	3.5	3.9	2.4	3.0	3.5	3.1	2.4	3.0	3.5	---	2.4
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.8	3.4	2.3	2.3	3.3	4.0	---	2.7	3.3	3.9	4.3	2.6	3.3	3.9	3.4	2.6	3.3	3.9	---	2.6
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.9	3.4	2.3	2.3	3.6	4.2	---	2.9	3.5	4.1	4.5	2.8	3.5	4.1	3.6	2.8	3.5	4.1	---	2.8
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-105 Behaviour Factor Calculated for 8-Storey 4Bays and 6m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.2	3.0	2.0	1.6	2.3	3.1	---	1.7	2.3	3.1	2.7	1.7	2.2	3.1	2.2	1.7	2.3	3.1	---	1.7
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.6	5.0	3.0	2.7	3.9	5.4	---	2.9	3.9	5.4	4.7	2.9	3.8	5.3	3.7	2.9	3.9	5.4	---	2.9
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.9	5.3	3.0	2.9	4.3	6.0	---	3.2	4.3	6.0	5.2	3.2	4.3	5.9	4.1	3.2	4.3	6.0	---	3.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.9	5.4	2.9	2.9	4.4	6.1	---	3.3	4.4	6.1	5.3	3.3	4.3	6.0	4.2	3.2	4.4	6.1	---	3.3
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.9	5.4	2.9	2.9	4.4	6.1	---	3.3	4.4	6.1	5.4	3.3	4.4	6.0	4.2	3.3	4.4	6.1	---	3.3
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-106 Behaviour Factor Calculated for 8-Storey 4Bays and 6m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.1	**2.6**	**1.8**	**1.5**	**2.2**	**2.8**	---	**1.6**	**2.2**	**2.8**	**2.6**	**1.6**	**2.1**	**2.7**	**2.0**	**1.5**	**2.2**	**2.8**	---	**1.6**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.3	**5.5**	**3.6**	**3.2**	**4.7**	**6.0**	---	**3.4**	**4.7**	**6.0**	**5.5**	**3.4**	**4.6**	**5.8**	**4.3**	**3.3**	**4.7**	**6.0**	---	**3.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.6	**5.8**	**3.4**	**3.3**	**5.3**	**6.6**	---	**3.8**	**5.3**	**6.6**	**6.1**	**3.8**	**5.1**	**6.4**	**4.7**	**3.7**	**5.2**	**6.6**	---	**3.8**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.7	**5.9**	**3.4**	**3.4**	**5.8**	**7.4**	---	**4.2**	**5.8**	**7.3**	**6.7**	**4.2**	**5.6**	**7.1**	**5.2**	**4.1**	**5.8**	**7.3**	---	**4.2**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.7	**5.9**	**3.3**	**3.4**	**6.1**	**7.6**	---	**4.4**	**6.0**	**7.5**	**6.9**	**4.3**	**5.8**	**7.3**	**5.4**	**4.2**	**5.9**	**7.5**	---	**4.3**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.114-Storey 4Bays and 6m Length of the Bay

Table 9-107 Behaviour Factor Calculated for 4-Storey 4Bays and 6m Length of the Bay, q=2, q_{acc}=2

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.8	**3.1**	**2.6**	**2.3**	**2.9**	**3.2**	---	**2.4**	**2.9**	**3.2**	**3.9**	**2.4**	**2.9**	**3.2**	**3.1**	**2.4**	**2.9**	**3.2**	---	**2.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.8	**3.1**	**2.6**	**2.4**	**2.9**	**3.2**	---	**2.4**	**2.9**	**3.2**	**4.0**	**2.4**	**2.9**	**3.2**	**3.2**	**2.4**	**2.9**	**3.2**	---	**2.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.8	**3.1**	**2.7**	**2.4**	**3.0**	**3.3**	---	**2.5**	**3.0**	**3.3**	**4.0**	**2.5**	**3.0**	**3.3**	**3.2**	**2.5**	**2.9**	**3.2**	---	**2.5**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.9	**3.1**	**2.6**	**2.4**	**3.0**	**3.3**	---	**2.5**	**3.0**	**3.3**	**4.1**	**2.5**	**3.0**	**3.3**	**3.3**	**2.5**	**3.0**	**3.3**	---	**2.5**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.9	**3.2**	**2.6**	**2.4**	**3.0**	**3.3**	---	**2.6**	**3.0**	**3.3**	**4.1**	**2.5**	**3.0**	**3.3**	**3.3**	**2.5**	**3.0**	**3.3**	---	**2.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-108 Behaviour Factor Calculated for 4-Storey 4Bays and 6m Length of the Bay, q=3, 4, 5, 6, 7, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.5	**1.9**	**1.3**	**1.2**	**1.5**	**1.9**	---	**1.2**	**1.5**	**1.9**	**2.0**	**1.2**	**1.5**	**1.9**	**1.6**	**1.2**	**1.5**	**1.9**	---	**1.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.1	**3.9**	**2.7**	**2.5**	**3.2**	**4.0**	---	**2.6**	**3.2**	**4.0**	**4.2**	**2.6**	**3.2**	**4.0**	**3.3**	**2.6**	**3.2**	**4.1**	---	**2.6**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.1	**4.0**	**2.7**	**2.5**	**3.2**	**4.1**	---	**2.6**	**3.2**	**4.1**	**4.2**	**2.6**	**3.2**	**4.1**	**3.4**	**2.6**	**3.3**	**4.2**	---	**2.6**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.2	**4.0**	**2.7**	**2.5**	**3.3**	**4.2**	---	**2.7**	**3.3**	**4.2**	**4.3**	**2.6**	**3.3**	**4.2**	**3.5**	**2.6**	**3.3**	**4.2**	---	**2.7**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.2	**4.0**	**2.7**	**2.5**	**3.3**	**4.2**	---	**2.7**	**3.3**	**4.2**	**4.3**	**2.6**	**3.3**	**4.2**	**3.5**	**2.6**	**3.3**	**4.2**	---	**2.7**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.122-Storey 4Bays and 6m Length of the Bay

Table 9-109 Behaviour Factor Calculated for 2-Storey 4Bays and 6m Length of the Bay, q=2, 3, 4, 5, 6, 7, q_{acc}=4.5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.7	**3.8**	**3.3**	**3.3**	**4.2**	**4.3**	---	**3.7**	**4.2**	**4.3**	**5.8**	**3.7**	**4.2**	**4.3**	**4.7**	**3.7**	**4.2**	**4.3**	---	**3.7**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.9	**3.9**	**3.4**	**3.4**	**4.4**	**4.4**	---	**3.8**	**4.4**	**4.4**	**6.1**	**3.8**	**4.4**	**4.4**	**4.9**	**3.8**	**4.4**	**4.4**	---	**3.8**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.9	**4.0**	**3.4**	**3.5**	**4.5**	**4.6**	---	**3.9**	**4.5**	**4.6**	**6.2**	**3.9**	**4.5**	**4.6**	**5.0**	**3.9**	**4.5**	**4.6**	---	**3.9**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.0	**4.0**	**3.5**	**3.5**	**4.6**	**4.7**	---	**4.0**	**4.6**	**4.7**	**6.4**	**4.0**	**4.6**	**4.7**	**5.1**	**4.0**	**4.6**	**4.7**	---	**4.0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.9	**4.0**	**3.4**	**3.5**	**4.7**	**4.8**	---	**4.1**	**4.6**	**4.7**	**6.5**	**4.1**	**4.6**	**4.7**	**5.2**	**4.1**	**4.6**	**4.7**	---	**4.1**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.1312-Storey 4Bays and 8m Length of the Bay

Table 9-110 Behaviour Factor Calculated for 12-Storey 4Bays and 8m Length of the Bay, q=2, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.7	4.1	3.4	3.4	4.1	4.5	---	3.7	4.1	4.5	5.8	3.7	4.1	4.5	4.6	3.7	4.1	4.5	---	3.7
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.5	4.9	4.1	4.0	5.0	5.6	---	4.6	5.0	5.6	7.1	4.6	5.0	5.6	5.7	4.6	5.1	5.6	---	4.6
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.6	5.1	4.2	4.2	5.5	6.0	---	4.9	5.5	6.0	7.7	4.9	5.5	6.0	6.2	4.9	5.5	6.0	---	4.9
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.6	5.1	4.1	4.2	5.8	6.4	---	5.3	5.8	6.4	8.2	5.2	5.8	6.4	6.6	5.2	5.8	6.4	---	5.3
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.5	5.0	4.0	4.1	6.3	6.9	---	5.7	6.2	6.8	8.7	5.6	6.2	6.8	7.0	5.6	6.2	6.8	---	5.6
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-111 Behaviour Factor Calculated for 12-Storey 4Bays and 8m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.9	2.0	1.8	1.8	2.0	2.0	---	1.8	2.0	2.0	2.9	1.8	2.0	2.0	2.3	1.8	2.0	2.0	---	1.8
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.3	2.4	2.2	2.1	2.4	2.5	---	2.2	2.4	2.5	3.5	2.2	2.4	2.5	2.8	2.2	2.4	2.5	---	2.2
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	2.4	2.4	2.2	2.1	2.5	2.5	---	2.2	2.5	2.5	3.5	2.2	2.5	2.5	2.8	2.2	2.4	2.5	---	2.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	2.4	2.4	2.2	2.1	2.5	2.5	---	2.2	2.5	2.5	3.6	2.2	2.5	2.5	2.9	2.2	2.5	2.5	---	2.2
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	2.4	2.4	2.2	2.1	2.5	2.5	---	2.2	2.5	2.5	3.6	2.2	2.5	2.5	2.9	2.2	2.5	2.5	---	2.2
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-112 Behaviour Factor Calculated for 12-Storey 4Bays and 8m Length of the Bay, q=4, q_{acc}=4

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.7	3.0	2.4	2.3	2.8	3.1	---	2.3	2.8	3.1	3.8	2.3	2.8	3.1	3.0	2.3	2.8	3.1	---	2.3
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.2	3.4	2.7	2.6	3.3	3.6	---	2.7	3.3	3.6	4.4	2.7	3.3	3.6	3.5	2.7	3.3	3.6	---	2.7
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.2	3.5	2.7	2.6	3.3	3.6	---	2.7	3.3	3.6	4.4	2.7	3.3	3.6	3.5	2.7	3.3	3.6	---	2.7
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.2	3.5	2.7	2.6	3.3	3.6	---	2.8	3.3	3.6	4.4	2.8	3.3	3.6	3.6	2.8	3.3	3.6	---	2.8
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.2	3.5	2.7	2.6	3.3	3.6	---	2.8	3.3	3.6	4.5	2.8	3.3	3.6	3.6	2.8	3.3	3.6	---	2.8
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-113 Behaviour Factor Calculated for 12-Storey 4Bays and 8m Length of the Bay, q=5, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.9	4.2	3.4	3.4	4.6	5.0	---	4.0	4.6	5.0	6.2	4.0	4.6	5.0	4.9	4.0	4.6	5.0	---	4.0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.6	4.9	4.0	4.0	5.5	5.9	---	4.8	5.5	5.9	7.3	4.8	5.5	5.9	5.9	4.8	5.5	5.9	---	4.8
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.8	5.1	4.1	4.1	6.0	6.5	---	5.2	6.0	6.5	8.0	5.2	6.0	6.5	6.4	5.2	6.0	6.5	---	5.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.8	5.2	4.1	4.2	6.5	7.0	---	5.7	6.5	7.0	8.6	5.6	6.5	7.0	6.9	5.6	6.5	7.0	---	5.6
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.7	5.1	4.0	4.1	7.0	7.5	---	6.1	6.8	7.4	9.2	6.0	6.9	7.4	7.3	6.0	6.9	7.4	---	6.0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-114 Behaviour Factor Calculated for 12-Storey 4Bays and 8m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.1	**4.3**	**3.3**	**3.2**	**5.3**	**5.6**	---	**4.2**	**5.3**	**5.6**	**6.2**	**4.2**	**5.2**	**5.5**	**4.9**	**4.1**	**5.3**	**5.7**	---	**4.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.1	**5.4**	**4.1**	**4.0**	**6.8**	**7.2**	---	**5.4**	**6.8**	**7.2**	**8.0**	**5.4**	**6.7**	**7.1**	**6.3**	**5.3**	**6.9**	**7.3**	---	**5.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.3	**5.7**	**4.2**	**4.2**	**7.5**	**8.0**	---	**6.0**	**7.5**	**8.0**	**8.8**	**6.0**	**7.4**	**7.9**	**6.9**	**5.9**	**7.6**	**8.1**	---	**6.0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.4	**5.7**	**4.2**	**4.3**	**8.2**	**8.7**	---	**6.5**	**8.1**	**8.7**	**9.5**	**6.4**	**8.0**	**8.5**	**7.5**	**6.3**	**8.2**	**8.7**	---	**6.5**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.3	**5.6**	**4.0**	**4.2**	**8.8**	**9.4**	---	**7.0**	**8.7**	**9.2**	**10.1**	**6.9**	**8.5**	**9.1**	**8.0**	**6.8**	**8.8**	**9.3**	---	**6.9**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-115 Behaviour Factor Calculated for 12-Storey 4Bays and 8m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.5	**4.8**	**3.3**	**3.2**	**7.5**	**8.0**	---	**5.3**	**7.4**	**7.9**	**7.2**	**5.2**	**5.0**	**5.3**	**3.8**	**3.5**	**7.4**	**7.9**	---	**5.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.9	**6.3**	**4.3**	**4.2**	**10.1**	**10.7**	---	**7.1**	**10.0**	**10.6**	**9.7**	**7.0**	**6.7**	**7.1**	**5.2**	**4.7**	**10.0**	**10.7**	---	**7.1**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	6.1	**6.5**	**4.4**	**4.3**	**10.8**	**11.5**	---	**7.6**	**10.7**	**11.4**	**10.4**	**7.6**	**7.2**	**7.7**	**5.6**	**5.1**	**10.7**	**11.5**	---	**7.6**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	6.1	**6.5**	**4.4**	**4.3**	**11.6**	**12.3**	---	**8.2**	**11.4**	**12.2**	**11.0**	**8.1**	**7.7**	**8.2**	**5.9**	**5.4**	**11.4**	**12.2**	---	**8.1**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	6.1	**6.5**	**4.2**	**4.3**	**12.3**	**13.1**	---	**8.7**	**12.0**	**12.8**	**11.6**	**8.5**	**8.1**	**8.6**	**6.3**	**5.7**	**12.1**	**12.9**	---	**8.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.148-Storey 4Bays and 8m Length of the Bay

Table 9-116 Behaviour Factor Calculated for 8-Storey 4Bays and 8m Length of the Bay, q=2, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.3	**3.5**	**2.3**	**2.2**	**4.0**	**4.3**	---	**2.6**	**4.0**	**4.3**	**3.9**	**2.6**	**3.6**	**3.8**	**2.8**	**2.3**	**4.0**	**4.3**	---	**2.7**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	6.1	**6.5**	**4.1**	**4.0**	**8.0**	**8.5**	---	**5.3**	**8.0**	**8.5**	**7.8**	**5.2**	**7.1**	**7.6**	**5.6**	**4.7**	**8.1**	**8.6**	---	**5.3**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	6.6	**7.0**	**4.2**	**4.3**	**9.5**	**10.1**	---	**6.2**	**9.4**	**10.0**	**9.2**	**6.2**	**8.4**	**8.9**	**6.6**	**5.5**	**9.5**	**10.2**	---	**6.3**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	6.6	**7.0**	**4.2**	**4.3**	**9.5**	**10.2**	---	**6.3**	**9.5**	**10.1**	**9.3**	**6.2**	**8.4**	**9.0**	**6.6**	**5.5**	**9.6**	**10.2**	---	**6.3**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	6.6	**7.0**	**4.2**	**4.3**	**9.6**	**10.2**	---	**6.3**	**9.5**	**10.1**	**9.3**	**6.2**	**8.4**	**9.0**	**6.6**	**5.5**	**9.6**	**10.3**	---	**6.3**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-117 Behaviour Factor Calculated for 8-Storey 4Bays and 8m Length of the Bay, q=3, q_{acc}=3

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.4	**2.0**	**2.0**	**1.7**	**2.7**	**2.2**	---	**1.8**	**2.7**	**2.2**	**2.9**	**1.8**	**2.5**	**2.1**	**2.2**	**1.7**	**2.7**	**2.2**	---	**1.8**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.6	**3.0**	**2.7**	**2.4**	**4.1**	**3.4**	---	**2.8**	**4.1**	**3.4**	**4.4**	**2.8**	**3.8**	**3.2**	**3.3**	**2.6**	**4.1**	**3.4**	---	**2.8**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.1	**3.5**	**2.9**	**2.8**	**5.1**	**4.3**	---	**3.5**	**5.1**	**4.3**	**5.5**	**3.5**	**4.8**	**4.0**	**4.1**	**3.3**	**5.1**	**4.3**	---	**3.5**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.3	**3.6**	**2.9**	**2.9**	**5.7**	**4.8**	---	**3.9**	**5.6**	**4.7**	**6.0**	**3.8**	**5.3**	**4.4**	**4.6**	**3.6**	**5.6**	**4.7**	---	**3.8**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.3	**3.6**	**2.8**	**2.9**	**6.0**	**5.1**	---	**4.1**	**5.9**	**5.0**	**6.4**	**4.0**	**5.6**	**4.7**	**4.8**	**3.8**	**5.9**	**5.0**	---	**4.0**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-118 Behaviour Factor Calculated for 8-Storey 4Bays and 8m Length of the Bay, q=4, q_{acc}=4

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.9	**2.0**	**1.6**	**1.3**	**2.0**	**2.1**	---	**1.4**	**2.0**	**2.1**	**2.2**	**1.4**	**2.0**	**2.0**	**1.8**	**1.4**	**2.0**	**2.1**	---	**1.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	2.7	**2.8**	**2.1**	**1.8**	**2.9**	**3.0**	---	**2.0**	**2.9**	**3.0**	**3.2**	**2.0**	**2.8**	**2.9**	**2.5**	**1.9**	**2.8**	**3.0**	---	**2.0**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.1	**3.2**	**2.2**	**2.1**	**3.5**	**3.6**	---	**2.4**	**3.5**	**3.6**	**3.9**	**2.4**	**3.4**	**3.6**	**3.1**	**2.4**	**3.5**	**3.6**	---	**2.4**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.2	**3.3**	**2.3**	**2.2**	**3.7**	**3.9**	---	**2.6**	**3.7**	**3.9**	**4.2**	**2.6**	**3.7**	**3.8**	**3.3**	**2.5**	**3.7**	**3.8**	---	**2.6**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.2	**3.3**	**2.2**	**2.2**	**3.8**	**4.0**	---	**2.6**	**3.8**	**3.9**	**4.2**	**2.6**	**3.7**	**3.9**	**3.3**	**2.6**	**3.8**	**3.9**	---	**2.6**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-119 Behaviour Factor Calculated for 8-Storey 4Bays and 8m Length of the Bay, q=5, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.0	**2.0**	**1.6**	**1.3**	**2.1**	**2.2**	---	**1.4**	**2.1**	**2.2**	**2.3**	**1.4**	**2.0**	**2.1**	**1.8**	**1.4**	**2.1**	**2.2**	---	**1.4**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.3	**3.4**	**2.5**	**2.2**	**3.6**	**3.7**	---	**2.4**	**3.6**	**3.7**	**3.9**	**2.4**	**3.5**	**3.6**	**3.0**	**2.3**	**3.6**	**3.7**	---	**2.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.9	**4.1**	**2.6**	**2.6**	**4.7**	**4.9**	---	**3.1**	**4.7**	**4.9**	**5.1**	**3.1**	**4.6**	**4.8**	**4.0**	**3.1**	**4.7**	**4.9**	---	**3.2**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.0	**4.2**	**2.6**	**2.7**	**5.0**	**5.2**	---	**3.3**	**5.0**	**5.2**	**5.4**	**3.3**	**4.9**	**5.0**	**4.2**	**3.2**	**5.0**	**5.2**	---	**3.3**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.0	**4.2**	**2.6**	**2.7**	**5.1**	**5.3**	---	**3.4**	**5.1**	**5.3**	**5.5**	**3.4**	**5.0**	**5.1**	**4.3**	**3.3**	**5.1**	**5.3**	---	**3.4**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-120 Behaviour Factor Calculated for 8-Storey 4Bays and 8m Length of the Bay, q=6, q_{acc}=6

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.9	**3.1**	**2.2**	**1.9**	**3.3**	**3.5**	---	**2.2**	**3.3**	**3.5**	**3.4**	**2.2**	**3.0**	**3.2**	**2.5**	**2.0**	**3.2**	**3.4**	---	**2.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	5.3	**5.7**	**3.7**	**3.6**	**6.6**	**7.1**	---	**4.5**	**6.6**	**7.1**	**6.9**	**4.5**	**6.0**	**6.5**	**5.0**	**4.1**	**6.5**	**7.0**	---	**4.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	5.9	**6.3**	**3.9**	**4.0**	**8.1**	**8.7**	---	**5.4**	**8.1**	**8.7**	**8.4**	**5.4**	**7.4**	**7.9**	**6.1**	**5.0**	**7.9**	**8.5**	---	**5.3**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	6.0	**6.4**	**3.9**	**4.0**	**8.6**	**9.2**	---	**5.8**	**8.5**	**9.2**	**8.9**	**5.8**	**7.8**	**8.4**	**6.5**	**5.3**	**8.4**	**9.0**	---	**5.7**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	6.0	**6.4**	**3.9**	**4.0**	**8.6**	**9.2**	---	**5.8**	**8.5**	**9.2**	**8.9**	**5.8**	**7.8**	**8.4**	**6.5**	**5.3**	**8.4**	**9.0**	---	**5.7**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-121 Behaviour Factor Calculated for 8-Storey 4Bays and 8m Length of the Bay, q=7, q_{acc}=7

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.5	**2.5**	**2.1**	**1.9**	**3.0**	**3.0**	---	**2.3**	**3.0**	**3.0**	**3.4**	**2.3**	**2.7**	**2.7**	**2.5**	**2.0**	**3.0**	**3.0**	---	**2.3**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	6.9	**6.9**	**7.2**	**5.2**	**7.2**	**7.2**	---	**5.4**	**7.2**	**7.2**	**8.2**	**5.4**	**6.4**	**6.4**	**5.9**	**4.8**	**7.2**	**7.2**	---	**5.4**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	8.4	**8.4**	**7.1**	**6.3**	**7.9**	**7.9**	---	**6.0**	**7.9**	**7.9**	**9.0**	**5.9**	**7.0**	**7.0**	**6.4**	**5.3**	**7.9**	**7.9**	---	**5.9**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	8.4	**8.4**	**7.1**	**6.3**	**7.9**	**7.9**	---	**6.0**	**7.9**	**7.9**	**9.0**	**5.9**	**7.0**	**7.0**	**6.4**	**5.3**	**7.9**	**7.9**	---	**5.9**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	8.4	**8.4**	**7.1**	**6.3**	**7.9**	**7.9**	---	**6.0**	**7.9**	**7.9**	**9.0**	**5.9**	**7.0**	**7.0**	**6.4**	**5.3**	**7.9**	**7.9**	---	**5.9**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.154-Storey 4Bays and 8m Length of the Bay

Table 9-122 Behaviour Factor Calculated for 4-Storey 4Bays and 8m Length of the Bay, q=2, q_{acc}=2

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.1	**3.0**	**2.5**	**2.1**	**3.3**	**3.2**	---	**2.2**	**3.3**	**3.2**	**3.5**	**2.2**	**3.1**	**3.0**	**2.7**	**2.1**	**3.3**	**3.2**	---	**2.2**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.6	**4.5**	**3.8**	**3.1**	**5.0**	**4.8**	---	**3.3**	**5.0**	**4.8**	**5.3**	**3.3**	**4.7**	**4.5**	**4.0**	**3.1**	**5.0**	**4.8**	---	**3.3**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.7	**4.5**	**3.8**	**3.1**	**5.1**	**4.9**	---	**3.4**	**5.1**	**4.9**	**5.4**	**3.4**	**4.8**	**4.6**	**4.1**	**3.2**	**5.1**	**4.9**	---	**3.4**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.7	**4.5**	**3.7**	**3.1**	**5.2**	**5.0**	---	**3.5**	**5.2**	**5.0**	**5.5**	**3.4**	**4.9**	**4.7**	**4.1**	**3.3**	**5.2**	**5.0**	---	**3.5**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.7	**4.5**	**3.5**	**3.1**	**5.3**	**5.1**	---	**3.5**	**5.3**	**5.1**	**5.6**	**3.5**	**5.0**	**4.8**	**4.2**	**3.3**	**5.3**	**5.1**	---	**3.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-123 Behaviour Factor Calculated for 4-Storey 4Bays and 8m Length of the Bay, q=3, 4, 5, 6, 7, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	1.9	**2.4**	**1.6**	**1.4**	**2.0**	**2.5**	---	**1.5**	**2.0**	**2.5**	**2.4**	**1.5**	**2.0**	**2.5**	**1.9**	**1.5**	**2.0**	**2.5**	---	**1.5**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.2	**4.0**	**2.6**	**2.4**	**3.5**	**4.4**	---	**2.7**	**3.5**	**4.4**	**4.3**	**2.7**	**3.5**	**4.4**	**3.4**	**2.6**	**3.5**	**4.4**	---	**2.6**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	3.3	**4.1**	**2.6**	**2.4**	**3.6**	**4.5**	---	**2.7**	**3.6**	**4.5**	**4.3**	**2.7**	**3.6**	**4.5**	**3.5**	**2.7**	**3.6**	**4.5**	---	**2.7**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	3.3	**4.1**	**2.6**	**2.5**	**3.6**	**4.6**	---	**2.7**	**3.6**	**4.5**	**4.4**	**2.7**	**3.6**	**4.5**	**3.5**	**2.7**	**3.6**	**4.5**	---	**2.7**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	3.3	**4.1**	**2.6**	**2.5**	**3.7**	**4.6**	---	**2.8**	**3.7**	**4.6**	**4.5**	**2.8**	**3.7**	**4.6**	**3.6**	**2.8**	**3.7**	**4.6**	---	**2.8**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.10.162-Storey 4Bays and 8m Length of the Bay

Table 9-124 Behaviour Factor Calculated for 2-Storey 4Bays and 8m Length of the Bay, q=2, q_{acc}=2

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.7	**5.0**	**3.7**	**3.5**	**5.5**	**5.7**	---	**4.0**	**5.5**	**5.7**	**6.2**	**4.0**	**5.3**	**5.6**	**4.8**	**3.9**	**5.5**	**5.7**	---	**4.0**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.8	**5.1**	**3.8**	**3.6**	**5.6**	**5.9**	---	**4.1**	**5.6**	**5.9**	**6.4**	**4.1**	**5.4**	**5.7**	**5.0**	**4.0**	**5.6**	**5.9**	---	**4.1**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.9	**5.1**	**3.8**	**3.6**	**5.7**	**6.0**	---	**4.2**	**5.7**	**6.0**	**6.5**	**4.2**	**5.6**	**5.9**	**5.1**	**4.1**	**5.7**	**6.0**	---	**4.2**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.9	**5.2**	**3.8**	**3.6**	**5.9**	**6.2**	---	**4.4**	**5.9**	**6.2**	**6.7**	**4.4**	**5.7**	**6.0**	**5.2**	**4.2**	**5.9**	**6.2**	---	**4.3**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.9	**5.2**	**3.7**	**3.6**	**6.1**	**6.4**	---	**4.5**	**6.0**	**6.4**	**6.9**	**4.5**	**5.9**	**6.2**	**5.4**	**4.3**	**6.0**	**6.4**	---	**4.5**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-125 Behaviour Factor Calculated for 2-Storey 4Bays and 8m Length of the Bay, q=3, 4, 5, 6, 7, q_{acc}=5

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	4.1	**4.2**	**3.6**	**3.4**	**4.6**	**4.7**	---	**3.8**	**4.6**	**4.7**	**6.0**	**3.8**	**4.6**	**4.7**	**4.8**	**3.8**	**4.6**	**4.7**	---	**3.8**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.2	**4.3**	**3.7**	**3.5**	**4.7**	**4.8**	---	**3.9**	**4.7**	**4.8**	**6.2**	**3.9**	**4.7**	**4.8**	**4.9**	**3.9**	**4.7**	**4.8**	---	**3.9**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.3	**4.4**	**3.7**	**3.6**	**4.9**	**5.0**	---	**4.0**	**4.9**	**5.0**	**6.4**	**4.0**	**4.9**	**5.0**	**5.1**	**4.0**	**4.9**	**5.0**	---	**4.1**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.3	**4.4**	**3.7**	**3.6**	**5.0**	**5.1**	---	**4.2**	**5.0**	**5.1**	**6.5**	**4.2**	**5.0**	**5.1**	**5.2**	**4.2**	**5.0**	**5.1**	---	**4.2**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.3	**4.4**	**3.7**	**3.6**	**5.1**	**5.2**	---	**4.2**	**5.1**	**5.2**	**6.6**	**4.2**	**5.1**	**5.2**	**5.3**	**4.2**	**5.1**	**5.2**	---	**4.2**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.11 APPENDIX-G-2

Behaviour Factor Calculations for *Non-Conventional* Structures

9.11.1 8-Storey 3Bays and 6m Length of the Bay

Table 9-126 Behaviour Factor Calculated for 8-Storey Bolted Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.8	3.2	2.6	2.4	3.9	4.5	---	3.3	4.1	4.7	4.9	3.5	3.2	3.7	3.1	2.7	3.9	4.5	---	3.3
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.9	4.5	3.5	3.3	5.7	6.5	---	4.8	6.0	6.8	7.1	5.0	4.6	5.3	4.4	3.9	5.6	6.4	---	4.7
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.3	4.9	3.8	3.6	6.6	7.5	---	5.5	6.9	7.9	8.3	5.8	5.4	6.2	5.1	4.5	6.5	7.5	---	5.5
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.4	5.0	3.8	3.7	7.3	8.4	---	6.2	7.7	8.8	9.1	6.4	6.0	6.8	5.7	5.0	7.2	8.3	---	6.1
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.4	5.0	3.7	3.7	7.8	8.9	---	6.5	8.1	9.2	9.6	6.8	6.3	7.2	6.0	5.3	7.6	8.7	---	6.4
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-127 Behaviour Factor Calculated for 8-Storey Welded Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.2	3.6	2.8	2.8	5.2	5.8	---	4.5	5.1	5.7	6.0	4.5	4.0	4.5	3.8	3.5	5.2	5.8	---	4.6
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.8	4.3	3.4	3.4	6.3	7.1	---	5.6	6.2	7.0	7.4	5.5	4.9	5.5	4.6	4.3	6.3	7.1	---	5.6
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.0	4.5	3.5	3.6	7.0	7.8	---	6.1	6.9	7.7	8.1	6.1	5.4	6.1	5.1	4.8	7.0	7.9	---	6.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.1	4.7	3.6	3.6	7.7	8.6	---	6.7	7.5	8.4	8.9	6.6	5.9	6.6	5.6	5.2	7.6	8.6	---	6.7
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.1	4.6	3.5	3.6	8.2	9.2	---	7.2	8.0	9.0	9.5	7.0	6.3	7.1	6.0	5.5	8.1	9.1	---	7.1
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.11.2 4-Storey 3Bays and 6m Length of the Bay

Table 9-128 Behaviour Factor Calculated for 4-Storey Bolted Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.0	2.9	2.4	2.1	4.6	4.5	---	3.3	4.3	4.2	4.5	3.1	3.0	3.0	2.5	2.2	4.6	4.5	---	3.3
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.4	4.4	3.6	3.2	7.2	7.1	---	5.2	6.8	6.7	7.1	4.9	4.8	4.7	4.0	3.5	7.2	7.1	---	5.3
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.8	4.7	3.8	3.5	8.3	8.1	---	6.0	7.7	7.6	8.1	5.6	5.5	5.4	4.6	4.0	8.3	8.1	---	6.0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.9	4.8	3.8	3.6	9.1	9.0	---	6.6	8.5	8.4	8.9	6.2	6.0	5.9	5.0	4.4	9.1	9.0	---	6.6
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.9	4.8	3.5	3.5	10.5	10.3	---	7.6	9.7	9.5	10.1	7.0	6.8	6.7	5.7	5.0	10.4	10.2	---	7.5
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-129 Behaviour Factor Calculated for 4-Storey Welded Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.2	3.4	2.5	2.3	5.8	6.2	---	4.1	5.0	5.3	4.8	3.5	3.3	3.5	2.6	2.4	5.7	6.1	---	4.0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.0	4.2	3.0	2.8	7.3	7.7	---	5.1	6.2	6.6	6.0	4.4	4.2	4.4	3.2	2.9	7.1	7.6	---	5.0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.2	4.5	3.2	3.0	8.0	8.5	---	5.7	6.9	7.3	6.7	4.9	4.6	4.8	3.5	3.2	7.9	8.3	---	5.6
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.4	4.7	3.2	3.1	8.8	9.3	---	6.2	7.5	8.0	7.3	5.3	5.0	5.3	3.9	3.5	8.6	9.1	---	6.1
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.4	4.7	3.2	3.1	9.6	10.1	---	6.8	8.2	8.6	7.9	5.8	5.4	5.8	4.2	3.8	9.3	9.9	---	6.6
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.11.3 2-Storey 3Bays and 6m Length of the Bay

Table 9-130 Behaviour Factor Calculated for 2-Storey Bolted Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	2.8	2.8	2.2	1.6	4.2	4.3	---	2.4	3.3	3.4	2.9	1.9	2.5	2.6	1.8	1.4	4.2	4.3	---	2.4
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	3.6	3.7	2.8	2.1	5.6	5.8	---	3.2	4.4	4.6	3.9	2.5	3.3	3.4	2.4	1.9	5.6	5.8	---	3.2
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.2	4.3	3.0	2.4	7.0	7.3	---	4.0	5.5	5.7	4.9	3.2	4.2	4.3	2.9	2.4	7.0	7.2	---	4.0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	4.6	4.7	2.9	2.6	8.5	8.7	---	4.9	6.7	6.9	5.9	3.8	5.0	5.2	3.5	2.9	8.4	8.7	---	4.8
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	4.7	4.8	2.5	2.7	10.0	10.3	---	5.7	7.8	8.0	6.9	4.5	5.9	6.0	4.1	3.4	9.8	10.1	---	5.6
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-131 Behaviour Factor Calculated for 2-Storey Welded Beam Splices

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	3.7	3.6	3.0	2.4	5.9	5.6	---	3.8	5.3	5.0	5.0	3.4	3.6	3.5	2.8	2.4	5.9	5.6	---	3.8
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	4.4	4.2	3.4	2.8	7.1	6.7	---	4.6	6.3	6.0	6.0	4.1	4.3	4.1	3.3	2.8	7.0	6.6	---	4.5
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	4.8	4.5	3.6	3.1	8.1	7.7	---	5.2	7.2	6.8	6.9	4.7	5.0	4.7	3.8	3.2	8.0	7.6	---	5.2
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	5.0	4.7	3.5	3.2	9.2	8.8	---	6.0	8.2	7.8	7.8	5.3	5.6	5.4	4.3	3.7	9.1	8.6	---	5.9
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	5.0	4.7	3.1	3.2	10.3	9.8	---	6.7	9.1	8.6	8.7	5.9	6.3	5.9	4.8	4.1	10.1	9.6	---	6.5
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.12 APPENDIX-G-3

Behaviour Factor Acceptance Percentile for *Conventional* Structures

9.12.1 3Bays and 6m Length of the Bay

Table 9-132 Behaviour Factor Acceptance Percentile for 12S-3B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	0	0	17	---	0	0	17	17	0	0	0	0	0	0	17	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0	50	0	0	0	100	---	0	0	100	100	0	0	100	50	0	0	100	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0	100	0	0	67	100	---	0	67	100	100	0	67	100	67	0	67	100	---	0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0	100	0	0	67	83	---	33	67	83	83	33	67	83	67	17	67	83	---	33
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0	100	0	0	67	67	---	33	67	67	83	33	67	67	67	17	67	67	---	33
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-133 Behaviour Factor Acceptance Percentile for 8S-3B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	0	0	0	---	0	0	0	0	0	0	0	0	0	0	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0	17	0	0	0	33	---	0	0	33	33	0	0	33	0	0	0	33	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0	33	0	0	0	50	---	0	0	50	50	0	0	50	0	0	0	50	---	0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0	67	0	0	17	67	---	0	17	67	50	0	17	67	17	0	17	67	---	0
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	17	67	0	0	17	67	---	17	17	67	33	0	17	67	17	0	17	67	---	0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-134 Behaviour Factor Acceptance Percentile for 4S-3B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	17	0	17	17	0	0	---	17	0	0	0	17	0	0	0	17	0	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0	0	0	17	0	83	---	17	0	83	83	17	0	83	0	17	0	83	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0	83	0	17	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0	83	0	17	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0	83	0	17	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-135 Behaviour Factor Acceptance Percentile for 12/8/4/2S-3B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	29	25	4	4	25	29	---	29	25	29	4	29	25	25	25	29	25	29	---	25
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	25	42	0	4	25	79	---	29	25	79	54	29	25	79	38	29	25	79	---	25
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	25	79	0	4	42	83	---	25	42	83	58	25	42	83	42	25	42	83	---	25
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	25	88	0	4	46	83	---	33	46	83	54	33	46	83	46	29	46	83	---	33
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	29	88	0	4	46	79	---	38	46	79	50	33	46	79	46	29	46	79	---	33
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.12.2 3Bays and 8m Length of the Bay

Table 9-136 Behaviour Factor Acceptance Percentile for 12S-3B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	0	50	50	---	0	50	50	100	0	33	33	50	0	50	50	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	83	100	33	17	67	67	---	83	67	67	17	83	83	83	67	67	67	50	---	83
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	100	100	50	50	50	33	---	100	50	33	0	100	67	50	50	83	50	33	---	100
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	100	100	50	50	50	33	---	83	50	33	0	100	67	50	50	83	50	33	---	83
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	83	100	33	50	33	33	---	83	33	33	0	83	50	50	50	67	33	33	---	83
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-137 Behaviour Factor Acceptance Percentile for 8S-3B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	0	17	0	---	0	17	0	17	0	0	0	0	0	17	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	50	83	17	17	50	33	---	33	50	33	83	33	50	67	67	17	50	17	---	17
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	67	100	17	17	33	33	---	67	33	33	33	67	67	33	100	33	33	33	---	67
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	67	100	17	17	33	17	---	83	33	17	17	83	50	17	100	67	33	17	---	83
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	67	100	17	17	33	17	---	83	33	17	17	83	50	17	83	67	33	17	---	83
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-138 Behaviour Factor Acceptance Percentile for 4S-3B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	**0**	**0**	**17**	**0**	**0**	---	**17**	**0**	**0**	**0**	**17**	**0**	**0**	**0**	**17**	**0**	**0**	---	**0**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0	**83**	**0**	**0**	**0**	**83**	---	**0**	**0**	**83**	**83**	**0**	**0**	**83**	**0**	**0**	**0**	**83**	---	**0**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0	**83**	**0**	**0**	**0**	**83**	---	**0**	**0**	**83**	**83**	**0**	**0**	**83**	**0**	**0**	**0**	**83**	---	**0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0	**83**	**0**	**0**	**0**	**83**	---	**0**	**0**	**83**	**83**	**0**	**0**	**83**	**0**	**0**	**0**	**83**	---	**0**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0	**83**	**0**	**0**	**0**	**83**	---	**0**	**0**	**83**	**83**	**0**	**0**	**83**	**0**	**0**	**0**	**83**	---	**0**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-139 Behaviour Factor Acceptance Percentile for 2S-3B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	**83**	**0**	**0**	**83**	**83**	---	**0**	**83**	**83**	**83**	**0**	**83**	**83**	**83**	**0**	**83**	**83**	---	**0**
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	83	**83**	**0**	**0**	**83**	**83**	---	**0**	**83**	**83**	**0**	**0**	**83**	**83**	**83**	**0**	**83**	**83**	---	**0**
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	83	**83**	**0**	**0**	**83**	**83**	---	**0**	**83**	**83**	**0**	**0**	**83**	**83**	**83**	**0**	**83**	**83**	---	**0**
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	83	**83**	**0**	**0**	**83**	**83**	---	**83**	**83**	**83**	**0**	**83**	**83**	**83**	**83**	**83**	**83**	**83**	---	**83**
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	83	**83**	**0**	**0**	**83**	**83**	---	**83**	**83**	**83**	**0**	**83**	**83**	**83**	**83**	**83**	**83**	**83**	---	**83**
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-140 Behaviour Factor Acceptance Percentile for 12/8/4/2S-3B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	21	0	4	38	33	---	4	38	33	50	4	29	29	33	4	38	33	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	54	88	13	8	50	67	---	29	50	67	46	29	54	79	54	21	50	58	---	25
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	63	92	17	17	42	58	---	42	42	58	29	42	54	63	58	29	42	58	---	42
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	63	92	17	17	42	54	---	63	42	54	25	67	50	58	58	58	42	54	---	63
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	58	92	13	17	38	54	---	63	38	54	25	63	46	58	54	54	38	54	---	63
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.12.3 4Bays and 6m Length of the Bay

Table 9-141 Behaviour Factor Acceptance Percentile for 12S-4B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	17	33	17	0	17	67	---	0	17	67	67	0	17	67	17	0	17	67	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	17	83	17	0	17	67	---	17	17	67	67	17	17	67	17	17	17	67	---	17
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	17	83	17	0	50	33	---	17	50	33	67	17	50	33	50	17	50	33	---	17
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	17	83	17	0	50	33	---	33	50	33	50	33	50	33	50	33	50	33	---	33
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	17	83	17	0	50	33	---	33	50	33	50	33	50	33	50	33	50	33	---	33
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-142 Behaviour Factor Acceptance Percentile for 8S-4B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	0	0	0	---	0	0	0	0	0	0	0	0	0	0	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	17	50	17	0	33	67	---	0	33	67	17	0	17	67	17	0	33	67	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	17	67	17	0	33	50	---	17	33	50	83	17	17	50	17	17	33	50	---	17
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	17	67	17	0	50	83	---	17	50	83	83	17	33	83	17	17	50	83	---	17
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	17	67	17	17	83	83	---	17	50	83	83	17	33	83	50	17	50	83	---	17
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-143 Behaviour Factor Acceptance Percentile for 4S-4B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	17	0	0	---	17	0	0	0	17	0	0	0	17	0	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0	0	0	17	0	83	---	17	0	83	83	17	0	83	0	17	0	83	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0	83	0	17	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0	83	0	17	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0	83	0	17	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-144 Behaviour Factor Acceptance Percentile for 12/8/4/2S-4B-6m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	29	33	4	4	29	42	---	29	29	42	17	29	29	42	29	29	29	42	---	25
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	33	58	8	4	38	79	---	33	38	79	42	33	33	79	33	33	38	79	---	29
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	33	83	8	4	46	67	---	33	46	67	58	33	42	67	42	33	46	67	---	33
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	33	83	8	4	50	75	---	38	50	75	54	38	46	75	42	38	50	75	---	38
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	33	83	8	8	58	75	---	38	50	75	54	38	46	75	50	38	50	75	---	38
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.12.4 4Bays and 8m Length of the Bay

Table 9-145 Behaviour Factor Acceptance Percentile for 12S-4B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	33	0	0	67	67	---	17	67	67	83	17	50	50	50	17	67	67	---	17
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	83	100	33	33	83	83	---	67	83	83	33	67	100	100	83	50	83	67	---	67
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	83	100	33	33	67	50	---	67	67	50	33	67	83	67	67	50	67	50	---	67
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	100	100	33	33	50	33	---	67	50	33	33	67	67	50	50	50	50	33	---	67
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	100	100	17	33	33	33	---	33	33	33	33	50	50	33	50	67	33	33	---	50
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-146 Behaviour Factor Acceptance Percentile for 8S-4B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	17	0	0	0	17	0	---	0	17	0	17	0	17	0	0	0	17	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	67	67	33	17	50	50	---	17	50	50	67	17	50	67	67	17	50	50	---	17
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	50	100	33	33	50	50	---	67	50	50	33	67	67	50	67	33	50	50	---	67
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	83	100	33	33	50	50	---	50	50	50	33	50	67	50	83	33	50	50	---	50
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	83	100	33	33	50	50	---	50	50	50	33	50	67	50	83	17	50	50	---	50
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-147 Behaviour Factor Acceptance Percentile for 4S-4B-8m

	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	0	0	0	17	0	0	---	17	0	0	0	17	0	0	0	17	0	0	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	0	83	0	0	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	0	83	0	0	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	0	83	0	0	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	0	83	0	0	0	83	---	0	0	83	83	0	0	83	0	0	0	83	---	0
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-148 Behaviour Factor Acceptance Percentile for 2S-4B-8m

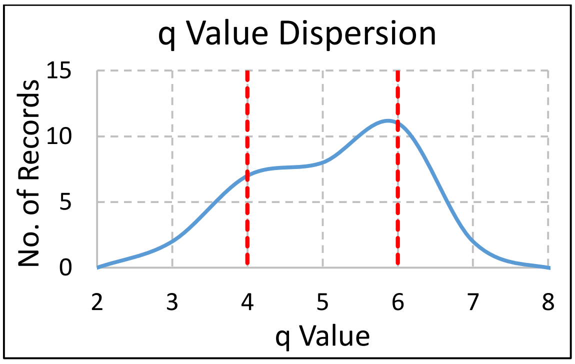
	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	67	67	0	0	83	83	---	0	83	83	17	0	83	83	83	0	83	83	---	0
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	67	67	0	0	83	83	---	0	83	83	0	0	83	83	83	0	83	83	---	0
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	67	67	0	0	83	83	---	67	83	83	0	67	83	83	83	67	83	83	---	67
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	67	67	0	0	83	83	---	67	83	83	0	67	83	83	83	67	83	83	---	67
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	67	67	0	0	83	83	---	83	83	83	0	83	83	83	83	83	83	83	---	83
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

Table 9-149 Behaviour Factor Acceptance Percentile for 12/8/4/2S-4B-8m

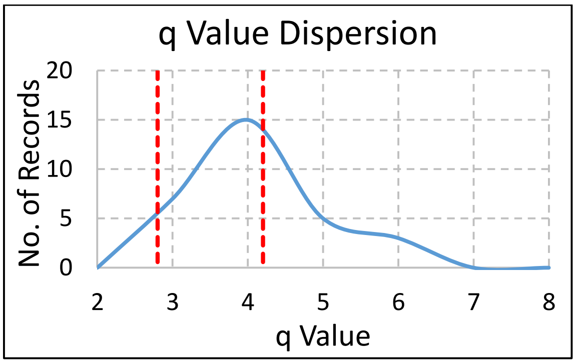
	Fy-dy-1				Fy-dy-2				Fy-dy-3				Fy-dy-4				Fy-dy-5			
dm-1	q1	q2	q3	q4	q5	q6	---	q7	q8	q9	q10	q11	q12	q13	q14	q15	q16	q17	---	q18
	21	25	0	4	42	38	---	8	42	38	29	8	38	33	33	8	42	38	---	4
dm-2	q19	q20	q21	q22	q23	q24	---	q25	q26	q27	q28	q29	q30	q31	q32	q33	q34	q35	---	q36
	54	79	17	13	54	75	---	21	54	75	46	21	58	83	58	17	54	71	---	21
dm-3	q37	q38	q39	q40	q41	q42	---	q43	q44	q45	q46	q47	q48	q49	q50	q51	q52	q53	---	q54
	50	88	17	17	50	67	---	50	50	67	38	50	58	71	54	38	50	67	---	50
dm-4	q55	q56	q57	q58	q59	q60	---	q61	q62	q63	q64	q65	q66	q67	q68	q69	q70	q71	---	q72
	63	88	17	17	46	63	---	46	46	63	38	46	54	67	54	38	46	63	---	46
dm-5	q73	q74	q75	q76	q77	q78	---	q79	q80	q81	q82	q83	q84	q85	q86	q87	q88	q89	---	q90
	63	88	13	17	42	63	---	42	42	63	38	46	50	63	54	42	42	63	---	46
	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4	F1-d1-1	F1-d1-2	F1-d1-3	F1-d1-4

9.13 APPENDIX-H

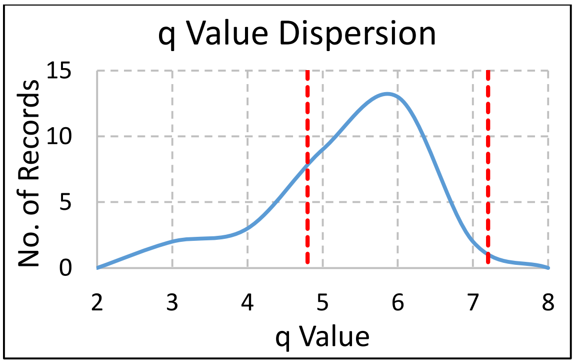
Behaviour Factor Value Dispersion for *Conventional* Structures



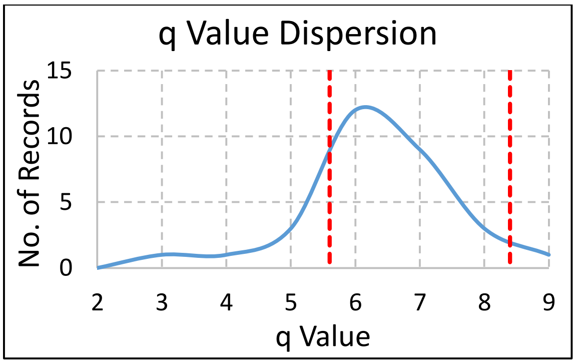
q_{des}=2, 5 q_{acc}=5



q_{des}=3, 4 q_{acc}=3.5

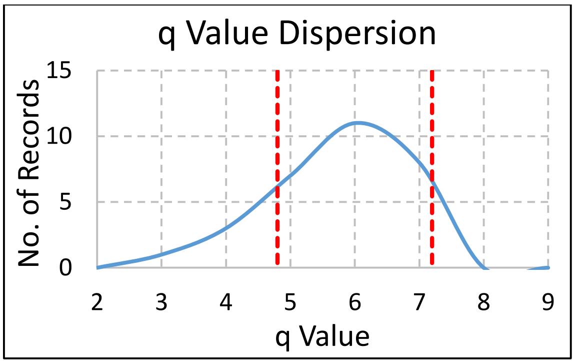


q_{des}=6 q_{acc}=6

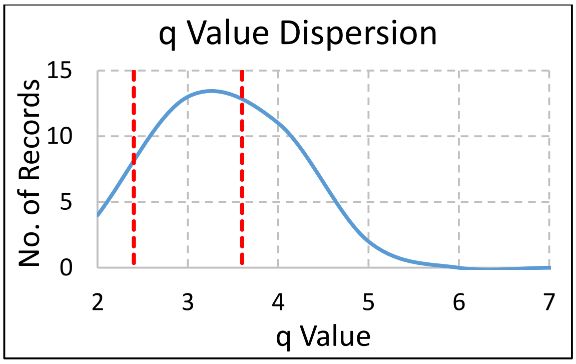


q_{des}=7 q_{acc}=7

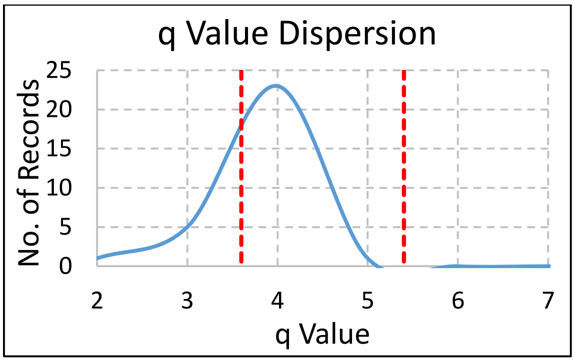
Figure 9-40 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 12S-3B-6m



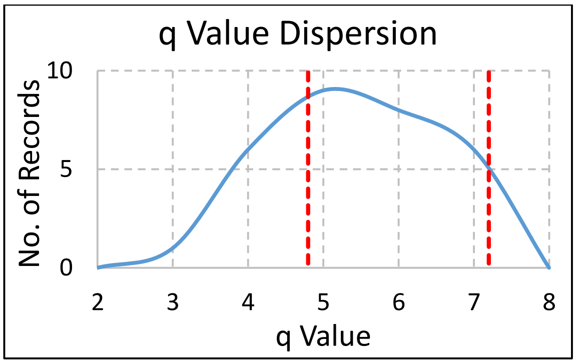
q_{des}=2, q_{acc}=2



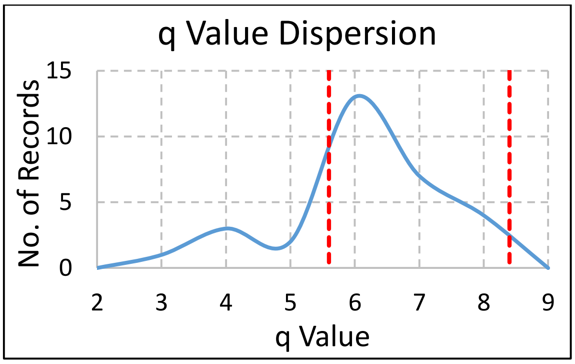
q_{des}=3 q_{acc}=3



q_{des}=4,5 q_{acc}=4.5

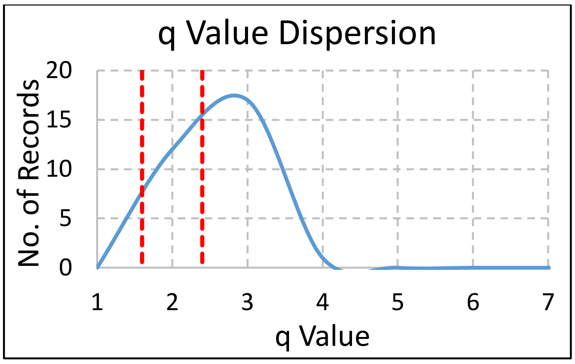


q_{des}=6 q_{acc}=6

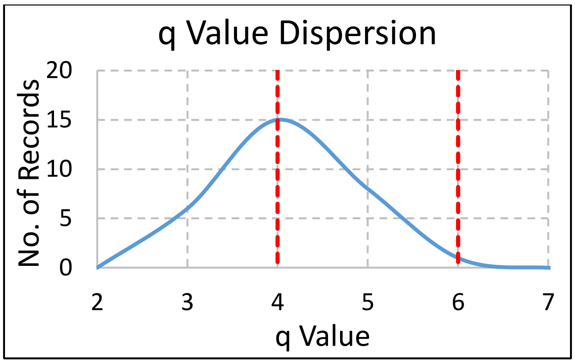


q_{des}=7 q_{acc}=7

Figure 9-41 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 8S-3B-6m

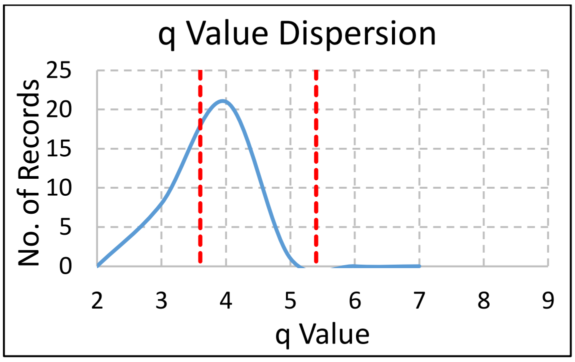


q_{des}=2, q_{acc}=2



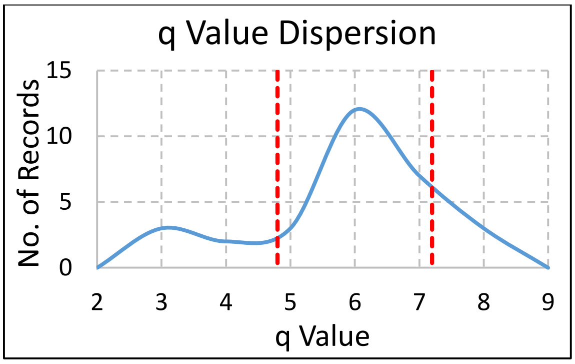
q_{des}=3,4,5,6,7 q_{acc}=5

Figure 9-42 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% of their Design Behaviour Factor for 4S-3B-6m

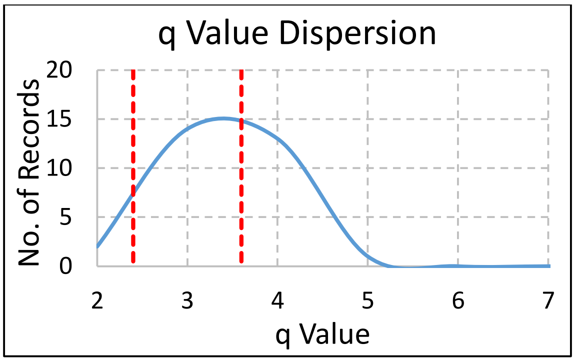


q_{des}=2,3,4,5,6,7 q_{acc}=4.5

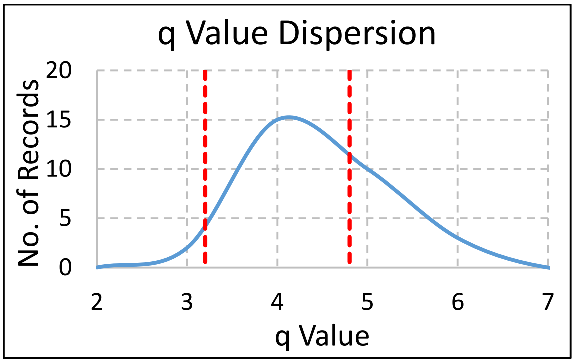
Figure 9-43 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 2S-3B-6m



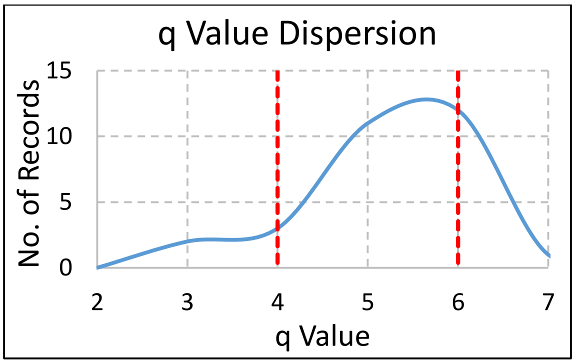
q_{des}=2, q_{acc}=6



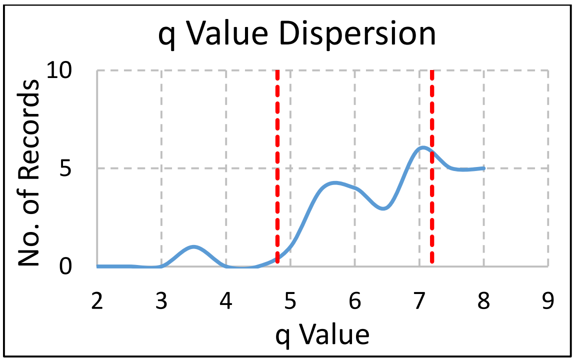
q_{des}=3 q_{acc}=3



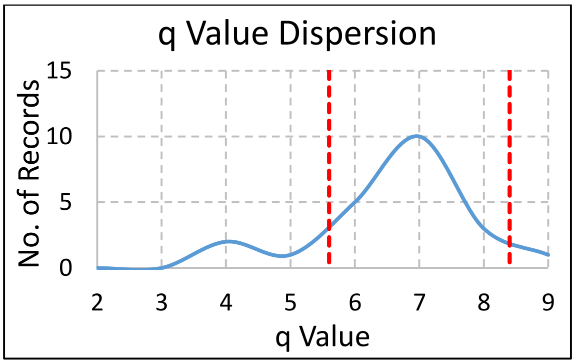
q_{des}=4 q_{acc}=4



q_{des}=5 q_{acc}=5

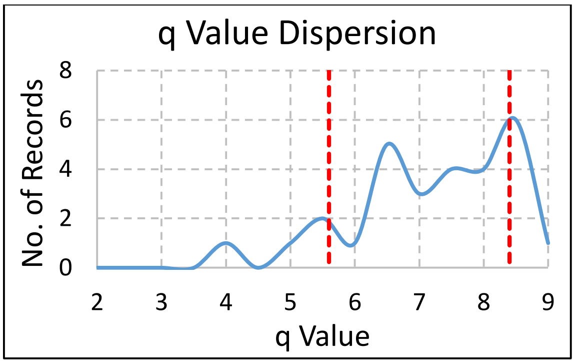


q_{des}=6 q_{acc}=6

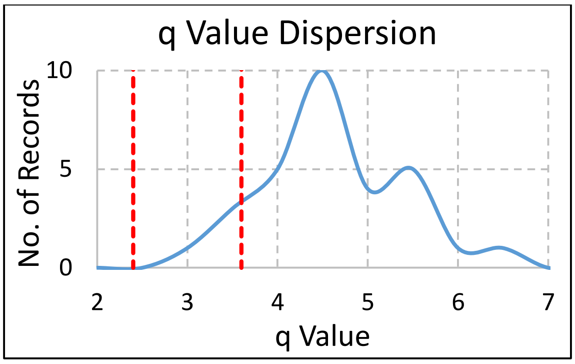


q_{des}=7 q_{acc}=7

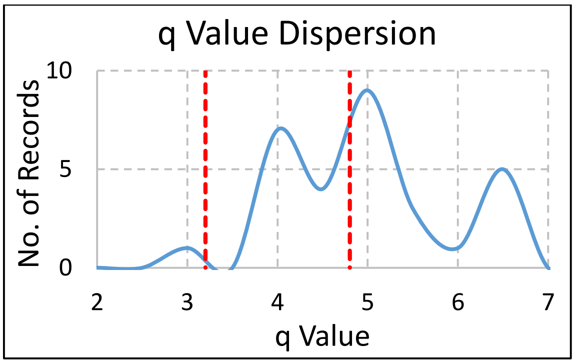
Figure 9-44 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 12S-3B-8m



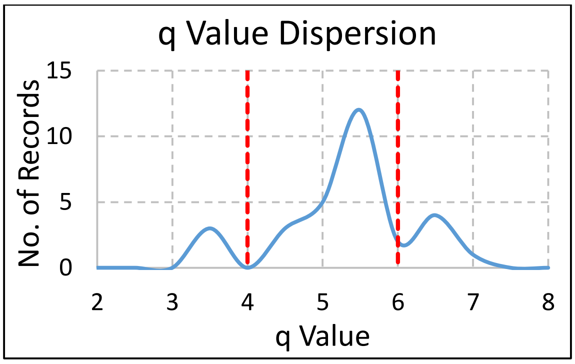
q_{des}=2, q_{acc}=7



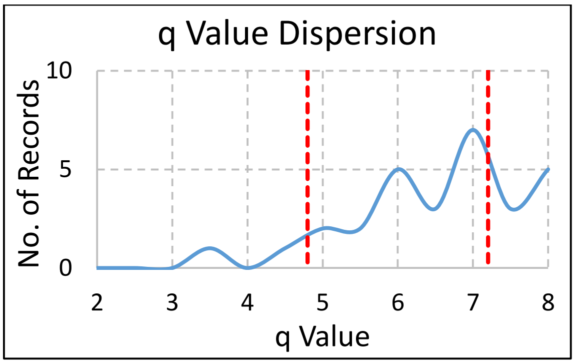
q_{des}=3 q_{acc}=3



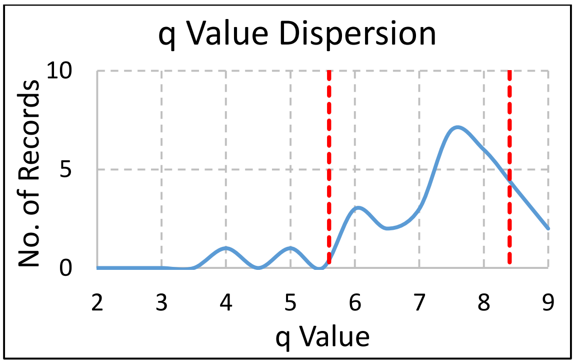
q_{des}=4 q_{acc}=4



q_{des}=5 q_{acc}=5

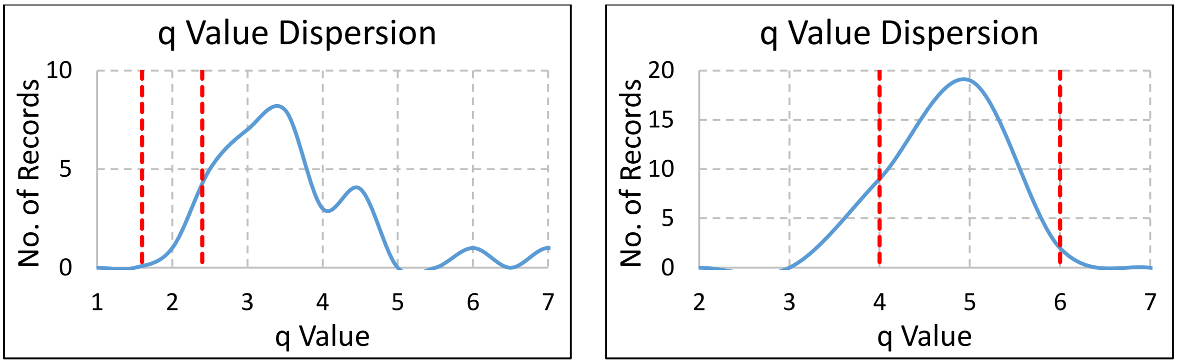


q_{des}=6 q_{acc}=6



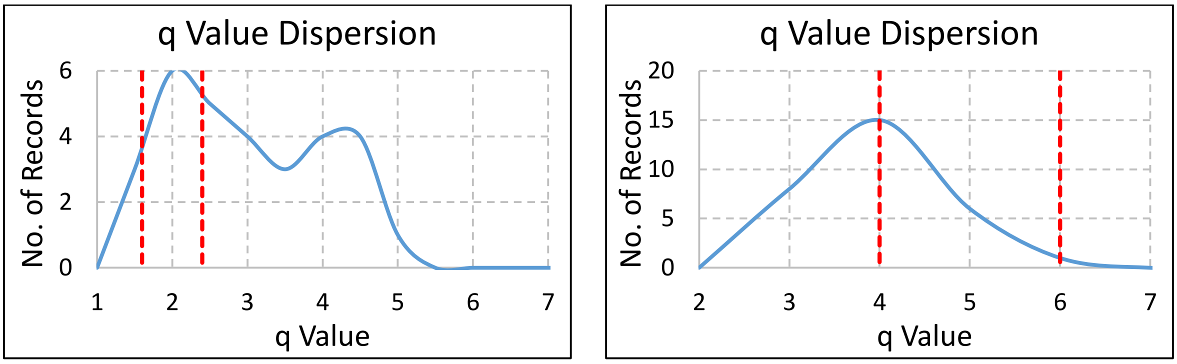
q_{des}=7 q_{acc}=7

Figure 9-45 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 8S-3B-8m



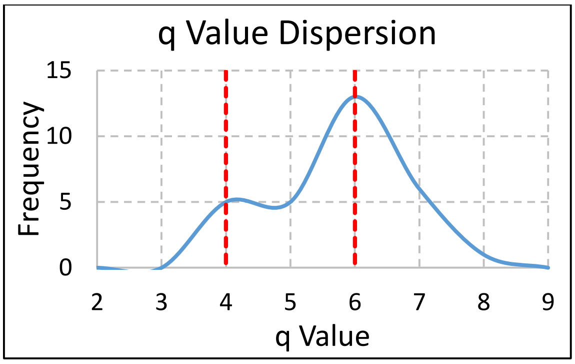
q_{des}=2, q_{acc}=2 q_{des}=3,4,5,6,7 q_{acc}=5

Figure 9-46 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 4S-3B-8m

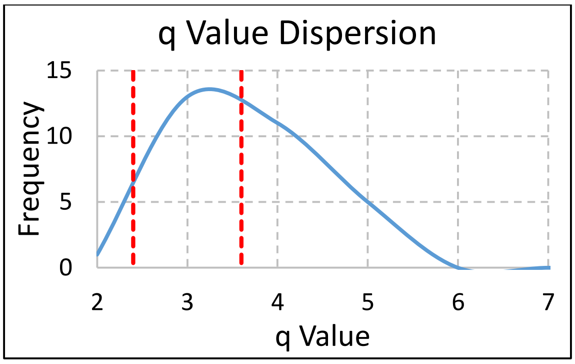


q_{des}=2, q_{acc}=2 q_{des}=3,4,5,6,7 q_{acc}=5

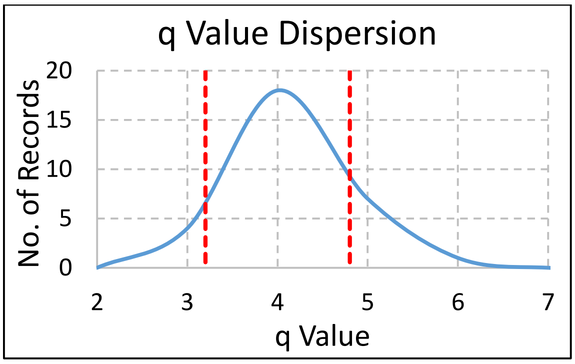
Figure 9-47 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 2S-3B-8



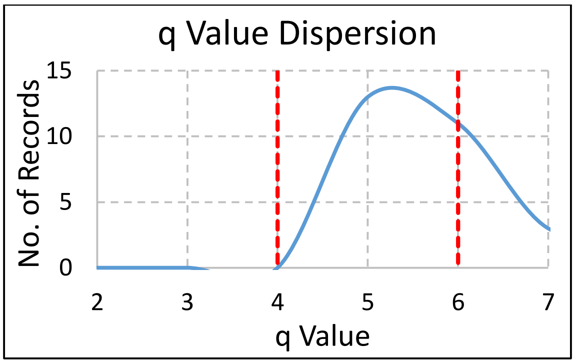
q_{des}=2, q_{acc}=5



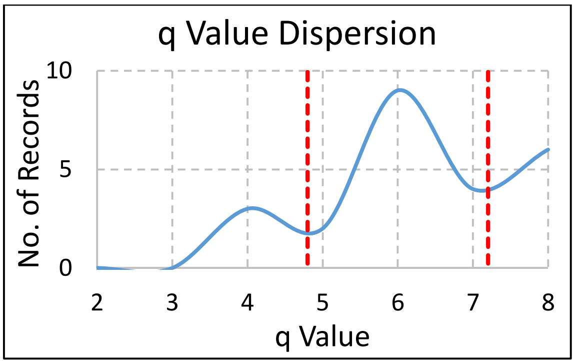
q_{des}=3 q_{acc}=3



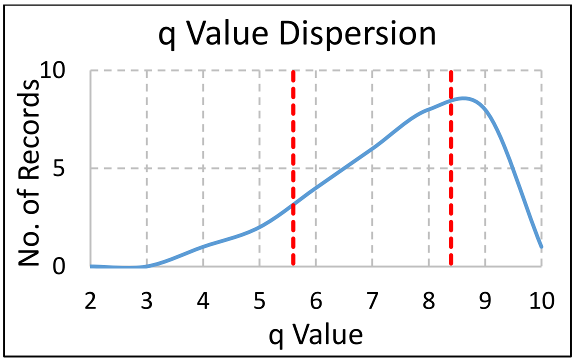
q_{des}=4 q_{acc}=4



q_{des}=5 q_{acc}=5

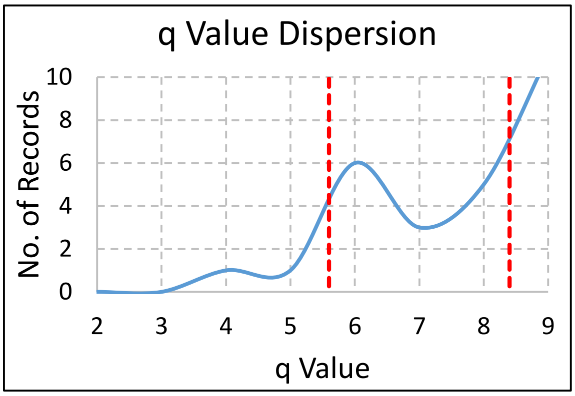


q_{des}=6 q_{acc}=6

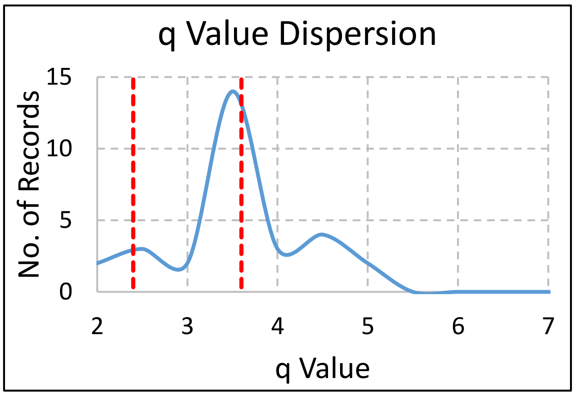


q_{des}=7 q_{acc}=7

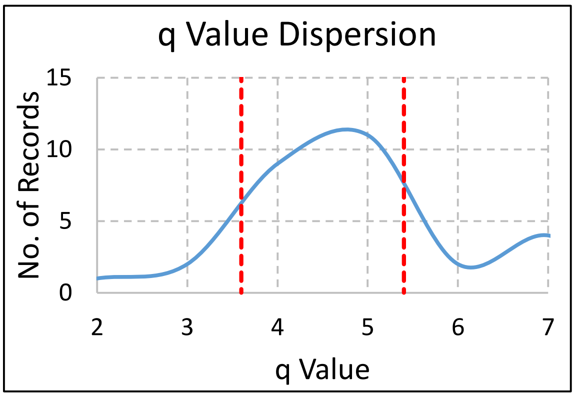
Figure 9-48 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 12S-4B-6m



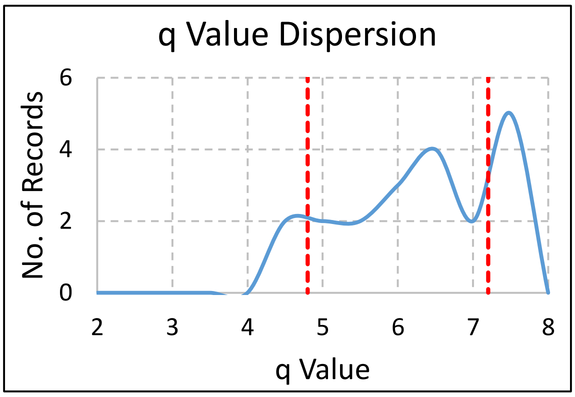
q_{des}=2, q_{acc}=7



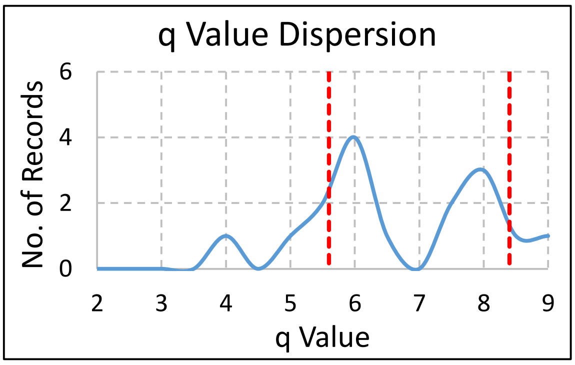
q_{des}=3 q_{acc}=3



q_{des}=4,5 q_{acc}=4.5

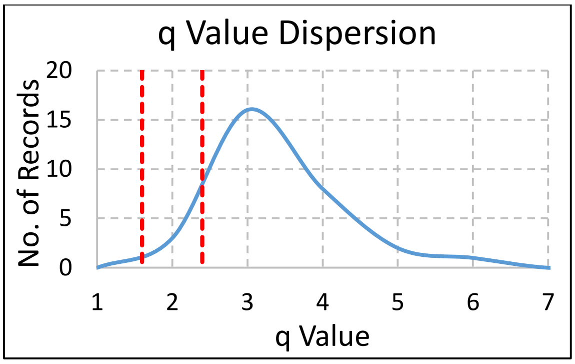


q_{des}=6 q_{acc}=6

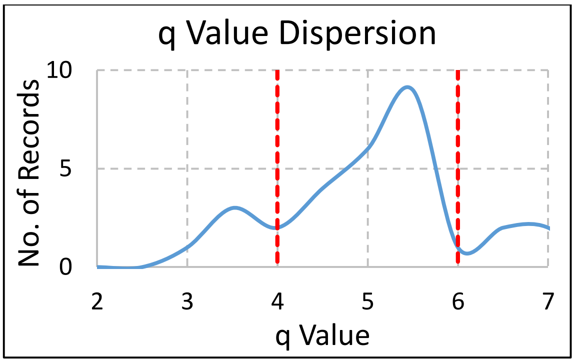


q_{des}=7 q_{acc}=7

Figure 9-49 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 8S-4B-6m

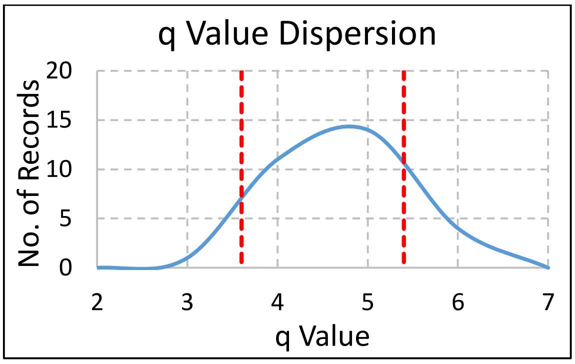


q_{des}=2, q_{acc}=2



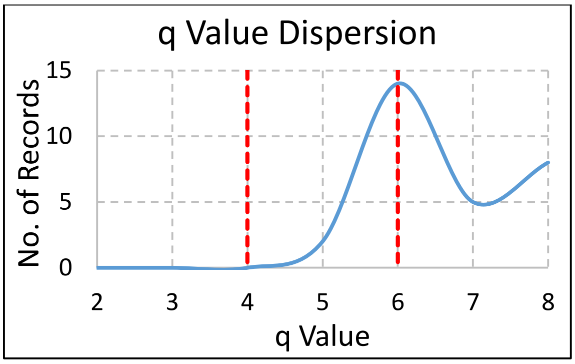
q_{des}=3,4,5,6,7 q_{acc}=5

Figure 9-50 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 4S-4B-6m

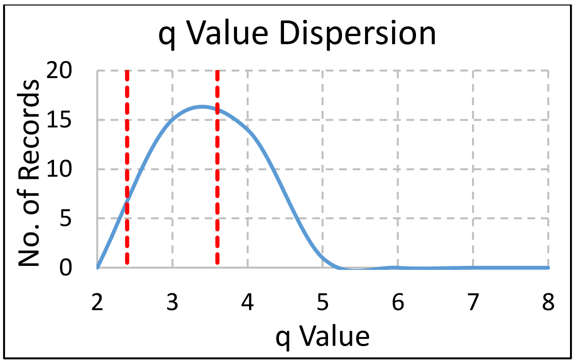


q_{des}=2,3,4,5,6,7 q_{acc}=4.5

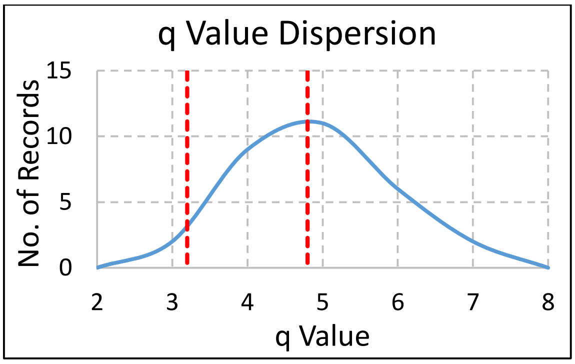
Figure 9-51 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 2S-4B-6m



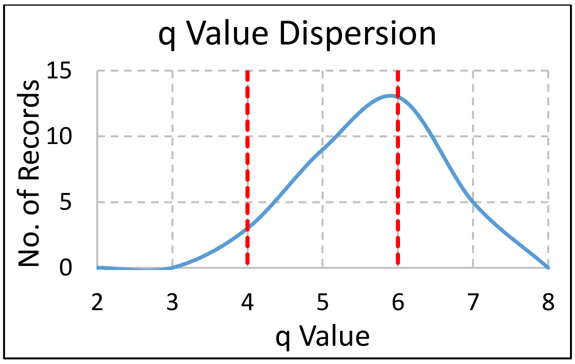
q_{des}=2, q_{acc}=5



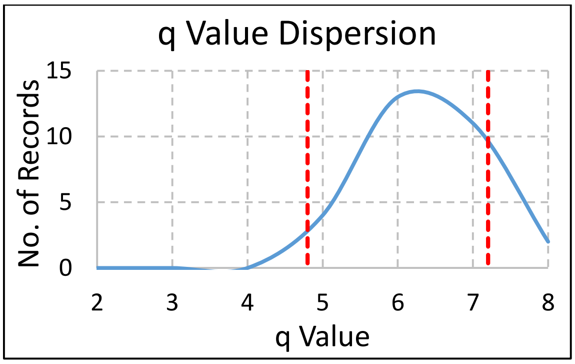
q_{des}=3 q_{acc}=3



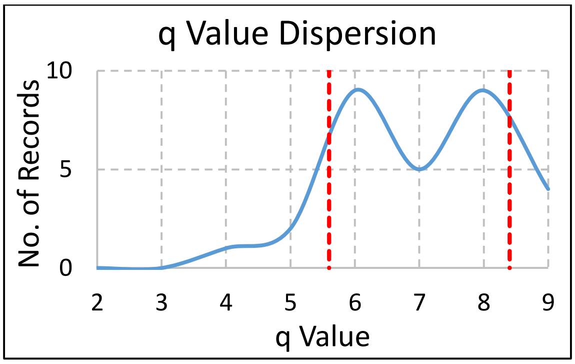
q_{des}=4 q_{acc}=4



q_{des}=5 q_{acc}=5

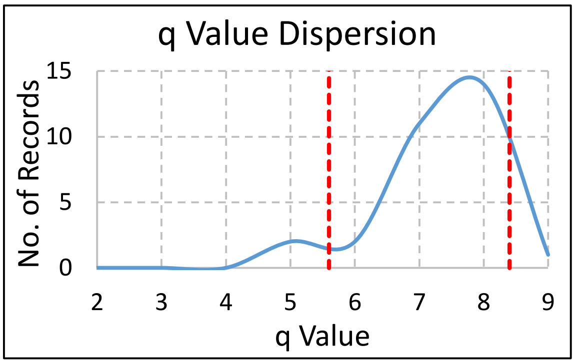


q_{des}=6 q_{acc}=6

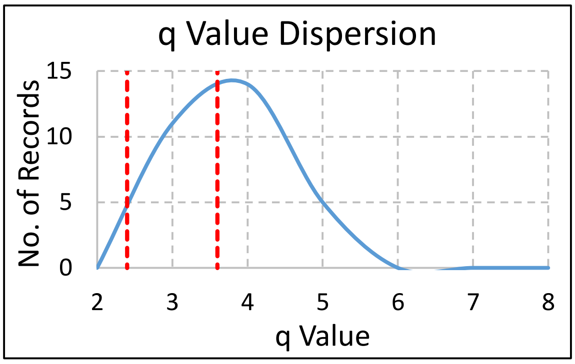


q_{des}=7 q_{acc}=7

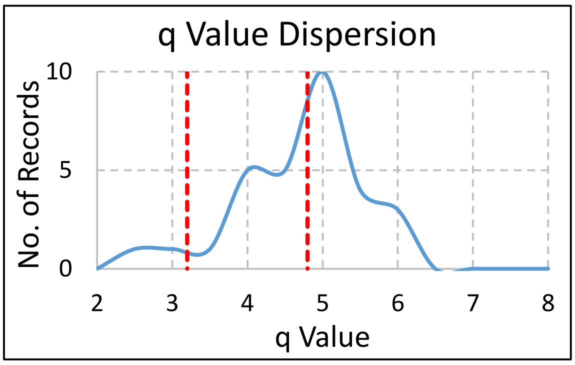
Figure 9-52 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 12S-4B-8m



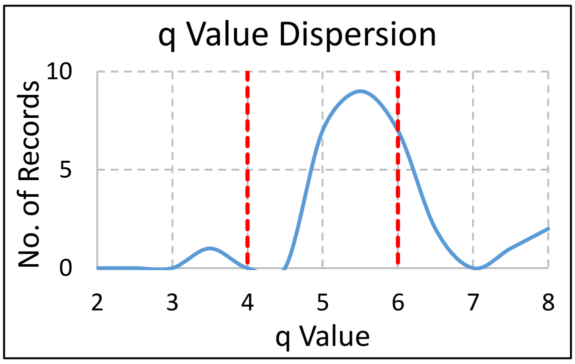
q_{des}=2, q_{acc}=7



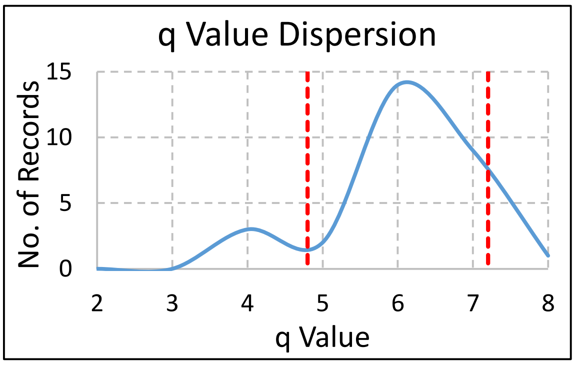
q_{des}=3 q_{acc}=3



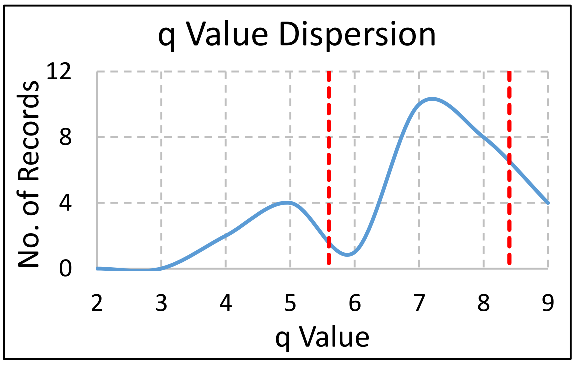
q_{des}=4 q_{acc}=4



q_{des}=5 q_{acc}=5

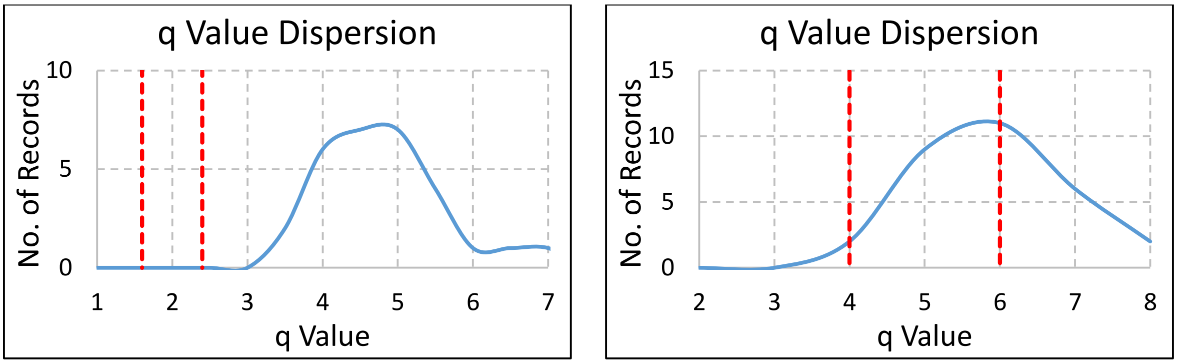


q_{des}=6 q_{acc}=6



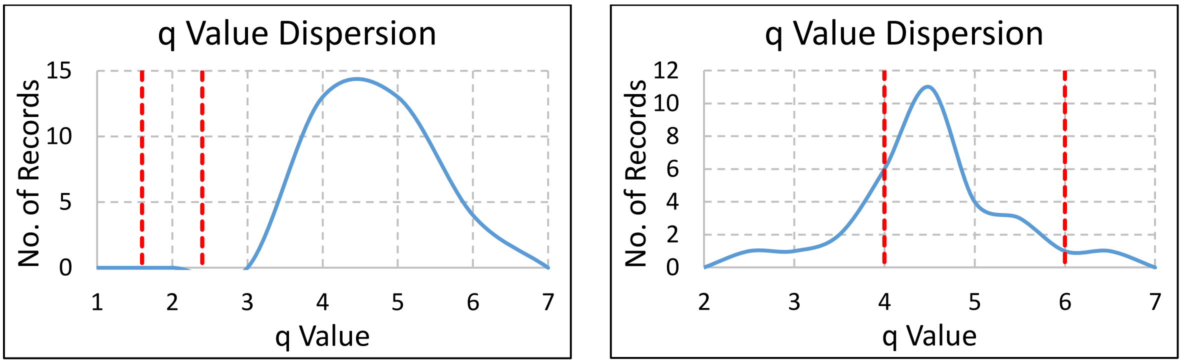
q_{des}=7 q_{acc}=7

Figure 9-53 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 8S-4B-8m



q_{des}=2, q_{acc}=2 q_{des}=3,4,5,6,7 q_{acc}=5

Figure 9-54 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 4S-4B-8m



q_{des}=2 q_{acc}=2 q_{des}=3,4,5,6,7 q_{acc}=5

Figure 9-55 q-Value Dispersion (Obtained by 30 IDAs Record) with Respect to ±20% (the dashed red lines) of their Design Behaviour Factor for 2S-4B-8m