

INELASTIC DEFORMATION DEMANDS ON
MOMENT-RESISTING FRAME STRUCTURES

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ABSTRACT

INELASTIC DEFORMATION DEMANDS ON MOMENT-RESISTING FRAME STRUCTURES

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Interstory drift ratio is an important parameter for the determination of the structural performance under strong ground motions. A probabilistic procedure is proposed in this study to estimate the inelastic maximum interstory drift ratio. The procedure considers the uncertainties associated with the strong ground motions and structural behavior. Elastic and inelastic response history analyses of reinforced-concrete, moment-resisting frames are used together with a near-fault strong ground motion data set to derive the probabilistic procedure. The elastic and inelastic response history analysis results are evaluated in a statistical manner to present the probabilistic approach proposed here. The method presented basically makes use of the fundamental mode properties of the frame systems and modifies the elastic maximum interstory drift ratio by a modifying factor that is determined from the idealized lateral strength capacity (pushover analysis) of the structure. As a part of this thesis, the performance of recently improved nonlinear static procedures that are used in estimating the deformation demands on structural systems are also evaluated using the single- and multi-degree-of-freedom response history analyses results obtained during the conduct of the study.

Keywords: Interstory drift, elastic and inelastic response, pushover analysis, near-fault records, probability

ÖZ

ÇERÇEVE SİSTEMLİ YAPILARDAKİ İNELASTİK DEFORMASYON TALEPLERİ

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Yüksek Lisans, İnşaat Mühendisliği Bölümü

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Kat arası ötelenme oranı, yapıların kuvvetli yer hareketleri altında performanslarını belirlemede önemli bir parametredir. Bu çalışmada, inelastik maksimum kat arası ötelenme oranını belirleyebilmek için olasılık hesaplarına dayalı yaklaşık bir yöntem önerilmektedir. Bu yöntem, hem kuvvetli yer hareketlerinde hem de yapı davranımlarında var olan belirsizlikleri göz önünde bulundurmaktadır. Yöntemi geliştirebilmek için, betonarme çerçeve sistemli yapıların, seçilen yakın mesafe kuvvetli yer hareketleri altında zaman ortamında elastik ve inelastik analizleri yapılmıştır. Bu analizlerin sonuçları istatistiksel bir şekilde incelenerek yöntem oluşturulmuştur. Yöntem, çerçeve sistemlerin temel mod özelliklerinden yararlanarak sistemin idealize edilmiş yanal dayanım kapasitesinden (itme analizi) elde edilen bir faktörü kullanır ve elastik kat arası ötelenme oranını bu faktörle modifiye ederek sonuca ulaşır. Tezin bir bölümü olarak, tek dereceli sistemlerin ve çok dereceli sistemlerin zaman ortamında yapılan elastik ve inelastik analizlerinden elde edilen sonuçlarından yararlanılarak, yapıların deformasyon taleplerini belirlemede kullanılan ve yakın zaman önce iyileştirilen doğrusal olmayan statik yöntemlerin detaylı incelemeleri de yapılmıştır.

Anahtar Kelimeler: Kat arası ötelenme, elastik ve inelastik davranım, itme analizi, yakın mesafe kayıtları, olasılık

To My Family

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LIST OF SYMBOLS

a	: regression constant for C_1
a_1, b_1	: regression constants of Giovenale et al. (2004)
$A - K$	: regression constants of CSM
ADRS	: acceleration-displacement response spectrum
$\mathbf{c}$	: damping matrix
c_1, c_2, c_3, c_4	: coefficients of Akkar et al. (2005a) method
C_0, C_1, C_2, C_3	: coefficients of DCM
C_m	: effective mass factor
COC	: correlation coefficient
COV	: coefficient of variation
CSM	: capacity spectrum method
d	: closest site-to-source distance
$D(t), \dot{D}(t), \ddot{D}(t)$	: relative displacement, velocity and acceleration values of a SDOF system
DCM	: displacement coefficient method
$E[\text{error}]$	: mean error
g	: gravitational acceleration
$GSDR_{sh}$	: maximum ground story drift ratio of a shear beam
h	: story height
H	: building height
I	: moment of inertia
IDR	: interstory drift ratio
k	: number of stories
$\mathbf{k}$	: stiffness matrix

k_1	: initial stiffness
l	: length of column/beam element
L_1	: modal earthquake excitation factor
m	: total mass of the building
$\mathbf{m}$	: mass matrix
M_1	: first-mode modal mass
M_w	: moment magnitude
MDOF	: multi-degree-of-freedom
$MIDR$	: maximum interstory drift ratio
$MIDR_e$	: elastic maximum interstory drift ratio
$MIDR_{ie}$	: inelastic maximum interstory drift ratio
$MRDR$	: maximum roof drift ratio
$MRDR_e$	: elastic maximum roof drift ratio
$MRDR_{ie}$	: inelastic maximum roof drift ratio
n	: total number of ground motions
NEHRP	: national earthquake hazards reduction program
PGA	: peak ground acceleration
PGV	: peak ground velocity
PSA	: pseudo-spectral acceleration
$q_n(t)$	: modal coordinates of the n th mode
$Q, Q_{ym}, Q_{om}, Q_{yc}, T$	: site conditions
R	: normalized lateral strength
RHA	: response history analysis/analyses
S_d	: spectral displacement
$S_{d,e}$	: elastic spectral displacement
$S_{d,ie}$	: inelastic spectral displacement
SDOF	: single-degree-of-freedom
T	: oscillator period

T_0	: initial period of CSM
T_1	: fundamental period
$T_{1,idealized}$	: idealized fundamental period
T_c	: corner period
T_e	: effective fundamental period of DCM
T_{eff}	: effective period of CSM
T_p	: pulse period
T_{pv}	: dominant pseudo-spectral velocity period
u	: displacement value
u_{max}	: maximum value of the displacement
$\mathbf{u}(t), \dot{\mathbf{u}}(t), \ddot{\mathbf{u}}(t)$	: relative displacement, velocity and acceleration vectors
$\ddot{u}_g(t)$	: earthquake induced ground acceleration
V	: base shear force
V_s	: shear wave velocity
V_y	: yield base shear
α	: post-yield stiffness ratio
β_0	: initial damping of CSM
β_{eff}	: effective damping of CSM
Δ	: lateral displacement
Δ_i	: lateral displacement of the i th floor
Δ_{i+1}	: lateral displacement of the $(i+1)$ th floor
Δ_{roof}	: roof displacement
$\Delta_{roof,e}$	: elastic roof displacement
$\Delta_{roof,ie}$	: inelastic roof displacement

$\varepsilon_1, \varepsilon_2, \varepsilon_3, \varepsilon_4,$	
$\varepsilon_{11}, \varepsilon_{12}, \varepsilon_{13}, \varepsilon_{21}, \varepsilon_{22}, \varepsilon_{23}$	
$\varepsilon_{31}, \varepsilon_{32}, \varepsilon_{33}, \varepsilon_{41}, \varepsilon_{42}, \varepsilon_{43}$	: regression constants of the proposed procedure
ϕ_1	: first mode vector
ϕ_1^{roof}	: elastic fundamental mode roof displacement in modal coordinates
ϕ_n, ϕ_j	: n th and j th mode vectors
γ	: normalized strength capacity
γ_1, γ_2	: modification factors of Akkar et al. (2005a) method
Γ_n	: n th mode participation factor
Γ_1	: modal participation factor
η	: base shear coefficient
λ_{MIDR_e}	: log-normal mean of $MIDR_e$
$\lambda_{MIDR_{ie}}$	: log-normal mean of $MIDR_{ie}$
$\lambda_{\Omega(\alpha, R, k)}$	: log-normal mean of $\Omega(\alpha, R, k)$
μ	: ductility
$\mu_{initial}$	: initial ductility
$\mu_{computed}$	: computed ductility
ν	: regression constant of the proposed procedure
ρ	: beam-to-column stiffness ratio
σ_{error}	: standard deviation of error
$\sum W$	: total weight of the building
ω_n	: n th mode natural frequency
Ω	: modifying factor to calculate $S_{d,ie}$ by using $S_{d,e}$
ξ_n	: n th mode damping ratio
ζ_{MIDR_e}	: log-normal standard deviation of $MIDR_e$

$\zeta_{\Omega(\alpha, R, k)}$	: log-normal standard deviation of $\Omega(\alpha, R, k)$
$\zeta_{MIDR_e}^2$	: log-normal variance of $\zeta_{MIDR_e}^2$
$\zeta_{MIDR_{ie}}^2$	: log-normal variance of $\zeta_{MIDR_{ie}}^2$
ζ_{Ω}^2	: log-normal variance of Ω

CHAPTER 1

INTRODUCTION

1.1 General

Deformation demand on structural systems is the key parameter for structural damage. Its estimation plays an important role for the structural performance assessment against seismic excitation. One way of estimating deformation demands on structural systems is to conduct rigorous response history analyses (RHA) that becomes impractical when the structure behaves beyond its elastic capacity. The modeling of the structural system, lack of adequate knowledge in nonlinear structural behavior as well as the limited capacity of the structural analyses programs in handling the complex structural behavior are the main handicaps for conducting RHA to determine the nonlinear deformation demands on structural systems (Chopra, 2004). In most cases, nonlinear RHA is beyond the capacity of practicing engineers and it is mainly utilized by the academia as a benchmark to learn more about the inherent complexity in inelastic structural response. A great effort has been devoted by many professional academic agencies or establishments (e.g., Applied Technology Council, Federal Emergency Management Agency, Pacific Earthquake Engineering Research Center, Mid-America Earthquake Center, Earthquake Engineering Research Center, etc.) to introduce simplified procedures for the estimation of nonlinear deformation demands on structures. Various reports and guidelines have already been published as a result of this effort (e.g., ATC-40 (ATC, 1996); FEMA-273/274 (ATC, 1997); FEMA-356, (ASCE, 2000); FEMA-440 (ATC, 2005)) that provide a significant information about the nonlinear system behavior as well as propose various procedures to determine the most likely deformation demands on structures due to inelastic action. Nevertheless, these

procedures are criticized by many researchers pointing out their deficiencies in determining the structural performance against seismic action.

The main subject of this study is to propose a probabilistic method to estimate the inelastic maximum interstory drift ratio that is an important parameter for assessing the structural damage in frame buildings under earthquakes. This method considers the inherent uncertainties associated with the earthquake action and the behavior of structural systems. The secondary objective of this study is to describe the nonlinear deformation demands on frame buildings in a statistical manner considering the influences of lateral strength capacity, post-yield stiffness ratio and important features of near-fault ground motions that result in significant maximum interstory displacements when compared to far-fault events. The statistical results derived from the secondary objective serve for the probabilistic procedure presented here. As a part of this thesis, the verification of recently improved approximate procedures for estimating global deformation demands is realized in order to point out some deficiencies that are believed to be surmounted partially by the proposed procedure. The study has made use of 78 near-fault ground motions, 24 model frames with 3- to 9-story levels to achieve these objectives.

1.2 Previous research

Many studies have shown that interstory drift ratio, defined as the difference in lateral displacements between two consecutive stories normalized by the story height is the key response parameter as it correlates well with the damage sustained by buildings during strong ground shaking (Algan, 1982; Sözen, 1983; Moehle, 1984, 1994; Qi and Moehle, 1991; Güllkan and Sözen, 1999; Bozorgnia and Bertero, 2001, Ghobarah, 2004a). Therefore, a significant amount of research has been done to

propose approximate methods for estimating the interstory drift demands on structural systems.

To the best of author's knowledge Westergaard (1933) is the pioneer study that describes the lateral deformation demands for blast type loading using a shear beam model. Later Jennings and Newmark (1960) as well as Rosenblueth et al. (1968) used the shear beam model for defining the variation of lateral displacement in structural systems. The practicality of shear beam model, its fairly accurate drift estimations for frame buildings with relatively very stiff girders can be counted as main reasons for its popularity. This model has also been used by Chopra and Chintanapakdee (2001) together with the conventional response spectrum method to determine the drift demands on shear-dominant frame behavior. Iwan (1997) used dispersive wave approach to continuous shear beam model and introduced "drift spectrum" terminology to the engineering literature, a tool that is complementary to response spectrum particularly for near-fault records with pulse signals. Gülkan and Akkar (2002) also used the continuous shear beam model and considered the fundamental-mode behavior to propose an alternative methodology to compute drift spectrum. Heidebrecht and Naumoski (1997) and Heidebrecht and Rutenberg (2000) are other researchers who made use of shear beam behavior to estimate interstory drift demands on structural systems.

Accurate estimations of lateral deformations of many buildings can be obtained by using shear beam models. However, there are also some cases that shear beam model assumption yields unrealistic behaviors (Rinne, 1960; Blume, 1968; Chopra and Cruz, 1986; Uang and Maarouf, 1993). For moment-resisting frames where the lateral stiffness provided by the columns is significant relative to the beams or where axial deformations are significant, shear beam model assumption does not give accurate results. Similarly, the models in which shear type deformations are completely neglected (i.e., structures deforming in flexure) do not yield accurate

lateral deformation results. Note that the use of shear beam model forces the maximum interstory drift to occur at the ground story level that is not true for general frame behavior. Thus, the most accurate building behavior can be achieved by considering both shear and flexural type deformations in building models.

The combined effect of shear and flexure in structural models was initially used by Khan and Sbarounis (1964). Later, Csonka (1965), Rosman (1967), Heidebrecht and Stafford Smith (1973), Fajfar and Stojnik (1981), Miranda (1999), Zalka (2000), Miranda and Reyes (2002), Potzta and Kollár (2003), Akkar et al. (2005a) and Miranda and Akkar (2006) conducted studies in the estimation of drift demands for structural systems under the influence of shear and flexure. Heidebrecht and Stafford Smith (1973), used the solution of a fourth order partial differential equation that represents the behavior of wall and frame type structures to estimate the lateral displacements and internal stresses under earthquake action. Heidebrecht and Naumoski (1997) and Heidebrecht and Rutenberg (2000) proposed methods to determine the maximum interstory drift ratio of the building models in the elastic range that respond in the first mode. Miranda (1999) used the fourth order differential equation derived by Heidebrecht and Stafford Smith (1973) and proposed an approximate method to determine the elastic and inelastic maximum interstory drift ratio in building models responding primarily in the fundamental mode and having uniform lateral stiffness along their heights. As a follow up study Miranda and Reyes (2002) extended the method by Miranda (1999) to cases in which the lateral stiffness does not remain constant along the building height. They showed that the maximum interstory drift does not differ significantly for the uniform and nonuniform lateral stiffness cases in the elastic range. This study is also confined to the elastic systems responding in the first mode. Gülkan and Akkar (2002) defined an alternative expression for drift spectrum by considering only the shear beam fundamental mode of vibration and using response spectrum ordinates. They estimated the ground story drift within an error bound of $\pm 10\%$ for shear

frames with fundamental periods less than 2 s subjected to near-fault ground motions. Later, Akkar et al. (2005a) modified the procedure by Güllkan and Akkar (2002) for the combined shear and flexure behavior in frame systems. The procedure proposed in Akkar et al. (2005a) uses beam-to-column stiffness ratio to consider the contributions from shear and flexure and estimates the maximum ground story and interstory drift ratio. Similar to most of the studies presented above, this procedure also considers the first-mode dominant elastic frame behavior.

In an attempt to describe drift demands, Gupta and Krawinkler (2000) proposed a procedure to calculate seismic drift demands for frame structures from the spectral displacement demand at the first mode of the structure. The spectral displacement demand is related to the seismic drift demand by using three modification factors to account for multi-degree-of-freedom (MDOF) effects, inelasticity effects and $P - \Delta$ effects. This procedure and the one presented in Miranda (1999) form the basis of the current Displacement Coefficient Method presented in detail in the FEMA-273/274 (ATC, 1997) and the following FEMA-356 (ASCE, 2000) documents. More recently, Medina and Krawinkler (2005) proposed a method by using regression techniques to estimate the elastic and inelastic drift demands for moment-resisting non-degrading frame structures subjected to ground motions without pulse signals.

The most recent study for computing elastic interstory drift spectrum for general frame behavior in the elastic range is proposed by Miranda and Akkar (2006) using the continuum model by Heidebrecht and Stafford Smith (1973) that consists of a combination of a flexural beam and a shear beam. This method is based on modal analysis technique and showed that for most cases consideration of at most 6 modes would yield “exact” maximum interstory drifts when compared to the results of RHA. This study also showed that Iwan’s (1997) shear beam model would yield significantly conservative drifts for short-period structures.

The literature cited in above paragraphs indicate that most of the models used in estimating maximum interstory drift are based on (a) fundamental-mode assumption, (b) elastic behavior assumption. The fundamental-mode assumption seems to yield fairly good results for short- to medium-rise structures. The elastic drift results, on average, can be acceptable except for short-period structures as within this spectral period range deformation demands are very sensitive to the changes in ductility and lateral strength capacity of the structure. Moreover, the estimation of nonlinear drift becomes more important for buildings subjected to near-fault ground motions. The distinct features of near-fault records (high peak ground velocity, pulse signal and fling step) may alter the drift demands significantly as indicated in many studies (Anderson and Bertero, 1987; Hall et al., 1995; Iwan, 1997; Alavi and Krawinkler, 2000, 2004; Báez and Miranda, 2000; Iwan et al., 2000; Cuesta et al., 2003; Akkar et al., 2004; Akkar et al., 2005a).

In this study, maximum interstory drift ratio of building models will be calculated considering that the building models behave beyond their elastic limits and the proposed method in Akkar et al. (2005a) study will be modified to account for the nonlinear frame behavior and the uncertainty associated with the interaction between building nonlinear behavior and ground motion features.

1.3 Conduct of the study

Reinforced-concrete moment-resisting frames of 3-, 5-, 7- and 9-stories are used in this study. They are subjected to ground motions with and without pulse signals of different peak ground velocity values. In Chapter 2, the selection and grouping of strong ground motion records are explained in detail. Their important seismological features also presented within the context of this chapter. The information regarding to the model buildings used in this study, their important dynamic characteristics

together with the corresponding equivalent single-degree-of-freedom (SDOF) system representations are also presented in Chapter 2. The equivalent SDOF system representation of the model buildings are computed using pushover analysis that is discussed in detail in this chapter.

Chapter 3 reports the results of RHA for building models and corresponding equivalent SDOF systems in a statistical manner. The median and different percentile variations of lateral displacement profiles, interstory drift demands as well as their corresponding dispersions along the building height both in the elastic and inelastic ranges are displayed. The variation between normalized strength capacity versus maximum interstory drift ratio and maximum roof drift ratio are investigated and the information on the normalized lateral strength of corresponding equivalent SDOF systems are given. The correlations between the deformation demands computed from the building models and corresponding equivalent SDOF systems are investigated. The log-normal distribution assumption for nonlinear demand variations of building models and equivalent SDOF systems are also verified in Chapter 3. This chapter also evaluates the current approach in transforming the SDOF deformation demands to corresponding MDOF deformation demands that is presented in the approximate nonlinear static procedures.

Chapter 4 verifies the current nonlinear static procedures for estimating the deformation demands on structural systems. The improved Displacement Coefficient Method and the equivalent linear method presented in the FEMA-440 document (ATC, 2005) are evaluated using the building models and ground motions used in this study. The results are presented in a statistical manner. The observations highlighted in this chapter are used in Chapter 5 for the probabilistic method proposed in this study.

In Chapter 5, the probabilistic approach for estimating inelastic maximum interstory drift ratio demands is derived by modifying Akkar et al. (2005a) method. The basic probabilistic concepts used in handling the variations in lateral strength and post-yield stiffness are described in detail. A case study is also given at the end of this chapter to illustrate the proposed procedure.

Chapter 6 summarizes the conclusions of this study and provides complementary studies for further improvements related to the proposed procedure.

CHAPTER 2

GROUND MOTIONS AND BUILDING MODELS

2.1 Brief description of the ground motions

A total of 78 strong ground motions, recorded on soil sites whose average shear wave velocity values range between $180 \text{ m/s} < V_s < 750 \text{ m/s}$ in the upper 30 m soil profile were used in this study. The records were downloaded from the COSMOS Virtual Data Center (COSMOS, 2004) and represent moderate to large events with moment magnitudes varying between $5.7 \leq M_w \leq 7.6$. The selected strong ground motions can be designated as near-fault records as their closest site-to-source distances vary between $1 \text{ km} < d < 25 \text{ km}$.

Larger deformation demands of near-fault ground motions recorded in the proximity of causative source are of concern in the design and seismic performance evaluation of structures. Seismological studies conducted on the near-fault ground-motion records have put forward to 2 important features of such records; (a) Higher ground velocity with respect to the far-fault ground motions, (b) Pulse-dominant waveforms that reflect the characteristics of rupture and wave propagation process during the main event (Somerville et al., 1997; Bray and Rodriguez-Marek, 2004). The influence of these seismological features in the near-fault ground-motion records should be considered separately. Four different ground-motion groups that are presented in Tables 2.1 to 2.4 were assembled based on these particular ground-motion features that are specific to near-fault events. The criteria used while assembling these groups are explained in the succeeding paragraphs.

Table 2.1 Group A ground motions, $40 \text{ cm/s} \leq PGV < 60 \text{ cm/s}$

Record #	Earthquake	Magnitude (M_w)	Fault	Station	Component	Site *	d^{**} (km)	PGA^{**} (cm/s^2)	PGV^{**} (cm/s)
1	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	Taichung - Chungming School, TCU051	360	NEHRP - D	7.0	230.0	40.6
2	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	Taichung - Taichung City, TCU082	360	NEHRP - D	4.5	182.1	41.0
3	Imperial Valley, 10/15/79	6.5	SS (right)	Dogwood Rd., Diff. Array, El Centro	NS	Alluvium; Q	5.6	473.6	41.1
4	Imperial Valley, 10/15/79	6.5	SS (right)	Aeropuerto Mexicali	45	Deep Alluvium	3.2	284.9	42.0
5	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	Chiayi - Meishan School, CHY006	360	NEHRP - D	14.5	351.8	42.1
6	Cape Mendocino, 04/25/92	7.0	thrust/reverse	Rio Dell - 101/Painter St. Overpass	360	NEHRP - C	18.5	538.5	42.6
7	Landers, 06/28/92	7.3	SS (right)	Joshua Tree - Fire Station	90	Shallow Alluvium Over Granite Bedrock; Q	10.0	278.4	42.7
8	Imperial Valley, 10/15/79	6.5	SS (right)	Bonds Corner	140	Alluvium; Q	4.4	578.0	44.3
9	Imperial Valley, 10/15/79	6.5	SS (right)	Bonds Corner	230	Alluvium; Q	4.4	762.4	45.0
10	Imperial Valley, 10/15/79	6.5	SS (right)	McCable School, El Centro Array #11	S50W	Alluvium; Q	12.4	362.5	45.2
11	Imperial Valley, 10/15/79	6.5	SS (right)	Community Hospital, Keystone Rd., El Centro Array #10	N40W	Alluvium; More Than 300 m Q	8.7	226.6	46.0
12	Cape Mendocino, 04/25/92	7.0	thrust/reverse	Petrolia	0	Alluvium	9.5	578.1	48.3
13	Imperial Valley, 10/15/79	6.5	SS (right)	James Rd., El Centro Array #5	S40E	Alluvium; More Than 300 m Q	5.2	539.8	49.7
14	Kocaeli, 8/18/99	7.4	SS (right)	Düzce	SN	Alluvium	17.1	307.8	50.7
15	Northridge, 01/17/94	6.7	thrust/reverse	Pacoima - Kagel Canyon	360	Tertiary Sandstone	10.6	424.2	50.9
16	Düzce, 11/12/99	7.1	SS (right)	Bolu	NS	Alluvium	20.4	722.1	55.2
17	Loma Prieta, 10/18/89	7.0	oblique	Corralitos - Eureka Canyon Rd.	0	Landslide Deposits	2.8	617.7	55.2
18	Northridge, 01/17/94	6.7	thrust/reverse	14145 Mulholland Rd., Beverly Hills, CA	N09E	NEHRP - D	19.6	419.3	57.9
19	Northridge, 01/17/94	6.7	thrust/reverse	17645 Saticoy St.	S00E	NEHRP - D	13.3	428.7	59.8
20	Northridge, 01/17/94	6.7	thrust/reverse	7769 Topanga Canyon Blvd., Canoga Park	S16W	Alluvium; Qym Q	15.7	381.0	59.8

* Abbreviations for site conditions: NEHRP - C: $360 \text{ m/s} < V_s < 750 \text{ m/s}$, Q: Quaternary ($V_s = 333 \text{ m/s}$), NEHRP - D: $180 \text{ m/s} < V_s < 360 \text{ m/s}$, T: Tertiary ($V_s = 406 \text{ m/s}$), Qym: Holocene, medium-grained sediment

** Abbreviations designated by d , PGA and PGV : closest site-to-source distance, peak ground acceleration and velocity, respectively

Table 2.2 Group B ground motions, $20 \text{ cm/s} \leq PGV < 40 \text{ cm/s}$

Record #	Earthquake	Magnitude (M_w)	Fault	Station	Component	Site *	d^{**} (km)	PGA^{**} (cm/s^2)	PGV^{**} (cm/s)
1	Whittier Narrows, 10/01/87	6.1	thrust/reverse	Los Angeles - Oregon Park	360	Alluvium; Qom Q	14.2	420.1	21.8
2	Northridge, 01/17/94	6.7	thrust/reverse	Los Angeles - UCLA Grounds	360	Alluvium; Qom Q (Medium - Grained Sediment (Sand))	22.9	464.6	21.9
3	Northridge, 01/17/94	6.7	thrust/reverse	Los Angeles - UCLA Grounds	90	Alluvium; Qom Q (Medium - Grained Sediment (Sand))	22.9	272.4	22.0
4	Imperial Valley, 10/15/79	6.5	SS (right)	Community Hospital, Keystone Rd., El Centro Array #10	S50W	Alluvium; More Than 300 m	8.7	117.3	22.9
5	Loma Prieta, 10/18/89	7.0	oblique	Gilroy - Gavilan Coll	337	Terrace Deposit Over Sandstone	3.0	310.0	23.0
6	Northridge, 01/17/94	6.7	thrust/reverse	6850 Coldwater Canyon Ave., North Hollywood	S00W	Alluvium; Qyc Q	12.5	296.0	23.1
7	Imperial Valley, 10/15/79	6.5	SS (right)	Aeropuerto Mexicali	315	Deep Alluvium	3.2	249.9	23.9
8	Northridge, 01/17/94	6.7	thrust/reverse	Los Angeles, Brentwood V.A. Ho	195	Alluvium; Qom Q	23.1	182.1	24.0
9	Parkfield, 06/27/66	6.1	SS (right)	Cholame, Shandon, Array No.5	N85E	Alluvium; Sandstone Q	7.1	425.7	25.4
10	Whittier Narrows, 10/01/87	6.1	thrust/reverse	7420 Jaboneria, Bell Gardens	N63W	Alluvium; Qym Q	16.4	215.9	28.0
11	Loma Prieta, 10/18/89	7.0	oblique	Gilroy - Gavilan Coll	67	Terrace Deposit Over Sandstone	3.0	349.1	28.9
12	Imperial Valley, 10/15/79	6.5	SS (right)	Casa Flores, Mexicali	270	Alluvium	9.8	414.7	31.5
13	Coyote Lake, 08/06/79	5.7	SS (right)	Gilroy Array No.2	140	Alluvium	6.0	248.9	31.9
14	Imperial Valley, 10/15/79	6.5	SS (right)	Keystone Rd., El Centro Array #2	S40E	Alluvium ($V_s=333 \text{ m/s}$ Quaternary)	16.2	309.4	32.7
15	Loma Prieta, 10/18/89	7.0	oblique	Gilroy #2 - Hwy 101/Bolsa Rd.	0	Alluvium	4.5	344.2	33.3
16	Loma Prieta, 10/18/89	7.0	oblique	Gilroy #3 - Gilroy Sewage Plant	0	Alluvium	6.3	531.7	34.5
17	Loma Prieta, 10/18/89	7.0	oblique	Saratoga - 1 - School Gym	270	Alluvium	13.7	347.3	37.2
18	Imperial Valley, 10/15/79	6.5	SS (right)	Anderson Rd., El Centro Array #4	S40E	Alluvium; More Than 300 m	8.3	480.8	38.1
19	Loma Prieta, 10/18/89	7.0	oblique	Gilroy #2 - Hwy 101/Bolsa Rd.	90	Alluvium	4.5	316.3	39.2
20	Morgan Hill, 04/24/84	6.1	SS (right)	Halls Valley	240	Alluvium	2.5	305.8	39.6

* Abbreviations for site conditions: Qom: Pleistocene, medium-grained sediment, Q: Quaternary ($V_s = 333 \text{ m/s}$), Qyc: Holocene, coarse grained sediment, Qym: Holocene, medium-grained sediment
** Abbreviations designated by d , PGA and PGV : closest site-to-source distance, peak ground acceleration and velocity, respectively

Table 2.3 Group C ground motions, $40 \text{ cm/s} \leq PGV < 60 \text{ cm/s}$

Record #	Earthquake	Magnitude (M_w)	Fault	Station	Component	Site *	d^{**} (km)	PGA^{**} (cm/s^2)	PGV^{**} (cm/s)	T_p^{**} (s)	T_{pT}^{**} (s)
1	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU087	0	NEHRP - C	3.4	111.3	39.9	4.28	4.0
2	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU087	90	NEHRP - C	3.4	119.0	42.2	8.74	8.7
3	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	NSY	0	NEHRP - C	9.1	114.1	41.9	4.53	4.8
4	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CHY035	90	NEHRP - D	15.2	246.1	44.0	1.19	1.4
5	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU053	0	NEHRP - D	5.5	131.5	45.8	6.62	6.1
6	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU104	0	NEHRP - C	12.5	86.1	47.4	6.93	5.5
7	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU057	0	NEHRP - C	11.2	100.5	47.5	6.98	7.5
8	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU059	90	NEHRP - D	16.5	157.7	50.9	6.66	6.3
9	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU064	0	NEHRP - D	12.2	113.0	54.3	6.71	6.8
10	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro Array Sta 10 - El Centro Community Hospital - Imperial & Ross	320	Alluvium; More Than 300 m	8.7	226.6	46.0	2.05	1.5
11	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro Array Sta 10 - El Centro Community Hospital - Imperial & Ross	50	Alluvium; More Than 300 m	8.7	166.1	49.6	5.11	6.4
12	Imperial Valley, 10/15/79	6.5	SS (right)	Holville - Post Office	315	Alluvium	8.8	209.4	51.2	4.28	3.4
13	Imperial Valley, 10/15/79	6.5	SS (right)	Holville - Post Office	225	Alluvium	8.8	243.0	51.9	6.39	4.0
14	Loma Prieta, 10/18/89	7.0	oblique	CSMIP: Gilroy Array STA 3 Sewage Plant	90	Alluvium	6.3	362.0	43.8	2.46	2.0
15	Loma Prieta, 10/18/89	7.0	oblique	Saratoga, CA	90	Alluvium; More Than 300 m	4.1	316.2	43.6	2.13	3.9
16	Loma Prieta, 10/18/89	7.0	oblique	Corralitos, CA	90	Landslide Deposits	2.8	469.4	47.5	0.70	0.8
17	Cape Mendocino/Petaluma, 04/25/92	7.0	thrust/reverse	Rio Dell - HWY 101 & Painter St. Overpass	270	Deep Stiff Soil	18.5	378.3	44.7	1.17	1.3
18	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	TCU036	90	NEHRP - D	12.7	134.5	57.5	5.14	5.0
19	Northridge, 01/17/94	6.7	thrust/reverse	Newhall, CA Sun Oil Co W Pico Canyon Blvd	N44W	Nonmarine Deposit; T	9.4	348.2	59.9	2.05	1.8
20	Loma Prieta, 10/18/89	7.0	oblique	Gilroy Array STA 2 Hwy 101 & Bolsa Rd	90	Alluvium	4.5	316.3	39.2	1.34	1.5

* Abbreviations for site conditions: NEHRP - C: $360 \text{ m/s} < V_s < 750 \text{ m/s}$, NEHRP - D: $180 \text{ m/s} < V_s < 360 \text{ m/s}$, T: Tertiary ($V_s = 406 \text{ m/s}$)

** Abbreviations designated by d , PGA and PGV : closest site-to-source distance, peak ground acceleration and velocity, respectively

Table 2.4 Group D ground motions, $PGV \geq 60$ cm/s

Record #	Earthquake	Magnitude (M_w)	Fault	Station	Component	Site *	d^{**} (km)	PGA^{**} (cm/s^2)	PGV^{**} (cm/s)	T_p (s)	T_{mp} (s)
1	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Taichung, Taiwan TCU128	90	NEHRP - C	9.1	140.4	60.2	7.44	7.9
2	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Taichung, Taiwan TCU128	0	NEHRP - C	9.1	163.5	62.2	4.63	4.7
3	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB, Taiwan WCK	90	NEHRP - D	13.3	338.6	60.4	3.37	3.2
4	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Taichung, Taiwan TCU102	0	NEHRP - D	1.2	167.5	65.7	3.60	2.4
5	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Chiayi, Taiwan CHY101	90	NEHRP - D	13.3	326.5	65.9	4.27	3.2
6	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Taichung, Taiwan TCU103	90	NEHRP - D	2.4	126.5	67.4	6.03	7.3
7	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Taichung, Taiwan TCU101	90	NEHRP - D	1.9	207.3	74.5	9.49	8.2
8	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro Array Sta. 6 - Huston Road - 551 Huston Rd.	140	Alluvium; More Than 300 m	3.5	332.4	66.5	2.83	2.4
9	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro, CA - Differential Array	270	Alluvium	5.6	339.3	74.4	5.89	2.6
10	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro Array Sta. 4 - Anderson R - 2905 Anderson Rd.	230	Alluvium; More Than 300 m	8.3	349.2	79.3	5.13	4.0
11	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro, CA - Meloland Route 8 Overpass	270	Alluvium	3.1	292.3	91.8	3.17	3.0
12	Imperial Valley, 10/15/79	6.5	SS (right)	El Centro Array Sta 5 - James Road - 2801 James	270	Alluvium; More Than 300 m	5.2	360.4	95.9	3.83	3.6
13	Chi-Chi, Taiwan, 09/20/99	7.6	thrust/reverse	CWB: Taichung, Taiwan TCU063	0	NEHRP - D	10.3	129.9	82.1	4.61	4.5
14	Cape Mendocino/Petrolia, 04/25/92	7.0	thrust/reverse	Petrolia, CA - General Store	90	Alluvium	H15.9	649.4	89.5	1.93	0.7
15	Parkfield, 06/28/66	6.1	SS (right)	Cholame 2WA	65	Alluvium; 45 m; Sandstone Q	3.5	479.6	78.1	0.90	0.7
16	Tabas, Iran 09/16/78	7.3	thrust/reverse	9101 Tabas Station	NS	NEHRP-C	2.1	1037.0	98.1	4.50	5.0
17	Erzincan, Turkey 03/13/92	6.5	SS (right)	Erzincan Station	N189	Alluvium	2.0	390.4	106.2	2.06	2.0
18	Kocaeli, Turkey 08/17/99	7.4	SS (right)	Düzce Meteoroloji Müdürlüğü Station	EW	Alluvium	17.1	366.5	56.4	1.64	1.4

* Abbreviations for site conditions: NEHRP - C: 360 m/s< V_s <760 m/s, NEHRP - D: 180 m/s< V_s <360 m/s, Q: Quaternary ($V_s = 333$ m/s)

** Abbreviations designated by d , PGA and PGV : closest site-to-source distance, peak ground acceleration and velocity, respectively

The strong-ground motion records listed in Tables 2.1 and 2.2 represent near-fault ground motions with different velocity amplitude ranges and they do not contain pulse signals in their waveforms. Higher ground velocity amplitudes of near-fault ground-motion records have been recognized and described in the empirical ground motion prediction equations derived for estimating peak ground velocity (PGV) (e.g., Joyner and Boore, 1981; Campbell, 1997). Recent studies such as Akkar and Özen (2005) and Wald et al. (1999) have shown that this ground-motion parameter can be considered as an unbiased parameter to relate deformation demands on structures in particular for nonlinear structural behavior. The ground motions listed in Tables 2.1 and 2.2 are designated as Group A and B throughout the study, including 20 strong ground-motion records in each group with peak ground velocities ranging between 40-60 cm/s and 20-40 cm/s, respectively.

The significance of pulse signal that is considered as one of the most important features of the near-fault records has been the subject of research for the last 3 decades (e.g., Housner and Trifunac, 1967; Boore and Zoback, 1974). This issue has been brought back to the attention of engineering community by Somerville et al. (1997) and Abrahamson (2000) indicating that fault rupture propagating towards a site at high velocities would result a dominant high amplitude pulse in the fault-normal direction and a fling step in the fault-parallel direction. Anderson and Bertero (1987), Hall et al. (1995), Iwan (1997), Alavi and Krawinkler (2000, 2004), Baéz and Miranda (2000), Iwan et al. (2000), Cuesta et al. (2003) and Akkar et al. (2005a) are among the researchers who described the distinct features of pulse effect on structural systems. Records with clear pulse signals that were chosen for this study are presented in Tables 2.3 and 2.4. The ground motions listed in Table 2.3 contain 20 pulse-dominant records with $40 \text{ cm/s} \leq PGV < 60 \text{ cm/s}$, and this group is called as Group C in this study. The rest of the pulse-dominant ground motions (i.e., 18 records) is defined as Group D and their important seismological features are listed in Table 2.4. Group D records have $PGV > 60 \text{ cm/s}$. Site conditions

displayed in Tables 2.1 to 2.4 that are designated by Q, Qym, Qom, Qyc and T simply refer to soil site classes with different upper 30 m shear wave velocities. The reader is referred to Park and Elrick (1998) and Tinsley and Fumal (1985) for a detailed description of these site classes. The shear wave velocity values of NEHRP site classes were taken from the FEMA-450 document (BSSC, 2003).

The parameters T_p and T_{pv} which are listed in the last two columns of Tables 2.3 and 2.4 are the pulse periods and the dominant pseudo-spectral velocity periods of the earthquakes, respectively. The pulse period values are obtained from the velocity time series of the ground motions. It is the duration of the largest amplitude pulse cycle in the velocity trace. The spectral period that corresponds to the maximum pseudo-spectral velocity, PSV , amplitude is described as T_{pv} in this study. The relation between T_p and T_{pv} are plotted in Figure 2.1. The strong correlation between T_p and T_{pv} indicates the dependence of spectral quantities on the pulse-dominant waveforms that has been addressed by many researchers (e.g., Akkar et al., 2005a; Bray and Rodriguez-Marek, 2004).

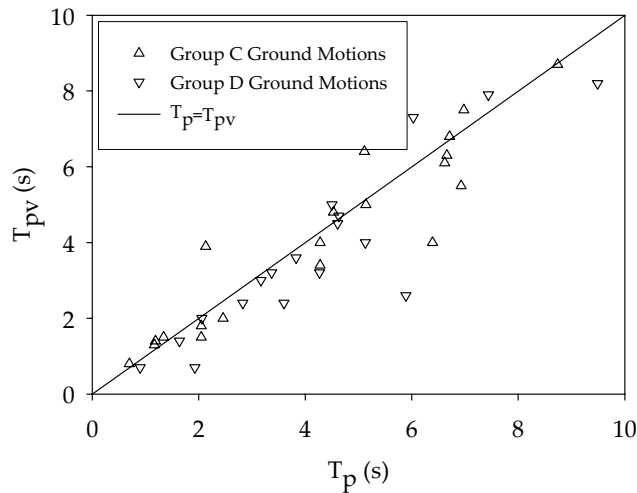


Figure 2.1 Correlation between T_p and T_{pv}

Figure 2.2 displays the scatter diagrams for the ground-motion groups used in this study to observe the correlations between the important seismological parameters. The plots also present histograms of the chosen seismological parameters (i.e., distance, magnitude, peak ground acceleration and peak ground velocity) to distinguish the variations of these parameters quantitatively for each ground-motion group. The following observations are highlighted from the plots presented in Figure 2.2.

1. Group D ground motions have the shortest distance range. The distance distributions of Groups A, B and C are similar to each other.
2. Group C and D ground motions have records from larger magnitude earthquakes with fairly similar distributions compared to the other ground motion bins. Group D ground motions have the highest magnitude variation. The magnitude distribution of Group A ground motions follows Groups C and D. Group B ground motions have the lowest magnitude values compared to the other groups.
3. Peak ground velocity distributions of Groups A and C records are almost the same. This is expected as both groups contain records with $40 \text{ cm/s} \leq PGV < 60 \text{ cm/s}$. The only difference between Group A and Group C records is the pulse-dominant signals in Group C ground motions. It should be noted that the distance distributions of these two groups are also similar.
4. The values of PGV are the highest for Group D records associated with the shortest distance range and the highest magnitude values when compared to other groups. The values of PGV are the lowest for Group B ground motions associated with the lowest magnitude range.

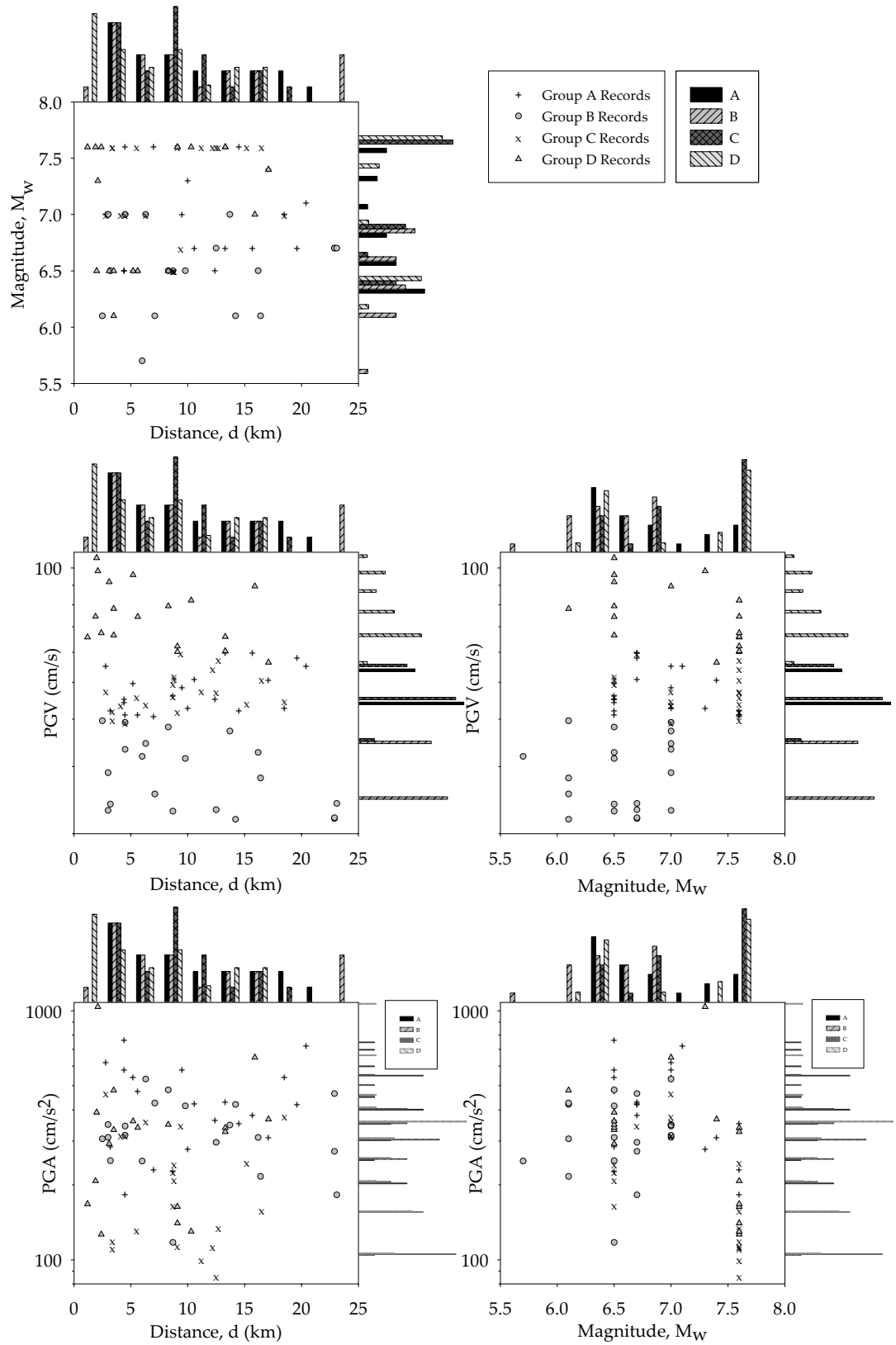


Figure 2.2 Histograms of some ground-motion parameters

5. Groups C and D have lower *PGA* distributions relative to the *PGA* distributions of Groups A and B.
6. *PGV* values tend to increase with increasing magnitude. This is not the case for *PGA*. Abrahamson and Silva (1997) indicated that near-fault ground motions recorded from large magnitude events yield lower high frequency parameters such as *PGA* due to soil nonlinearity. This is consistent with the observations made for the ground motions presented in this study.

2.2 Design spectra

The representative design spectra of each ground-motion group were computed using the median pseudo-acceleration spectra (*PSA*) for each group in order to design moment-resisting frames that are used throughout this study. The procedure described in the FEMA-356 document (ASCE, 2000) was employed for this purpose. This method requires short- and long-period *PSA* values to smooth the jagged variation in the actual spectrum. The short-period design spectrum parameter that controls the constant-acceleration plateau is taken as the pseudo-spectral acceleration at 0.2 s (i.e., $PSA_{0.2}$) confirming that it should not be less than the 90% of the actual *PSA* values at any period. The long-period design spectrum parameter that controls the descending branch is the spectral value at 1.0 s (i.e., $PSA_{1.0}$). The long-period design pseudo-spectral acceleration divided by the period values (i.e., $PSA_{1.0}/T$) should not be less than the 90% of the actual *PSA* values at any period. The long- and short-period pseudo-spectral acceleration parameters can be modified if they do not confirm the required limitations. The corner period, T_c , between the constant-acceleration plateau and the descending branch is the intersection of $PSA_{0.2}$ and $PSA_{1.0}/T$ curves. The median design spectra for each group are shown separately in Figure 2.3 together with the median and 84-

percentile response spectra of the ground-motion groups. The design spectra and the median *PSA* are in good agreement suggesting the success of the fits. The four design spectra are presented together on the same plot in Figure 2.4.

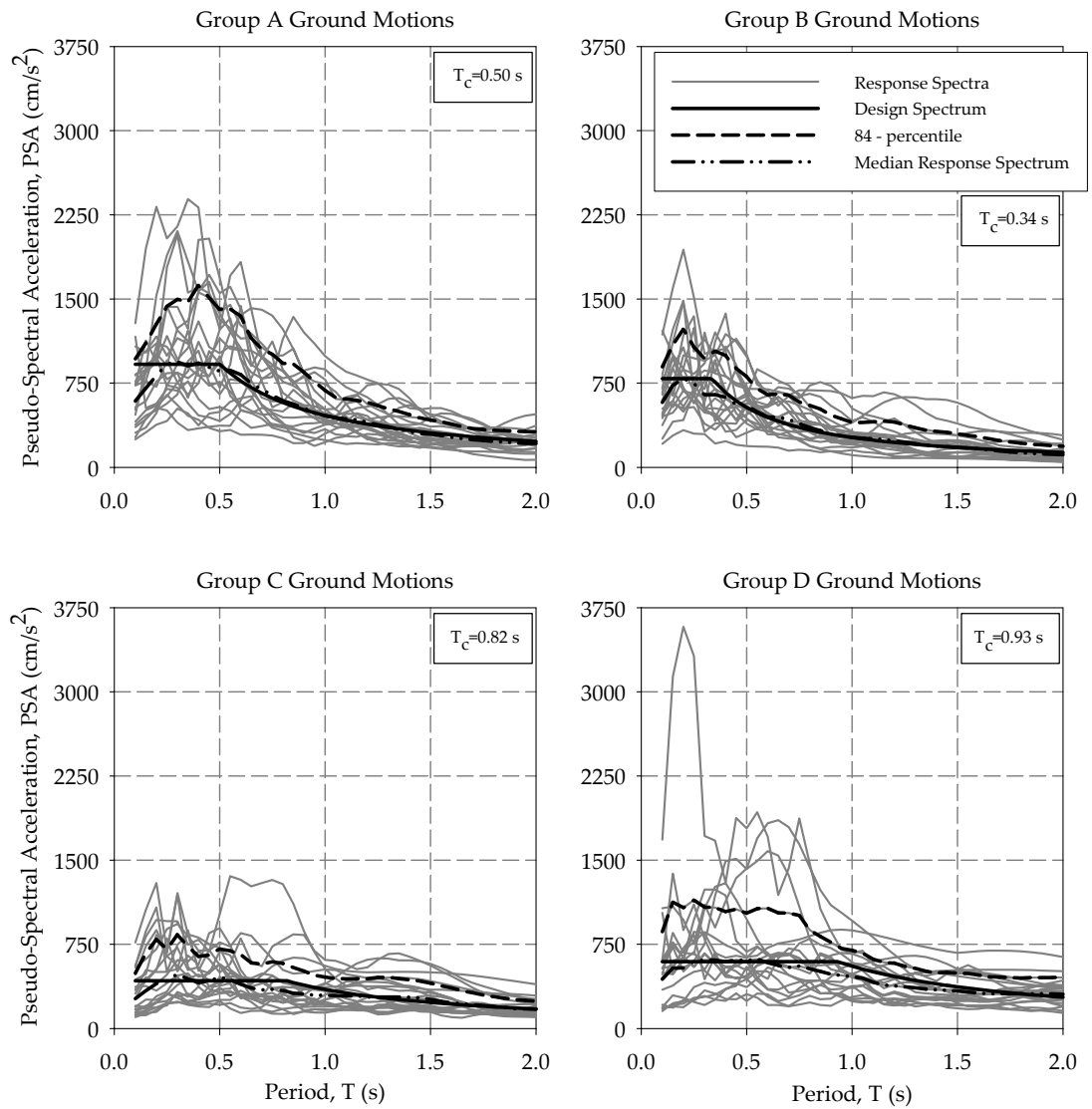


Figure 2.3 Design and response spectra plotted separately for all groups

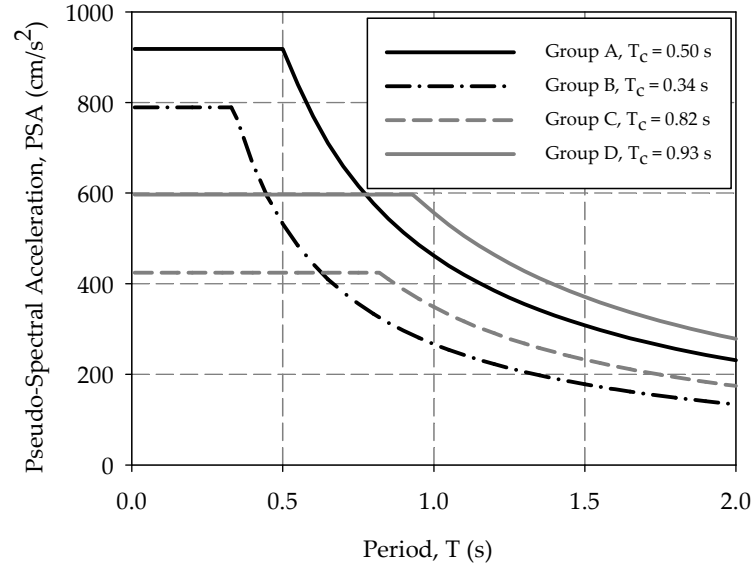


Figure 2.4 Design spectra of the ground-motion records

The following observations can be highlighted when the information revealed from Figure 2.2 is combined with the plots presented in Figures 2.3 and 2.4.

1. The level of constant-acceleration plateau is compatible with the median values of PGA for each ground-motion group. The PGA distributions presented for each group in Figure 2.2 indicate $PGA_A > PGA_B > PGA_D > PGA_C$. This inequality is reflected to the constant acceleration plateau values displayed in Figure 2.4.
2. When Groups A and B (records without pulse) and Groups C and D (records with pulse) are considered within each other separately, the groups with higher PGV (i.e., Group A for no-pulse and Group D for with pulse) are associated with higher PSA for the spectral period range presented.

3. The corner period values are consistent with the level of PGV presented for each group. Larger PGV levels yield the larger T_c value.
4. Group C and D ground motions constitute the larger magnitude events followed by Groups A and B as presented in Figure 2.2. Consequently median design spectra components become rich in the long-period range with increasing magnitude that is consistent with the theoretical source spectrum behavior (e.g., Brune, 1970). When the PGV versus magnitude variation is considered (i.e., larger magnitude implies larger PGV as described in Figure 2.2) one can also state that the ground motions with high PGV are more likely to have rich long-period components.

2.3 Building models

A total of 6 building groups were modeled in this study. Each group consists of 4 reinforced-concrete, moment-resisting frames with 3-, 5-, 7- and 9-stories. The compressive strength of concrete is 20 MPa and the yield strength of steel is 220 MPa. The frames have 3 bays with a constant span length of 5 m and the story heights of the model buildings were chosen as 3 m. Rigid diaphragm assumption was made while modeling the buildings and floor masses were lumped at the beam-column joints at each story level. Uniformly distributed dead and live loads that are reduced at the roof level by 25% were assigned to each story. The magnitude of dead loads vary between 30 kN/m to 70 kN/m whereas live load magnitudes are between 7 kN/m and 15 kN/m.

The model buildings were designed using the median design spectra presented in Figure 2.4. The dimensions of the structural members and the reinforcement detailing of the frames conformed to the provisions of ACI-318 (ACI, 2002). The

lateral earthquake loads were reduced by a response modification factor of 8 that is defined for “special moment-resisting frames” according to the provisions of the FEMA-450 document (BSSC, 2003). The drift limits imposed by the FEMA-450 document (BSSC, 2003) and the Turkish Earthquake Code (1998) were also checked for the serviceability limit requirements. The main goal for the model buildings was to achieve a non-degrading and ductile nonlinear response associated with strong column-weak beam behavior under the ground motions chosen in this study.

The relation between the model buildings and the ground-motion groups whose median design spectra were used during the design stage is presented in Table 2.5. This table also displays the fundamental period, T_1 , range for each building group. The preliminary information displayed in Table 2.5 indicates that Group 1, Group 2, Group 4 and Group 5 building models were designed for records without pulse signals (Group A and B ground motions). Group 3 and Group 6 frames were designed using the design spectra of Group C and D records, respectively, that resemble near-fault records with dominant pulse signals.

Table 2.5 Model building and ground-motion groups

Building Group	Strong Ground-Motion Group Used in Design	Number of Stories	Fundamental period, T_1 (s), range
Group 1	Group A (20 Records)	3, 5, 7, 9	0.27 – 0.82
Group 2	Group B (20 Records)	3, 5, 7, 9	0.33 – 0.92
Group 3	Group C (20 Records)	3, 5, 7, 9	0.56 – 1.29
Group 4	Group A (20 Records)	3, 5, 7, 9	0.38 – 1.17
Group 5	Group B (20 Records)	3, 5, 7, 9	0.47 – 1.31
Group 6	Group D (18 Records)	3, 5, 7, 9	0.41 – 1.05

Table 2.5 indicates that Group 1 and Group 4 model buildings were designed by using the median design spectrum of Group A ground motions. The fundamental

periods of Group 4 model buildings are greater than the fundamental periods of Group 1 model buildings. Similarly, Group 2 and Group 5 model buildings were designed by using the median design spectrum of Group B ground motions. The fundamental periods of Group 5 model buildings are greater than those of Group 2 model buildings. Therefore, Group 4 and Group 5 model buildings represent more flexible moment-resisting frames relative to those in Group 1 and Group 2, respectively. The variation in the fundamental periods of these model buildings were achieved by modifying the floor masses at each story level. In a similar fashion, Group 3 and Group 6 mimic relatively flexible and rigid moment-resisting frames, respectively, that are designed for near-fault, pulse-dominant records.

Some important geometrical and dynamic properties of the model buildings are presented in Table 2.6. The lateral stiffness along the building height is uniform for Group 1 and 4 model buildings, whereas, the lateral stiffnesses of other model buildings decrease uniformly along their heights. The lateral stiffness reduction was applied for 5-, 7- and 9-story building models. A reduction in the lateral stiffness was held regularly at every 2 story after the first 3 stories. The column and beam dimensions were reduced in such a way that the total reduced cross-sectional area of column and beam members of a story was not less than the 80% of the total cross-sectional area of column and beam members in the lower story. The idealized fundamental period of the equivalent SDOF system of each building model, $T_{1,idealized}$, is also displayed in Table 2.6 and its computation is explained in Section 2.6.

Table 2.6 Some important geometrical and dynamic properties of model buildings

MODEL BUILDING	STORY NO.	GROUP 1						GROUP 2							
		COLUMNS (cm)	BEAMS (cm)	ρ	T ₁ (s)	T _{1,idealized} (s)	% mass (mode 1)	Σmass (tons)	COLUMNS (cm)	BEAMS (cm)	ρ	T ₁ (s)	T _{1,idealized} (s)	% mass (mode 1)	Σmass (tons)
3-STORY	1	55x55	30x65						50.8x50.8	30x55					
	2	55x55	30x65	0.405	0.27	0.27	86.77	135	50.8x50.8	30x55	0.342	0.33	0.31	86.09	135
	3	55x55	30x65						50.8x50.8	30x55					
5-STORY	1	55x55	30x65						50.8x50.8	30x55.9					
	2	55x55	30x65						50.8x50.8	30x55.9					
	3	55x55	30x65	0.405	0.45	0.44	83.98	232	50.8x50.8	30x55.9	0.354	0.55	0.55	82.07	232
	4	55x55	30x65						48.3x48.3	30x50.3					
	5	55x55	30x65						48.3x48.3	30x50.3					
7-STORY	1	55x55	30x65						55x55	30x65					
	2	55x55	30x65						55x55	30x65					
	3	55x55	30x65						55x55	30x65					
	4	55x55	30x65	0.405	0.63	0.64	82.56	330	52.1x52.1	30x58.4	0.435	0.68	0.67	79.69	330
	5	55x55	30x65						52.1x52.1	30x58.4					
	6	55x55	30x65						49.5x49.5	30x53.3					
	7	55x55	30x65						49.5x49.5	30x53.3					
9-STORY	1	55x55	30x65						55x55	30x65					
	2	55x55	30x65						55x55	30x65					
	3	55x55	30x65						55x55	30x65					
	4	55x55	30x65						52.1x52.1	30x58.4					
	5	55x55	30x65	0.405	0.82	0.82	81.69	428	52.1x52.1	30x58.4	0.366	0.92	0.88	77.28	428
	6	55x55	30x65						49.5x49.5	30x53.3					
	7	55x55	30x65						49.5x49.5	30x53.3					
	8	55x55	30x65						47x47	30x50.8					
	9	55x55	30x65						47x47	30x50.8					

Table 2.6 (continued)

MODEL BUILDING	STORY NO.	GROUP 3						GROUP 4							
		COLUMNS (cm)	BEAMS (cm)	ρ	T ₁ (s)	T _{1,idealized} (s)	% mass (mode 1)	Σmass (tons)	COLUMNS (cm)	BEAMS (cm)	ρ	T ₁ (s)	T _{1,idealized} (s)	% mass (mode 1)	Σmass (tons)
3-STORY	1	40x40	26.7x40						55x55	30x65					
	2	40x40	26.7x40	0.300	0.56	0.55	85.69	135	55x55	30x65	0.405	0.38	0.37	87.04	303
	3	40x40	26.7x40						55x55	30x65					
5-STORY	1	40x40	35x45						55x55	30x65					
	2	40x40	35x45						55x55	30x65					
	3	40x40	35x45	0.561	0.78	0.78	82.83	232	55x55	30x65	0.405	0.64	0.65	83.82	523
	4	38x38	35x42						55x55	30x65					
	5	38x38	35x42						55x55	30x65					
7-STORY	1	45x45	30x48.8						55x55	30x65					
	2	45x45	30x48.8						55x55	30x65					
	3	45x45	30x48.8						55x55	30x65					
	4	42.7x42.7	30x43.9	0.408	1.05	1.00	79.63	330	55x55	30x65	0.405	0.90	0.90	82.24	744
	5	42.7x42.7	30x43.9						55x55	30x65					
	6	40.5x40.5	30x39.5						55x55	30x65					
	7	40.5x40.5	30x39.5						55x55	30x65					
9-STORY	1	45x45	30x53.3						55x55	30x65					
	2	45x45	30x53.3						55x55	30x65					
	3	45x45	30x53.3						55x55	30x65					
	4	42.7x42.7	30x48.3						55x55	30x65					
	5	42.7x42.7	30x48.3	0.458	1.29	1.23	77.25	428	55x55	30x65	0.405	1.17	1.16	80.91	964
	6	40.5x40.5	30x43.2						55x55	30x65					
	7	40.5x40.5	30x43.2						55x55	30x65					
	8	38.4x38.4	30x38.9						55x55	30x65					
	9	38.4x38.4	30x38.9						55x55	30x65					

Table 2.6 (continued)

MODEL BUILDING	STORY NO.	GROUP 5						GROUP 6							
		COLUMNS (cm)	BEAMS (cm)	ρ	T ₁ (s)	T _{1,idealized} (s)	% mass (mode 1)	Σmass (tons)	COLUMNS (cm)	BEAMS (cm)	ρ	T ₁ (s)	T _{1,idealized} (s)	% mass (mode 1)	Σmass (tons)
3-STORY	1	50.8x50.8	30x55						47x47	30x47					
	2	50.8x50.8	30x55	0.342	0.47	0.47	86.34	303	47x47	30x47	0.287	0.41	0.41	85.28	135
	3	50.8x50.8	30x55						47x47	30x47					
5-STORY	1	50.8x50.8	30x55.9						53.3x53.3	29.2x50.8					
	2	50.8x50.8	30x55.9						53.3x53.3	29.2x50.8					
	3	50.8x50.8	30x55.9	0.354	0.71	0.69	81.00	442	53.3x53.3	29.2x50.8	0.213	0.59	0.59	81.86	232
	4	48.3x48.3	30x50.3						50.8x50.8	29.2x48.3					
	5	48.3x48.3	30x50.3						50.8x50.8	29.2x48.3					
7-STORY	1	55x55	30x65						50.8x50.8	38.1x55.9					
	2	55x55	30x65						50.8x50.8	38.1x55.9					
	3	55x55	30x65						50.8x50.8	38.1x55.9					
	4	52.1x52.1	30x58.4	0.435	0.96	0.93	79.36	743	48.3x48.3	38.1x53.3	0.516	0.76	0.76	81.18	330
	5	52.1x52.1	30x58.4						48.3x48.3	38.1x53.3					
	6	49.5x49.5	30x53.3						45.7x45.7	38.1x50.8					
	7	49.5x49.5	30x53.3						45.7x45.7	38.1x50.8					
9-STORY	1	55x55	30x65						50.8x50.8	38.1x53.3					
	2	55x55	30x65						50.8x50.8	38.1x53.3					
	3	55x55	30x65						50.8x50.8	38.1x53.3					
	4	52.1x52.1	30x58.4						48.3x48.3	38.1x50.8					
	5	52.1x52.1	30x58.4	0.366	1.31	1.25	76.54	963	48.3x48.3	38.1x50.8	0.414	1.05	1.03	78.83	428
	6	49.5x49.5	30x53.3						45.7x45.7	38.1x48.3					
	7	49.5x49.5	30x53.3						45.7x45.7	38.1x48.3					
	8	47x47	30x50.8						43.2x43.2	38.1x45.7					
	9	47x47	30x50.8						43.2x43.2	38.1x45.7					

Table 2.6 also lists the beam-to-column stiffness ratios, ρ , of the model buildings that is used extensively in this study. This parameter was proposed by Blume (1968) to describe the general behavior of frame type structural systems for all practical purposes as the relative stiffnesses of beams and columns determine the structural behavior of the buildings. It is the ratio of sum of the beam rigidities to column rigidities at the story closest to the mid-height of the building that is shown in Equation (2.1).

$$\rho = \frac{\sum_{beams} (I/l)_{beam}}{\sum_{columns} (I/l)_{column}} \quad (2.1)$$

where I : moments of inertia of the column and beam elements

l : lengths of the column and beam elements

$(I/l)_{beam}$: rigidities of beam elements

$(I/l)_{column}$: rigidities of column elements

It should be noted that an average beam rigidity was calculated in the computation of ρ if the beam rigidities above and below the mid-story level show variations due to changes in lateral stiffness.

Equation (2.1) indicates that $\rho = 0$ represents the pure flexural deformation, whereas $\rho = \infty$ represents the pure shear deformation. The beam-to-column stiffness ratios presented in Table 2.6 range between 0.2 and 0.6 that is of typical for moment-resisting frame behavior (Blume, 1968).

2.4 Fundamental period, T_1 , and building height, H , relationships

The fundamental periods of the model buildings change between 0.27 s and 1.31 s as displayed in Tables 2.5 and 2.6, depending on the building height, H , and number of stories. Figure 2.5 displays the mean and $\pm$ standard deviation relation of building height versus fundamental period described by Chopra and Goel (2000a). The scatter for T_1 versus H of the model buildings used in this study is also shown in the same figure. It is observed that the fundamental periods of the model buildings are in good agreement with the period bounds suggested by this empirical relationship except for Group 1 model buildings. The fundamental periods of this group are slightly lower than those proposed by the empirical relationship. Nevertheless, the general picture displayed in Figure 2.5 depicts that the fundamental period versus building height scatter encompasses the number of stories of the building models described. This can be useful in generalizing the observations made on the moment-resisting building behavior for the number of stories covered in this study.

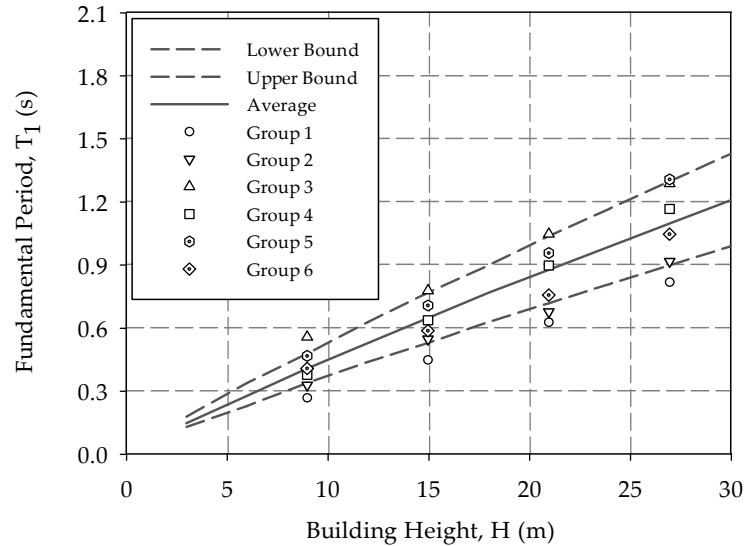


Figure 2.5 Fundamental period – building height relationships

2.5 Nonlinear static analyses (pushover analyses) of the model buildings

Nonlinear static analyses of model buildings were performed by the structural analysis program IDARC-2D (Valles et al., 1996) to compute their capacity envelopes that were also used in describing the corresponding equivalent SDOF system for each frame.

The high percentages of the modal masses contributing to the first mode as displayed in Table 2.6 suggest a first-mode dominant structural behavior. Therefore, the model buildings were pushed laterally in the form of inverted triangular load shape, which is consistent with the first-mode dominant behavior. The lateral load levels that are proportional to the floor masses were increased incrementally until the failure mode was achieved in each model. Since the model buildings were designed to conform to the code provisions, the failure mechanism was expected to follow beam failure pattern that implies strong column-weak beam behavior. The typical failure modes presented for the model buildings in Figure 2.6 indicated a hybrid failure pattern that was dominated by the beam mechanism. In this figure, cross signs represent the cracking of concrete or the initial yield of steel, and circle signs represent the plastic hinges developed or local failures.

The global capacity curves of the model buildings were computed in terms of base shear coefficient, η , (i.e., base shear, V , divided by the total weight of the building, $\sum W$) versus roof displacement, Δ_{roof} . These pushover curves were idealized according to the procedure described in the ATC-40 document (ATC, 1996). The bilinear idealization of the pushover curve is constructed by drawing a straight line (i.e., initial branch) from the origin that represents the initial stiffness, k_1 , of the model building. A second straight line is drawn from the last point of the pushover curve towards the initial branch. The slope of this second line (i.e., post-yielding branch) is adjusted such that the areas under the original pushover curve and the

corresponding bilinear idealization are equal to each other. The intersection of initial and post-yielding branch is designated as the “yield point” for the idealized bilinear curve. The ratio of the post-yielding to initial branch slope is defined as the post-yield stiffness ratio, α .

The capacity curves for Group 1 building models are shown in the first row plots of Figure 2.7. These plots also display the idealized bilinear curves fitted by the ATC-40 (ATC, 1996) procedure. The second row plots in Figure 2.7 show the same information in terms of pseudo-spectral acceleration versus spectral displacement, S_d , (i.e., ADRS format) that is described in detail in Section 2.6. The capacity curves and the corresponding ADRS formats for the rest of the building groups are displayed in Figures A.1 to A.5 in Appendix A. The adequacy of first-mode assumption for the computation of pushover curves was tested by comparing the maximum base shear coefficient versus maximum roof displacement results that were computed from the nonlinear RHA that are described in Chapter 3 in detail. The nonlinear RHA were also conducted by IDARC-2D (Valles et al., 1996). These are superimposed on the corresponding pushover curve as scatter plots. The comparisons between the base shear-roof displacement variations and the pushover curves suggest that fundamental-mode assumption is sufficient to envelop the global capacity of models. In all cases the pushover curves represent a lower bound for the values of nonlinear dynamic analysis; a generally expected phenomenon as noted by other researchers (e.g., Ghobarah, 2004b; Gupta and Krawinkler, 2000).

It should be noted that, a consensus has still not been reached among the researchers for the most appropriate pushover technique to idealize the genuine linear/nonlinear capacity of the structural systems against earthquake action. The first-mode dominant dynamic properties presented and the comparisons between pushover curves and nonlinear RHA are believed to provide sufficient evidence for

the adequacy of the approximations made while computing the pushover curves in this study. For detailed information on different pushover methods and their applications, the reader may refer to Oğuz (2005), Gupta and Kunnath (2000), Chopra and Goel (2002) and Aydınoğlu (2004).

The scatter plots superimposed on the pushover curves also indicate that model buildings are subjected to significant nonlinearity for most cases. In the second row plots of Figure 2.7 and Figures A.1 to A.5, the reduced design spectral acceleration values used during the design stage of the model buildings are displayed as dashed lines. (Recall that a response modification factor of 8 was used to reduce the elastic design spectra that is proposed by the seismic provisions of FEMA-450 (BSSC, 2003) for moment-resisting frame behavior). The substantially high yielding capacities of model buildings when compared to the corresponding reduced design accelerations indicate a significant overstrength factor. This might be considered as another verification that the model buildings conform to the seismic design provisions of the modern codes.

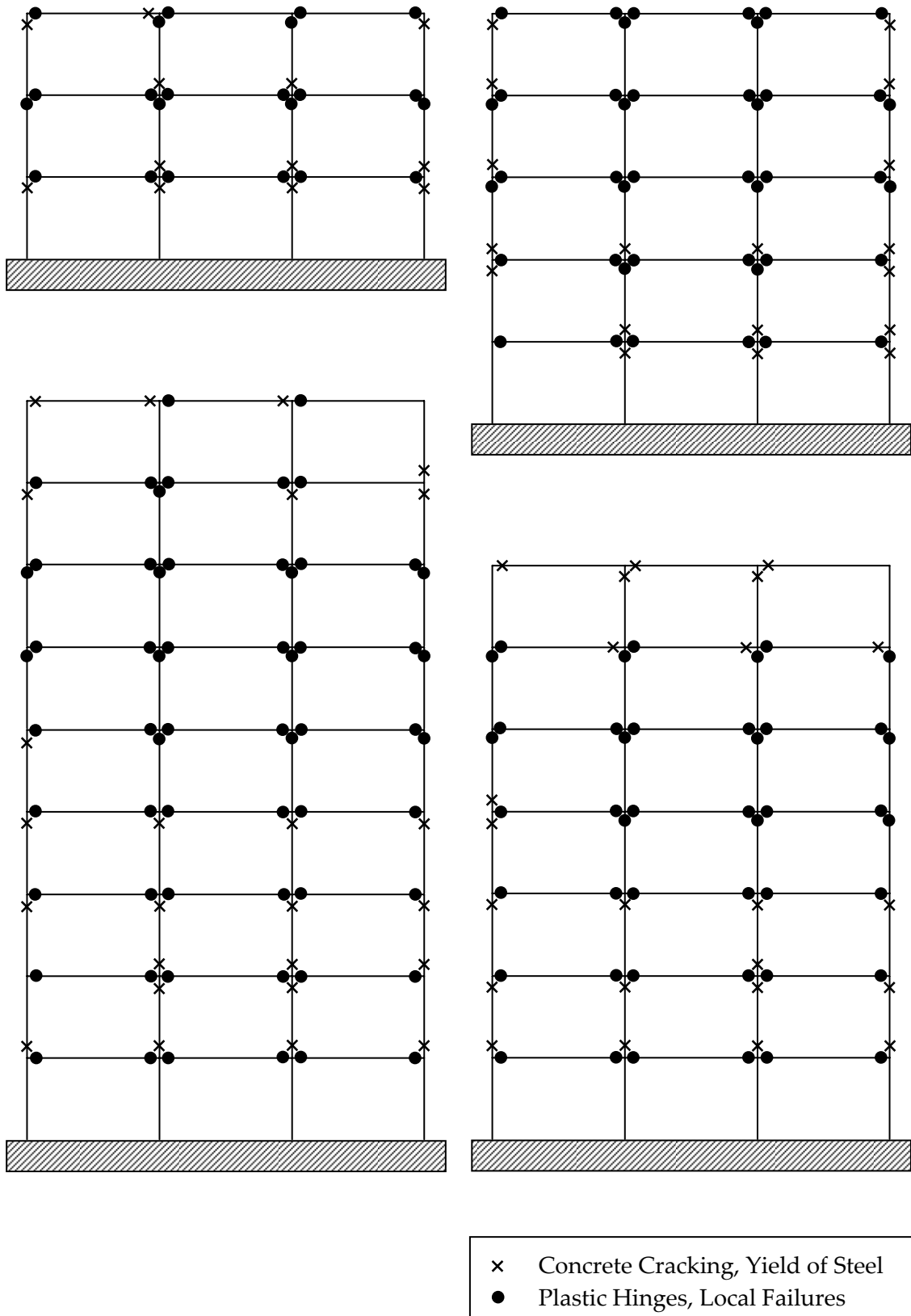


Figure 2.6 Failure mechanisms of typical 3-, 5-, 7- and 9-story model buildings

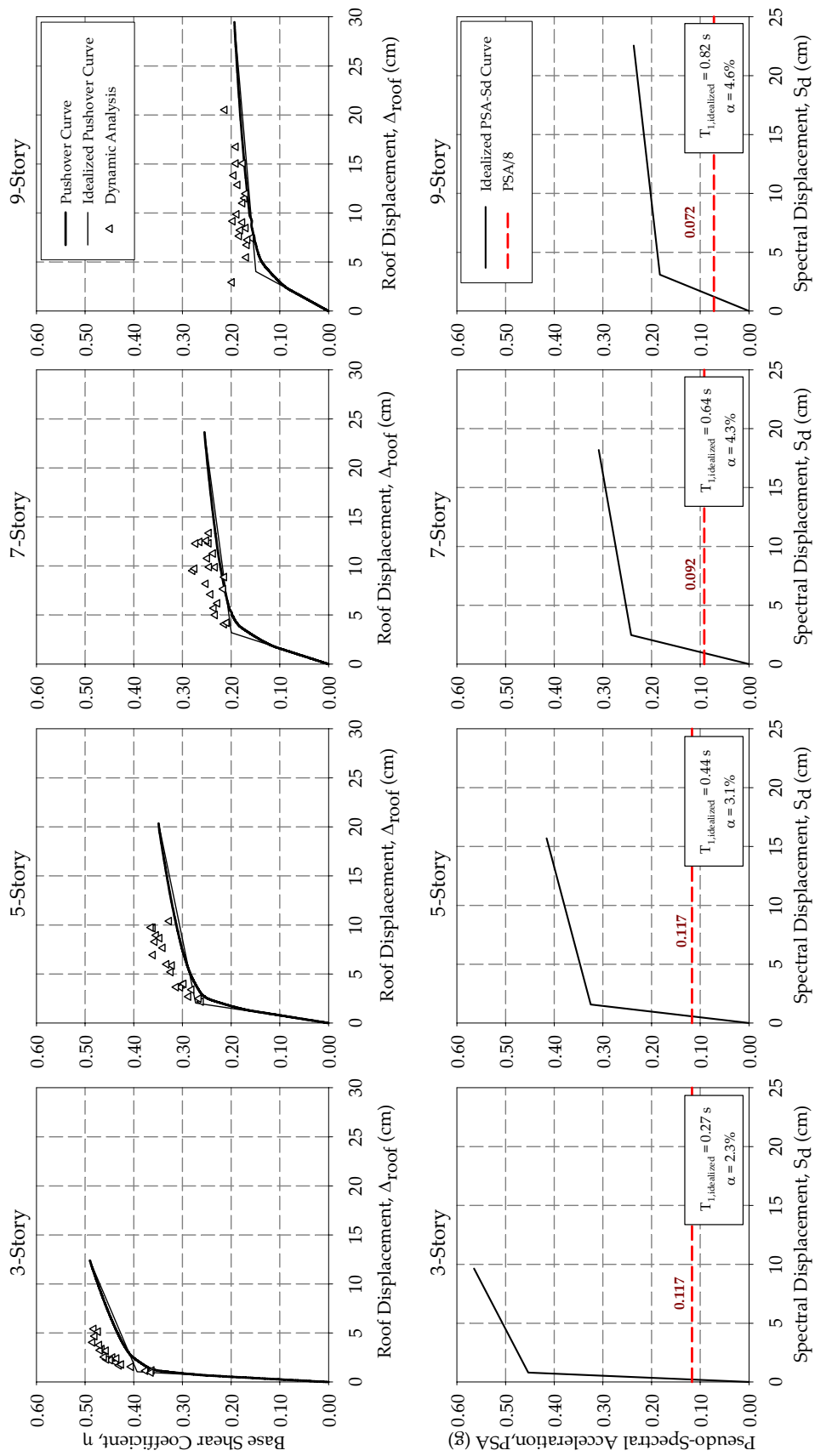


Figure 2.7 Pushover curves and ADRS of Group 1 model buildings

2.6 Conversion of the MDOF systems to the equivalent SDOF systems

Idealized bilinear pushover curves were converted to the ADRS format (i.e., Acceleration-Displacement Response Spectrum) by using Equations (2.2) and (2.3). The parameters ϕ_1^{roof} and Γ_1 represent the elastic fundamental mode roof displacement in modal coordinates and the modal participation factor, respectively. The gravitational acceleration is designated by the variable, g .

$$S_d = \frac{\Delta_{roof}}{\phi_1^{roof} \Gamma_1} \quad (2.2)$$

$$PSA(g) = \frac{\eta \sum W}{\Gamma_1^2 g} \quad (2.3)$$

In Equation (2.3) the denominator term should also contain the first-mode modal mass, M_1 ($= \phi_1^T \mathbf{m} \phi_1$; ϕ_1 is the first mode vector) that is equal to 1 for this study due to mass normalized mode shapes. Thus, this term is not shown in Equation (2.3) for the sake of simplicity.

The physical significance of the Equations (2.2) and (2.3) are illustrated in Figure 2.8. These equations are trivial and they can be found in most of the modern seismic guidelines for seismic performance and design of buildings (e.g., FEMA-356 (ASCE, 2000), ATC-40 (ATC, 1996), etc.). They are based on the assumption that the structures are primarily deformed under the first-mode influence and the elastic fundamental mode shape, ϕ_1 , and the modal participation factor, Γ_1 , can still represent the general dynamic characteristics of the structure in the post-elastic range. The following paragraphs briefly describe the derivation of these equations for the sake of completeness. The reader is referred to Chopra (2001) or Clough and Penzien (1993) for further discussion.

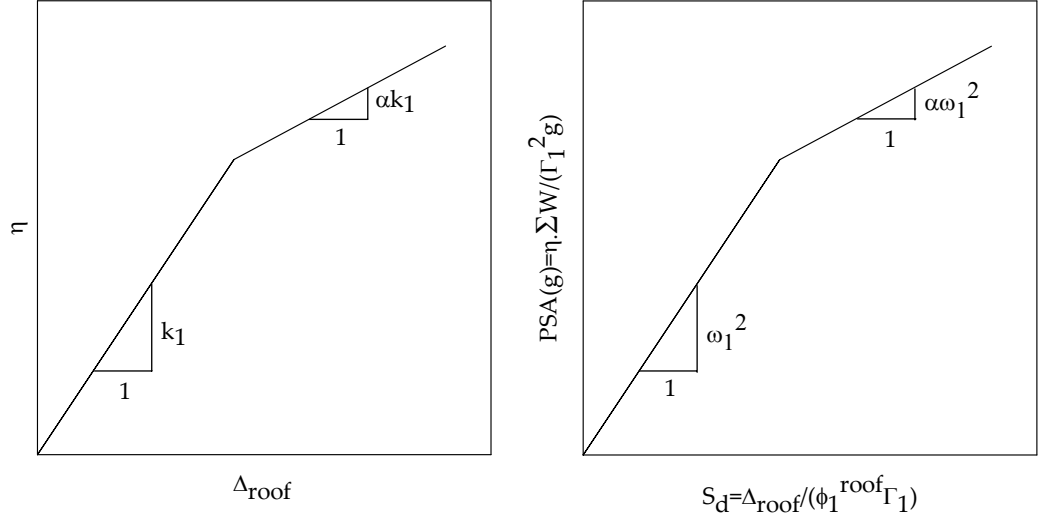


Figure 2.8 Conversion of a pushover curve to ADRS format

The coupled differential equation governing the response of a MDOF system to earthquake induced ground acceleration, $\ddot{u}_g(t)$, can be expressed as

$$\mathbf{m}\ddot{\mathbf{u}}(t) + \mathbf{c}\dot{\mathbf{u}}(t) + \mathbf{k}\mathbf{u}(t) = -\mathbf{m}\mathbf{1}\ddot{u}_g(t) \quad (2.4)$$

where $\mathbf{u}(t)$, $\dot{\mathbf{u}}(t)$ and $\ddot{\mathbf{u}}(t)$ are the relative displacement, velocity and acceleration vectors of the MDOF system, respectively. The matrices $\mathbf{m}$, $\mathbf{c}$ and $\mathbf{k}$ represent the coupled mass, damping and stiffness matrices, respectively. The vector $\mathbf{1}$ indicates that a unit static translation of the base produces a unit displacement.

The above coupled differential equation can be uncoupled by using the principle of modal superposition and the orthogonality of modal vectors as shown in Equation (2.5), respectively.

$$\mathbf{u}(t) = \sum_{n=1}^N \phi_n q_n(t) ; \phi_n^T \mathbf{m} \phi_j = 0 \text{ if } n \neq j \quad (2.5)$$

In Equation (2.5), $q_n(t)$ represents the modal coordinates of the n th mode whereas ϕ_n and ϕ_j represent the n th and j th mode vectors.

The uncoupled form of Equation (2.4) is given in Equation (2.6) that is obtained by using Equation (2.5). The variables ξ_n , ω_n and Γ_n represent the n th mode damping ratio, natural frequency and modal participation factor, respectively.

$$\ddot{q}_n(t) + 2\xi_n\omega_n \dot{q}_n(t) + \omega_n^2 q_n(t) = -\Gamma_n \ddot{u}_g(t) \quad (2.6)$$

For an oscillator of ω_n and Γ_n , the governing equation of motion is expressed as

$$\ddot{D}(t) + 2\xi_n\omega_n \dot{D}(t) + \omega_n^2 D(t) = -\ddot{u}_g(t) \quad (2.7)$$

where $D(t)$ is the relative displacement of the SDOF system

When Equations (2.6) and (2.7) are compared, one can obtain

$$\frac{q_n(t)}{\Gamma_n} = D(t) \quad (2.8)$$

When Equations (2.5) and (2.8) are considered only for the fundamental mode,

$$u = \phi_1 \Gamma_1 D(t) \quad (2.9)$$

The maximum absolute value $u_{\max}$ of the relative displacements along the building height is

$$u_{\max} = \phi_1 \Gamma_1 S_d \quad (2.10)$$

where S_d is the spectral displacement of the SDOF system.

If roof displacement, Δ_{roof} , is the main parameter sought, Equation (2.10) takes a similar form to that of Equation (2.2) by replacing ϕ_1 with ϕ_1^{roof} .

In a similar manner one can write the maximum base shear force, V , by considering only the first-mode behavior as (Clough and Penzien, 1993).

$$V = \frac{L_1^2}{M_1} PSA \quad (2.11)$$

where L_1 : modal earthquake excitation factor

PSA : pseudo– spectral acceleration corresponding to first-mode period

M_1 : modal mass for the first mode

Given $\Gamma_1 = \frac{L_1}{M_1}$, Equation (2.11) reduces to the below expression when the model mass, M_1 , is mass normalized which is the case in this study.

$$V = \Gamma_1^2 PSA \quad (2.12)$$

When both sides of Equation (2.12) is normalized by the total building weight, $\sum W$, and PSA is solved in terms of gravitational acceleration, g , one can obtain Equation (2.3).

Equations (2.2) and (2.3) were used to convert the pushover curves into their equivalent, idealized bilinear SDOF systems in terms of PSA and S_d (i.e., ADRS format). These are presented in the second rows of Figures 2.7 and A.1 to A.5.

The slopes of the initial branch of the idealized ADRS curves correspond to the square of the elastic natural circular vibration frequency, ω_1^2 , of the equivalent SDOF systems. They can be used to represent the idealized periods, $T_{1,idealized}$, of the model buildings

$$T_{1,idealized} = \frac{2\pi}{\omega_1} \quad (2.13)$$

The fundamental periods of the model buildings and the corresponding idealized periods follow very close values to each other as displayed in Table 2.6.

2.7 Distributions of T_p and PSA for the model buildings

Many researchers (e.g., Hall et al., 1995; Iwan, 1997; Alavi and Krawinkler, 2004; Iwan et al., 2000; Cuesta et al., 2003) have indicated that the dominant pulse signal in the near-fault ground motions play a role in the linear and nonlinear behavior of structural systems. In particular, Alavi and Krawinkler (2004) indicated that the higher stories yield earlier for frames with fundamental periods larger than the pulse period, T_p . They also stated that when the fundamental period is less than the pulse period, one should expect the maximum deformation demands to occur in the lower stories regardless of the base shear strength.

Baéz and Miranda (2000) indicated that records with pulse signals shift the SDOF nonlinear deformation demands towards longer vibration periods. Iwan et al. (2000)

indicated that nonlinear deformation demands of oscillators with periods less than the pulse period are substantially larger than their elastic counterparts. Later the above observations were verified in a statistical manner by Akkar et al. (2004) for a large number of near-fault ground motions. Akkar et al. (2004) showed the variation of inelastic to elastic spectral displacements for oscillator periods normalized to pulse periods in order to bring forward the influence of T_p on SDOF deformation demands. They showed that for oscillator periods that are approximately less than $0.8T_p$, the nonlinear deformation demands amplify substantially with decreasing lateral strength. Using a large near-fault ground-motion data set, Akkar et al. (2005a) showed that the maximum interstory drift ratio demands also increase substantially for moment-resisting frames of fundamental periods in the vicinity of near-fault pulse period.

The possible influence of pulse period on building Groups 3 and 6 that were designed for ground motions with dominant pulse signals is investigated by computing the cumulative probability distributions of pulse periods of the ground motions listed in Tables 2.3 and 2.4. Figure 2.9 presents this plot together with the log-normal distribution fit that reveal a fairly good match. The figure also contains the 10% confidence intervals computed by using Kolmogorov-Smirnov goodness-of-fit check (Ang and Tang, 1975). The dashed vertical lines in Figure 2.9 represent the fundamental period interval covered by the building models subjected to pulse dominant records. When the overall information revealed in this figure is evaluated one can state that the chances of fundamental periods getting closer to pulse periods of ground motions is considerably small (i.e., $T_1 < T_p$ for most cases). Based on the observations presented in Alavi and Krawinkler (2004) the nonlinear maximum interstory drift ratio demands would mostly occur at the lower portion of frame buildings for this case suggesting a fundamental-mode dominant behavior for the model buildings (Gupta and Kunnath, 2000).

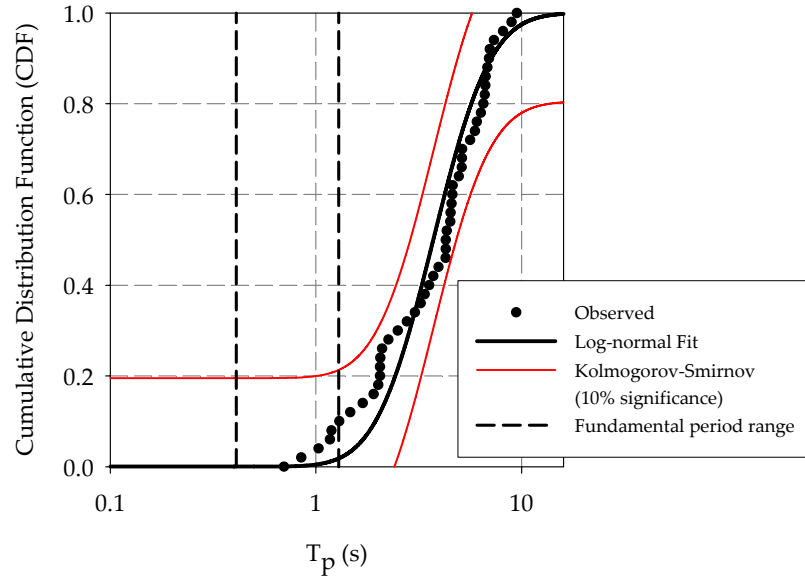


Figure 2.9 T_p distributions of ground motions with dominant pulse signals

Figure 2.10 plots the histograms of pseudo-spectral acceleration values at the fundamental periods for each ground-motion group. These histograms show that $PSA(T_1)$ tends to attain smaller values for increasing story number for all building groups except for the Group 6 building models that are designed for Group D ground motions. The decrease in $PSA(T_1)$ for increasing fundamental period values is an expected phenomenon for well-designed structures. The fundamental periods of Group 6 frames are less than approximately 1 s and the median pseudo-spectral accelerations and design spectra presented in Figures 2.3 and 2.4 clearly show that the spectral accelerations do not show a great variation within this period band, thus clarifying this exceptional case.

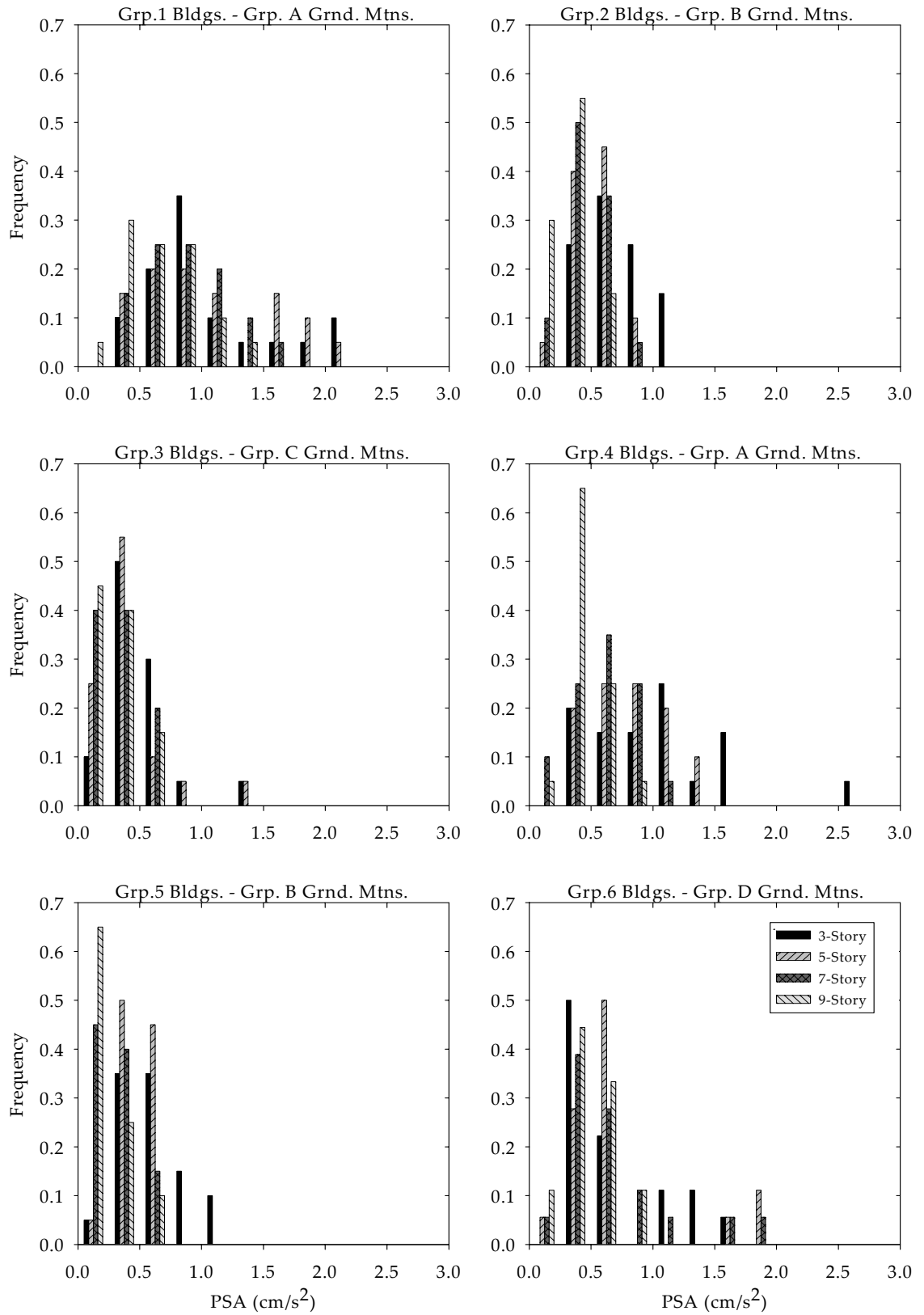


Figure 2.10 PSA histograms of the model buildings

CHAPTER 3

RESPONSE HISTORY ANALYSES

3.1 Introduction

Different types of degrading and non-degrading hysteretic models that define the behavior of concrete and steel structures have been studied by researchers and they are used in structural analyses. Because of their simplicity elastoplastic hysteretic models (bilinear model without a reserved post-elastic strength capacity) that are non-degrading in both stiffness and strength have been used extensively among the researchers. Other than the non-degrading models, there are stiffness degrading as well as stiffness and strength degrading hysteretic models. The reader is referred to Ibarra et al. (2005) and Sucuoğlu and Erberik (2004) for a detailed insight about the different types of hysteretic models.

In this study, bilinear and non-degrading hysteretic model, shown in Figure 3.1 was used both for MDOF and SDOF nonlinear RHA as the building models conform to the seismic provisions of modern design codes. The details of this discussion were given in Chapter 2. The simplification also enables this study for investigating some of the fundamental assumptions used in the current simplified procedures to estimate the nonlinear deformation demands of the structural systems. These are presented in the next chapter.

This chapter presents the results of MDOF and SDOF RHA conducted for the building models presented in Chapter 2. The results presented in this chapter are used further in Chapters 4 and 5 for the evaluation of current approximate

nonlinear static procedures and proposition of a new probabilistic method for estimating maximum interstory drift ratio demands.

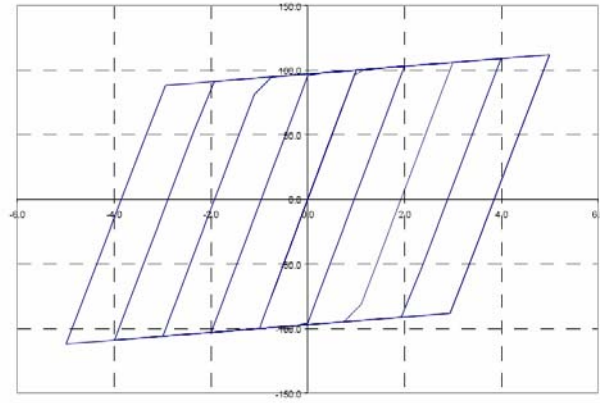


Figure 3.1 Bilinear hysteretic model (Valles et al., 1996)

3.2 Elastic and inelastic behavior of MDOF systems

Elastic and inelastic MDOF response history analyses were performed by using the structural analysis program IDARC-2D (Valles et al., 1996). A total of 944 response history analyses (i.e., 472 elastic and 472 inelastic) were performed to display the general structural behavior of building models under the chosen ground motions.

In general, the earthquake induced structural and non-structural damage are directly related to the lateral deformation demands on building systems. Lateral displacements at the story levels, interstory drift ratios and roof drift ratios are important engineering demand parameters to quantify the structural damage in buildings. These important engineering demand parameters are examined in the succeeding paragraphs for the model buildings presented in this study.

Figures 3.2 to 3.5 present the median and 84-percentile statistics for the maximum lateral displacement (Δ) profiles of the 3-, 5-, 7- and 9-story building models subjected to the ground-motion groups presented in Chapter 2. The statistics were computed assuming log-normal distribution and present the variation of lateral displacement profiles both for elastic and inelastic behavior.

The following observations are made from the lateral displacement profiles presented in Figures 3.2 to 3.5. It should be noted that these observations can apply only for non-degrading systems that are investigated in this study.

1. Median inelastic lateral displacements of 7- and 9-story models that are subjected to ground motions without pulse (i.e., Group A and B ground motions) are less than their elastic counterparts. Using records without pulse, a similar observation was also made by Seneviratna (1995) considering only the roof displacements. For the same number of stories, the model buildings of Group 3 and 6, which are subjected to pulse-dominant records (i.e., Group C and D ground motions) show higher median inelastic lateral displacements with respect to their elastic counterparts for almost all cases. The information revealed in Figure 2.9 suggested that in almost all cases the fundamental periods of model buildings are significantly smaller than the pulse periods for the ground motions chosen in this study. When this information is combined with the above observation, the remarks made in the previous studies (e.g., Iwan et al., 2000 and Akkat et al., 2004) are justified once again: the nonlinear deformation demands of structural systems with $T_1 < T_p$ are considerably amplified with respect to the elastic behavior.

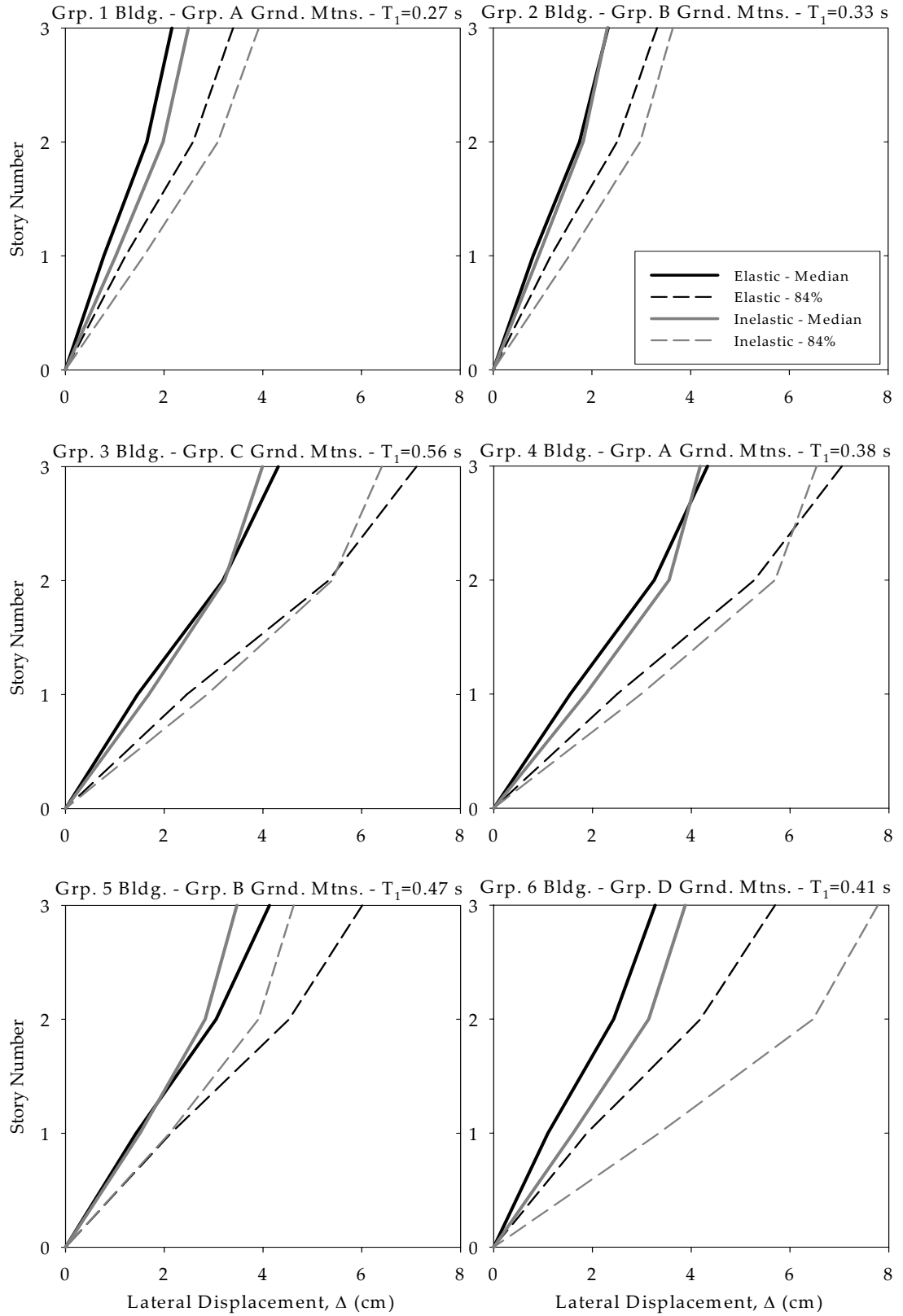


Figure 3.2 Lateral displacement profiles of 3-story model buildings

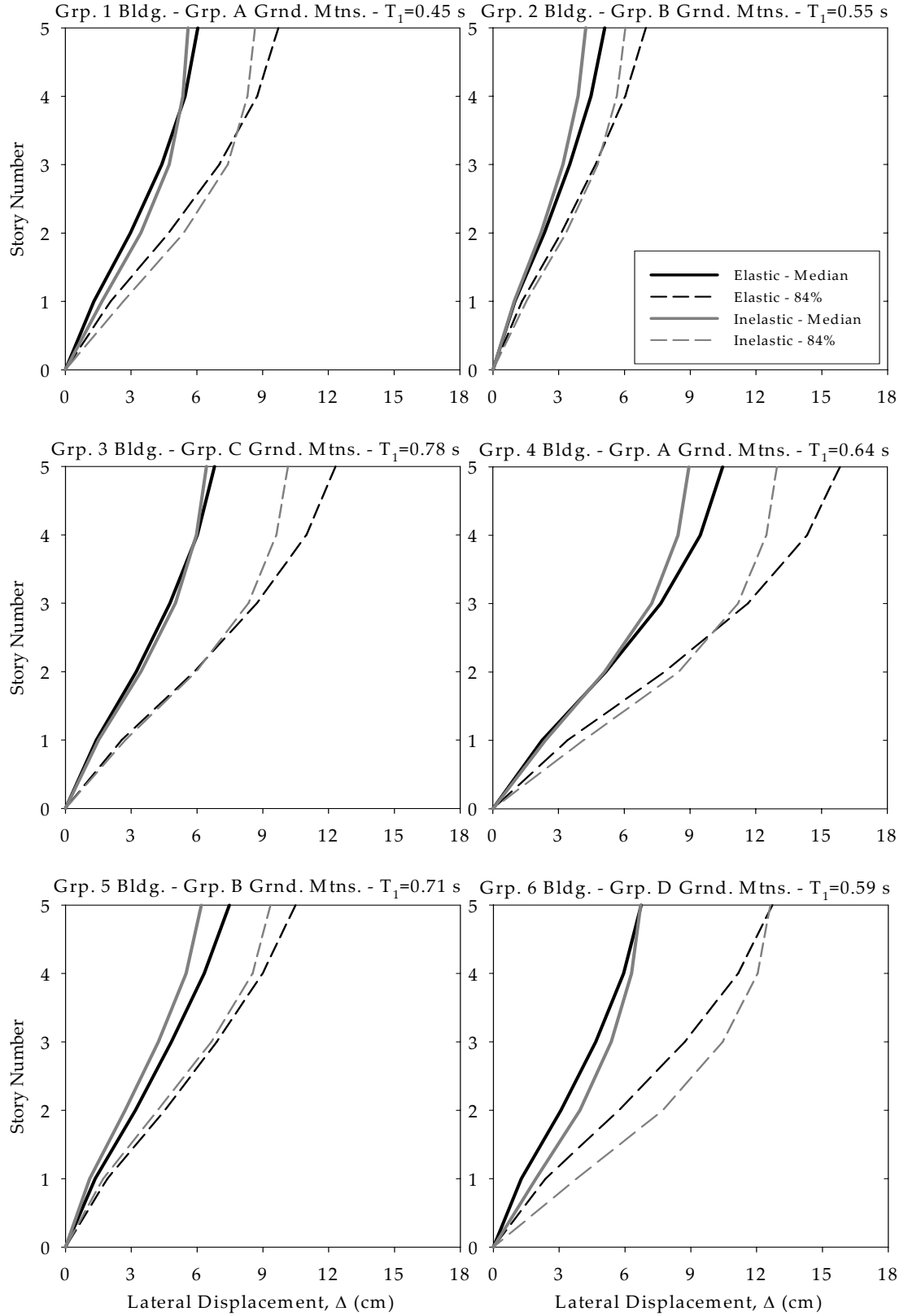


Figure 3.3 Lateral displacement profiles of 5-story model buildings

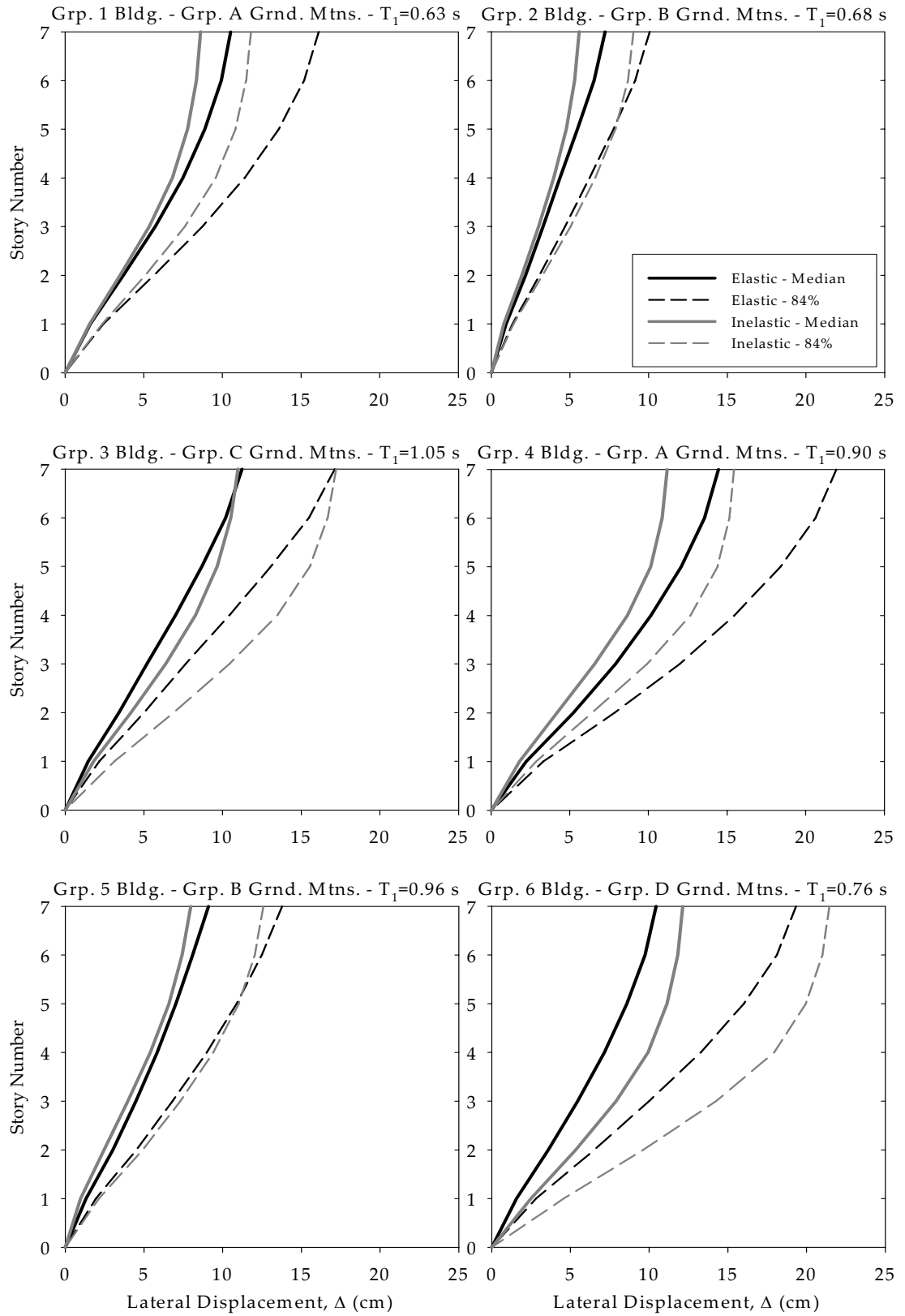


Figure 3.4 Lateral displacement profiles of 7-story model buildings

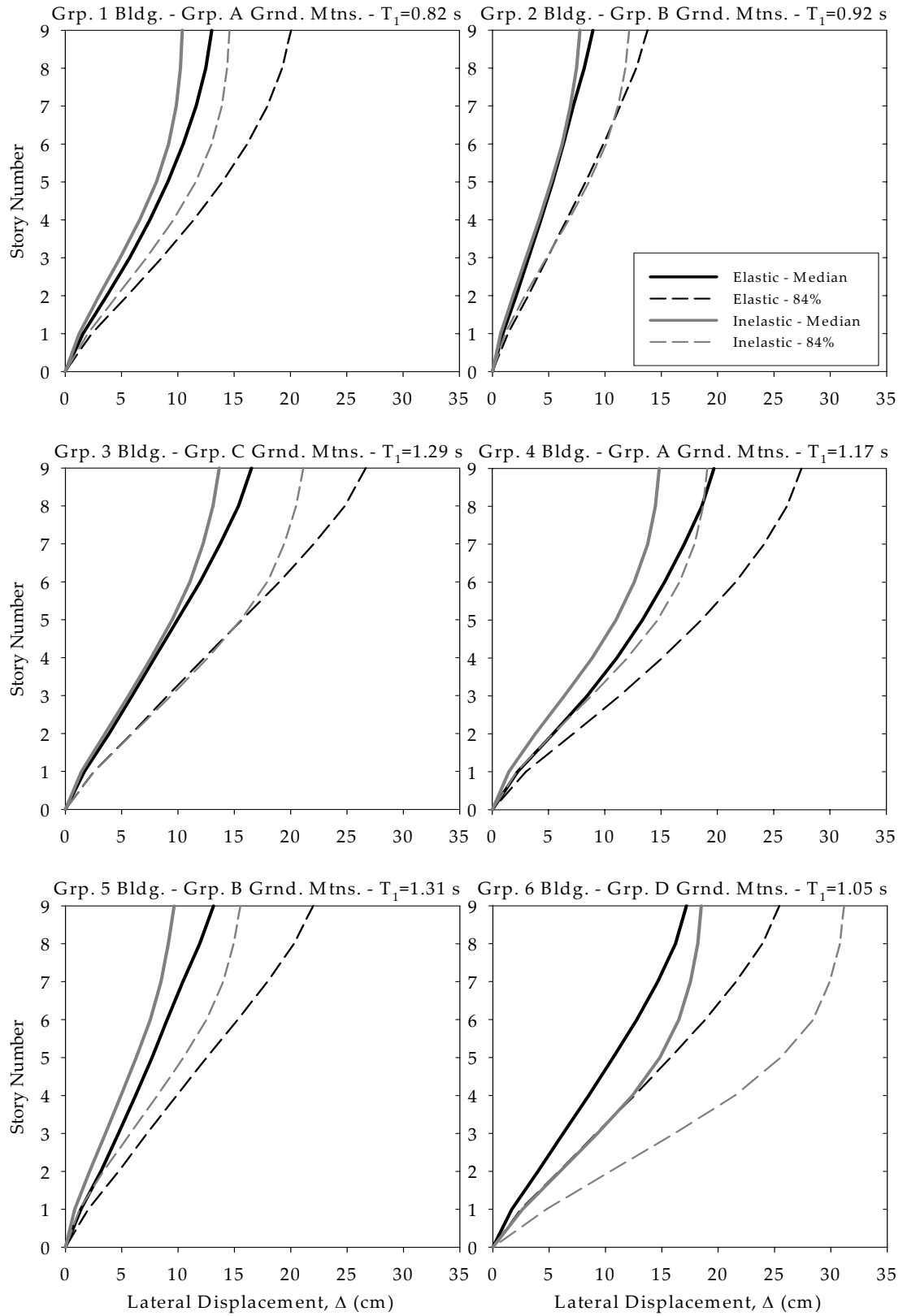


Figure 3.5 Lateral displacement profiles of 9-story model buildings

2. The inelastic lateral displacement patterns follow a more complex behavior for short building models. For low-rise stiff structures (e.g., 3-story building models of Groups 1, 2 and 4) that are subjected to records without pulse, the median nonlinear displacement demands are generally higher than the corresponding elastic demands. This is also the case for 3-story models in Groups 3 and 6 that are subjected to the records with pulse signals that can be explained through the discussion in item (1) on page 44: structures with $T_1 < T_p$ have larger nonlinear deformation demands with respect to their elastic demands. These observations suggest the vulnerability of short-period and stiff nonlinear systems to near-fault ground motions whether they are dominated by pulse or not.

3. In almost all cases presented, the median nonlinear roof displacement demand that is frequently used for seismic performance assessment of structures is less than or equal to its elastic counterpart. For 3-story model buildings, the elastic and inelastic median roof displacements are approximately equal to each other. The only exception is the Group 6 building models that are subjected to the most severe ground-motion group both in terms of magnitude, distance and PGV (i.e., Group D ground motions).

Many studies have shown that interstory drift ratio (IDR) is a structural deformation demand parameter that is best correlated with damage in buildings (e.g., Algan, 1982; Sözen, 1983; Moehle, 1984, 1994; Qi and Moehle, 1991; Güllkan and Sözen, 1999; Bozorgnia and Bertero, 2001; Ghobarah, 2004a). Interstory drift ratio is the lateral displacement difference between two consecutive stories normalized by the story height. Equation (3.1) shows the computation of IDR .

$$IDR = \frac{\Delta_{i+1} - \Delta_i}{h} \quad (3.1)$$

where Δ_{i+1} : lateral displacement of the $(i + 1)$ th floor

Δ_i : lateral displacement of the i th floor

h : story height

The interstory drift ratio for the consecutive stories was computed by finding the absolute maximum *IDR* during the entire excitation duration. The elastic and inelastic maximum interstory drift ratios along the building height are designated as $MIDR_e$ and $MIDR_{ie}$ in this study. Figures 3.6 to 3.9 present the median and 84-percentile elastic and inelastic absolute maximum of *IDR* values at each story level for all model buildings assuming log-normal distribution. The interstory drift ratios are presented in percent (i.e. $(\Delta_{i+1} - \Delta_i) / h * 100$).

The following observations are made from Figures 3.6 to 3.9.

1. The interstory drift ratio along the building height draws a more complex pattern than the lateral displacement profile variation. This is more significant for elastic behavior.
2. The median $MIDR_e$ migrates towards higher story levels for model buildings with uniformly decreasing lateral stiffness (i.e., Groups 2, 3, 5 and 6) regardless of the near-fault ground-motion type (i.e., records with or without pulse signals). This observation is particularly true as number of stories increases suggesting the influence of higher modes for these cases.

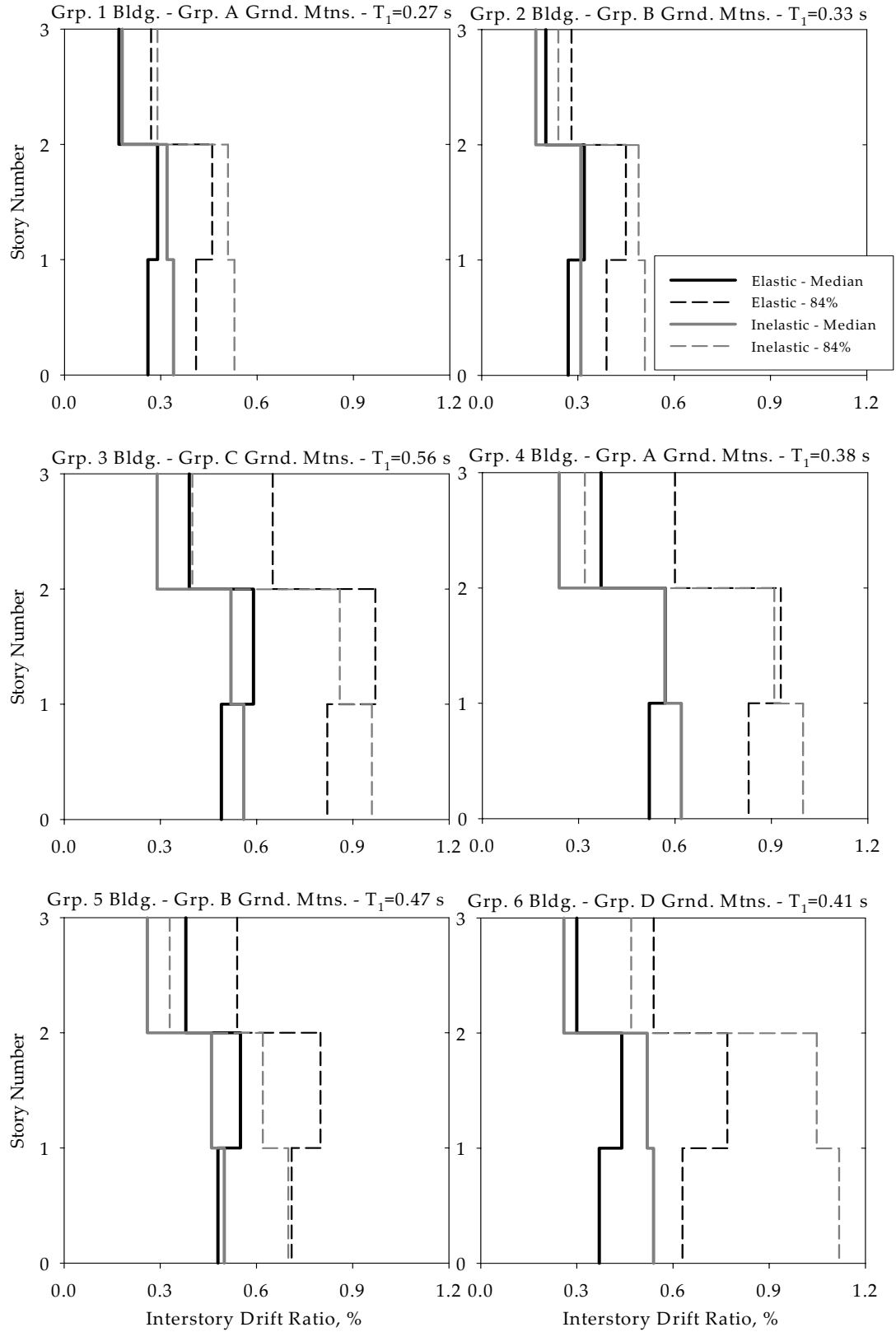


Figure 3.6 Interstory drift ratio profiles of 3-story model buildings

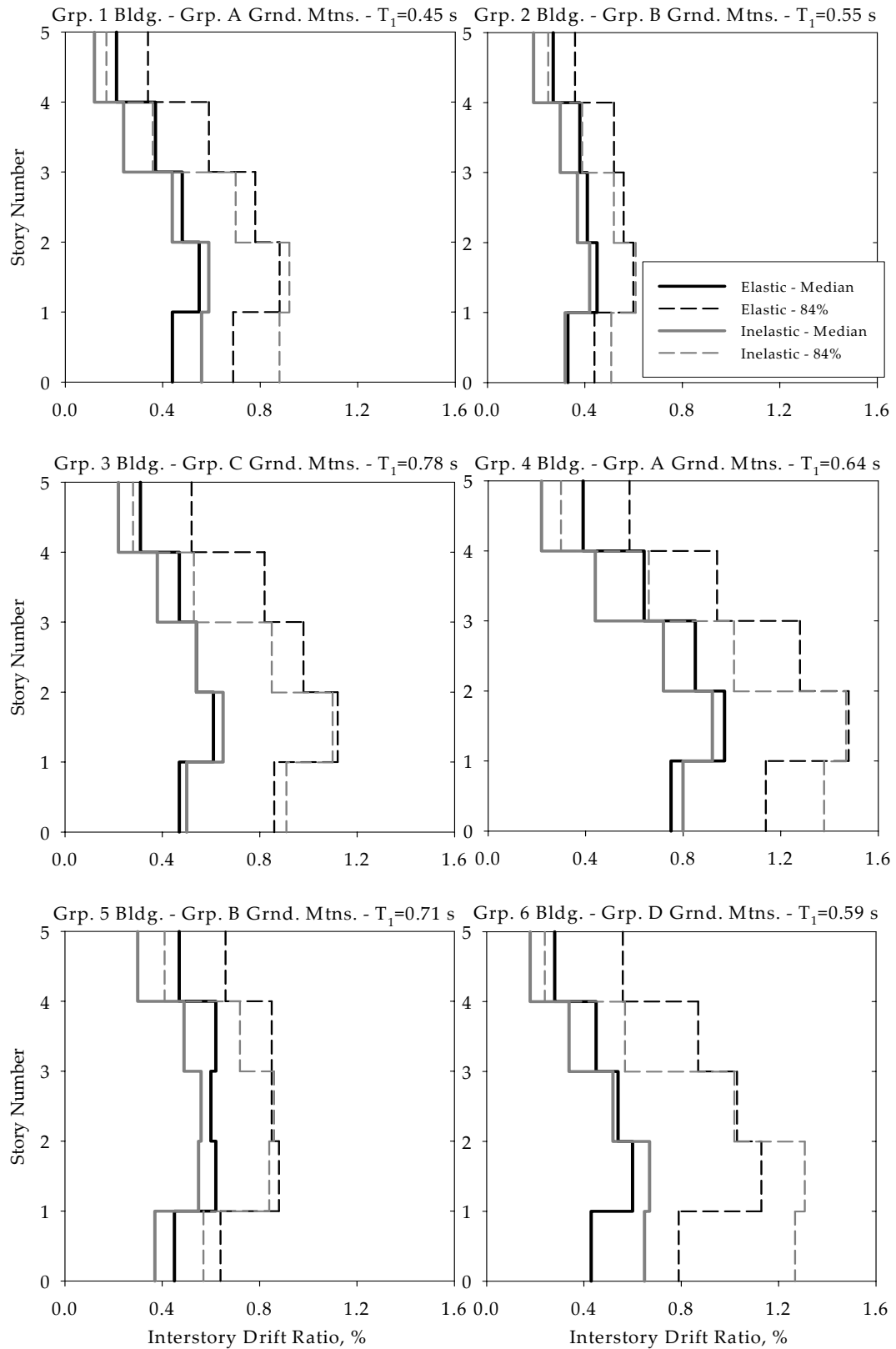


Figure 3.7 Interstory drift ratio profiles of 5-story model buildings

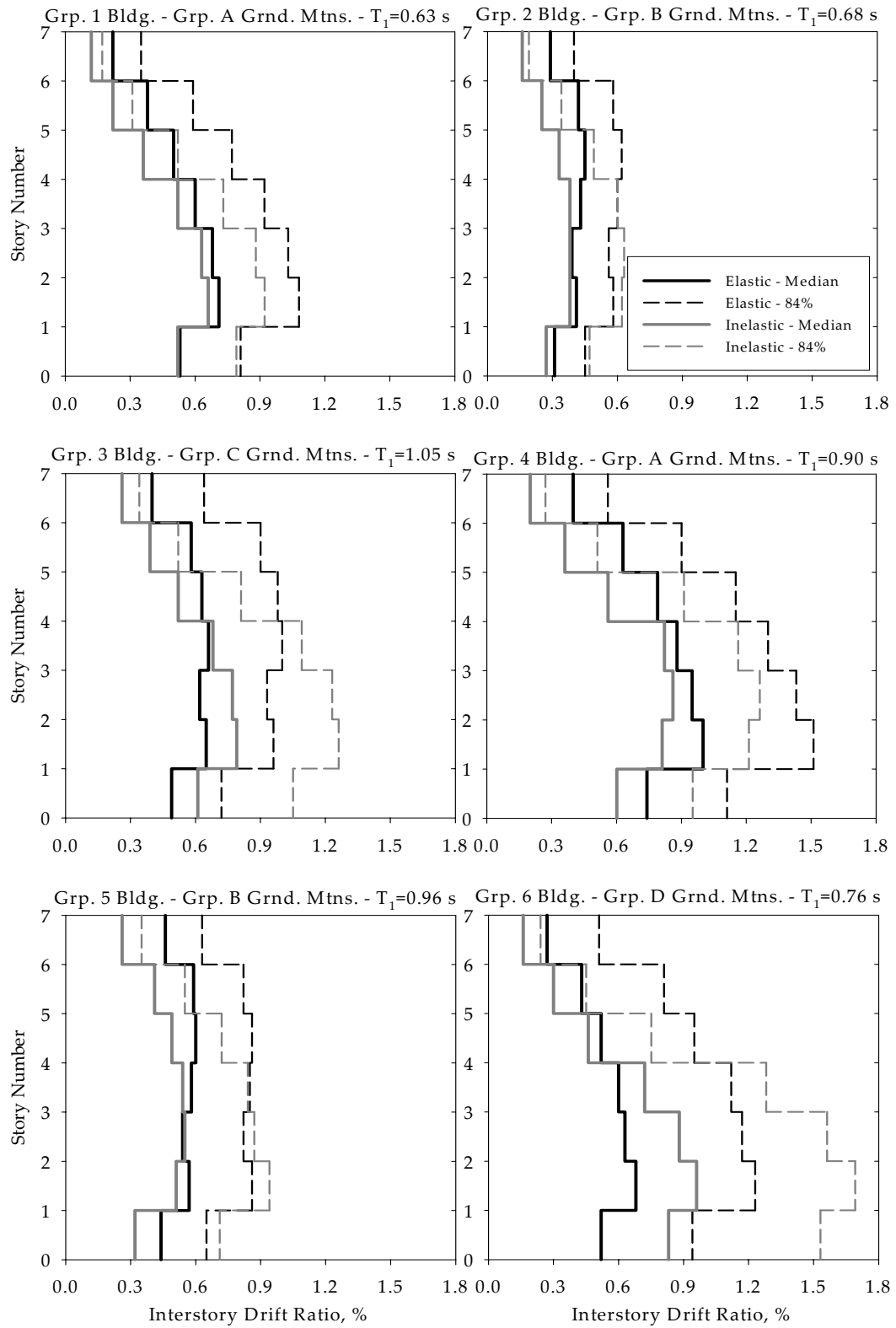


Figure 3.8 Interstory drift ratio profiles of 7-story model buildings

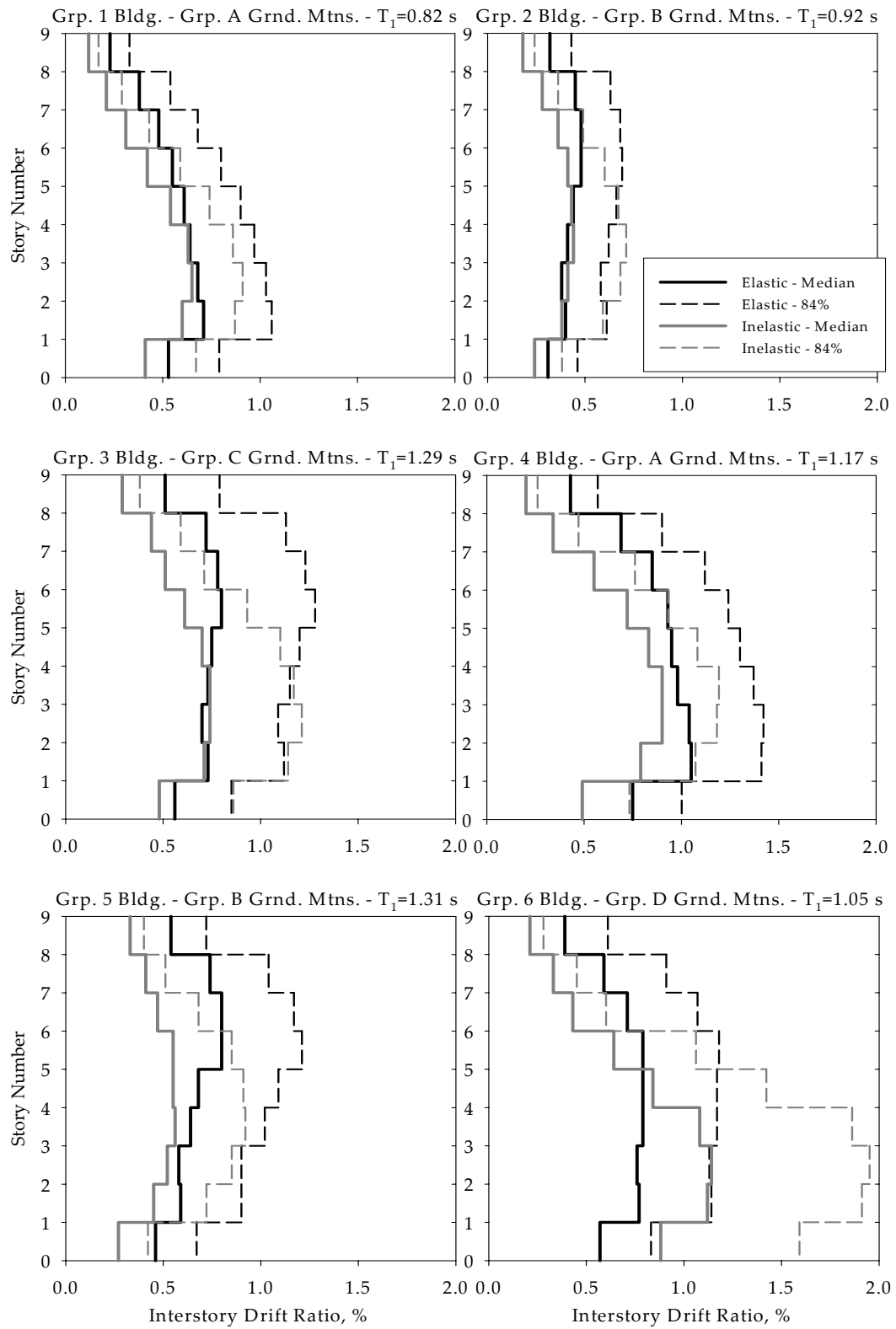


Figure 3.9 Interstory drift ratio profiles of 9-story model buildings

3. The observation in item (2) does not apply to the building models with uniform lateral stiffness (i.e., model buildings of Groups 1 and 4) where $MIDR_e$ occurs in the lower portion of the building height. The difference in $MIDR_e$ variation in items (2) and (3) might be due to the distribution of story shear capacities in the building models. The shear distributions for models with uniformly decreasing lateral stiffness may provoke the influence of wave trains traveling up and down along the building height causing distributions towards higher story levels (Alavi and Krawinkler, 2004).
4. The variations in interstory drift ratios along the building height are more predictable for nonlinear behavior. The median interstory drift ratio resembles a first-mode dominant pattern for structures behaving in the nonlinear range. For most cases, $MIDR_{ie}$ is located at the lower stories regardless of the lateral stiffness variation or near-fault ground-motion type.
5. The median $MIDR_{ie}$ is mostly greater than the median $MIDR_e$ for building models subjected to near-fault records with pulse. This can again be explained by the interaction between the fundamental period and pulse period as discussed in item (1) on page 44. For building models that are excited by Group A and B ground motions (i.e., records without pulse), the variations in $MIDR_{ie}$ and $MIDR_e$ are erratic.

3.3 Dispersions on elastic and inelastic story displacement profiles and interstory drift ratio profiles

Elastic and inelastic dispersions of lateral displacement profiles and interstory drift ratio profiles are displayed for 3-, 5-, 7- and 9-story building models in Figures 3.10 to 3.13, respectively. Coefficient of variation (COV) was used as the dispersion

measure. It is also a good approximation for log-normal standard deviation when COV attains values less or equal to 0.3 (Benjamin and Cornell, 1970). The following observations are made from these plots.

1. In general the elastic COV for lateral displacements and interstory drift ratios follow each other closely at lower stories. This observation is also valid for inelastic COV for lateral displacements and interstory drift ratios.
2. The COV of IDR at the roof level is significantly lower than the corresponding COV values at the lower portion of the buildings. This is particularly true for nonlinear behavior.
3. In general, the COV of lateral displacements tends to decrease towards roof particularly for nonlinear behavior.
4. When items (2) and (3) are evaluated together, one can conclude that the inelastic roof displacement demand parameter draws a less dispersive character with respect to the interstory drift demands (in particular $MIDR_{ie}$) as it is mostly located in the lower portion of the buildings.
5. In general, the COV of inelastic lateral displacements and interstory drift ratios for structures with uniformly decreasing lateral stiffnesses are larger (if not equal) than the corresponding elastic COVs. This observation is particularly valid at the lower story levels for the ground-motion groups used in this study.
6. For structures with uniform lateral stiffness in most cases, the COVs for inelastic lateral displacement and interstory drift ratios are equal or less than the corresponding COVs of elastic behavior.

7. In general, the higher COVs of the deformation demands suggest the importance of record-to-record variability that should be considered in a probabilistic manner during their estimation. This is going to be treated in detail while presenting the probabilistic approach in Chapter 5.

When the observations presented in items (1) to (7) are evaluated together with the remarks highlighted in Section 3.2, one can conclude that the estimation of *MIDR* is more complicated than the estimation of maximum roof displacement. The changes in building properties and near-fault ground-motion features play a role for the complicated variation of interstory drift demands. The statistical results reveal that the general tendency of nonlinear *MIDR* (i.e., $MIDR_{ie}$) seems to be more predictable but caution should be taken as these predictions might be associated with a significant dispersion particularly for structures with uniformly decreasing lateral stiffness.

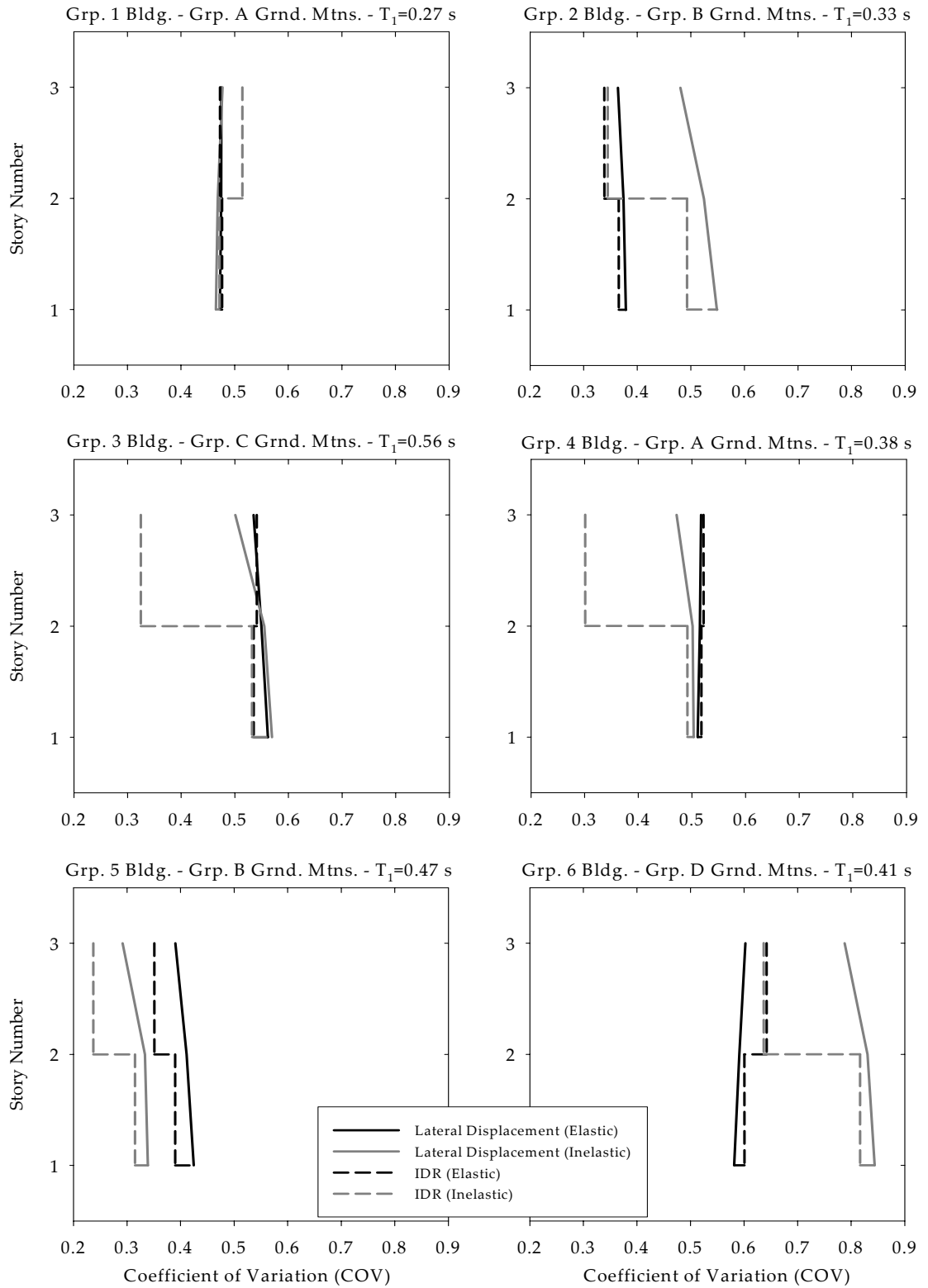


Figure 3.10 Dispersions in 3-story model buildings in terms of coefficient of variation

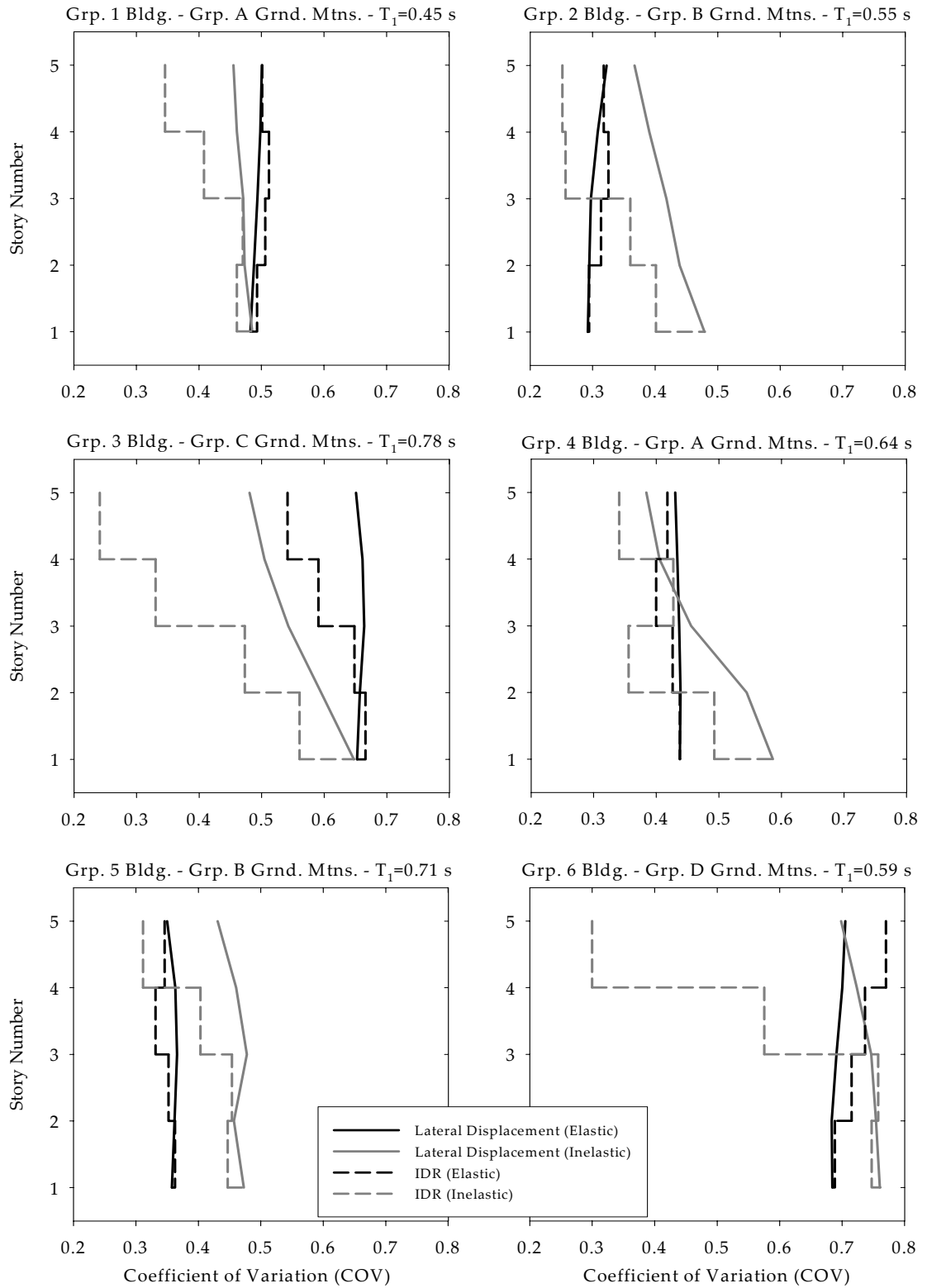


Figure 3.11 Dispersions in 5-story model buildings in terms of coefficient of variation

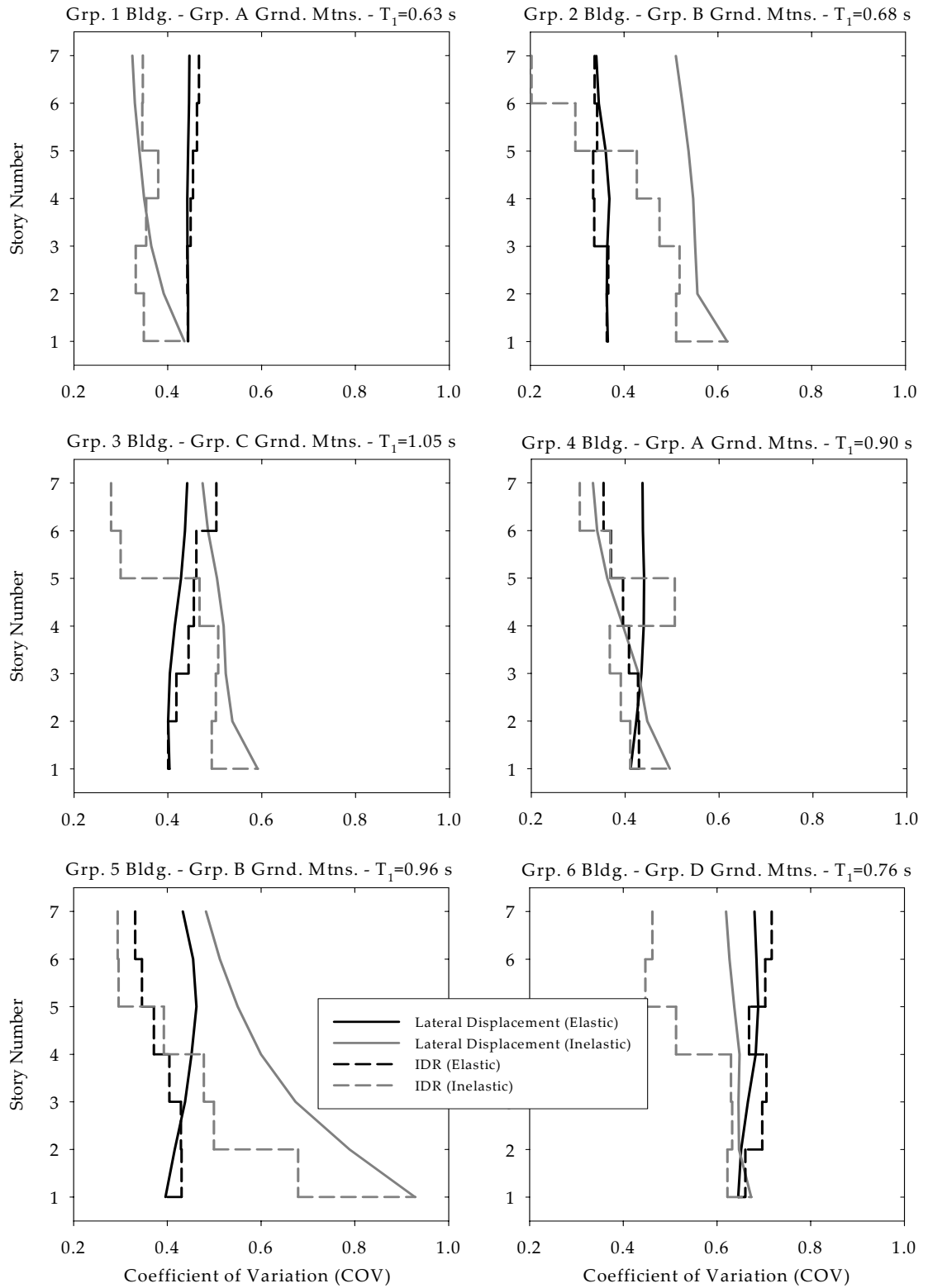


Figure 3.12 Dispersions in 7-story model buildings in terms of coefficient of variation

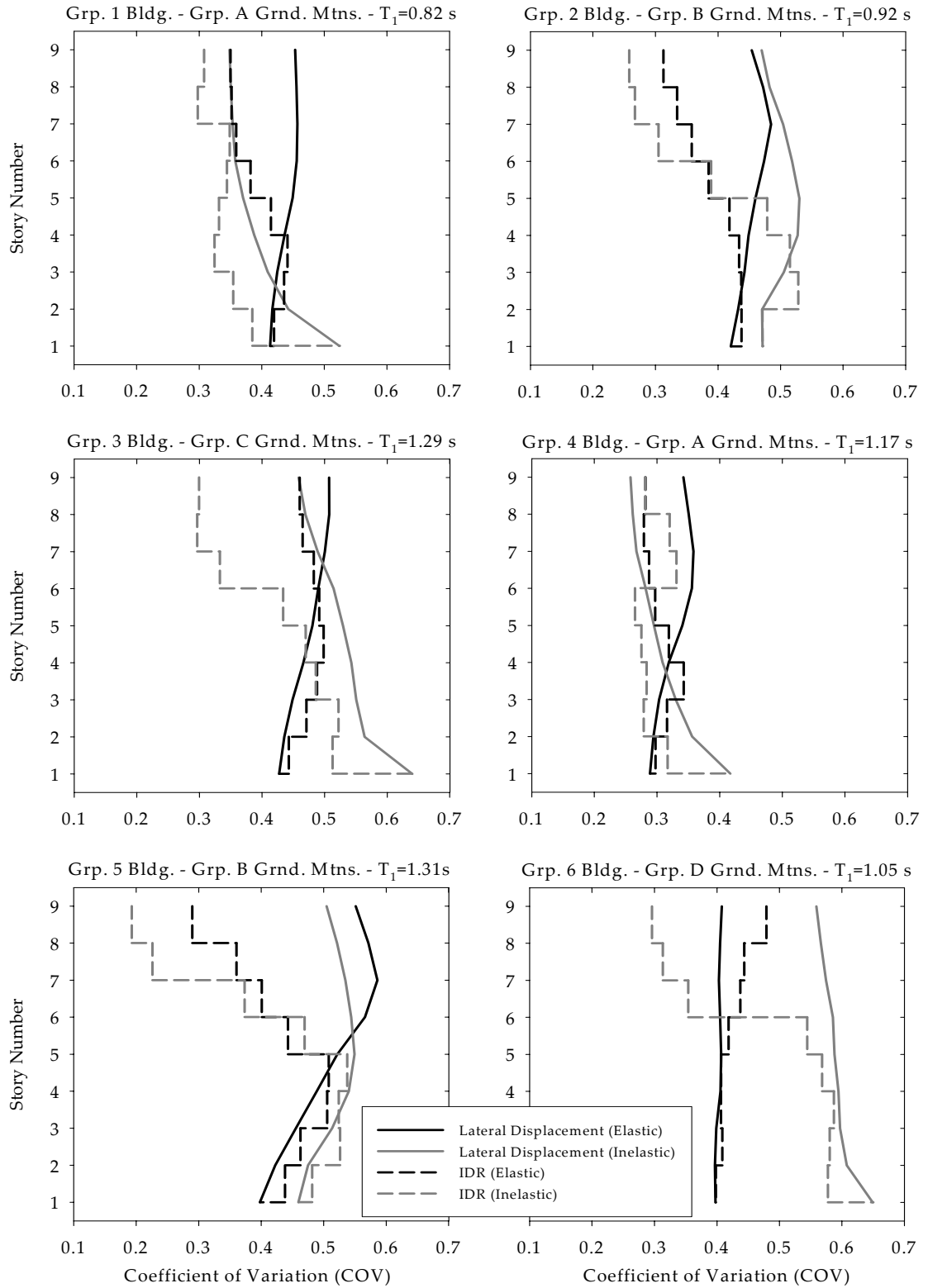


Figure 3.13 Dispersions in 9-story model buildings in terms of coefficient of variation

3.4 Empirical relationships between the maximum interstory drift ratio and maximum roof drift ratio

The elastic and inelastic maximum interstory drift ratios and roof displacements computed from the response history analyses were used to plot the scatter diagrams presented in Figure 3.14.

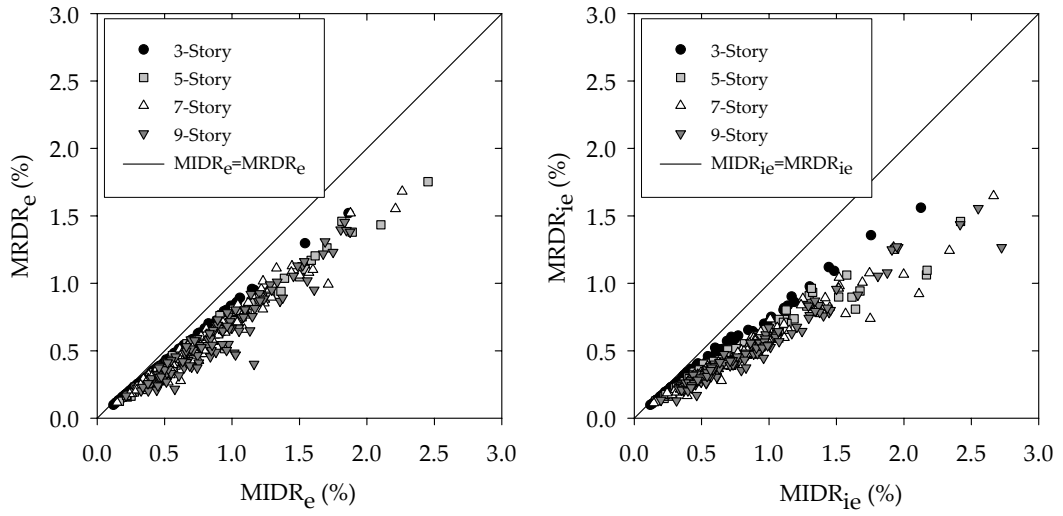


Figure 3.14 Scatter plots for $MRDR_e$ versus $MIDR_e$ and $MRDR_{ie}$ versus $MIDR_{ie}$

The left panel in Figure 3.14 shows the elastic maximum interstory drift ratio ($MIDR_e$) versus elastic maximum roof drift ratio ($MRDR$; maximum absolute roof displacement normalized by the total building height, H). The right panel in the same figure shows the same relations for nonlinear building behavior (i.e., $MIDR_{ie}$ and $MRDR_{ie}$). For the sake of generality the maximum interstory and roof drift ratios are given in percentage. The trends in these scatter plots show a clear relationship between the maximum interstory drift and maximum roof drift ratios both for elastic and inelastic ranges. If the dynamic and geometrical properties of

the structural systems (i.e., the fundamental period, lateral stiffness variation, strength capacity, number of stories, etc.) are disregarded together with the special seismological features of the ground motions used in this study, one can obtain the following empirical relationships for the estimation of these important demand parameters both in linear and nonlinear ranges.

$$MRDR_e = 0.73MIDR_e \quad (3.2)$$

$$MRDR_{ie} = 0.62MIDR_{ie} \quad (3.3)$$

Equations (3.2) and (3.3) indicate that the $MRDR$ for elastic systems is approximately 70% of the $MIDR$. This ratio reduces to 60% for nonlinear behavior. Equations (3.2) and (3.3) can be used in both ways if one can make a good estimate of either maximum interstory or maximum roof drift ratio. However, caution should be exercised for the predicted maximum demand as it lacks the structural properties and ground-motion features that play a significant role in the lateral deformation demands as discussed in the previous sections. A similar approach for the prediction of $MIDR_{ie}$ is proposed by Medina and Krawinkler (2005) that will be discussed in detail in Chapter 5.

3.5 Elastic and inelastic SDOF response history analyses

Response history analyses of SDOF systems were performed by using the computer program BISPEC (Hachem, 1999). The pushover curves that were idealized as bilinear non-degrading systems as explained in Chapter 2 were used to conduct the nonlinear response history analyses of the equivalent SDOF systems. The vibration periods (i.e., $T_{1,idealized}$) computed from the initial branch of the idealized ADRS curves, as well as the post-yield stiffness ratios, α , that were computed from the slope of post-yielding branch were used in the nonlinear RHA. For the sake of

practicality, 5% viscous damping was assigned to all equivalent SDOF systems. Similar to the MDOF response history analyses, 472 nonlinear equivalent SDOF analyses were realized that correspond to the 24 model buildings (6x4) and 78 ground-motion records. A total of 472 linear equivalent SDOF analyses were also conducted using only the $T_{1,idealized}$ and 5% viscous damping that represent the elastic counterparts of the nonlinear runs. The following subsections describe the relevant observations from these analyses.

3.5.1 Relationships between the spectral displacements and roof displacements

Various researchers proposed to use the spectral displacement and fundamental-mode properties of discrete and continuum systems to estimate the roof displacements (e.g., Seneviratna and Krawinkler, 1997; Gupta and Krawinkler, 2000; Miranda, 1999, etc). The basic premise in these procedures is the dominant fundamental-mode behavior and the proportional amplification of roof displacement with respect to the spectral displacement at the fundamental-mode period. The proportionality constant used is related to the product of the fundamental mode participation factor (Γ_1) and the fundamental mode eigenvector value at the roof (ϕ_1^{roof}) of a discrete or continuum model. The recent seismic performance assessment guidelines such as FEMA-273 (ATC, 1997) and FEMA-356 (ASCE, 2000) define this factor as “ C_0 ” that relates the spectral displacement of the SDOF system to the roof displacement of the corresponding MDOF system. The most basic relationship between these two deformation demand quantities for a non-degrading system is:

$$\Delta_{roof,ie} = C_0 S_{d,ie} \quad (3.4)$$

In Equation (3.4) the parameters $\Delta_{roof,ie}$ and $S_{d,ie}$ are the inelastic roof and spectral displacements at $T_{1,idealized}$, respectively.

For elastic MDOF behavior, Equation (3.4) becomes

$$\Delta_{roof,e} = C_0 S_{d,e} \quad (3.5)$$

where $\Delta_{roof,e}$ and $S_{d,e}$ are the elastic roof and spectral displacements, respectively. Neither FEMA-273 (ATC, 1997) nor FEMA-356 (ASCE, 2000) documents make a distinction in the values of C_0 for elastic and inelastic behavior. The most recent version of these documents, FEMA-440 (ATC, 2005), also addresses the same recommended values without making any modification. The following items are recommended by the above documents to compute the modifying factor C_0 .

- (a) Product of first modal participation factor and the modal roof displacement amplitude at the fundamental mode (i.e., $\Gamma_1 \phi_1^{roof}$)
- (b) The modal participation factor and the modal roof displacement at the fundamental mode calculated using a shape vector corresponding to the deflected shape of the building at the target roof displacement
- (c) The appropriate value from Table 3.2 of FEMA-356 document (ASCE, 2000).
(This table is presented as Table 3.1 in this study.)

Table 3.1 Values for the modification factor C_0 suggested by FEMA-356 document (ASCE, 2000)

	Shear Buildings		Other Buildings
# of Stories *	Triangular Load Pattern	Uniform Load Pattern	Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

* Linear interpolation is used to calculate the intermediate values.

Using the linear and nonlinear spectral displacements of the equivalent SDOF systems and the linear and nonlinear roof displacements computed from MDOF RHA, C_0 definitions proposed in items (a) and (c) that are frequently used in the engineering practice were evaluated. The parameters $\Gamma_1 \phi_1^{roof}$, $\Delta_{roof,e} / S_{d,e}$ and $\Delta_{roof,ie} / S_{d,ie}$ were computed from the RHA. The results are listed in Tables B.1 to B.6 in Appendix B. The statistics presented in the lower portion of each table are computed assuming log-normal distribution.

The information given in Tables B.1 to B.6 are also presented in Figure 3.15. The scatter diagrams for $\Delta_{roof,e} / S_{d,e}$ and $\Delta_{roof,ie} / S_{d,ie}$ are given in the left panel of this figure. Each vertical strip corresponds to the results of a particular building model. The scatter plots in the right panel shown by symbols filled with light gray color present the median value of each vertical strip for linear and nonlinear structural behaviors. The plots in the right panel also show the $\Gamma_1 \phi_1^{roof}$ values of each model frame (symbols filled with dark gray color). The empirical modifying factors that are listed in Table 3.1 are also superimposed on these plots as solid lines.

The immediate observation between the plots showing the scatter diagrams of $\Delta_{roof,e} / S_{d,e}$ and $\Delta_{roof,ie} / S_{d,ie}$ is the lesser dispersion in $\Delta_{roof,e} / S_{d,e}$ (i.e., elastic behavior). The increased dispersion of the same ratio for inelastic structural behavior could be attributed to the continuous alterations in the structural dynamic properties due to the cyclic behavior that results in a complicated interaction between the ground motion and the structure. The median $\Delta_{roof,e} / S_{d,e}$ ratios vary between approximately 1.3 and 1.5. These are generally in good agreement with the $\Gamma_1 \phi_1^{roof}$ and the alternatively tabulated constants of the FEMA-356 document (ASCE, 2000) except for the 3-story tabulated constants that display a slightly lower bound. The median $\Delta_{roof,ie} / S_{d,ie}$ ratios reveal that both $\Gamma_1 \phi_1^{roof}$ and the alternatively tabulated constants that depend on number of stories of buildings mostly constitute an upper bound for relating the equivalent SDOF displacements to the roof displacements. The median $\Delta_{roof,ie} / S_{d,ie}$ values for inelastic behavior are clustered around the value of 1.2 regardless of the number of stories or fundamental periods whereas other relationships suggest higher values than 1.2 (the exception is the 3-story FEMA-356 (ASCE, 2000) tabulated results). The concentration of plastic hinging towards lower stories that in turn affects the re-distribution of lateral displacement pattern along the building height can be one of the prime reasons for lower maximum roof to equivalent SDOF displacement ratios in the inelastic behavior. The results presented by the robust statistics reveal that the direct use of $\Gamma_1 \phi_1^{roof}$ or the recommended coefficients in Table 3.2 of FEMA-356 document (ASCE, 2000) would result overestimations of inelastic roof displacements for the building systems considered.

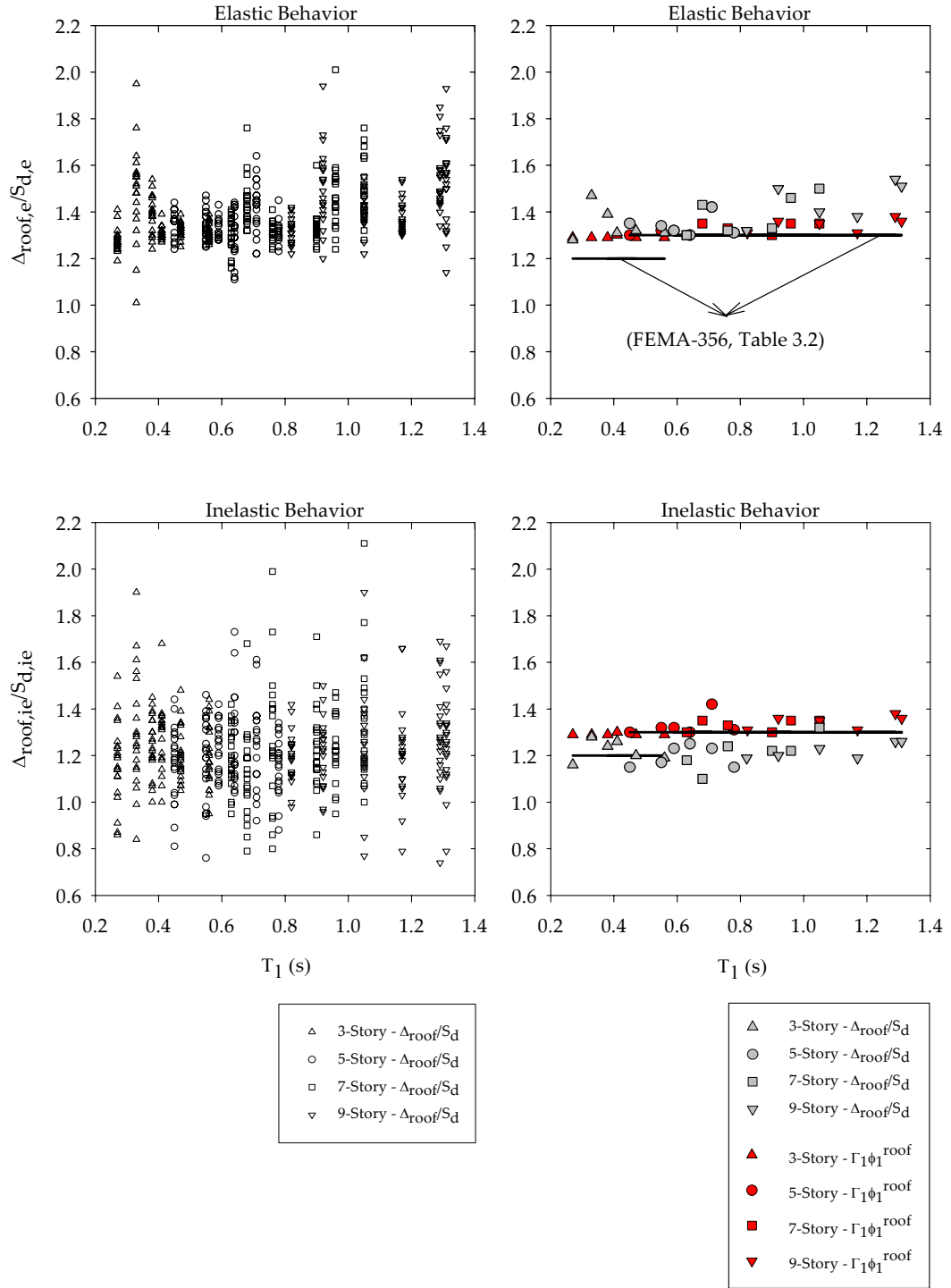


Figure 3.15 Various alternatives for relating SDOF demands to MDOF demands

3.5.2 Relationships between normalized strength capacity, maximum interstory drift and maximum roof drift ratios

Normalized strength capacities (i.e., γ) were computed for each model building according to the formula given below:

$$\gamma = \frac{mPSA}{V_y} \quad (3.6)$$

where m : total mass of the model building

PSA : elastic pseudo-acceleration at $T_{1,idealized}$

V_y : yield base shear strength computed from the idealized bilinear capacity curves

The normalized strength capacity, γ , can mimic the conventional response modification factor used in design codes if the contribution of overstrength is not considered. The increase in γ can be interpreted as the relative lateral strength decrease with respect to the elastic strength demand of the particular ground-motion record. The other advantage of γ is its close relationship between the so-called normalized lateral strength, R , used in the FEMA-273 (ATC, 1997), FEMA-356 (ASCE, 2000) and FEMA-440 (ATC, 2005) documents to describe the elastic to yielding strength capacity of SDOF systems.

The scatter plots presented in Figure 3.16 show the changes in the ratio of inelastic to elastic maximum roof drift ratio (i.e., $MRDR_{ie} / MRDR_e$) with respect to γ for the model buildings. Similarly, the scatter plots in Figure 3.17 present the same information for the ratio of inelastic to elastic maximum interstory drift ratios (i.e., $MIDR_{ie} / MIDR_e$). The ratios greater than 1 indicate that inelastic maximum

interstory or roof drift ratio values are greater than their elastic counterparts. The following observations are made from these figures.

1. In almost all cases increase in γ (i.e., decrease in the lateral strength capacity with respect to elastic strength obtained from pseudo-spectral acceleration) implies a fair decrease in the nonlinear roof displacements. The exceptional cases are some of the stiff buildings (i.e., 3-story models) that show an increase in nonlinear deformation demands with increasing γ . This might suggest the sensitivity of stiff systems to the changes in lateral strength capacity. The other exceptional case is the Group 6 building models that are subjected to the most severe ground-motion records as described in Chapter 2.
2. The nonlinear interstory drift ratio demands display a decreasing trend with increasing γ similar to nonlinear roof drift ratio as discussed in item (1). One should note that the variation of $MIDR_{ie} / MIDR_e$ scatters display a fairly more complex pattern. In particular $MIDR_{ie} / MIDR_e$ tends to be larger than 1 (or approximately follows a value of 1) for ground motions with pulse.
3. Another observation highlighted from these figures is that $MIDR_{ie} / MIDR_e$ ratios are slightly larger than the corresponding $MRDR_{ie} / MRDR_e$ ratios. This observation is clear in Figure 3.18 that presents $MIDR_{ie} / MIDR_e$ versus $MRDR_{ie} / MRDR_e$ scatters. Figure 3.18 once again indicates that the tendencies of these two ratio parameters are fairly the same with $MIDR_{ie} / MIDR_e$ yielding approximately 20% larger values than $MRDR_{ie} / MRDR_e$. One can observe this comment better from the plotted trend line in the same figure.

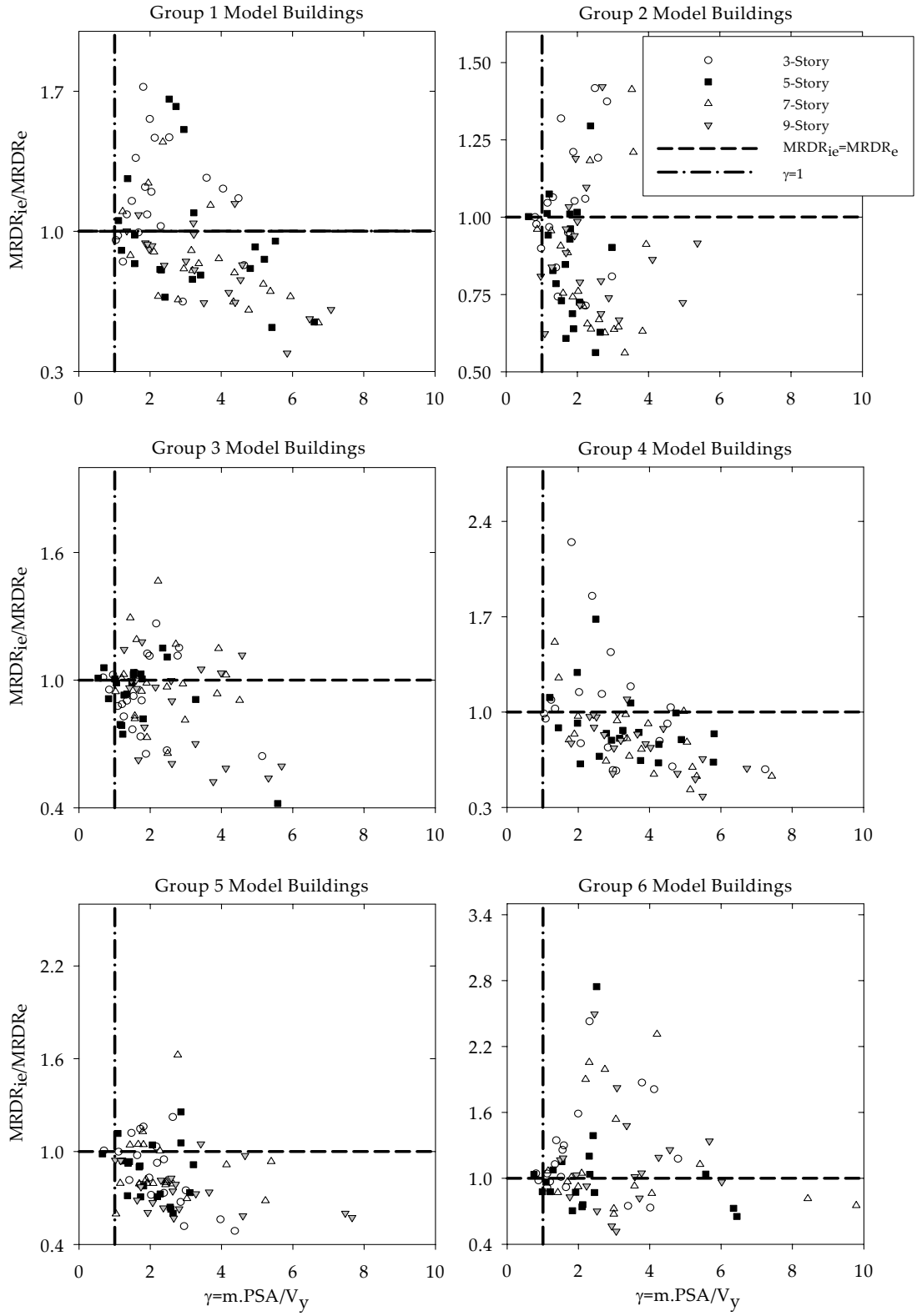


Figure 3.16 $MRDR_{ie} / MRDR_e$ versus γ plots for the model buildings

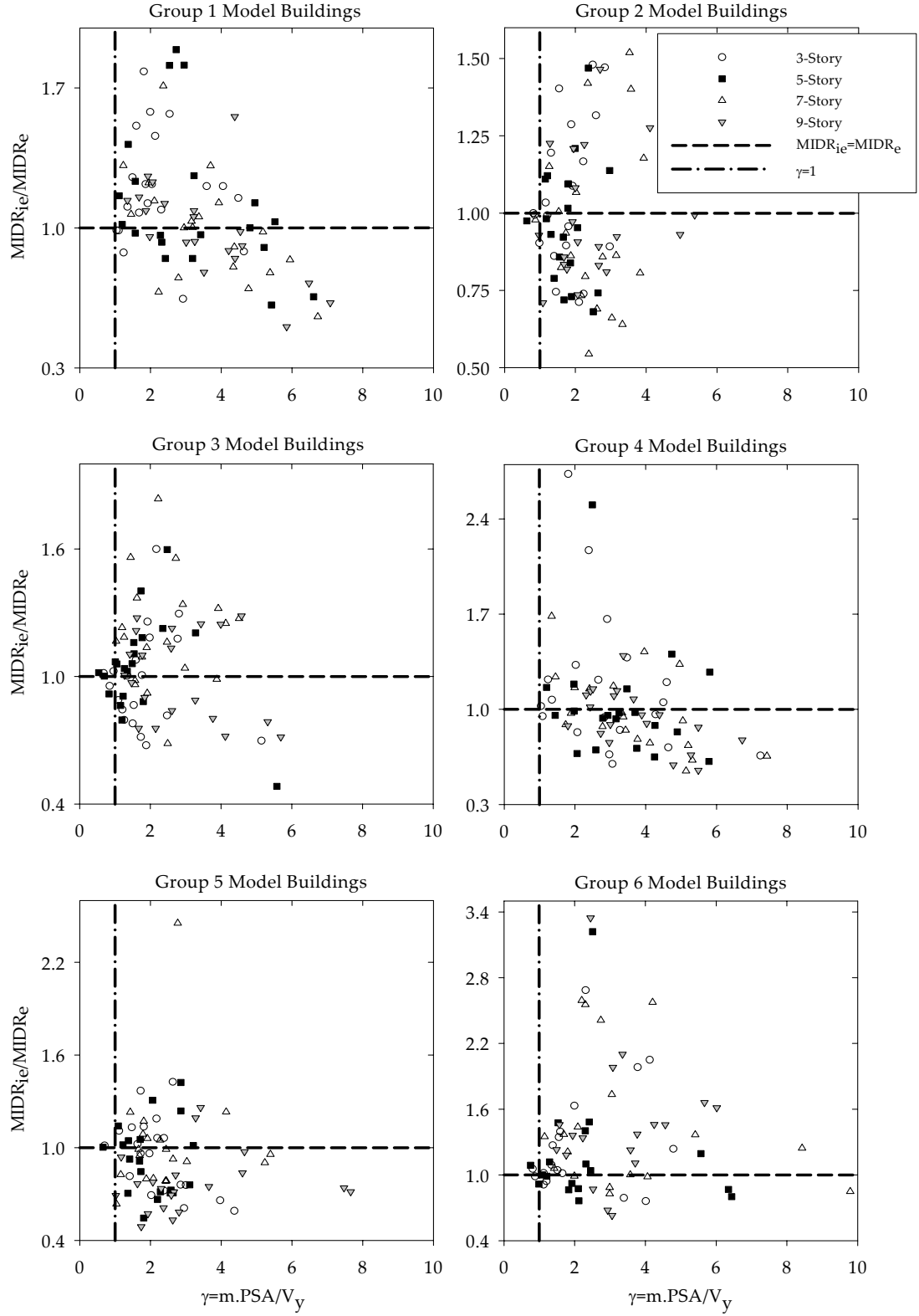


Figure 3.17 $MIDR_{ie} / MIDR_e$ versus γ plots for the model buildings

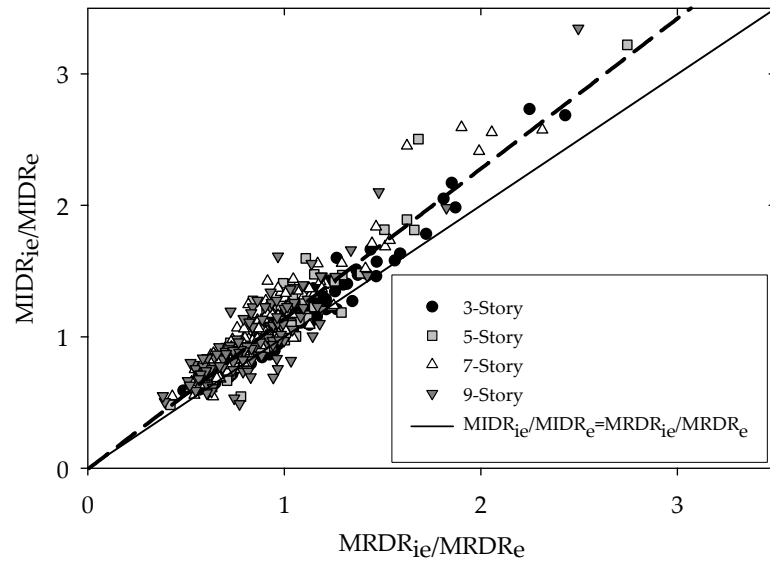


Figure 3.18 $MIDR_{ie} / MIDR_e$ versus $MRDR_{ie} / MRDR_e$ for the model buildings

3.5.3 Information on the normalized lateral strength, R , of the equivalent SDOF systems of model buildings

Normalized lateral strength, R , defined as the elastic to yield strength capacity of SDOF systems was computed for each equivalent SDOF system subjected to the ground motions used in this study. The observed distribution of R for each building group was verified for log-normal distribution. Probability papers for log-normal distribution and goodness-of-fit check (Kolmogorov-Smirnov at 10% significance level) were applied to the empirical data. A typical example for 5-story building of Group 4 is presented in Figure 3.19. The probability paper that is presented on the left shows that except for extreme values the observed R – values are in agreement with the theoretical log-normal distribution. The Kolmogorov-Smirnov goodness-of-fit check presented on the right indicates that the empirical data agrees with the log-normal distribution at the 10% significance level. The

cumulative distribution functions of R –values and the corresponding log-normal distributions are displayed in Figures B.1 to B.4 for all model frames in Appendix B.

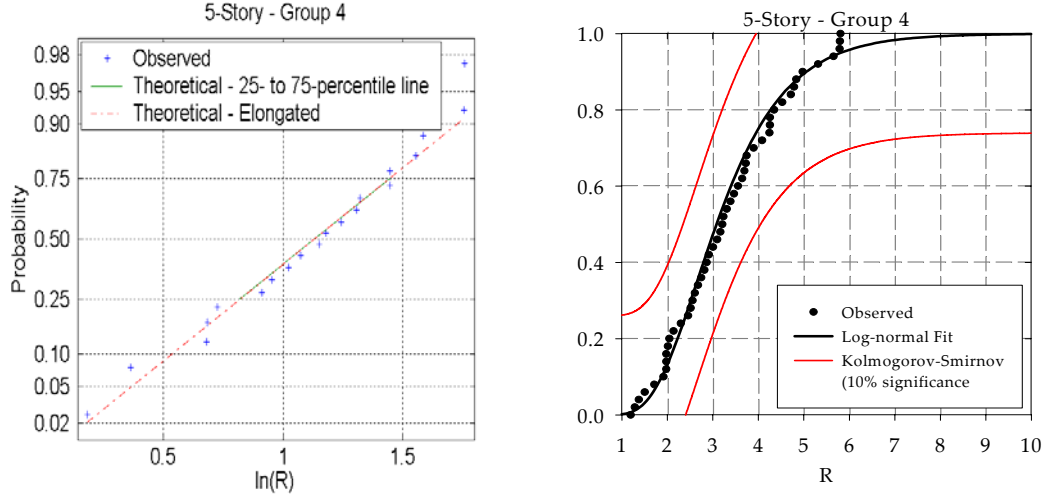


Figure 3.19 Model verification for the log-normality of R

Figure 3.20 shows the distribution of R –values together with the log-normal distributions for model buildings subjected to ground motions with and without pulse signals. The plots in the left panel show these distributions for building models that are subjected to the ground-motion groups without pulse signals (i.e., Group A and B ground motions). The plots in the right panel display the same information for building groups that are subjected to near-fault records with pulse signals (i.e., Group C and D ground motions). Each row in this figure corresponds to a different number of stories. The goodness-of-fit procedure to verify the theoretical log-normal distribution as presented in Figure 3.19 was also applied for the empirical data shown in this figure. The verification revealed that the log-normal distribution is acceptable at the 10% significance level. This observation will be used in Chapter 5 while describing the probabilistic approach presented for estimating the inelastic $MIDR$.

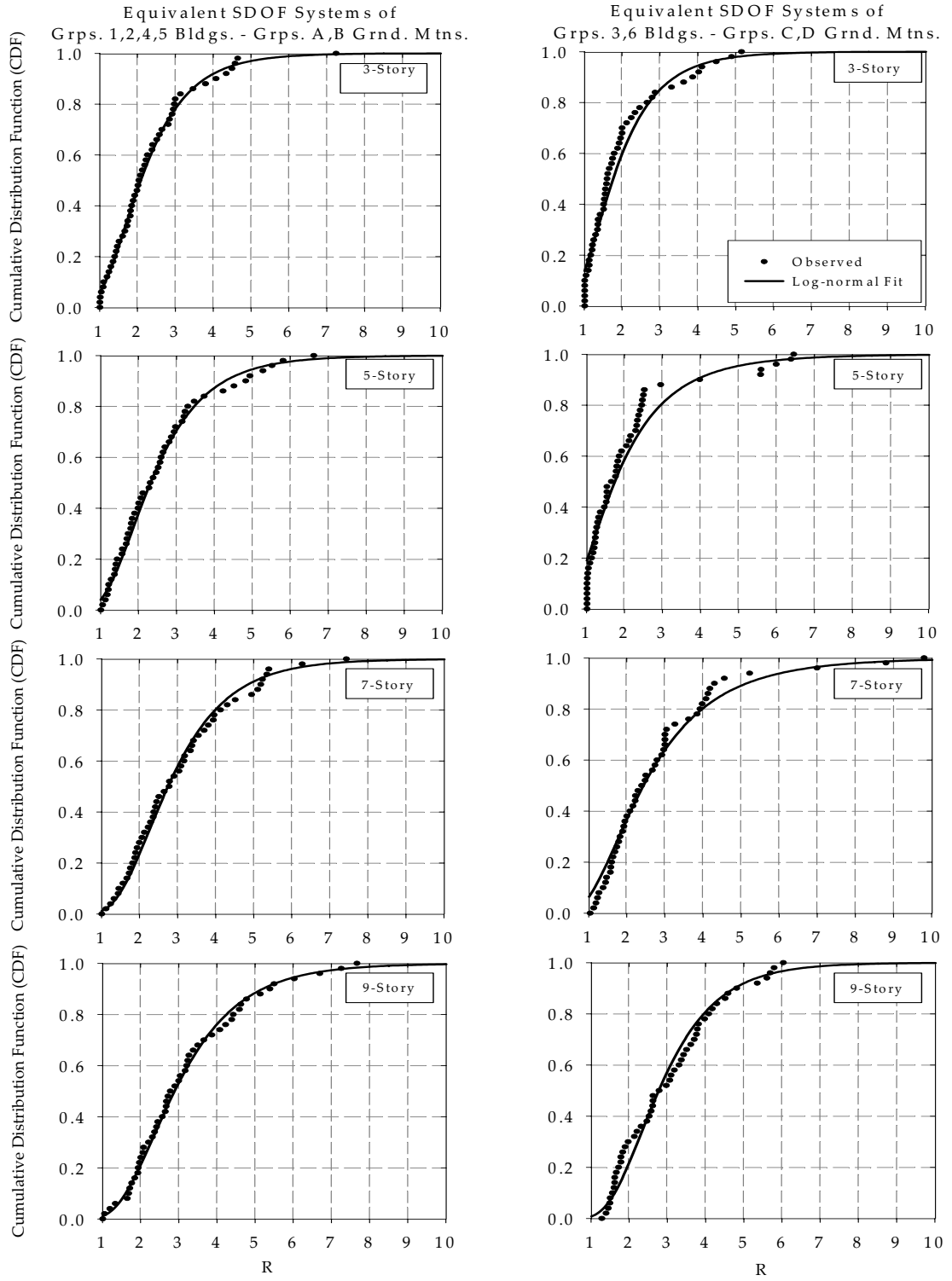


Figure 3.20 R – distributions for building groups subjected to ground-motion groups with (left panel) and without pulse (right panel) signals

Figure 3.21 shows the theoretical log-normal fits for R -distribution that are presented in Figures B.1 to B.4. Each panel presents the variation of R for a specific number of stories. The plots in this figure indicate that the increase in number of stories implies an increase in R -range. It is also noteworthy to indicate that for uniform lateral stiffness models (i.e., Groups 1 and 4) the variations of R follow a very close pattern for 5-, 7- and 9-story building models.

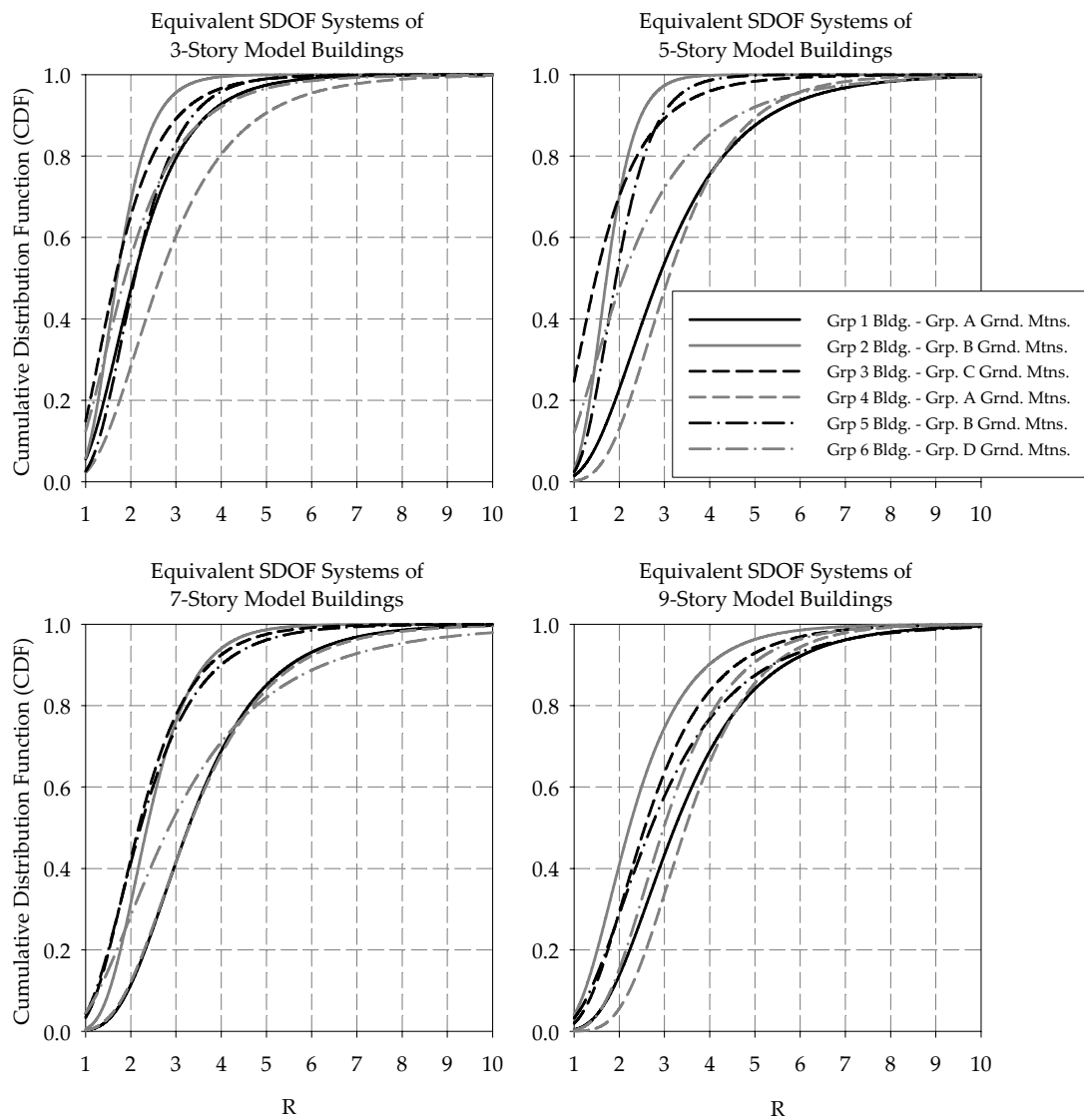


Figure 3.21 Log-normal fits assembled on the model buildings

3.6 Comparisons of SDOF and MDOF responses

The comparisons about the similarity of MDOF and corresponding equivalent SDOF behavior were performed by plotting inelastic to elastic S_d ratios against inelastic to elastic maximum interstory and roof drift ratios. The scatter plots for each building group are presented in Figures 3.22 and 3.23. The plots in Figure 3.22 describe this relationship for inelastic to elastic S_d variation against inelastic to elastic maximum roof drift ratio variation. Figure 3.23 gives the same information for inelastic to elastic maximum interstory drift ratio variation. The comparisons between these plots show that there is a fairly well correlation between the SDOF and MDOF responses given that the building groups are predominantly deforming under the first-mode behavior. The fairly well correlation between the inelastic to elastic SDOF and MDOF deformation demand ratios is presented by the correlation coefficients (i.e., COC) on each plot. This observation suggests that the variations in inelastic to elastic MDOF deformation demand can be extracted from the corresponding SDOF deformation ratios for structures primarily deforming under the first-mode influence. This will be used in the simplified procedure that is described in Chapter 5 of this study.

The discussions presented in the previous paragraph are carried one step further by computing the cumulative distribution functions for $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$. Comparative distribution plots between the $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ are presented between Figures B.5 to B.8. Similarly, Figures B.9 to B.12 show the same comparisons between $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$. These figures also show the theoretical log-normal distributions that are fitted to each actual distribution. In order to verify the log-normal fit assumption, the probability paper and Kolmogorov-Smirnov goodness-of-fit checks were applied once again to the empirical data. An example case for $S_{d,ie} / S_{d,e}$, $MIDR_{ie} / MIDR_e$, and

$MRDR_{ie} / MRDR_e$ is given in Figure 3.24. These plots also show that except for very extreme values the observed deformation demand values are in agreement with the theoretical log-normal distribution at 10% significance level. Although not shown in this report for the sake of brevity, the consistency in the variation of empirical data with log-normal distribution increases as the number of stories increases. This is clear in Figures B.5 to B.12 in Appendix B. Moreover, as presented in Figures B.5 to B.12, the log-normal fits between $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$ follow very similar patterns reinforcing the observations highlighted in Figures 3.22 and 3.23. Both $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$ ratios attain larger values for building models subjected to Group D ground motions that is the most severe ground-motion group in terms of magnitude, distance and PGV . It should be noted that the product of statistically independent log-normal variables is also log-normal. Thus, $S_{d,ie}$, $S_{d,e}$, $MRDR_{ie}$, $MRDR_e$, $MIDR_{ie}$ and $MIDR_e$ are conformed to be log-normal variables (Ang and Tang, 1975). The reasonably well log-normal distribution fits to the actual variation of the data and the close similarity between the inelastic to elastic MDOF maximum interstory drift ratio and SDOF deformation demands will be used in Chapter 5 while presenting the approximate procedure for estimating the inelastic maximum interstory drift ratio demands for frame buildings.

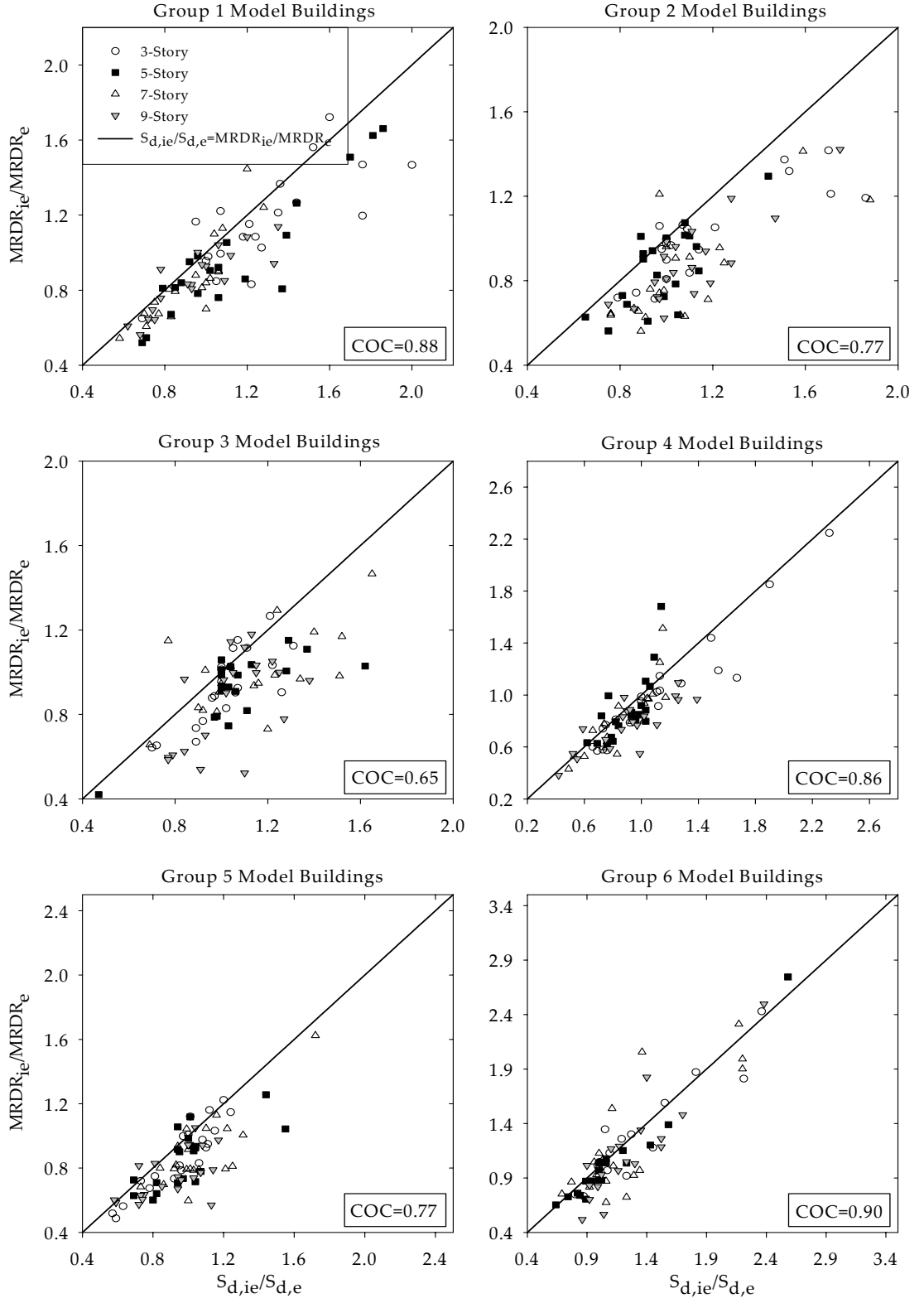


Figure 3.22 $MRDR_{ie} / MRDR_e$ versus $S_{d,ie} / S_{d,e}$ plots for the model buildings

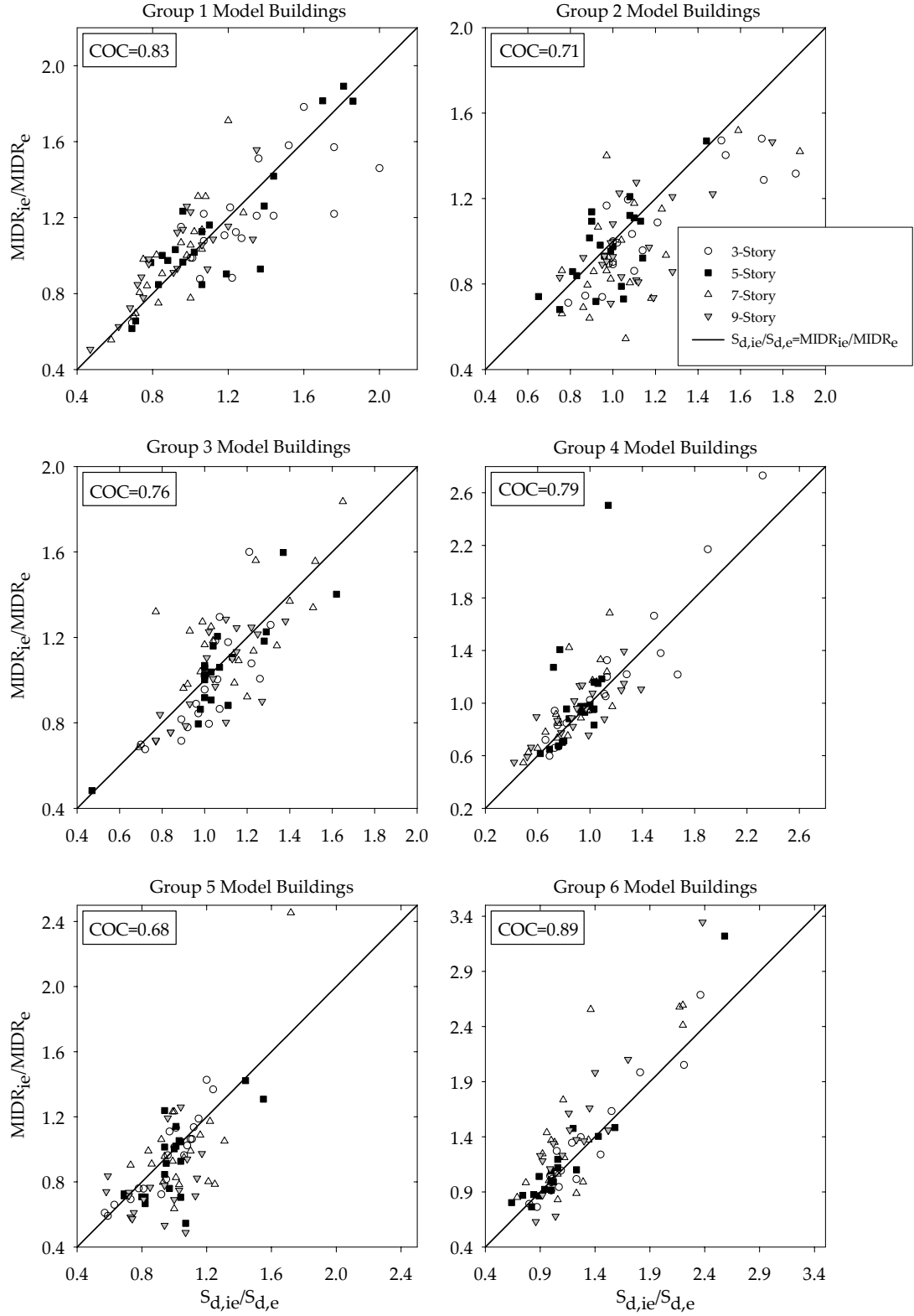


Figure 3.23 $MIDR_{ie} / MIDR_e$ versus $S_{d,ie} / S_{d,e}$ plots for the model buildings

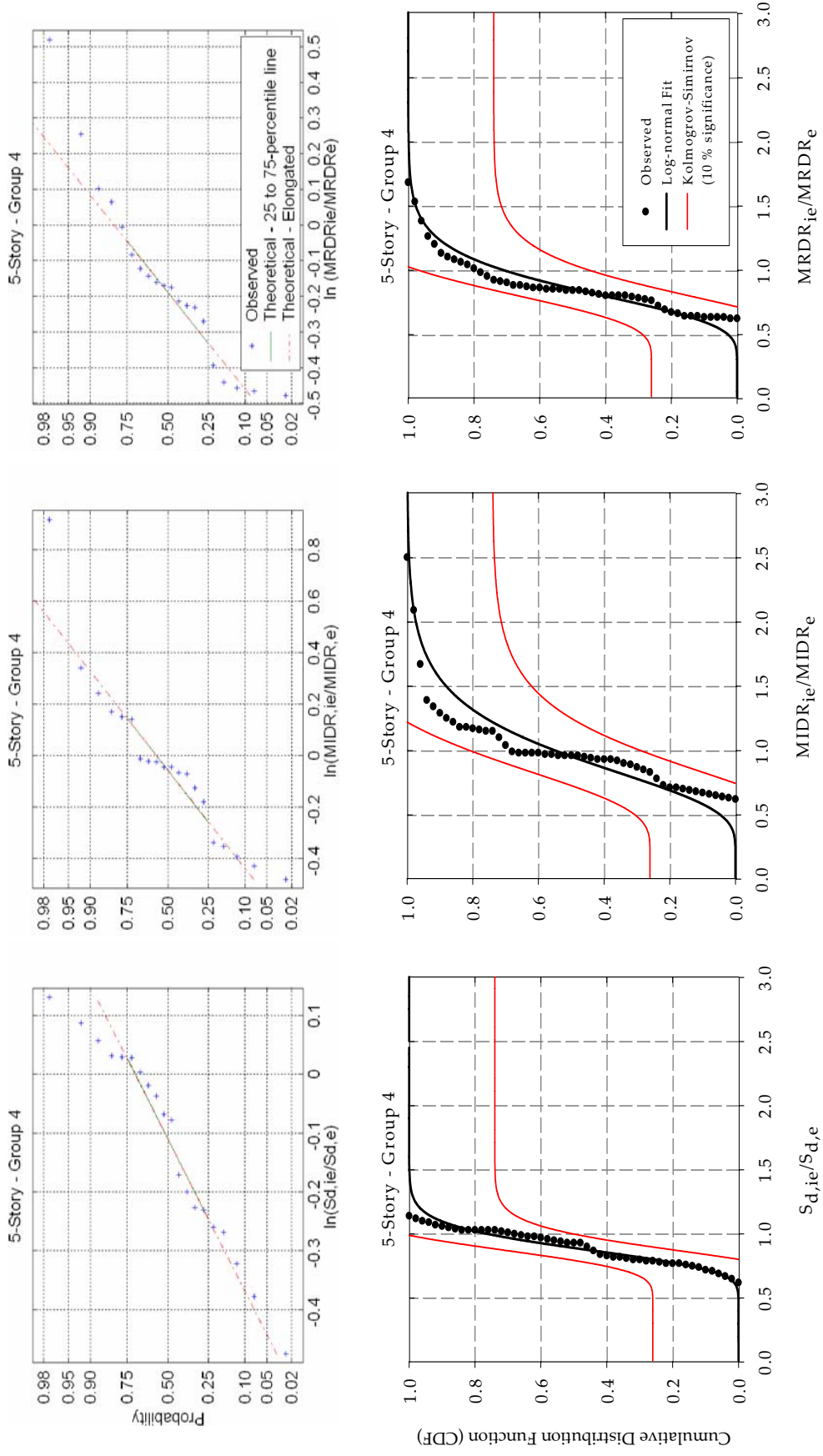


Figure 3.24 Model verification for the log-normality of $S_{d,ie} / S_{d,e}$, $MIDR_{ie} / MIDR_e$ and $MRDR_{ie} / MRDR_e$

Cumulative distribution functions of $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ ratios were also computed separately for Groups 1, 2, 4 and 5 model buildings (i.e., model buildings subjected to ground-motion groups without pulse; Groups A and B) and Groups 3 and 6 model buildings (i.e., model buildings subjected to ground-motion groups with pulse; Groups C and D). These are displayed in Figure 3.25. The plots on the left panel correspond to the model buildings subjected to ground motions without pulse signals, whereas the plots on the right panel correspond to the model buildings subjected to ground-motion groups with pulse signals. Similar figures that are plotted for the cumulative distributions of $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$ ratios are presented in Figure 3.26. The following observations are made by close examination of these figures.

1. Theoretical log-normal distribution follows the variation of observed data fairly well for all building models subjected to records with and without pulse. However, the log-normal distributions are in better agreement with the observed data for the building models subjected to ground motions that do not contain pulse.
2. Both $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$ ratios attain larger values for building models subjected to ground motions with pulse signals indicating the sensitivity of this ground-motion type with respect to others.
3. The inelastic to elastic deformation demand variations for SDOF and MDOF systems follow each other closely; an observation that is emphasized previously in this section.

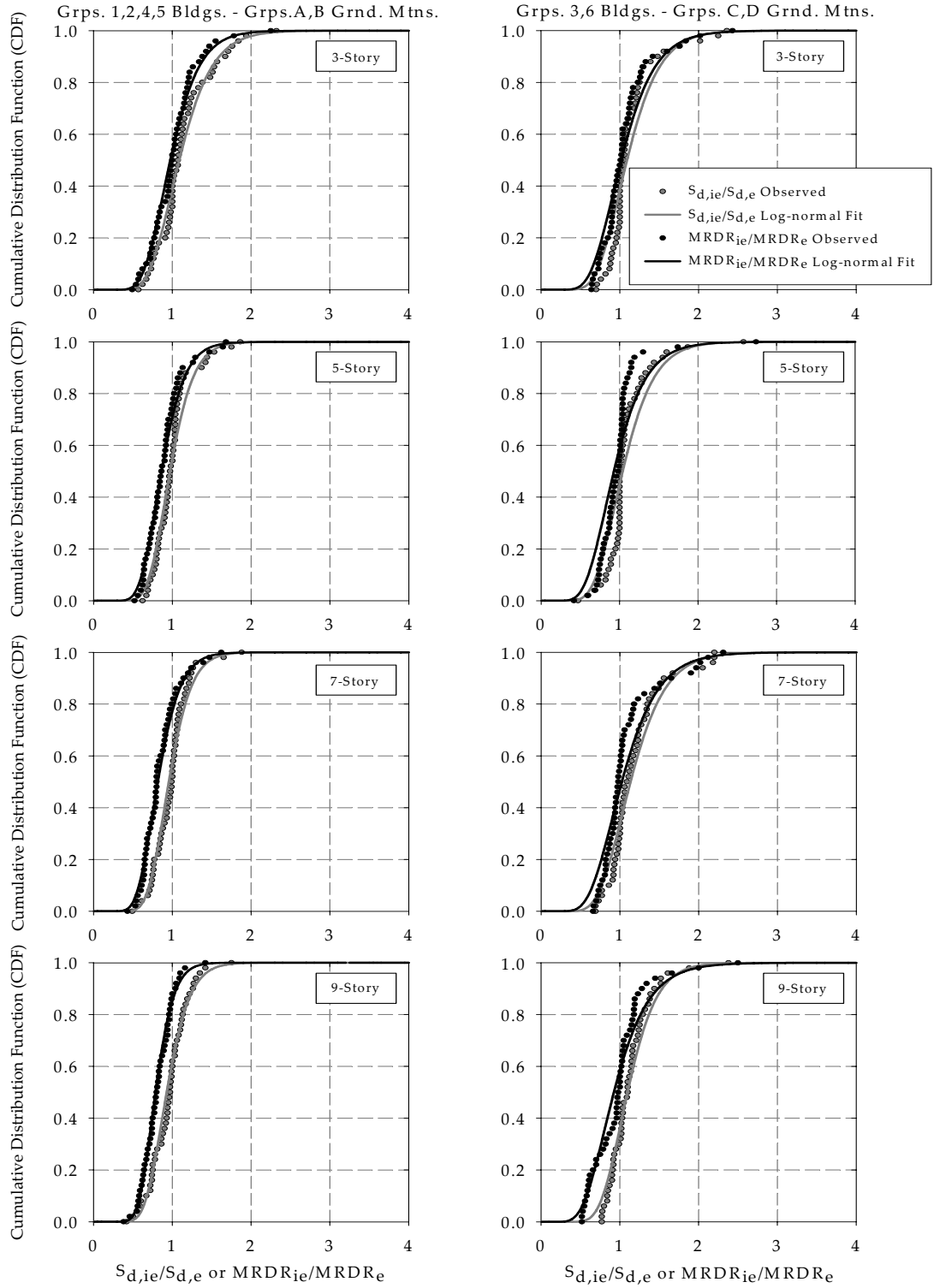


Figure 3.25 $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ distributions of building groups subjected to ground-motion groups with and without pulse signals

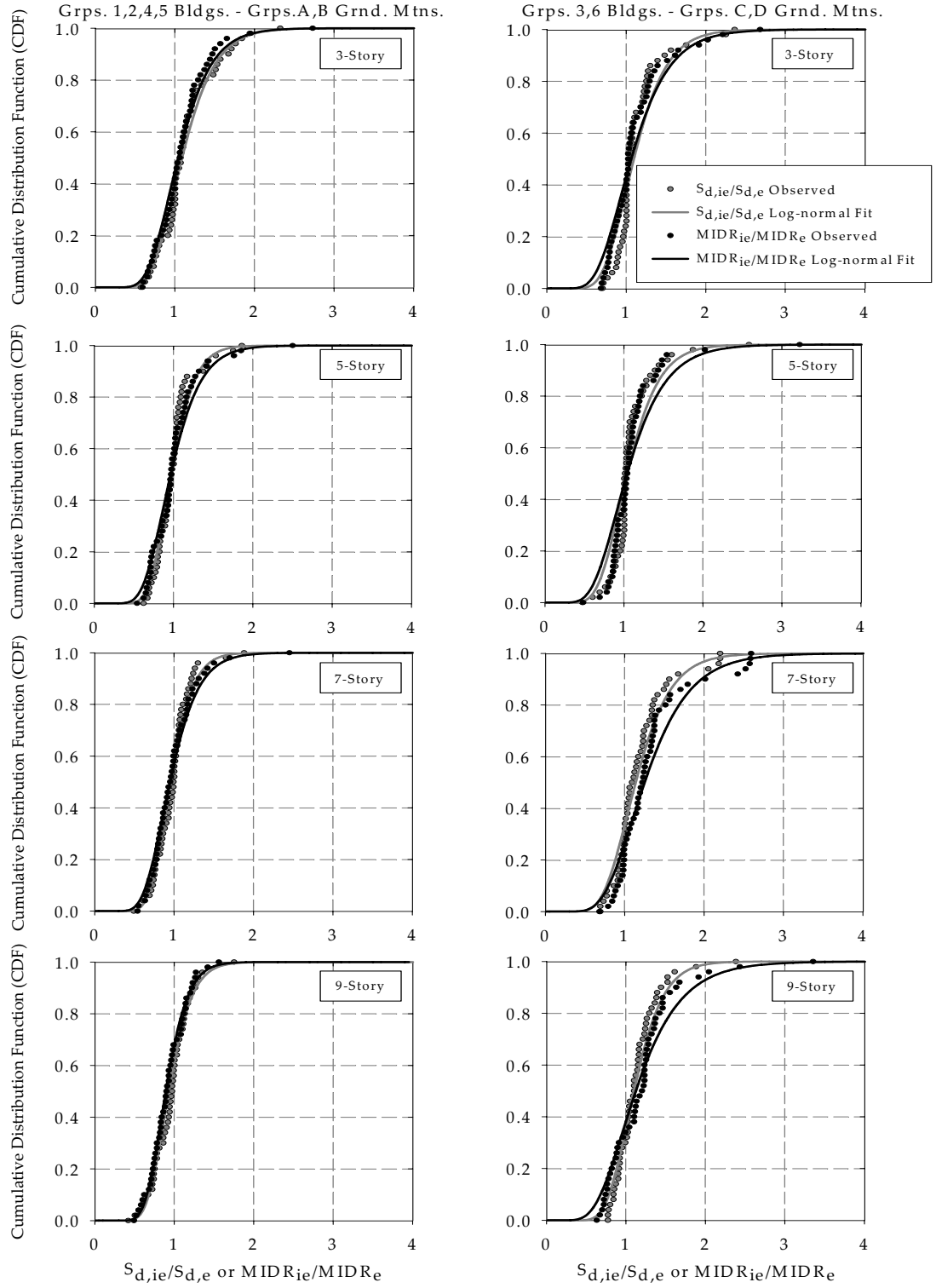


Figure 3.26 $S_{d,ie}/S_{d,e}$ and $MIDR_{ie}/MIDR_e$ distributions of building groups subjected to ground-motion groups with and without pulse signals

CHAPTER 4

EVALUATION AND COMPARISON OF NONLINEAR STATIC PROCEDURES

4.1 Introduction

The response of structural systems under strong ground shaking generally results in nonlinear behavior. Until recently, the prediction of nonlinear structural behavior has been mostly based on linear analysis procedures according to the traditional force-deformation relationships, and some adjustments have been made to overall building deformations and material behavior criteria to permit better consideration of the probable nonlinear characteristics of the seismic response. This is known as the force-based design approach. However, the increased implementation of displacement-based seismic assessment of structures during the last decade led to the proposition of approximate nonlinear static analysis procedures (i.e., NSPs) for predicting the seismic structural behavior. The NSPs are generally accepted as more reliable for characterizing the performance of a structure under ground shaking.

The nonlinear methods proposed in the FEMA-273/274 (ATC, 1997), its successor the FEMA-356 (ASCE, 2000) and the ATC-40 (ATC, 1996) documents constitute well-organized pioneer methods for the ongoing efforts in this field. Recently, the simplified procedures presented in these documents have been improved and the modified expressions for the estimation of nonlinear deformation demands on structural systems are presented in the FEMA-440 document (ATC, 2005). The basic premise in these documents is the estimation of deformation demands on structural systems by making use of nonlinear force-deformation curves that are generated from the pushover analysis. Then these force-deformation curves are converted into

ADRS (i.e., Acceleration-Displacement Response Spectrum) format. This initial step reduces the MDOF system to an equivalent SDOF system thus simplifying the detailed structural model as well as providing insightful information about the likely nonlinear behavior of the structure.

This chapter aims to give the basic outline of the NSPs that are presented in the documents stated in the above paragraph. Statistical results that evaluate the performance of these procedures are also presented within the context of this chapter using the results of nonlinear RHA presented in Chapter 3.

4.2 NSP proposed in FEMA-356 document (ASCE, 2000) (Displacement Coefficient Method, DCM)

In this procedure, several empirically derived factors are used to calculate the response of the MDOF system by modifying the linear elastic response of the equivalent SDOF system. The empirical factors used in DCM are primarily based on the normalized lateral strength, R , (i.e., ratio of elastic to yield strength) and the hysteretic behavior of the system. The general form of the equation proposed in DCM is displayed in Equation (4.1).

$$\Delta_{roof,ie} = C_0 C_1 C_2 C_3 S_{d,e} \quad (4.1)$$

Coefficient C_0 relates the spectral displacement of the equivalent SDOF system to the roof displacement of the MDOF system as explained in detail in Chapter 3. C_1 is the modification factor used in order to relate the expected maximum inelastic displacements to displacements calculated for linear elastic response. Coefficient C_2 is the modification factor used in order to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement

response. C_3 is the modification factor used in order to represent the increased displacements due to dynamic $P - \Delta$ effects. The reader is referred to Chapter 3 of the FEMA-356 document (ASCE, 2000) for more information on Displacement Coefficient Method.

4.3 NSP proposed in ATC-40 document (ATC, 1996) (Capacity Spectrum Method, CSM)

The basic assumption in the Capacity Spectrum Method, CSM, that is initially proposed by Freeman et al. (1975) is that the maximum inelastic deformation of a nonlinear SDOF system can be approximated by the maximum deformation of a linear elastic SDOF system that has a period and a damping ratio larger than the corresponding initial period and damping ratios of the nonlinear system. This is known as “equivalent linearization technique”. The Capacity Spectrum Method uses displacement ductility (μ) based empirical relationships to calculate the elongated period and the increased viscous damping of the equivalent linear system. The procedure simply matches the capacity curve of the equivalent SDOF system (in the form of acceleration-displacement response spectrum, ADRS) with the modified response spectrum (that mimics the nonlinear effects through equivalent linearization concept), to estimate the maximum response of the equivalent SDOF system through one of the several iteration procedures introduced in the ATC-40 document (ATC, 1996). The reader is referred to Chapter 8 of this document for more information on Capacity Spectrum Method.

The procedures presented in the ATC-40 (ATC, 1996) and FEMA-356 (ASCE, 2000) documents have been used widely among the researchers and engineers who

noticed some deficiencies of these two methods. The important deficiencies reported for these methods can be listed as:

1. Substantial differences in the deformation demand estimations between these two methods (e.g., Aschheim et al., 1998)
2. Numerical problems that would affect the reliability of the deformation demand estimations (e.g., Chopra and Goel, 2000b; Akkar and Miranda, 2006)
3. Serious accuracy problems in the SDOF deformation demand estimations when compared to the results from nonlinear RHA (e.g., Akkar and Miranda, 2006)

These two widely used procedures are recently updated by the ATC-55 project that principally aimed to improve the accuracy of the deformation demand estimations of these methods as well as the reduction of discrepancy in the predictions obtained from them. The regression equations used in the new DCM that estimate the expected inelastic SDOF deformation demands were derived from the statistical study conducted by Ruiz-García and Miranda (2003). The ductility dependent empirical relations used in the new CSM were presented by Guyader and Iwan (2006). The principal product of the ATC-55 project is the FEMA-440 report (ATC, 2005) that presents the improved versions of the DCM and CSM as well as a limited comparative study of these methods that will be mentioned in the following sections of this chapter.

4.4 NSPs proposed in the FEMA-440 document (ATC, 2005)

4.4.1 Improved Displacement Coefficient Method

The FEMA-440 document (ATC, 2005) brings the following modifications over DCM described in the FEMA-356 document (ASCE, 2000):

- (a) The coefficient C_1 is modified for estimating the maximum displacement of an elastoplastic SDOF oscillator from a linear elastic oscillator. The new C_1 also eliminates the recommended capping limitations by FEMA-356 document (ASCE, 2000) for relatively short-period structures.
- (b) The coefficient C_2 is modified by considering only the cyclic stiffness degradation (the former C_2 considered the stiffness degradation and strength deterioration hysteretic behavior).
- (c) The coefficient C_3 that considers the dynamic instability due to possible $P - \Delta$ effects is eliminated. Instead, a new expression is proposed that limits the normalized lateral strength (i.e., critical value of R) to avoid dynamic instability.

Therefore, the general form of the improved DCM takes the following format:

$$\Delta_{roof,ie} = C_0 C_1 C_2 S_{d,e} \quad (4.2)$$

The recommended C_1 and C_2 expressions of the improved DCM are as follows:

$$C_1 = 1 + \frac{R-1}{dT_e^2} \quad (4.3)$$

$$C_2 = 1 + \frac{1}{800} \left(\frac{R-1}{T_e} \right)^2 \quad (4.4)$$

In these formulas, T_e is the effective fundamental period of the SDOF system in seconds, the regression constant a is devised for considering the influence of different site classes. It attains a value of 60 for stiff site recordings used in this study. The parameter R is the normalized lateral strength computed by Equation (4.5).

$$R = \frac{PSA}{V_y / W} C_m \quad (4.5)$$

C_m is the effective mass factor that can be obtained from Table 3.1 of FEMA-356 document (ASCE, 2000). Alternatively, it can be taken as the effective modal mass calculated for the fundamental mode using an eigenvalue analysis.

The expression for C_1 does not consider the influence of earthquake magnitude based on the observations presented in Ruiz-García and Miranda (2003). The interaction between the SDOF deformation demands and important ground-motion parameters that are observed to play role in reducing the dispersion or improving the median deformation predictions (Akkar and Miranda, 2004; Akkar and Özen, 2005) are also disregarded in the coefficient C_1 . The new C_1 also neglects the influence of post-yield stiffness that is deemed to be important for the SDOF deformation demands in the short spectral period range for low strength systems (Chopra and Chintanapakdee, 2004; Ruiz-García and Miranda, 2003).

The coefficient C_2 that accounts for the changes in elastoplastic SDOF deformation demands produced by the departures due to stiffness degrading hysteretic behavior

is taken as 1 in this study since building models are assumed to represent non-degrading systems as discussed in Chapter 3.

4.4.2 Examination of the improved DCM coefficients for the model buildings in this study

The equation of improved DCM reduces to the form displayed in Equation (4.6) since coefficient C_2 is assumed as 1 for the model buildings of this study.

$$\Delta_{roof,ie} = C_0 C_1 S_{d,e} \quad (4.6)$$

The coefficient C_0 has been examined in detail in Chapter 3. This section principally examines the coefficient C_1 . Figure 4.1 shows the significance of ground-motion intensity and post-yield stiffness ratio, α , on C_1 . The plots in the left panel display the ratios of inelastic to elastic spectral displacements (i.e., $S_{d,ie} / S_{d,e}$) plotted against peak ground acceleration, PGA , of the ground motions used in the study. They are plotted for a relatively short-period oscillator of $T = 0.30$ s and $R = 2$. Note that $S_{d,ie} / S_{d,e}$ ratio represents the actual variation of C_1 . The right panel displays the same ratio against peak ground velocity, PGV , of the ground motions used in the study. They are plotted for $R = 4$ and a vibration period value of $T = 0.85$ s that is within the upper bound of the fundamental period range covered in this study. In the first row, the figures are plotted for $\alpha = 0\%$ whereas in the second row the post-yield stiffness ratio takes a value of 10%. The initial viscous damping is 5% for the sake of compatibility with C_1 . (Unless stated otherwise, the initial viscous damping in all dynamic analyses is taken as 5% that is compatible with the NSPs discussed in this study). These figures also show the variation of C_1 (horizontal constant solid line) proposed by the improved DCM.

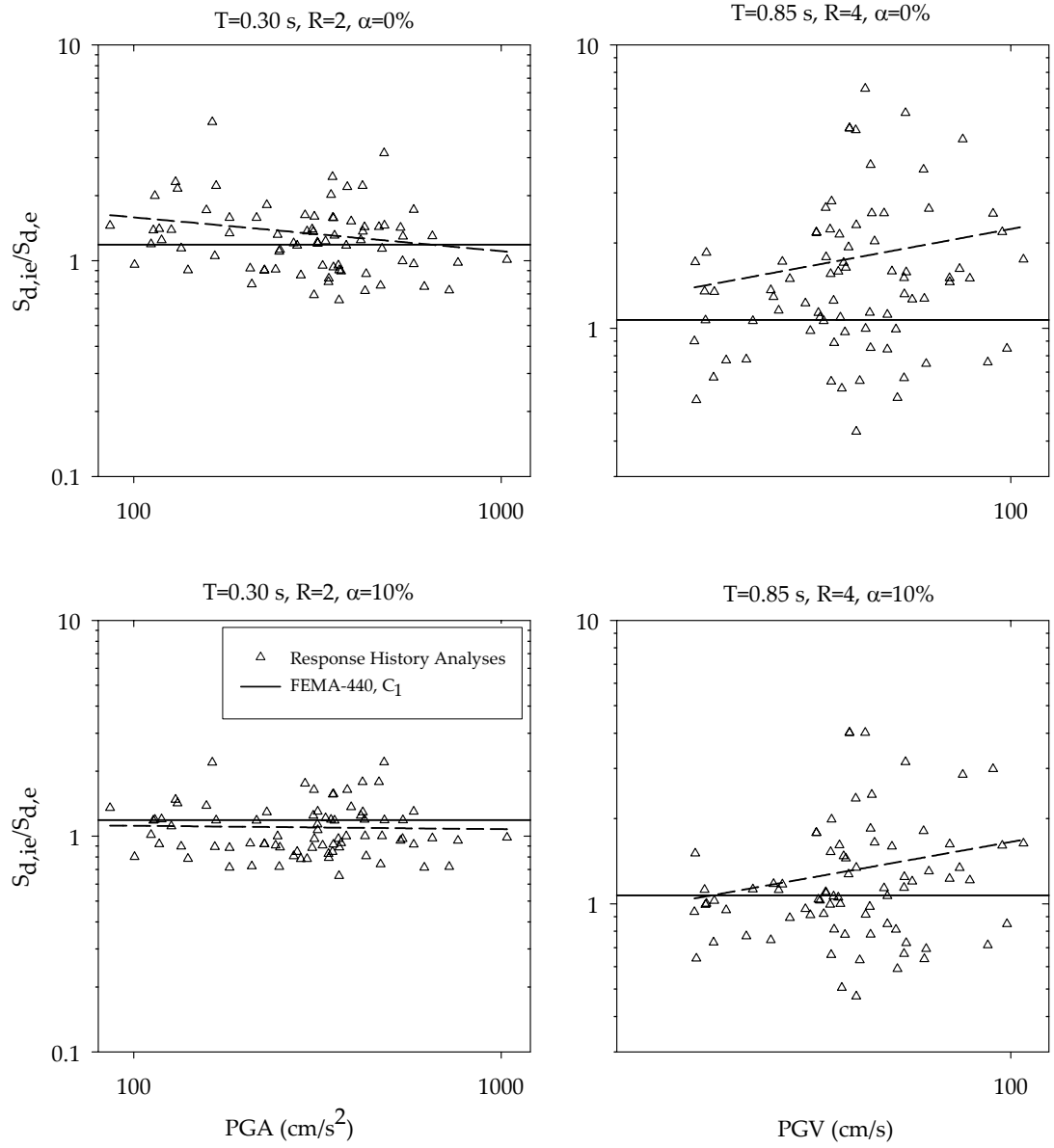


Figure 4.1 Variation of actual $S_{d,ie} / S_{d,e}$ as a function of PGA and PGV and comparisons with the improved C_1

The scatter plots presented in Figure 4.1 show a clear dependence of $S_{d,ie} / S_{d,e}$ ratios on the arbitrarily selected intensity measures. The variation of $S_{d,ie} / S_{d,e}$ is particularly sensitive to PGV as presented in the right-hand panels. Moreover, the changes in post-yield stiffness ratio, α , modify the general trends in $S_{d,ie} / S_{d,e}$ variation as presented by dashed lines that represent the tendency of actual $S_{d,ie} / S_{d,e}$ variation in each plot. The improved C_1 value considers neither the influence of post-yield stiffness nor ground-motion intensity. For this particular reason it attains a constant value for a given vibration period and R – value that may result in significant error in the deformation demand estimations.

Figures 4.2 to 4.5 present the scatter plots for Group A, B, C and D ground motions, respectively, for the actual variation of C_1 (i.e., $S_{d,ie} / S_{d,e}$) as a function of normalized lateral strength, R , varying between 1.5 to 8. These ratios are plotted for two distinct vibration periods, $T = 0.35$ s and $T = 1.30$ s in Figures 4.2 and 4.3, that approximately cover the lower and upper fundamental period values of the model buildings that are subjected to ground-motion groups without pulse signals (i.e., Groups 1, 2, 4 and 5 model buildings subjected to Group A and B ground motions). The plots in Figures 4.4 and 4.5 are displayed for $T = 0.40$ s and $T = 1.25$ s that constitute the lower and upper fundamental period values of Group 3 and 6 model buildings that are subjected to Group C and D ground motions with pulse-dominant signals. The plots are displayed for 3 distinct post-yield stiffness ratios of $\alpha = 0\%$, 6% and 10% to describe the influence of post-yield stiffness on inelastic SDOF deformation demands. The log-normal median variations of $S_{d,ie} / S_{d,e}$ as well as the changes in the improved C_1 for different R – levels are also shown on the plots in order to highlight the sensitivity of C_1 for different post-yield stiffness ratios.

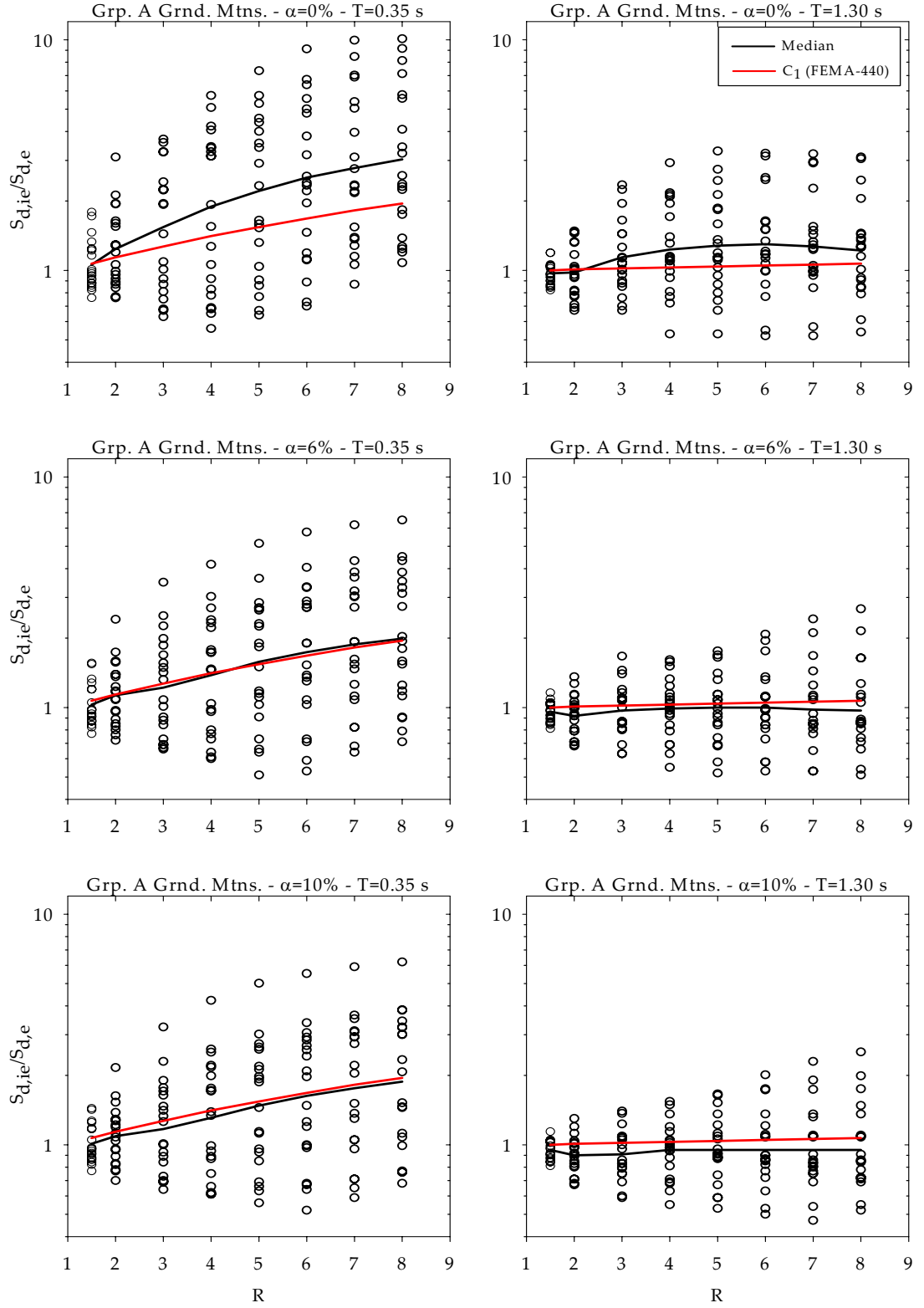


Figure 4.2 Sensitivity of $S_{d,ie}/S_{d,e}$ to α for SDOF systems subjected to Group A ground motions and comparisons with the improved C_1

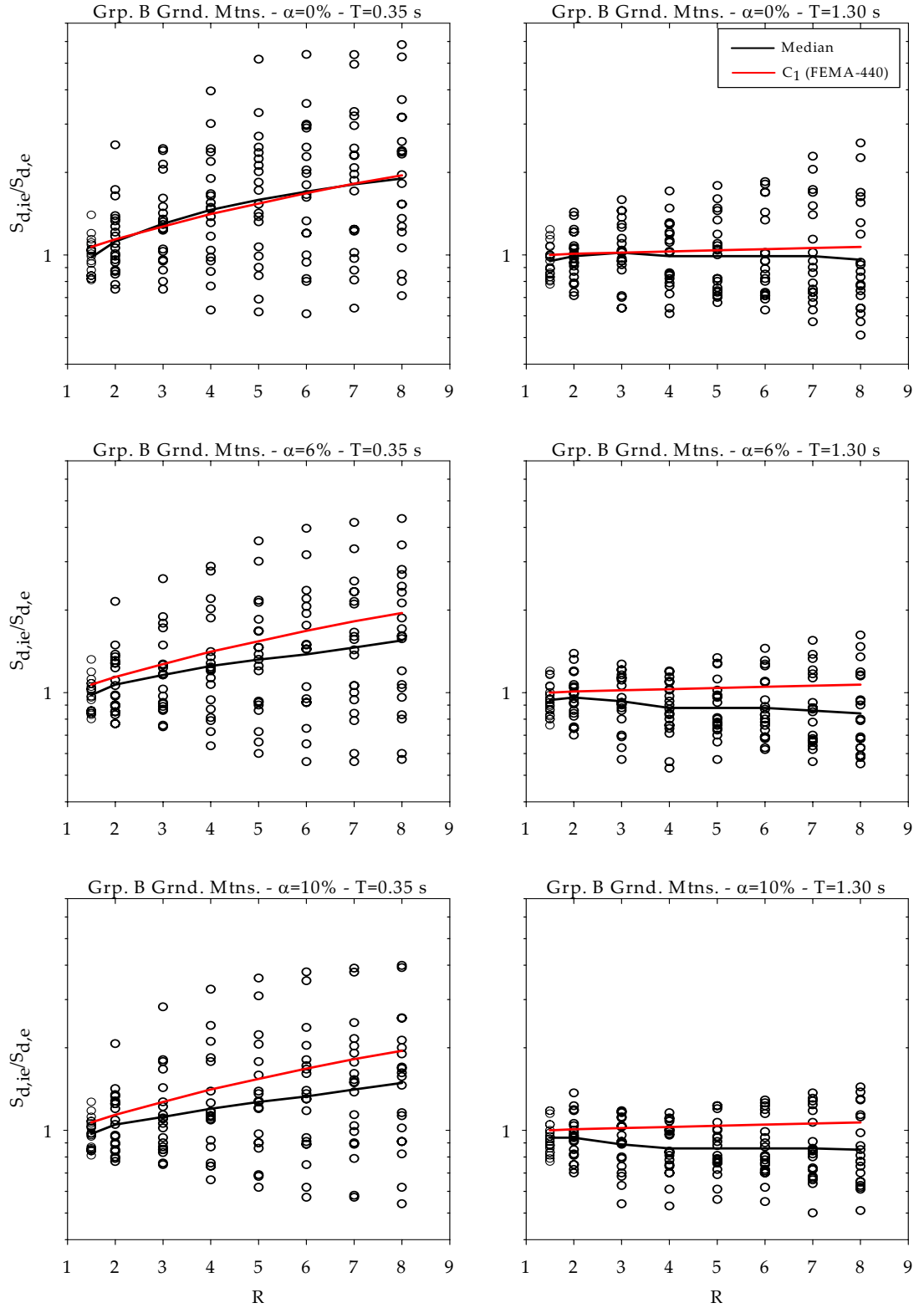


Figure 4.3 Sensitivity of $S_{d,ie}/S_{d,e}$ to α for SDOF systems subjected to Group B ground motions and comparisons with the improved C_1

The left panels in Figures 4.2 and 4.3 display the results for $T = 0.35$ s and the right panels display the results for $T = 1.30$ s. The first rows in each figure present the $S_{d,ie} / S_{d,e}$ ratios pertaining to $\alpha = 0\%$, whereas the second and the third rows correspond to $\alpha = 6\%$ and $\alpha = 10\%$ post-yield stiffness ratios, respectively.

The following observations are listed from Figures 4.2 and 4.3:

1. For ground-motion bin A, whose results are presented in Figure 4.2, C_1 underestimates the inelastic spectral displacements for elastoplastic behavior. However, as post-yield stiffness ratio takes values different than zero, C_1 slightly overestimates the inelastic spectral displacements for both lower and upper period values. This is because inelastic deformation demands decrease as α increases.
2. The plots in Figure 4.2 also reveal that median $S_{d,ie} / S_{d,e}$ ratios do not show significant variations for $\alpha = 6\%$ and $\alpha = 10\%$. This observation is parallel to the ones made by Ruiz-García and Miranda (2003) stating that the actual $S_{d,ie} / S_{d,e}$ variation is sensitive to the changes for $\alpha \leq 3$.
3. The plots in Figure 4.3 indicate that the improved C_1 describes the median variation of $S_{d,ie} / S_{d,e}$ fairly well for elastoplastic behavior whereas there is a significant deviation between the tendency of actual $S_{d,ie} / S_{d,e}$ and C_1 for non-zero post-yield stiffness ratios (i.e., $\alpha = 6\%$ and $\alpha = 10\%$). The discrepancy increases with increasing R . On average, the improved C_1 tends to overestimate the inelastic spectral displacement predictions when compared to the actual behavior as post-yield stiffness increases. The increase in α decreases the deformation demands whereas the improved C_1 that is insensitive to the variations in α does not consider this change.

Figures 4.4 and 4.5 display similar comparisons as of Figures 4.2 and 4.3 for Group C and D ground motions (records with dominant pulse signals). The left panels in these figures present the variations for $T = 0.40$ s whereas the right panels present the sensitivity of $S_{d,ie} / S_{d,e}$ for the upper period value $T = 1.25$ s. The plots for $\alpha = 0\%$, $\alpha = 6\%$ and $\alpha = 10\%$ are presented in the first, second and the third rows, respectively.

The following observations are highlighted from these figures:

1. The improved C_1 underestimates the median $S_{d,ie} / S_{d,e}$ variations for all post-yield stiffness ratios for Groups C and D ground motions that are dominated by pulse signals. Therefore, the improved C_1 modifying factor is more likely to underestimate the inelastic SDOF deformation demands for pulse-dominant records regardless of the variation in α .
2. The difference between the estimated C_1 values of FEMA-440 document (ATC, 2005) and the actual variation of $S_{d,ie} / S_{d,e}$ decreases as post-yield stiffness ratio, α , increases. As explained in the previous comments, the increase in α reduces inelastic deformation demands with respect to elastic deformation demands, thus resulting in smaller $S_{d,ie} / S_{d,e}$ ratios. The median variations of $S_{d,ie} / S_{d,e}$ do not change significantly for $\alpha = 6\%$ and $\alpha = 10\%$ that is similar to the one observed for ground-motion groups A and B that do not contain pulse-dominant signals.

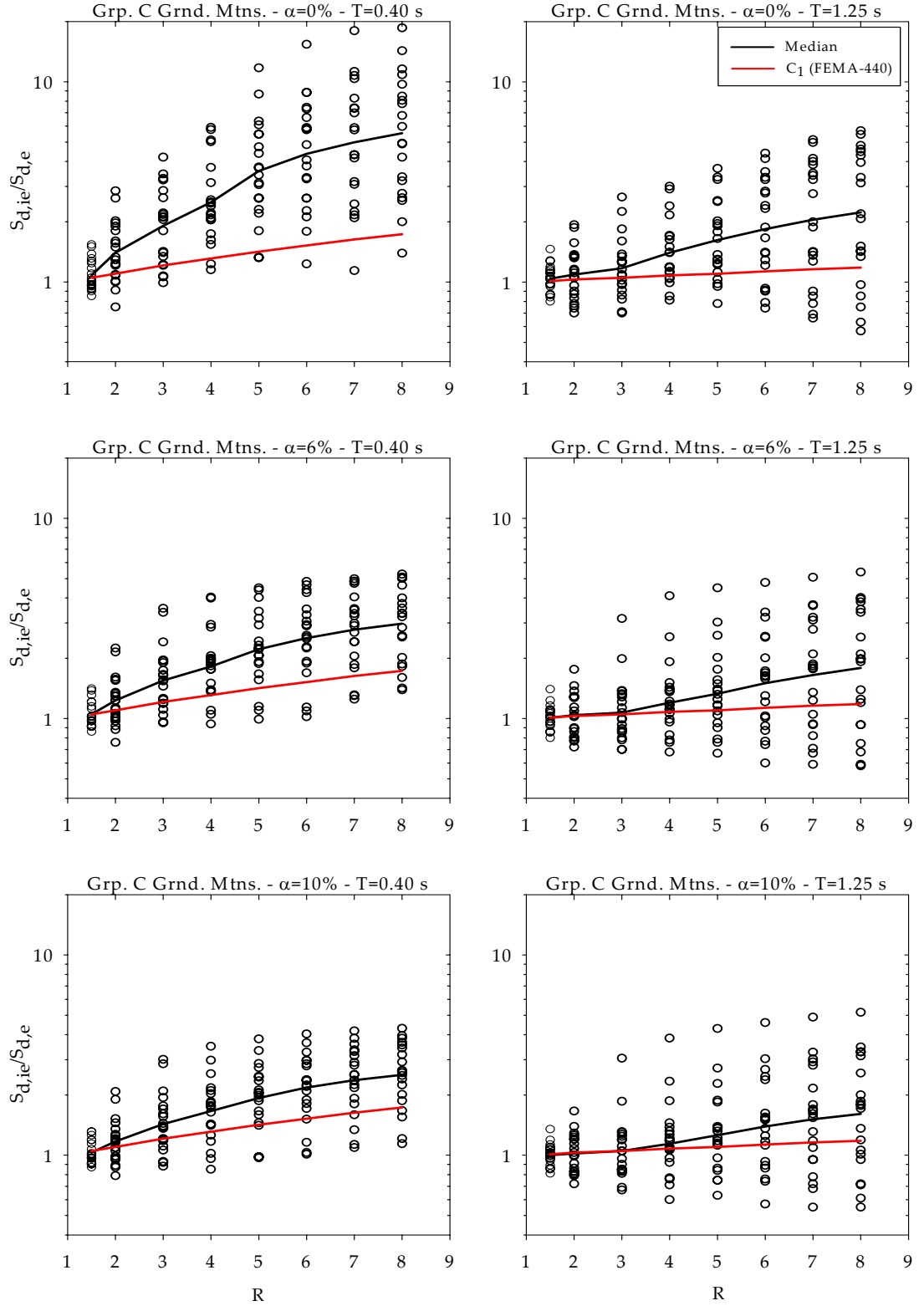


Figure 4.4 Sensitivity of $S_{d,ie}/S_{d,e}$ to α for SDOF systems subjected to Group C ground motions and comparisons with the improved C_1

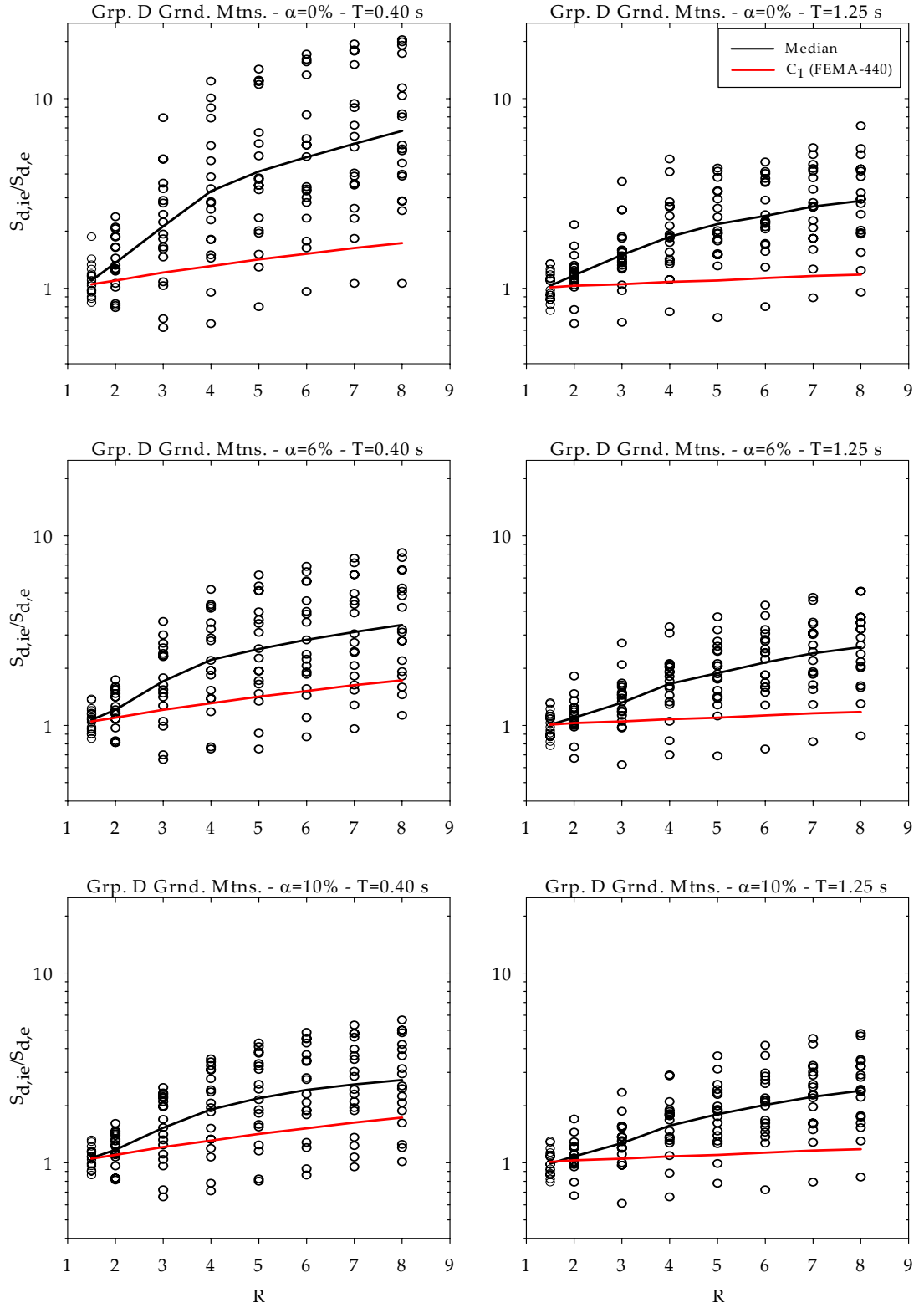


Figure 4.5 Sensitivity of $S_{d,ie}/S_{d,e}$ to α for SDOF systems subjected to Group D ground motions and comparisons with the improved C_1

4.4.3 Improved Capacity Spectrum Method

The improved CSM determines the equivalent linear parameters (i.e., effective period, T_{eff} , and effective damping, β_{eff}) through a statistical analysis that minimizes the extreme differences between the maximum response of an actual inelastic SDOF system and its equivalent linear counterpart (Guyader and Iwan, 2006). The improved method devises effective damping and period expressions for a wide range of post-yield stiffness ratios and hysteretic models (i.e., bilinear, stiffness degrading and in-cycle strength degrading). Inherently these expressions are functions of ductility, μ . In the improved CSM both T_{eff} and β_{eff} equations are discontinuous at two distinct ductility values (i.e., $\mu = 4$ and $\mu = 6.5$). The general format of the effective damping and effective period expressions are given in Equations (4.7).

For $\mu < 4.0$:

$$\beta_{eff} = A(\mu-1)^2 + B(\mu-1)^3 + \beta_0 \quad ; \quad T_{eff} = [G(\mu-1)^2 + H(\mu-1)^3 + 1]T_0 \quad (4.7.a)$$

For $4.0 \leq \mu < 6.5$:

$$\beta_{eff} = C + D(\mu-1) + \beta_0 \quad ; \quad T_{eff} = [I + J(\mu-1) + 1]T_0 \quad (4.7.b)$$

For $\mu \geq 6.5$:

$$\beta_{eff} = E \left[\frac{F(\mu-1)-1}{F(\mu-1)^2} \right] \left(\frac{T_{eff}}{T_0} \right)^2 + \beta_0 \quad ; \quad T_{eff} = \left\{ K \left[\sqrt{\frac{(\mu-1)}{1+L(\mu-2)}} - 1 \right] + 1 \right\} T_0 \quad (4.7.c)$$

where T_0 and β_0 are the initial period and viscous damping of the actual system. The variables A through K in the above equations are regression constants and they vary for different α – values and hysteretic models as listed in Tables 6.1 and 6.2 of the FEMA-440 document (ATC, 2005). The improved CSM also presents

alternative values for these constants that can be used for any case regardless of the post-yield stiffness ratio and hysteretic model.

Both effective damping and period expressions are functions of ductility that cannot be determined directly from the idealized capacity curves of the equivalent linear systems that are determined from pushover analyses. Thus, iterative or graphical procedures are required for the estimation of nonlinear SDOF deformation demand through its equivalent linear counterpart. The effective period expressions in the improved CSM do not follow the secant period approach that is the case in the former CSM. This fact somehow complicates the graphical solution and this study preferred direct iteration (i.e., Procedure A) technique to predict the maximum nonlinear equivalent SDOF displacements by the improved CSM. The procedure starts with an initial assumption for the displacement ductility ($\mu_{initial}$), computes the equivalent linear parameters using Equations (4.7) and estimates the maximum nonlinear SDOF displacement from the equivalent linear system. This estimation is used to compute the new displacement ductility ($\mu_{computed}$). The convergence between the initial and computed ductilities are then checked using a tolerance level

criterion that is compared with $\frac{|\mu_{computed} - \mu_{initial}|}{\mu_{computed}}$ for finalizing the computations. If

this ratio is less than the given tolerance level, the computations are terminated. Otherwise, the initial ductility is set to the value of $\mu_{computed}$ and the iterations continue until $\mu_{initial}$ and $\mu_{computed}$ converge to each other satisfying the pre-determined tolerance level. The detailed steps about this iterative scheme are described in the ATC-40 (ATC, 1996) and FEMA-440 (ATC, 2005) documents. The iterative schemes are numerical methods, and they aim to approach the root(s) of a function. The convergence between the consecutive assumption warrants the satisfactory approximation to the root that is dictated by the tolerance level. This particular iterative scheme is called “fixed point iteration” (Akai, 1994) and it may run into divergence problems or flip-flop behavior (Chopra and Goel, 2000b;

Akka and Miranda, 2006) that can be treated by some basic numerical analysis knowledge. Two distinct cases for the numerical problems are examined in the next section using the models presented in this study.

4.4.4 Examination of the flip-flop behavior of the improved CSM

A tolerance level of 5% was chosen in this study during the iterative scheme for estimating the nonlinear SDOF deformation demands by using the improved CSM. The flip-flop behavior (also called as non-convergent behavior) was observed in many cases. Two typical flip-flop cases experienced during the implementation of the improved CSM are presented in Figure 4.6. This plot shows the variation of initial and computed ductility using the improved CSM equations and it is a typical representation of the “fixed point iteration scheme” used in this study. The initial ductility assumption of $\mu_{initial}=2$ for the case displayed in the left panel diverges towards the ductility of 3 and then the flip-flop behavior occurs that results in an endless loop between two ductility points. This ill-case can be cured by applying the “bisection method” that would result a quick convergence to the root (i.e., peak SDOF displacement of the equivalent linear system). The second example on the right panel is another flip-flop case due to the discontinuity in Equations (4.7) at $\mu = 4$ that appears as an abrupt change in the functional form in the vicinity of $\mu = 4$. This case can also be handled by applying the bisection method. It should be noted that the bisection method was used to converge the inelastic displacement estimates for all flip-flop cases in this study.

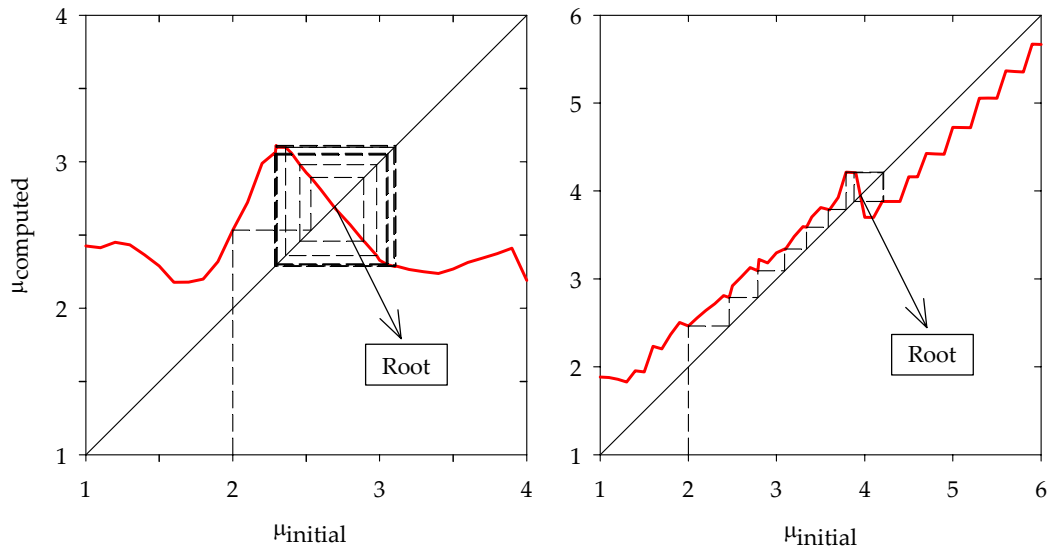


Figure 4.6 Flip-flop type numerical problems encountered in the improved CSM

4.5 Comparisons of improved NSPs proposed in FEMA-440 document (ATC, 2005)

4.5.1 Comparisons by other studies

A limited comparative study providing a preliminary evaluation for these methods based on the results from SDOF nonlinear RHA is presented in the Chapter 7 of the FEMA-440 document (ATC, 2005). The comparative results are confined to nine distinct bilinear oscillators with periods of 0.2 s, 0.5 s, and 1.0 s that are subjected to 13 strong ground motions. The results indicated that the mean SDOF deformation demand estimations of the improved DCM and CSM follow a fairly close pattern to the mean deformation demands computed from nonlinear RHA. These results also showed that the discrepancy between the methods is reduced with respect to the corresponding former methods.

A recent study by Kazaz et al. (2006) investigated the accuracy of the improved CSM in the FEMA-440 document (ATC, 2005) for an equivalent SDOF oscillator of a significantly stiff, high-lateral strength, wall-type structure subjected to 55 near-fault and far-fault ground motions recorded on rock sites. The mean error statistics depicted by Kazaz et al. (2006) reported an average of 20% unsafe spectral displacement predictions with respect to the nonlinear response history analysis results. Using a comprehensive ground-motion data set and wide-range of spectral periods, Akkar and Miranda (2005) also conducted a statistical study on the accuracy of the methods proposed by Guyader and Iwan (2006) and Ruiz-García and Miranda (2003). Note that these two studies form the basis of the improved CSM and DCM in the FEMA-440 document (ATC, 2005), respectively. Akkar and Miranda (2005) indicated that for elastoplastic behavior, on average, both methods would predict substantially different spectral displacements when vibration periods are approximately less than 0.6 s. The discrepancy between the predicted spectral displacements becomes more apparent for decreasing lateral strength capacities having a tendency towards a significant overestimation for the equivalent linear method proposed by Guyader and Iwan (2006). This observation is contradictory to the remarks made by Kazaz et al. (2006) as they reported unsafe spectral displacement predictions by the improved CSM for the particular stiff system they considered. The reason behind such conflicting results may be the differences in scenarios employed during the evaluation of NSPs. While Kazaz et al. (2006) established their results using the equivalent SDOF system of a high-strength capacity, very stiff, wall-type structure, Akkar and Miranda (2005) employed generic elastoplastic oscillator systems to compute the relevant statistics as a function of lateral strength and vibration period.

4.5.2 Comparisons by using the results of this study

Inelastic spectral displacements of the equivalent SDOF systems, maximum roof and interstory drift ratios computed from the nonlinear RHA were used to verify the performance of improved NSPs in the FEMA-440 document (ATC, 2005). Deformation demands computed from the nonlinear RHA were assumed to be exact for evaluation purposes and compared to the predicted deformation demands of the improved CSM and DCM. The nonlinear dynamic analyses of equivalent SDOF systems that were conducted for each equivalent SDOF system and ground-motion pair as described in detail in Chapter 3 were used in the verifications. The RHA were based on the corresponding R – values and the computation for a given equivalent SDOF system and particular ground motion is straightforward and it is described in the ATC-40 (ATC, 1996), FEMA-356 (ASCE, 2000) or FEMA-440 (ATC, 2005) documents. This is also described in Chapter 3 as well as the corresponding log-normal R – variations (Figures B.1 to B.4 and Figure 3.21). For the sake of completeness, the log-normal median variation of R with respect to the vibration periods of the equivalent SDOF systems (i.e., $T_{1,idealized}$) are highlighted once again in Figure 4.7. This information can be extracted from the relevant figures presented in Chapter 3. The median scatters are associated with the error bars that represent the 16- and 84-percentile values and display the dispersion about the median. The statistics pertaining to the building models that are subjected to pulse-dominant ground motions (i.e., Groups C and D) are separated from the rest by drawing an additional circle around the particular symbol chosen for each number of stories.

It is observed from Figure 4.7 that the median R – values for the equivalent SDOF systems are clustered between 1.5 and 3.5 with the maxima reaching to approximately 4.5 to 5.5 when dispersions about the median values are considered. These statistics indicate that the model buildings have relatively large lateral strength capacities.

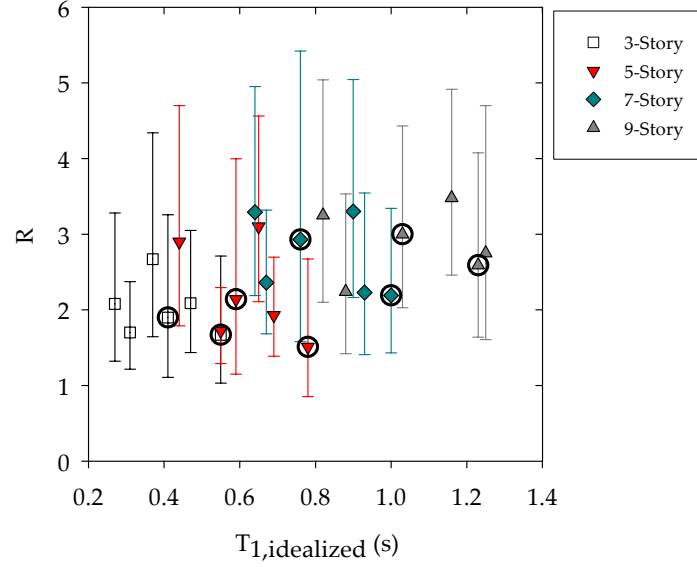


Figure 4.7 Log-normal median values of normalized lateral strength, R , associated with the 16- and 84-percentile error bars

The normalized lateral strengths computed for each particular case are directly implemented to the improved C_1 factor (i.e., Equation (4.3)) for the estimation of maximum nonlinear equivalent SDOF displacements (i.e., $(S_{d,ie})_{approx.}$) by modifying the maximum elastic displacement (i.e., $S_{d,e}$) of the same equivalent SDOF system. As stated previously, the constant a in the improved C_1 was taken as 60 because almost all ground motions are recordings from stiff soil site classes (i.e., NEHRP-D according to FEMA-450 (BSSC, 2003)). For the improved CSM, the iterative procedure described in the previous sections was employed to compute the $(S_{d,ie})_{approx.}$ for each particular case. The expressions presented in Equations (4.7) were implemented using the hysteretic model and post-yield stiffness independent values for constants A through K . A tolerance level of 5% was used in this study as the convergence criterion for estimating the nonlinear SDOF demands by the improved CSM. In non-convergent cases, the bisection method was used to handle

the numerical instability. The maximum roof displacements were estimated by modifying the approximate maximum SDOF displacements with $\Gamma_1 \phi_1^{roof}$ that is the common suggestion for both NSPs. The maximum interstory drift estimations were computed by multiplying the approximate maximum equivalent SDOF displacements with the product of the maximum of the difference of elastic fundamental mode vectors between two consecutive stories and the participation factor, Γ_1 . The maximum roof displacement estimates from the improved NSPs were then normalized by the total building height of each model to obtain the *MRDR* estimations. Similarly, the maximum interstory drift predictions were normalized by the story height for computing the estimated *MIDR*.

The error statistics were derived by computing the ratios of predicted response quantities from the NSPs to the corresponding exact values determined from the nonlinear RHA. The results of approximate NSPs are biased in the sense that the median ratios computed from these error terms differ from one (i.e., the NSPs underestimate the median response quantity if the median ratio is less than one and provide an overestimation if this ratio exceeds one). Given the building model and corresponding ground-motion bin, the median error and the pertaining dispersion (i.e., standard deviation) were computed assuming log-normal distribution. The variations of median error scatters with fundamental periods and idealized fundamental periods are presented in Figure 4.8 for the improved DCM (left panel) and improved CSM (right panel). The first row presents the median errors for the maximum displacement estimations of equivalent SDOF systems. The second and third rows display the period dependent errors for *MRDR* and *MIDR* estimations, respectively. The plots also display the error bars that represent the 16- and 84-percentile values for the error statistics. Similar to Figure 4.7, the statistics of pulse-dominant ground motions are distinguished from the rest by an additional circle around the symbols that represent different number of stories.

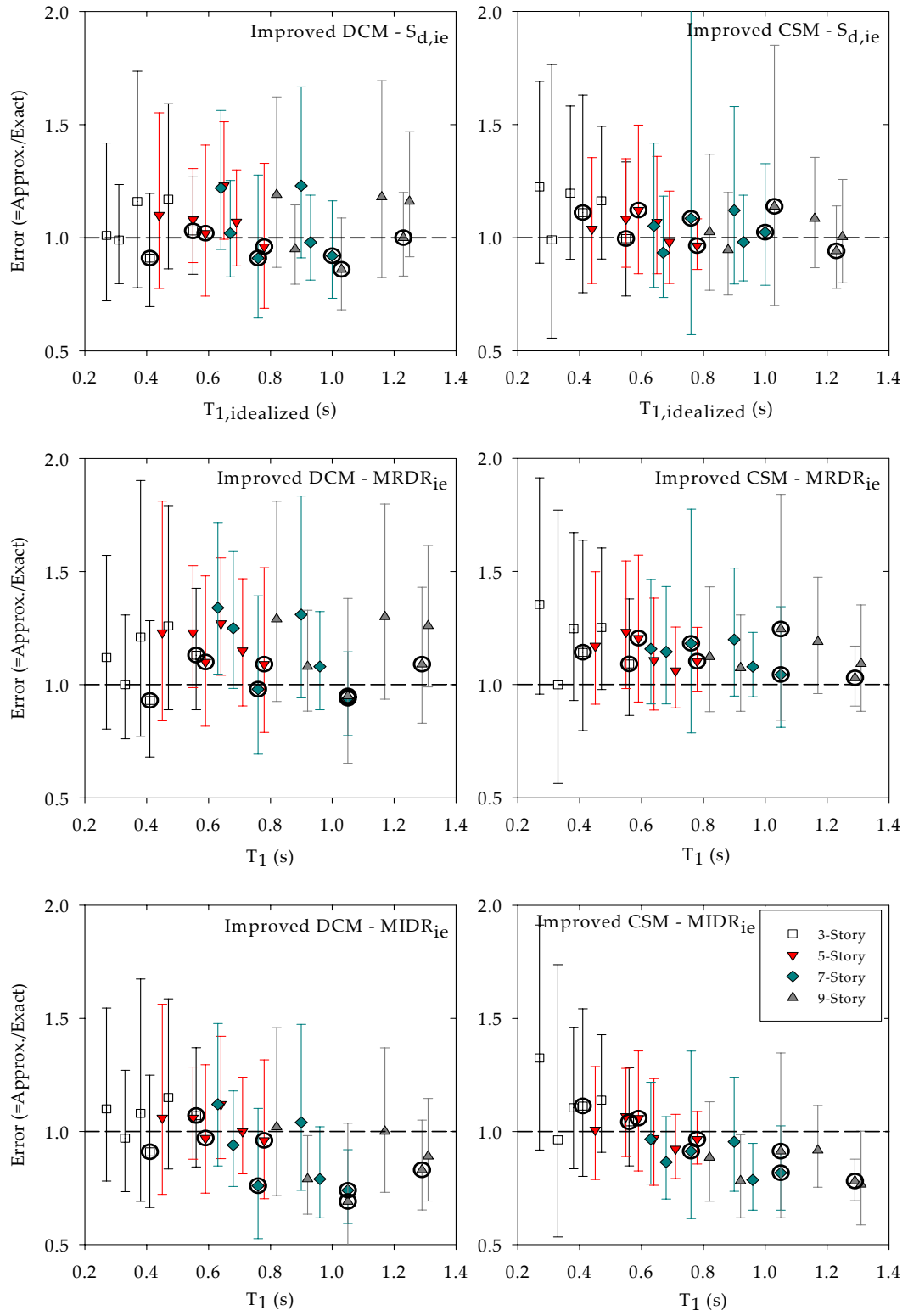


Figure 4.8 Log-normal median error statistics and the error bars showing 16- and 84-percentiles

The following observations are made from the examination of scatter plots presented in Figure 4.8.

1. The comparative median plots in Figure 4.8 indicate that the improved DCM would result more conservative estimations with respect to the improved CSM for the maximum displacements of equivalent SDOF systems. The estimations in the maximum SDOF displacements tend to unsafe side in the improved DCM for records with pulse particularly towards longer periods. The underestimation of deformation demands for pulse-dominant signals presented in Figures 4.4 and 4.5 also confirmed these unsafe predictions by the improved DCM. The dispersions about the median errors are considerable for the improved NSPs implying a noteworthy uncertainty that cannot be disregarded in these methods.
2. Both procedures overestimate the *MRDR* particularly for records without pulse. In almost all cases, the *MRDR* estimations are 10% larger with respect to those of the maximum equivalent SDOF system displacements. The conservative empirical MDOF-SDOF relationships implemented by these procedures (discussed in Chapter 3) and excessively safe equivalent SDOF displacement predictions are the major contributors to such *MRDR* predictions.
3. The improved NSPs tend to underestimate the *MIDR* particularly for increasing periods. The coefficient method draws a more erratic pattern in the median *MIDR* estimations with respect to the improved CSM. The improved CSM consistently tends towards unsafe *MIDR* predictions for increasing periods whereas the improved DCM predictions for median *MIDR* are mostly unsafe for 7- and 9-story building models particularly for pulse signal dominant records.

4. The close resemblance between the R – distributions displayed in Figure 4.7 and the equivalent maximum SDOF displacement and $MRDR$ error statistics for the improved DCM is noteworthy. The increase in the median R generally results in an increase in the median error for these demand parameters implying the sensitivity of improved DCM performance to the normalized lateral strength. This is not observed for the improved CSM whose predictions are based on the individual ground motions using μ – dependent empirical functions for estimating the equivalent linear system. On the other hand, improved DCM employs generic R – dependent empirical modifying coefficients representing the central tendency of a suite of ground motions. Figure 4.9 clarifies the independence of CSM with respect to the actual variations in R – values. This figure compares the actual variation of R with the actual μ – values obtained from the RHA (left panel) and the approximate μ computed at the end of maximum displacement estimated from the improved CSM (right panel). As noted from this figure the variation of approximate μ departs from the actual μ – R scatters particularly for large μ . Moreover, the $\mu_{computed}$ with respect to actual R reveals a more dispersive pattern when compared to the actual variation computed from nonlinear RHA. Particularly, the computed μ – values that are above 10 are unrealistically high such that a real structure can never sustain such large deformation demands.

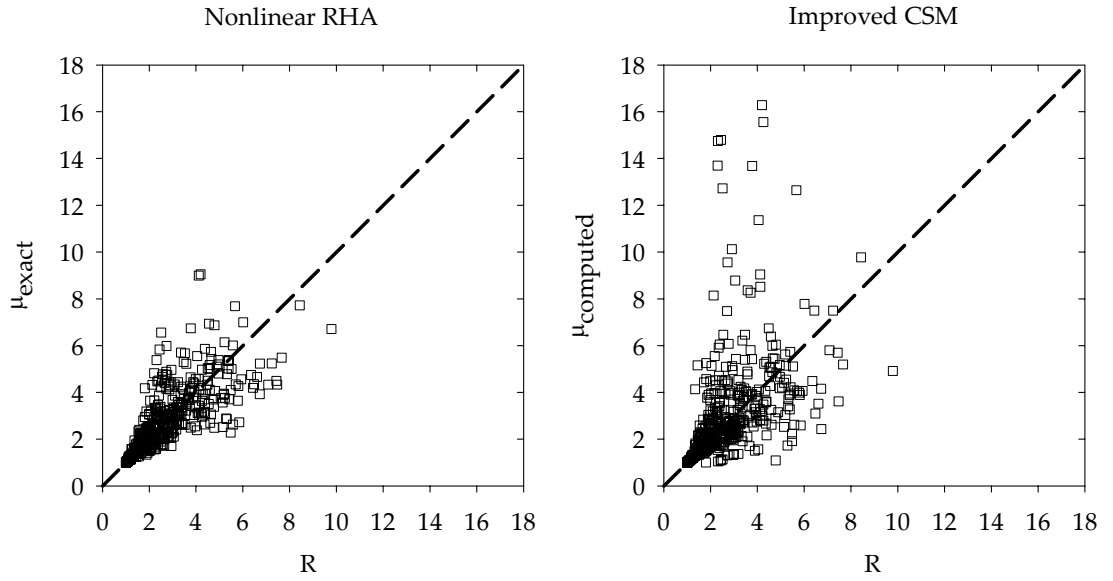


Figure 4.9 Comparison of actual variation of R with the actual and approximate μ – values

Figures 4.10 to 4.15 present the error statistics (i.e., approximate deformation demand normalized by the corresponding exact deformations computed from nonlinear RHA) with respect to the number of stories as a function of normalized lateral strength, R . Figures 4.10 and 4.11 present the scatter plots of $S_{d,ie}$ for improved DCM and improved CSM, respectively. In a similar fashion, Figures 4.12 and 4.13 present the scatter plots of $MRDR_{ie}$ and Figures 4.14 and 4.15 present the scatter plots of $MIDR_{ie}$ for improved DCM and improved CSM, respectively.

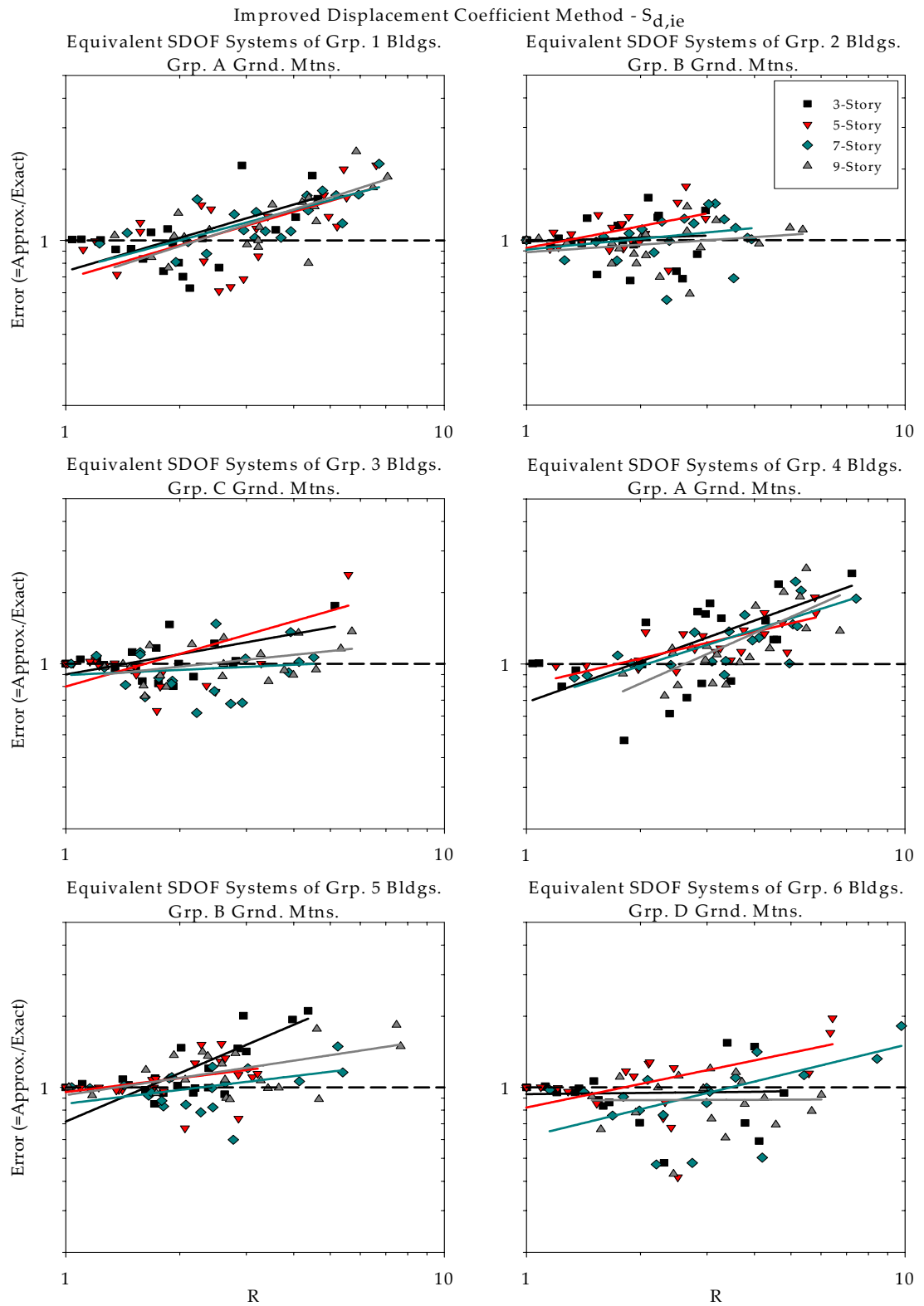


Figure 4.10 Error versus R relationships of $S_{d,ie}$ for improved DCM

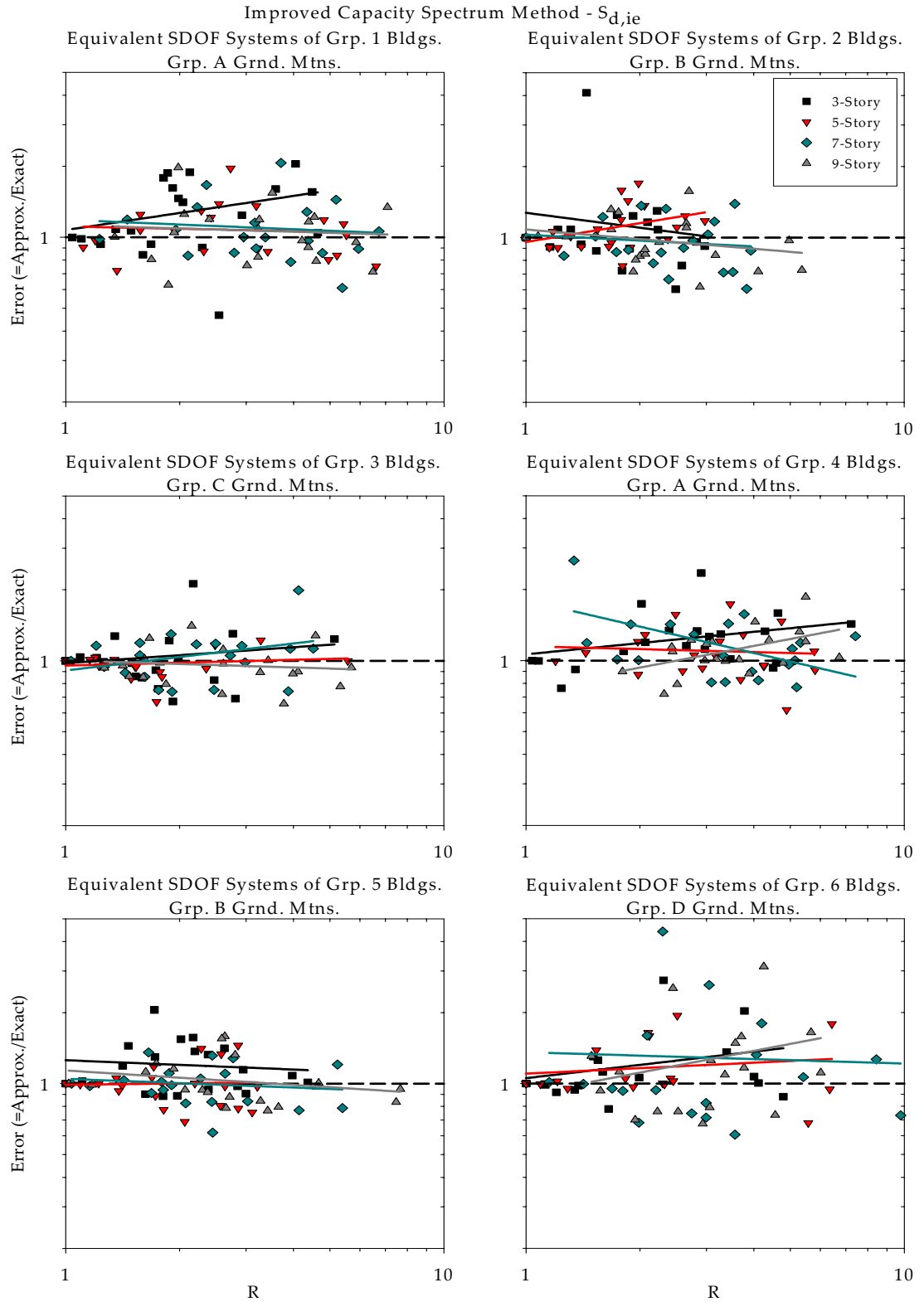


Figure 4.11 Error versus R relationships for $S_{d,ie}$ for improved CSM

Improved Displacement Coefficient Method - $MRDR_{ie}$

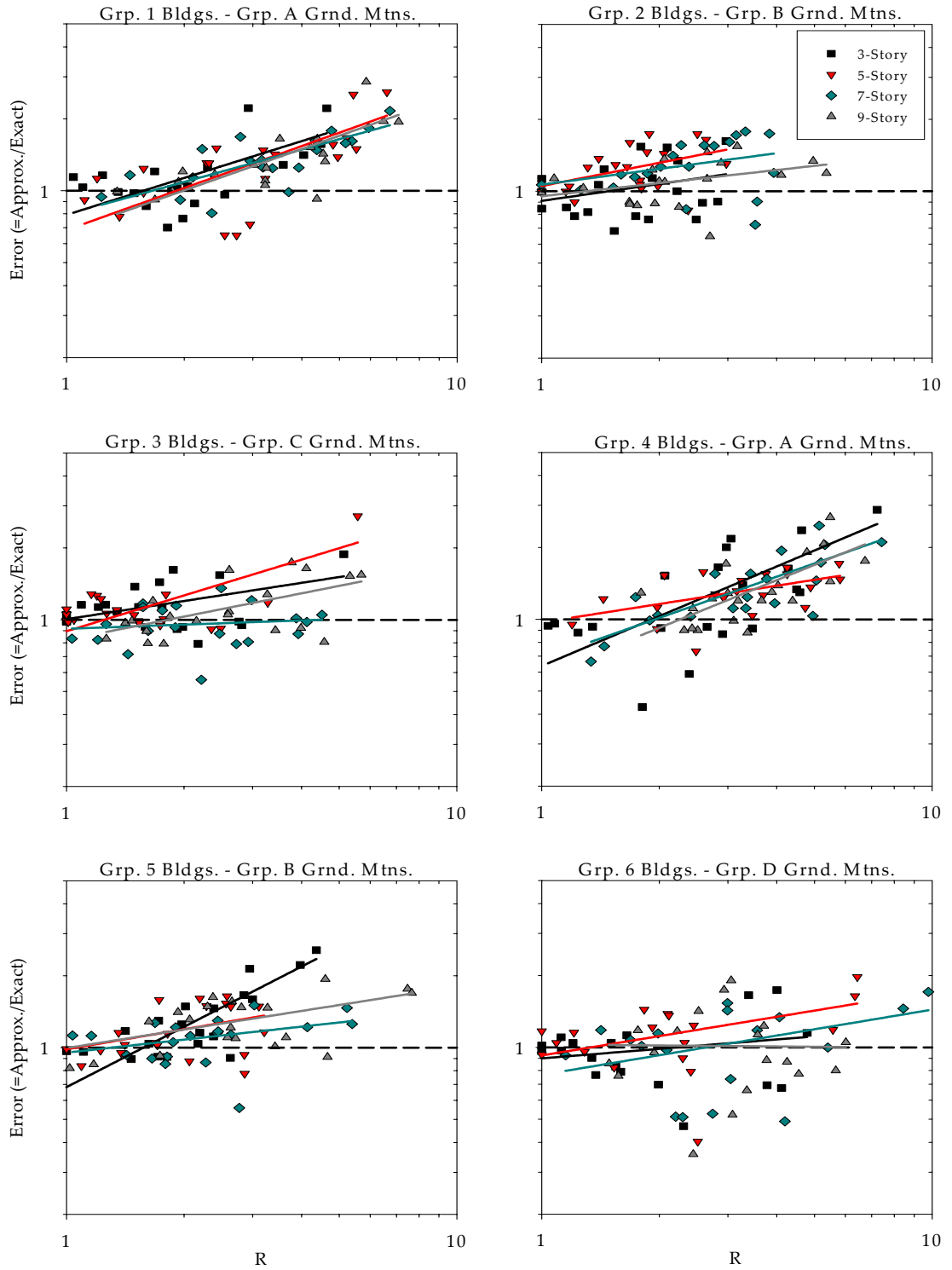


Figure 4.12 Error versus R relationships of $MRDR_{ie}$ for improved DCM

Improved Capacity Spectrum Method - $MRDR_{ie}$

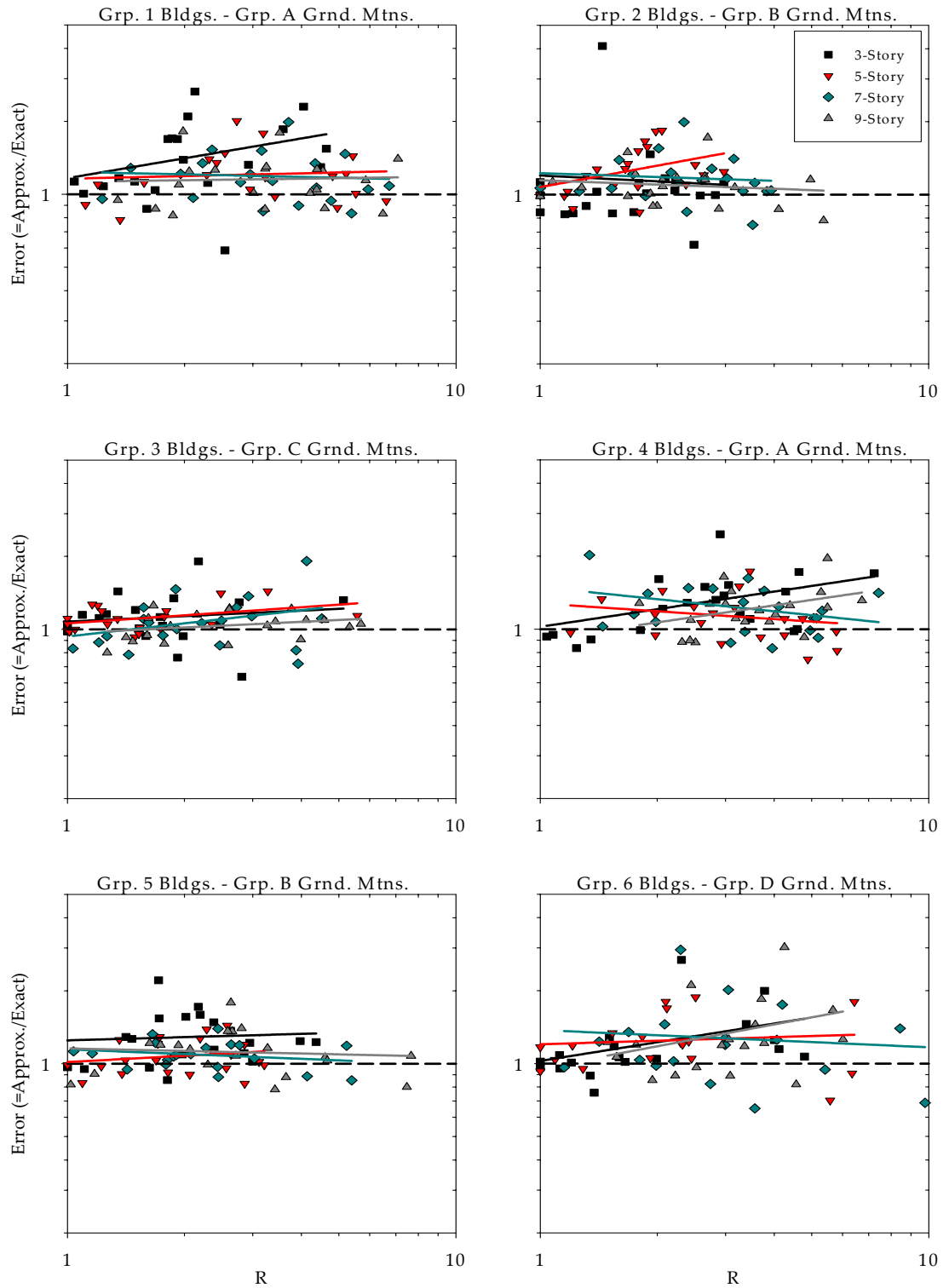


Figure 4.13 Error versus R relationships of $MRDR_{ie}$ for improved CSM

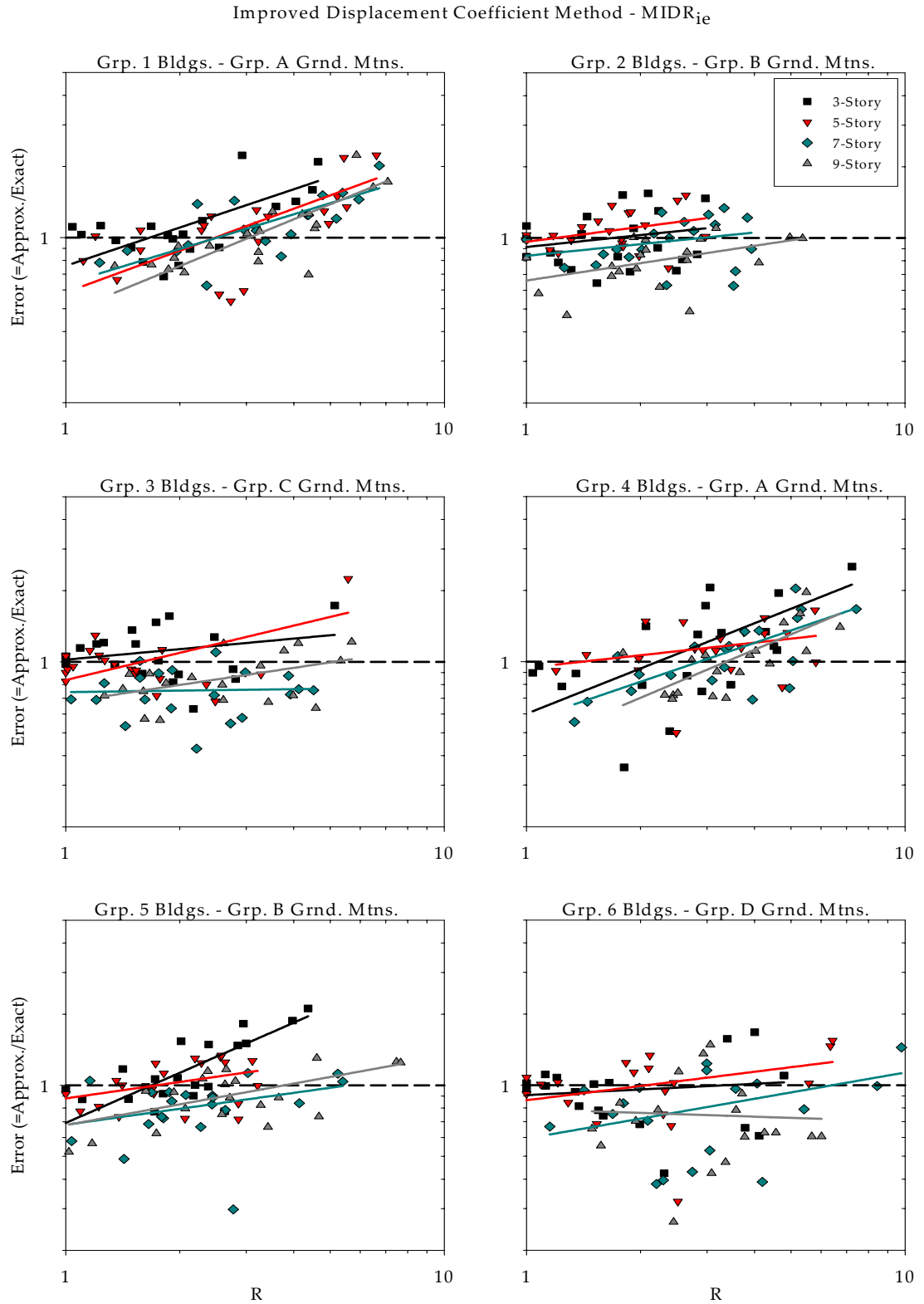


Figure 4.14 Error versus R relationships for $MIDR_{ie}$ for improved DCM

Improved Capacity Spectrum Method - $MIDR_{ie}$

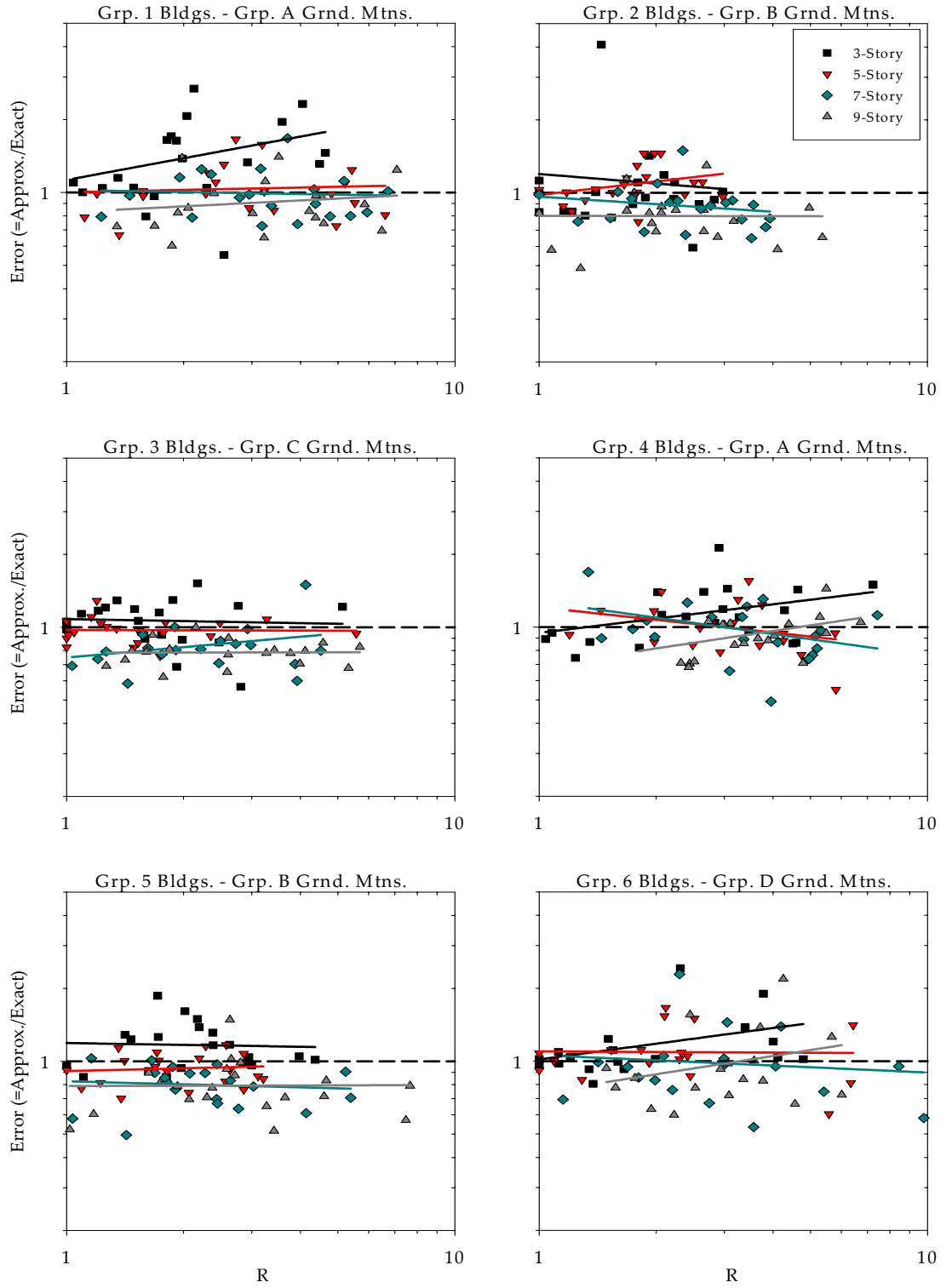


Figure 4.15 Error versus R relationships of $MIDR_{ie}$ for improved CSM

The following observations are made after the examination of these figures:

1. In general, the error by improved DCM, regardless of the number of stories, tends towards safe side with increasing R for maximum equivalent SDOF displacements. This is mostly valid for the ground-motion bins without pulse. For records with pulse the SDOF error trends are not as clear as the ones presented for records without pulse. The variation of error as a function of R does not show a particular trend for the improved CSM. This can be expected as this method is not devised particularly for R . Moreover, CSM estimates are done for each particular case that may also be another reason for such a dispersive behavior. It should be noted that the SDOF peak displacement estimations of DCM are more conservative than the corresponding CSM estimations that is also discussed in the previous figures.
2. The error trends for inelastic $MRDR$ estimations in the improved DCM also increase with increasing R similar to the error trends presented for maximum equivalent SDOF system displacements. Similar to item (1), the increase in error with increase in R is not observed for CSM. Both methods estimate $MRDR$ more conservatively than the maximum equivalent SDOF displacements. As a matter of fact, these statements are quite predictable from the observations in item (1) as both methods modify the SDOF estimations with the fundamental-mode properties of the building systems that is also discussed in the previous sections of this study.
3. The inelastic $MIDR$ estimations tend to unsafe side for CSM particularly when the number of stories increase. This is compatible with the median errors presented in Figure 4.8. The improved DCM generally tends towards safe inelastic $MIDR$ estimations particularly for records without pulse. The overestimations are slightly less with respect to the inelastic S_d and $MRDR$

estimations. The errors for the improved DCM indicate unsafe inelastic $MIDR$ estimations for records with pulse that is also similar to the median errors presented in Figure 4.8.

4.6 Error distribution functions of $S_{d,ie}$, $MRDR_{ie}$ and $MIDR_{ie}$ conditional to R

The lateral strength dependent error statistics for DCM that are discussed in Figures 4.10 to 4.15 encouraged this study to compute error distribution functions for $S_{d,ie}$, $MRDR_{ie}$ and $MIDR_{ie}$ conditional to R . The theoretical background for this second-order probabilistic approach is given in Giovenale et al. (2004).

The linear function between $\ln(Error)$ and $\ln(R)$ is used to represent the median error conditional to the normalized lateral strength, R , assuming log-normal distribution for both error and normalized lateral strength variables.

$$E[\ln(Error)|R] = \ln(a_1) + b_1 \ln(R) \quad (4.8)$$

where a_1 and b_1 are regression constants. The dispersion that is represented by the standard deviation of error conditional to R is:

$$\sigma_{\ln(Error)|R} = \sqrt{\frac{\sum (\ln(Error) - \ln(a_1 R^{b_1}))^2}{n - 2}} \quad (4.9)$$

where n represents the total number of ground motions for the particular ground-motion bin. In this study $n = 20$ except for Group D ground-motion bin that contains 18 records. Assuming $Error|R$ is log-normally distributed one can write the probability density function $f_{Error|R}$ as:

$$f_{Error|R} = \frac{1}{\sqrt{2\pi}Error.\sigma_{\ln(Error)|R}} \exp \left[-\frac{1}{2} \left(\frac{\ln(Error) - \ln(a_1 R^{b_1})}{\sigma_{\ln(Error)|R}} \right)^2 \right] \quad (4.10)$$

Equation (4.10) is used to derive error terms for a given probability of non-exceedance level conditional to R . For this purpose, the cumulative probability distributions (CDF) of errors for selected equivalent SDOF systems and building models used in this study are presented in Figures C.1 to C.4. For illustrative purposes, Figures C.1 and C.2 show the cumulative distributions for maximum equivalent SDOF systems corresponding to 3- and 9-story model buildings. Similarly, in Figures C.3 to C.4, the cumulative distributions for $MIDR$ of 3- and 9-story model buildings are presented for improved DCM.

The cumulative error distributions conditional to R are used to present the error statistics for different levels of non-exceedance probabilities and for different R – values. This way a more general description of the uncertainties associated in the improved DCM is made using the available error data presented in this study. The error result shown in Figures 4.16 to 4.18 are median error ratios (i.e., 50% probability of non-exceedance) indicating the central tendency of errors for inelastic S_d , $MRDR$ and $MIDR$, respectively. The conditional error statistics are given for $R = 2$ and $R = 4$ for building groups subjected to records without pulse (Building Groups 1, 2, 4 and 5) and with pulse (Building Groups 3 and 6). For the sake of completeness 10% and 90% probability of non-exceedance for the conditional error statistics are computed. These are given in Figures C.5 to C.10 for inelastic S_d , $MRDR$ and $MIDR$. The 10% probability of non-exceedance indicates that the error ratios presented have a 90% probability of exceedance (constitute the lower-bound error statistics conditional to different R – levels). The error statistics with 90% probability of non-exceedance constitute the upper-bound error statistics and indicate that the chances of exceeding the presented error ratios is only 10% for a

given level of R . It should be noted that the conditional error statistics presented are of concern of the improved DCM and they are confined to the ground motions selected and the building groups described in this study.

Regardless of the deformation demand parameter, the error plots indicate that the increase in R results in safer estimations. When median statistics are considered, the improved DCM tends to overestimate the deformation demands on short buildings significantly (3- and 5-story models in this study). The overestimations for $S_{d,ie}$ and $MRDR_{ie}$ are approximately 50% above the exact results computed from nonlinear RHA when $R = 4$. This observation is valid for both with and without pulse records. The level of median overestimation relatively decreases for $MIDR_{ie}$. The increase in number of stories results in a decrease in the level of overestimations (7- and 9-story building models). For records without pulse, the predictions for $S_{d,ie}$ and $MRDR_{ie}$ are still conservative but the estimations for records with pulse tend to unsafe side particularly for $MIDR_{ie}$. This is also the case for building models that are subjected to no-pulse records. In other words, $MIDR_{ie}$ estimations by the improved DCM tend to unsafe side regardless of the ground-motion type. When the upper limit of error statistics (i.e., 90% probability of non-exceedance) are considered that are presented in Figures C.8 to C.10, the level of overestimations increase significantly indicating how far the estimations might be off from the actual results.

The above observations indicate once again the importance of considering other important structural and ground-motion features for the estimation of inelastic deformation demands. A probabilistic approach for estimating $MIDR_{ie}$ that partially addresses this issue is presented in the next chapter.

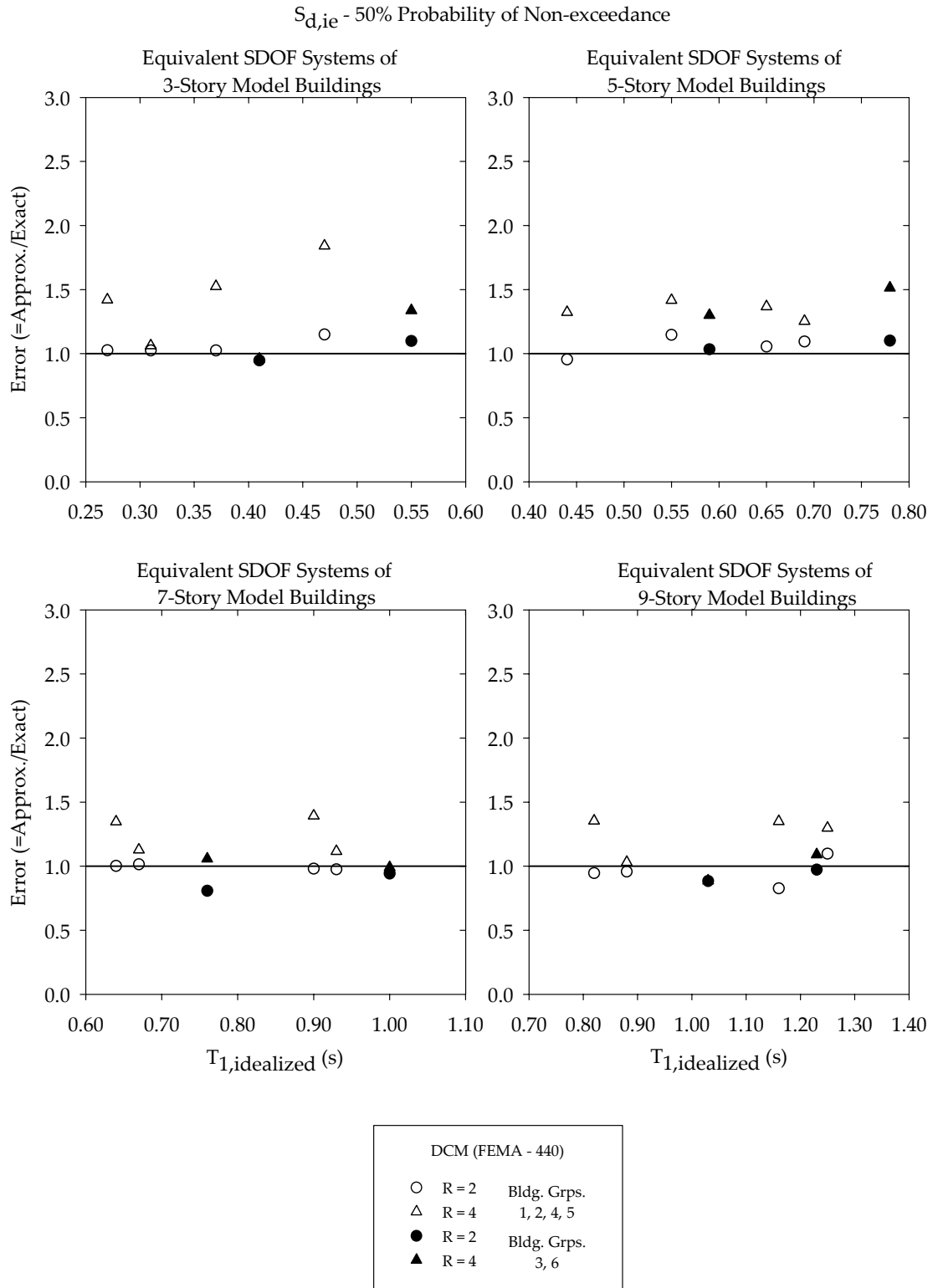


Figure 4.16 Error statistics for 50% probability of non-exceedance for improved DCM conditional to R for $S_{d,ie}$

MRDR_{ie} - 50% Probability of Non-exceedance

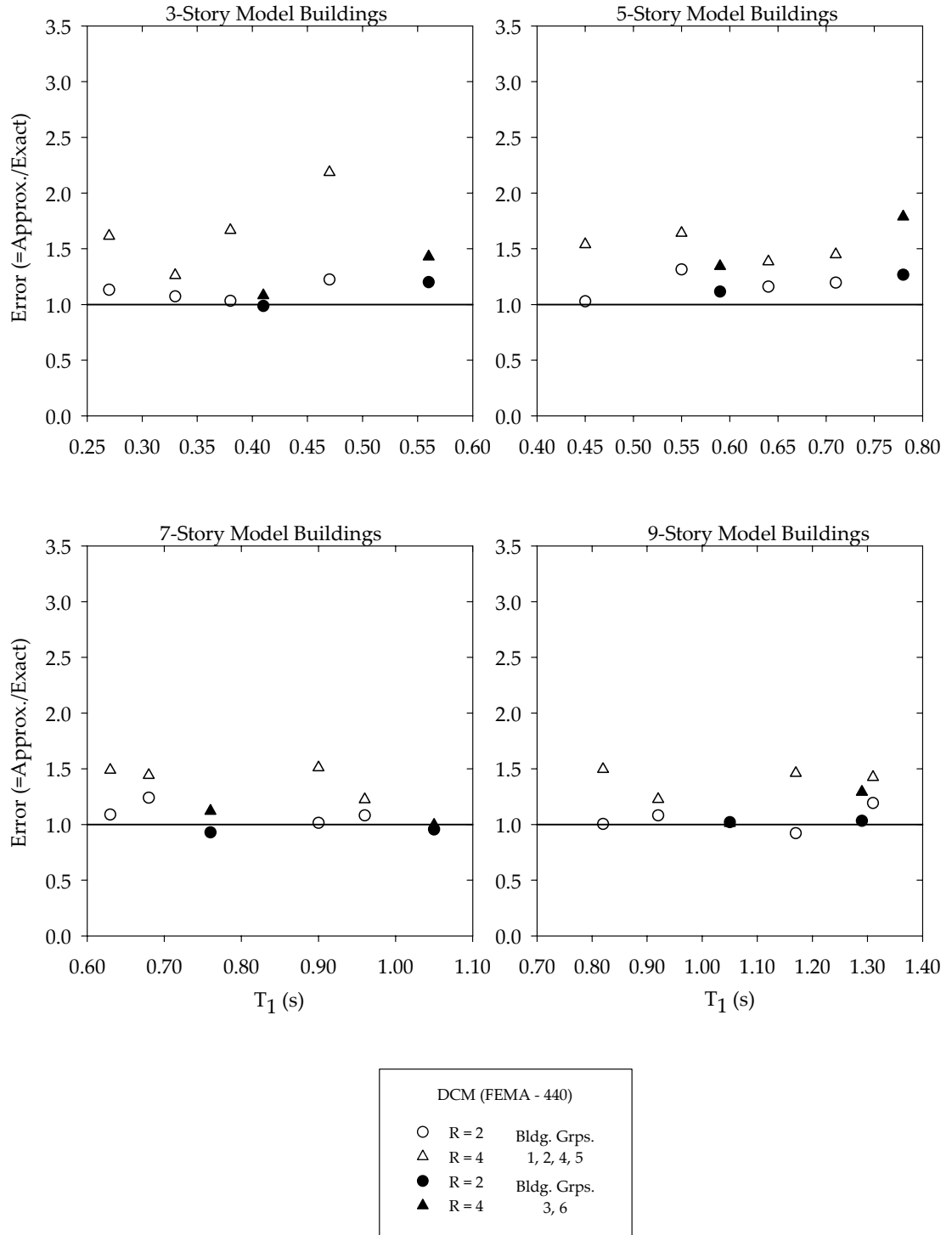


Figure 4.17 Error statistics for 50% probability of non-exceedance for improved DCM conditional to R for $MRDR_{ie}$

MIDR_{ie} - 50% Probability of Non-exceedance

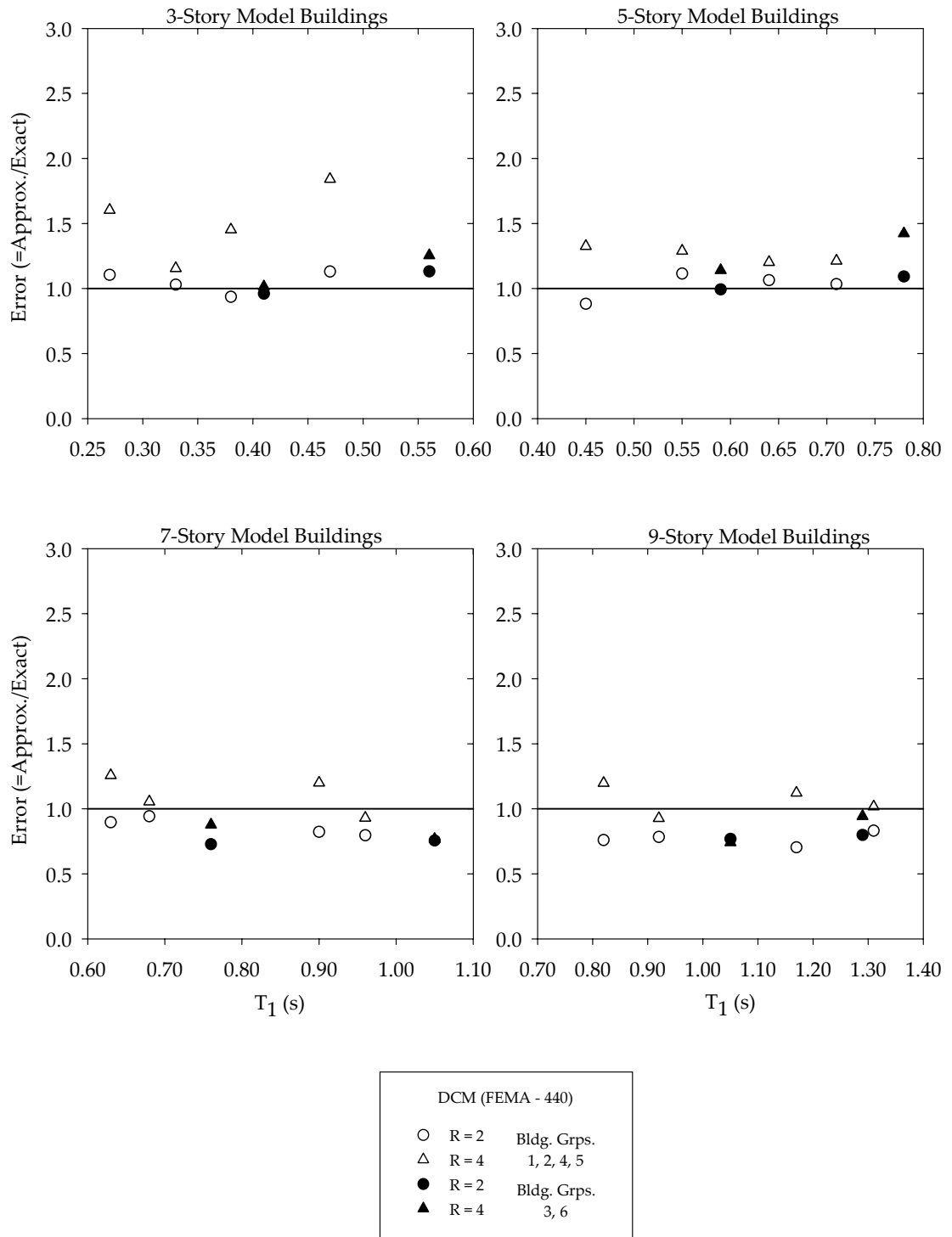


Figure 4.18 Error statistics for 50% probability of non-exceedance for improved DCM conditional to R for $MIDR_{ie}$

CHAPTER 5

A PROBABILISTIC APPROACH FOR ESTIMATING NONLINEAR DEFORMATION DEMANDS ON FRAME SYSTEMS

5.1 Introduction

The results of elastic and inelastic RHA for both MDOF and equivalent SDOF systems have been presented in Chapter 3. In Chapter 4, nonlinear static analysis procedures proposed by FEMA-440 document (ATC, 2005) have been examined in detail using the model buildings of this study. The observations in Chapters 3 and 4 have revealed several valuable information about linear and nonlinear responses of frame type structural systems. Observations presented in Chapter 3 are summarized below.

1. MDOF inelastic to elastic deformation demand variations have similar patterns to those of equivalent SDOF inelastic to elastic deformation demand variations. This information could be seen from Figures 3.22, 3.23 and Figures B.5 to B.12.
2. The distributions of $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$ fit well to log-normal distribution assumption (Figures B.5 to B.12 and Figures 3.25, 3.26).
3. Normalized lateral strength, R , is also in good correlation with log-normal distribution (Figures B.1 to B.4 and Figure 3.20).

4. Currently proposed factors by FEMA-440 document (ATC, 2005) for modifying the SDOF response to obtain the MDOF response are generally conservative.

The observations presented in Chapter 4 indicate:

1. Recently improved nonlinear static analysis procedures cannot capture the SDOF and MDOF deformation demands accurately for the following reasons:
 - (a) DCM does not account for the variation of C_1 with respect to different ground-motion intensity measures.
 - (b) DCM does not account for the variation of C_1 with respect to different post-yield stiffness ratios, α .
 - (c) CSM is an iterative procedure, based on displacement ductility and it does not capture the variation of R with respect to μ .

In this chapter, a new probabilistic approach for estimating the inelastic maximum interstory drift ratio demand (i.e., $MIDR_{ie}$) from the corresponding equivalent SDOF system is proposed. The elastic structural response is modified by a factor to predict the nonlinear response. The proposed modifying factor is a function of both normalized lateral strength, R , and post-yield stiffness ratio, α , and it accounts for the aleatory and epistemic uncertainties in a probabilistic manner. It is also a function of number of stories in the building.

The new procedure brings an improvement to the methodology presented by Akkar et. al (2005a) that estimates the maximum interstory drift ratio demands of elastic structures using $S_{d,e}$. Essentially, the proposed procedure modifies the elastic deformation demand imposed by $S_{d,e}$ by considering the variations in R and α .

5.2 Brief information on the procedure proposed by Akkar et al. (2005a)

The procedure proposed by Akkar et al. (2005a) calculates the elastic *MIDR* by modifying the ground story drift ratio expression presented in Güllkan and Akkar (2002) that utilizes the first-mode shear beam deformation pattern together with the corresponding elastic spectral displacement. The methodology uses beam-to-column stiffness ratio, ρ , proposed by Blume (1968) to account for the general moment-resisting frame behavior and modifies the local displacement demands computed for the shear beam behavior.

The general form of the equation proposed by Akkar et al. (2005a) is:

$$MIDR_e(T_1, \xi, \rho) = \gamma_1(\rho, T_1) \gamma_2(\rho, T_1) GSDR_{sh}(T_1, \xi) \quad (5.1)$$

where $GSDR_{sh}(T, \xi)$ is the maximum ground story drift ratio for a uniform continuous shear beam behaving in the fundamental mode (Güllkan and Akkar, 2002). The expression presented by Güllkan and Akkar (2002) is shown in Equation (5.2).

$$GSDR_{sh}(T_1, \xi) = 1.27 \sin\left(\frac{\pi h}{2H}\right) \frac{S_{d,e}(T_1, \xi)}{h} \quad (5.2)$$

where H and h represent the total building height and the story height, respectively.

The variable γ_1 in Equation (5.1) is the modification factor for correcting the shear frame ground story drift ratio to a more general ρ -based ground story drift ratio and it is calculated by using Equations (5.3) and (5.4).

$$\gamma_1(\rho, T_1) = c_1(\rho) + \frac{c_2(\rho)}{T_1} \quad (5.3)$$

$$c_1(\rho) = \frac{1}{1 + 0.35/\rho^{0.65}}, \quad c_2(\rho) = \frac{1}{8 + 25\rho^{0.4}} \quad (5.4)$$

The variable γ_2 accounts for the variation of maximum interstory drift ratio with respect to the ground story drift ratio for general elastic moment-resisting frame behavior. It is calculated by using Equations (5.5) and (5.6).

$$\gamma_2(\rho, T_1) = e^{(c_3(\rho) - c_4(\rho)/T_1)} \quad (5.5)$$

$$c_3(\rho) = \frac{1}{2\rho + 1}, \quad c_4(\rho) = \frac{0.07}{\rho^{0.25}} \quad (5.6)$$

Equations (5.1) to (5.6) suggest that the procedure proposed by Akkar et al. (2005a) modifies the elastic spectral displacement to obtain elastic *MIDR* using the modifying factors that account for the dynamic behavior of the structural system deforming in the first mode.

5.3 Evaluation of the procedure proposed by Akkar et al. (2005a) by using the model buildings of this study

The proposed probabilistic approach is primarily based on the procedure proposed by Akkar et al. (2005a). This section evaluates the validity of some of the descriptive statistics presented in Akkar et al. (2005a) for further enhancement of the methodology to estimate the nonlinear maximum interstory drift ratio demands.

The scatter plots in Figure 5.1 show the actual $\ln(MIDR_e)$ values for different ρ – values that are obtained by the elastic RHA in Akkar et al. (2005a) study. The

data presented were obtained from a total of 19 frame models with fundamental periods ranging between 0.2 to 2 s subjected to 89 records without pulse signals (Akkar et al., 2005a). The plots also display the logarithmic mean and $\pm$ standard deviations of the actual data superimposed with the mean and $\pm$ standard deviations of corresponding $MIDR_e$ estimations computed using the procedure presented in the same paper. The central tendencies (mean) and the dispersions ($\pm$ standard deviation) about the central tendencies computed from the actual data and estimations almost overlap each other. This observation when combined with the log-normality of actual elastic and inelastic $MIDR$ (stated in Chapter 3; Section 3.6) suggests that the elastic $MIDR$ estimations by Akkar et al. (2005a) procedure can also be assumed as log-normally distributed. Furthermore, the close resemblance between the actual data and estimations also suggest that the estimations by this procedure and the associated error statistics described in Akkar et al. (2005a) can be used confidently for this study. This is further investigated in the following paragraphs.

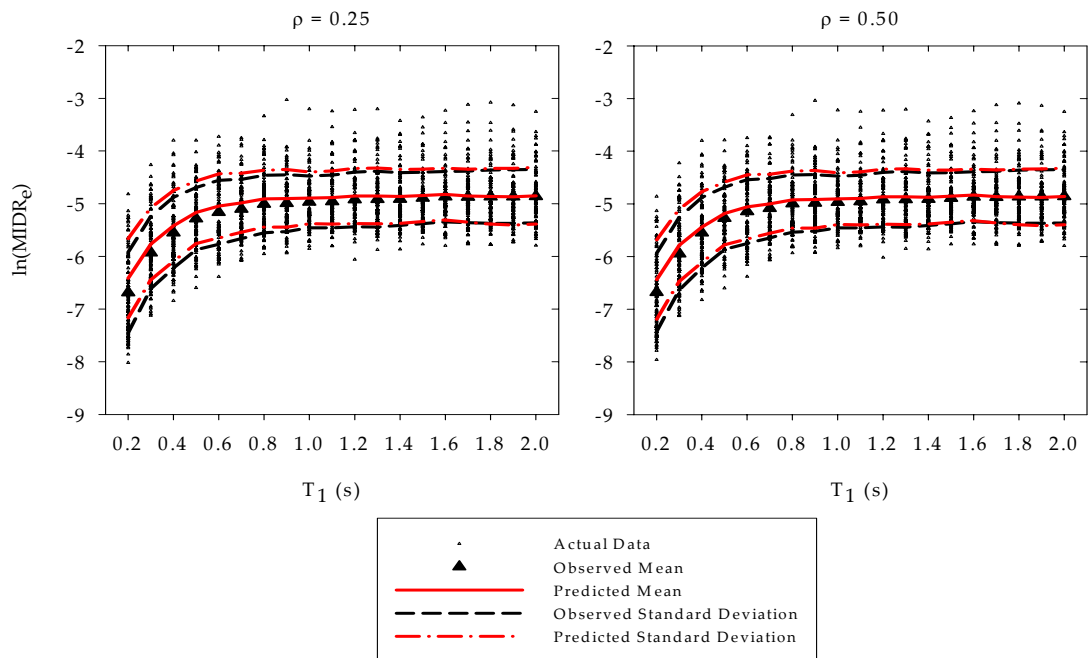


Figure 5.1 Actual and predicted logarithmic means and standard deviations by Akkar et al. (2005a) study for several ρ – values

The approximate $MIDR_e$ computed by the procedure presented in Akkar et al. (2005a) was normalized by the exact $MIDR_e$ determined from the RHA results presented in Chapter 3 to calculate the error statistics for $MIDR_e$ in a similar fashion as presented by Akkar et al. (2005a). (The approximate to exact $MIDR$ ratio is called “error” in the cited reference and it was used in deriving the pertinent error statistics for the verification of the proposed procedure for a comprehensive ground-motion data set that consists of 89 records without pulse and 56 records with pulse. In their evaluation Akkar et al. (2005a) used 190 frame models with fundamental periods and beam-to-column stiffness ratios, ρ , varying between 0.2-2.0 s and 0.125- ∞ , respectively). It should be noted that similar to Akkar et al. (2005a), the error statistics for pulse signal dominant records were computed for fundamental periods normalized by the pulse periods (i.e., T_1/T_p) for unbiased comparisons of the error statistics of this study and Akkar et al. (2005a). The error statistics were derived for a total of 20 ground motions for all building groups except for Group 6 that contains 18 near-fault records with pulse. Obviously, the number of ground motions used in this study is limited with respect to the ones presented in Akkar et al. (2005a).

The descriptive mean and standard deviations for error statistics computed for each model building-ground motion bin pair were compared with the ones presented in Akkar et al. (2005a). The comparative error statistics for $MIDR_e$ of this study and Akkar et al. (2005a) are presented in Tables 5.1 and 5.2 for no-pulse and with pulse ground-motion groups, respectively.

Table 5.1 Error statistics for building models subjected to ground motions that do not contain pulse signals

	T ₁ (s)	ρ	This Study		Akkar et al. (2005a)	
			E[error]	σ_{error}	E[error]	σ_{error}
Grp 1 3-St	0.27	0.405	1.177	0.045	1.209	0.020
Grp 1 5-St	0.45	0.405	1.097	0.020	1.114	0.013
Grp 1 7-St	0.63	0.405	1.097	0.058	1.104	0.025
Grp 1 9-St	0.82	0.405	1.051	0.088	1.086	0.041
Grp 2 3-St	0.33	0.342	1.164	0.018	1.163	0.013
Grp 2 5-St	0.55	0.354	1.115	0.058	1.113	0.019
Grp 2 7-St	0.68	0.435	1.066	0.152	1.097	0.029
Grp 2 9-St	0.92	0.366	0.942	0.182	1.080	0.054
Grp 4 3-St	0.38	0.405	1.225	0.034	1.132	0.010
Grp 4 5-St	0.64	0.405	1.108	0.037	1.103	0.025
Grp 4 7-St	0.90	0.405	1.058	0.085	1.078	0.054
Grp 4 9-St	1.17	0.405	1.023	0.114	1.064	0.067
Grp 5 3-St	0.47	0.342	1.202	0.024	1.119	0.014
Grp 5 5-St	0.71	0.354	1.081	0.146	1.107	0.028
Grp 5 7-St	0.96	0.435	0.988	0.180	1.072	0.060
Grp 5 9-St	1.31	0.366	0.838	0.224	1.059	0.084

Table 5.2 Error statistics for building models subjected to ground motions that contain pulse signals

T ₁ /T _p	ρ	This Study		Akkar et al. (2005a)	
		E[error]	σ_{error}	E[error]	σ_{error}
0.05	0.301	1.181	0.046	1.184	0.081
0.15	0.408	1.142	0.069	1.098	0.052
0.25	0.395	1.144	0.105	1.087	0.070
0.35	0.449	1.129	0.040	1.060	0.068
0.45	0.379	1.164	0.124	1.059	0.071
0.55	0.461	1.106	0.110	1.064	0.060
0.65	0.437	1.093	0.060	1.037	0.061
0.75	0.354	1.231	0.044	1.056	0.059
0.85	0.444	1.168	0.064	1.069	0.062
1.05	0.458	1.121	0.054	1.072	0.036
1.15	0.488	1.062	0.067	1.094	0.050

The statistics presented in Tables 5.1 and 5.2 indicate that the mean errors computed from this study are in good agreement with the ones presented in Akkar et al. (2005a). The higher dispersion (described by σ_{error}) in this study can be attributed to the relatively small ground-motion data set used.

The dependence of mean error statistics on the sample size is investigated quantitatively by generating discrete random samples of size 20 from the no-pulse $MIDR_e$ error statistics presented by Akkar et al. (2005a). This process presumes that the error data set of Akkar et al. (2005a) represents the actual population for the performance verification of the $MIDR_e$ estimations. The random realizations follow uniform distribution to ensure that the probability of each variable randomly selected from the population of size N (89 for this case) is equal to $1/N$. Thus, the randomly selected variables (error terms) do not repeat each other for randomly generated samples. Based on the list presented in Table 5.1, a total of 200 error samples of size 20 is generated from the error statistics of Akkar et al. (2005a) for $T_1 = 0.2$ to 1.3 s (with a period increment of 0.1 s) and $\rho = 0.25$ and 0.5 that practically covers the overall fundamental period and ρ -range of the model buildings used in this study. The mean error and standard deviation about the mean error is computed for each randomly generated sample of size 20 for each $T_1 - \rho$ pair. The scatters presented in Figure 5.2 display the simulation results for the mean error, $E[error]$, and the standard deviation about mean error, σ_{error} , using the procedure described above. Each column corresponds to one particular fundamental period. The scatters for each fundamental period contain $E[error]$ and σ_{error} of 400 independent realizations (200 realizations for each pre-selected beam-to-column stiffness ratio: $\rho = 0.25$ and 0.5). The shaded regions in Figure 5.2 display the upper and lower bounds for the mean error and standard deviation statistics. The circles presented on each plot correspond to the $E[error]$ and σ_{error} that are listed in Table 1 (i.e., mean error and the standard deviation about the mean

error of Akkar et al. (2005a) using the data in this study). The fairly good agreement between the mean error statistics of Akkar et al. (2005a) and this study is revealed once again from $E[error]$ plots except for a few cases whose mean error variations are off the upper and lower bounds of the simulations. The scatter plots for σ_{error} emphasize the significance of sample size while defining the uncertainty on the $MIDR_e$ estimations. The simulations from Akkar et al. (2005a) clearly show that the dispersion about σ_{error} becomes significant particularly for increasing periods. When the corresponding dispersion statistics of this study are superimposed on the simulations one can observe that σ_{error} of this study follows the pattern displayed by the simulations. For most cases, σ_{error} computed in this study follow the upper boundary of the simulations in a fairly close manner. However, for some medium and long range fundamental periods covered in this study the σ_{error} are significantly off the limits presented by the simulations.

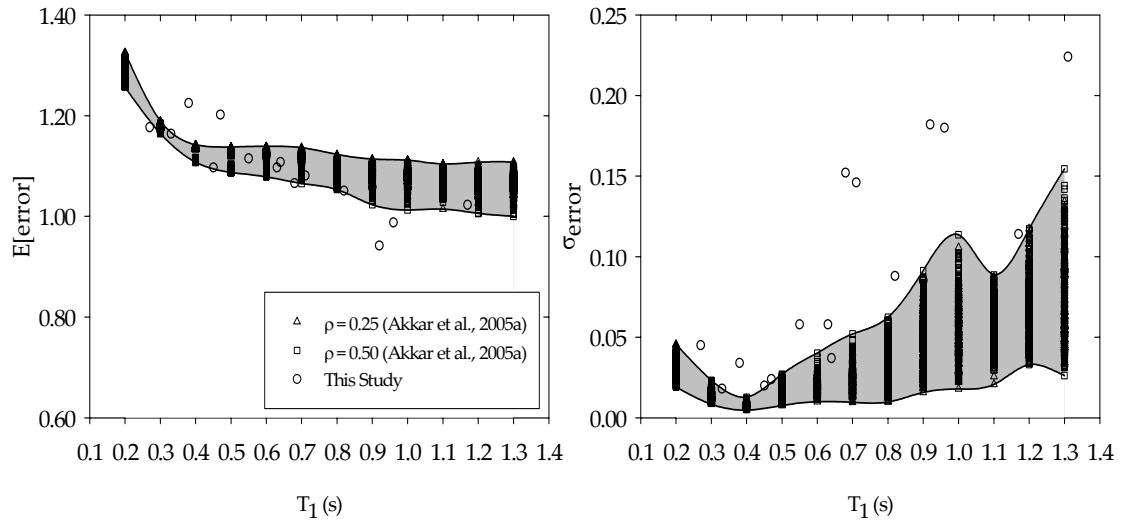


Figure 5.2 Comparison of mean error and standard deviation about the mean error using the statistics of this study and random realizations from Akkar et al. (2005a) for records without pulse

Following the above discussions, it was decided that the methodology presented in Akkar et al. (2005a) is general and it can be used within the context of this study. This decision accepts a certain level of risk that for some cases the error statistics by Akkar et al. (2005a) can exhibit some bias even if they are based on a significantly large ground motion and building model inventory. The lack of sufficient data for records with pulse did not permit a similar analysis. Thus, the comments made for the records without pulse cannot be validated for the generality of Akkar et al. (2005a) statistics for pulse-dominant ground motions. Figure 5.3 presents the log-normal standard deviations of Akkar et al. (2005a) approximate procedure from the actual data (i.e., ζ_{MIDR_e}) that are used while describing the proposed method in the next sections.

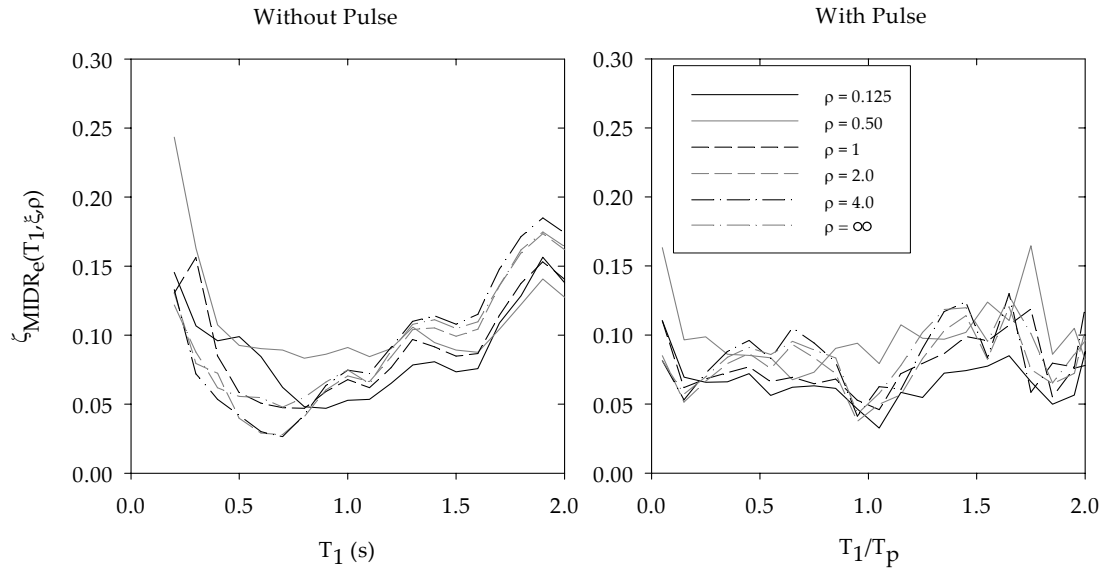


Figure 5.3 Log-normal standard deviations of $MIDR_e(T_1, \xi, \rho)$ for several ρ – values

5.4 A direct application of Akkar et al. (2005a) method for nonlinear structural systems

The literature is abundant for approximating the $MIDR_e$ demands on frame type structural systems. Using the fundamental-mode behavior, Miranda (1999), Heidebrecht and Rutenberg (2000), Güllkan and Akkar (2002), Miranda and Reyes (2002), Akkar et al. (2005a) have presented particular methodologies to estimate $MIDR_e$ for frames deforming either in shear or in mixed mode (i.e., shear and flexure). Miranda (1999) also extended his findings for inelastic behavior modifying the elastic drift estimations by a factor that is a function of ductility, μ . The basic methodology presented by Miranda (1999) has similarities with the one proposed by Akkar et al. (2005a); both procedures make use of the spectral displacement for estimating the $MIDR_e$. The procedure in Miranda (1999) extends the elastic estimations to inelastic $MIDR$ ($MIDR_{ie}$) estimations by using the aforementioned μ – dependent modifying factor for approximating $S_{d,ie}$ from the corresponding $S_{d,e}$. Miranda (1999) assumed that the rest of the parameters in the proposed procedure are the same for structures behaving either in the linear or nonlinear range. Recently Medina and Krawinkler (2005) proposed a new procedure to estimate $MIDR_{ie}$ by using the corresponding elastic values considering the lateral strength capacity of the structure.

In this section, the approximate procedure proposed by Akkar et al. (2005a) is verified for nonlinear building behavior by replacing the elastic spectral displacement term with the corresponding inelastic spectral displacement. The idealized bilinear pushover curves that are described in Chapter 2 were used to approximate the global nonlinear behavior of the structural models. Using the idealized bilinear curves, the spectral displacements for each building model-ground-motion pair were implemented in Equation (5.1). The inelastic spectral

displacements were computed for normalized lateral strength, R . The approximate inelastic $MIDR$ s computed from this process are compared with the ones obtained from the nonlinear RHA. Figure 5.4 presents the variation of approximate and exact inelastic $MIDR$ for no-pulse (i.e., $MIDR_{ie}$ values computed for Group 1, 2, 4, 5 model buildings) and with pulse (i.e., $MIDR_{ie}$ values computed for Group 3 and 6 model buildings) ground motions. The scatter plots also show the median, 16-percentile and 84-percentile values to present the overall tendency in the approximate and exact $MIDR_{ie}$ values. The plots in Figure 5.4 also include the recent approximate procedure proposed by Medina and Krawinkler (2005) that makes use of elastic $MRDR$ approximation and normalized lateral strength, R , to estimate the corresponding inelastic $MIDR$ values.

The comparisons in Figure 5.4 show that the $MIDR_{ie}$ estimations by Akkar et al. (2005a) follow the general trends described by those of nonlinear RHA. The approximations by Medina and Krawinkler (2005) show significant departures from $MIDR_{ie}$ values computed from nonlinear RHA. The procedure proposed by Medina and Krawinkler (2005) describes the variation of inelastic $MIDR$ through an empirical relationship that makes use of elastic $MRDR$ estimation. It considers the variation of R in an implicit manner. Therefore, it does not consider the particular variation of post-yield stiffness ratio, α , and normalized lateral strength, R , for each record that make a significant difference in the deformation demands on structural systems as presented in Chapter 4. In the current comparisons the direct use of $S_{d,ie}$ in the procedure by Akkar et al. (2005a) results in superior estimations when compared to the ones computed from Medina and Krawinkler (2005). This simple comparison once more reveals the importance of α and R in the nonlinear structural behavior.

One other important observation highlighted from the actual $MIDR_{ie}$ statistics (first column plots in Figure 5.4) is the existence of data that do not follow the general trend (statistics that are well above and below the 84-percentile and 16-percentile envelopes, respectively) that are believed to be due to the complicated ground-motion behavior that cannot be predicted adequately by central tendency plots. One way of handling such complex behavior is the utilization of probabilistic approaches in the estimation of deformation demand parameters that is the main objective of this chapter.

The correlation between the nonlinear RHA results and the estimations computed from the procedure proposed by Akkar et al. (2005a) in the inelastic range is displayed once again in Figure 5.5. The figure on the left panel displays the results for building models subjected to ground motions without pulse signals. The figure on the right panel displays the results for building models subjected to pulse dominant ground motions.

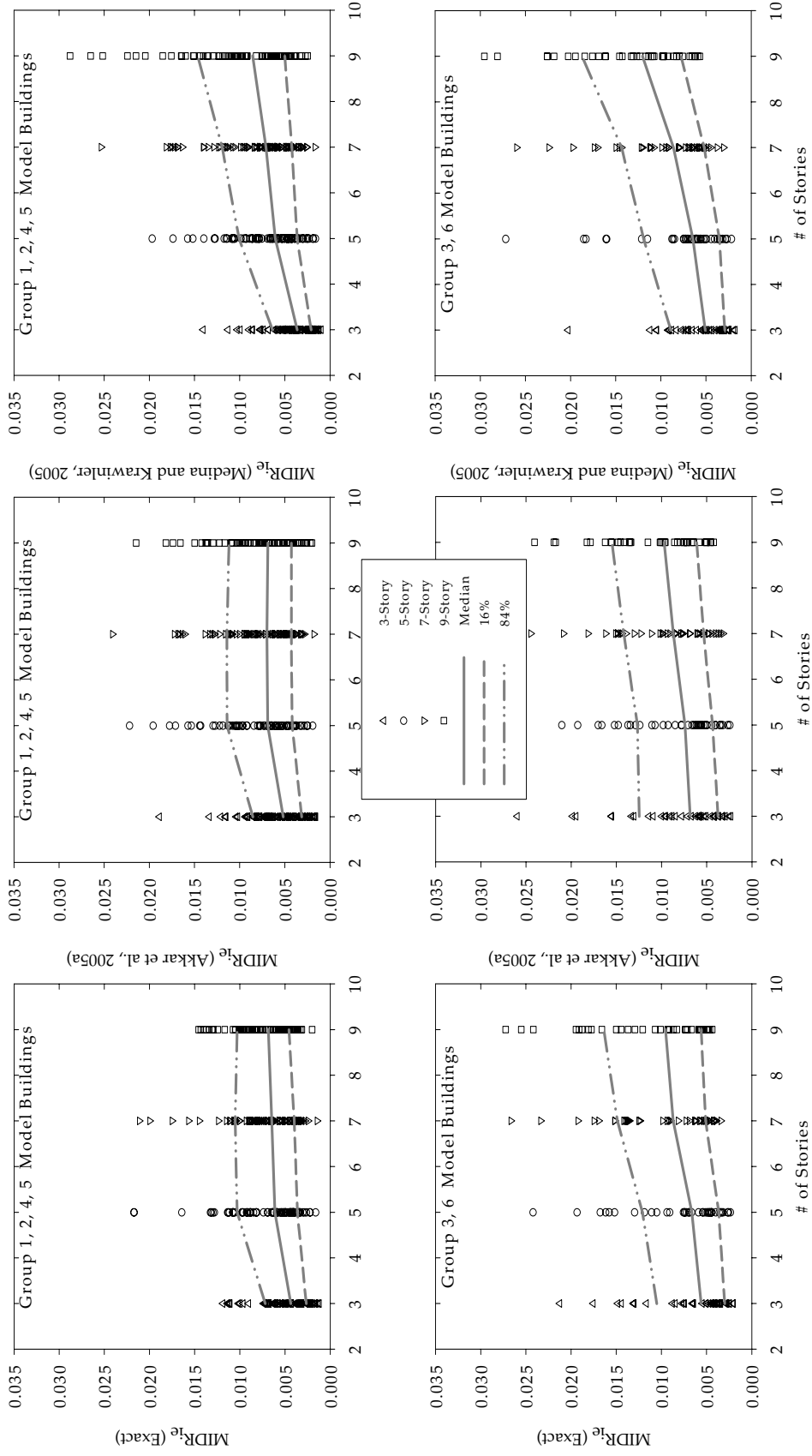


Figure 5.4 Comparisons of exact $MIDR_{ie}$ values with predictions computed from different approximate procedures

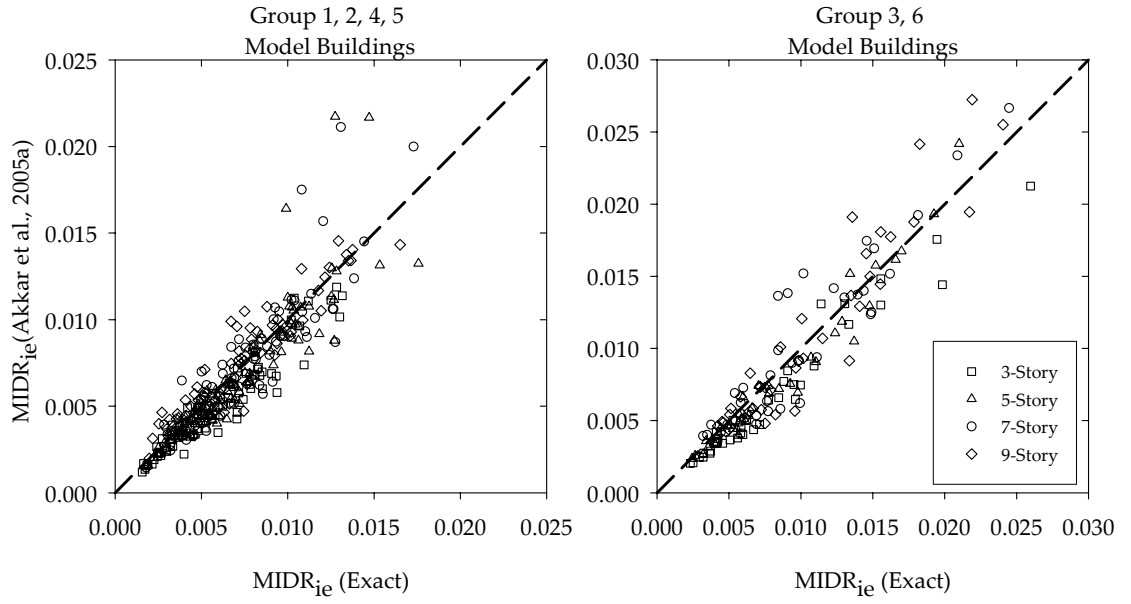


Figure 5.5 Correlation of exact and approximate $MIDR_{ie}$ values from Akkar et al. (2005a) procedure

The correlation between the exact and approximate $MIDR_{ie}$ is fairly well for both cases (i.e., no-pulse and pulse) except for a few points that do not follow the general trend in no pulse case. These plots emphasize once again that for fundamental-mode dominant building models, the procedure by Akkar et al. (2005a) would estimate the $MIDR_{ie}$ fairly well as long as the inelastic spectral displacements are accurately determined for that particular structural system.

5.5 Proposed probabilistic procedure

The discussions presented in the previous section indicated that the procedure by Akkar et al. (2005a) can be extended to inelastic $MIDR$ estimation for first-mode dominant frames as long as the inelastic spectral displacements are computed

accurately. One of the fundamental issues in the approximate procedures is their practicality that requires the estimation of spectral parameters from empirical expressions. Moreover, computation of inelastic spectral displacements for each particular case makes the use of Akkar et al. (2005a) procedure impractical for the seismic vulnerability analyses of large building stocks. This section presents a probabilistic approach that estimates the inelastic spectral displacements for a given R and α pair since these parameters play a role in the nonlinear deformation demands as presented in Chapter 4.

5.5.1 Determination of $\Omega = S_{d,ie} / S_{d,e}$ in a probabilistic manner

The probabilistic estimation of $S_{d,ie}$ is done by introducing a modifying factor, Ω , that establishes a relationship between elastic and inelastic spectral displacements as a function of R and α (i.e., $\Omega = S_{d,ie} / S_{d,e}$). The expression for Ω were derived for ground motions with and without pulse separately. The following procedure was used in the derivation of the modifying factor Ω .

1. The fundamental period versus building height relationship presented in Figure 2.5 was used to determine representative oscillator vibration periods for each of 3-, 5-, 7-, and 9-story buildings. Thus, the modifying factor, Ω , derived in this study is a function of number of stories of the building. The chosen vibration period intervals for 3-, 5-, 7- and 9-story buildings are 0.3-0.55 s, 0.5-0.8 s, 0.75-1.05 s, 0.9-1.3 s, respectively. For each vibration period interval, an increment of 0.05 s was used to capture the variation of Ω in detail. It should be noted once again that the fundamental periods of the model buildings used in this study are in good agreement with the fundamental mode-building height relationship derived by Chopra and Goel (2000a). This can give basis for general usage of the proposed procedure.

2. Nonlinear SDOF RHA for R varying between 1.5, 2, 3, 4, 5, 6, 7, 8 and $\alpha=0.0, 0.02, 0.04, 0.06, 0.08$ and 0.1 were performed for ground motions with and without pulse for all number of story based period intervals.
3. The above nonlinear RHA were repeated for the elastic oscillator behavior to compute the actual variation of Ω as a function of R, α and number of stories.
4. Based on the observations in Chapter 3 that suggested fairly good agreement of $S_{d,ie} / S_{d,e}$ (i.e., Ω) and R with log-normal probability distribution, the log-normal mean (i.e., $\lambda_{\Omega(\alpha,R,k)}$; k : number of stories) and standard deviations (i.e., $\zeta_{\Omega(\alpha,R,k)}$) were computed for Ω for each number of stories, α and R combination using the maximum likelihood estimator approach (Ang and Tang, 1975).
5. Using 2-stage regression analyses, the variation in $\lambda_{\Omega(\alpha,R,k)}$ and $\zeta_{\Omega(\alpha,R,k)}$ was determined to derive number of story based log-normal probability functions of Ω as a function of R and α . Equations (5.7) to (5.12) give the general expressions for $\lambda_{\Omega(\alpha,R,k)}$ and $\zeta_{\Omega(\alpha,R,k)}$ whereas Tables 5.3 and 5.4 give the regression coefficients that depend on number of stories of buildings for records with and without pulse, respectively. The expressions derived may seem to be complex at the first glance but a simple spreadsheet formulation can enable one to compute the different percentiles of Ω that depend on number of stories of the building as a function of R and α .

$$\lambda_{\Omega(\alpha,R,k)} = \varepsilon_1 + \varepsilon_2 R + \varepsilon_3 R^2 \quad (5.7)$$

$$\zeta_{\Omega(\alpha,R,k)} = \varepsilon_4 \nu^{1/R}, \quad \nu = \text{constant (number of stories and ground-motion type dependent)} \quad (5.8)$$

$$\varepsilon_1 = \frac{1}{\varepsilon_{11} + \varepsilon_{12}\alpha + \varepsilon_{13}\alpha^2} \quad (5.9)$$

$$\varepsilon_2 = \varepsilon_{21} + \varepsilon_{22}\alpha + \varepsilon_{23}\alpha^2 \quad (5.10)$$

$$\varepsilon_3 = \varepsilon_{31} + \varepsilon_{32}\alpha + \varepsilon_{33}\alpha^2 \quad (5.11)$$

$$\varepsilon_4 = \frac{1}{\varepsilon_{41} + \varepsilon_{42}\alpha^{\varepsilon_{43}}} \quad (5.12)$$

Figure 5.6 presents the mean and standard deviation values of the actual $\ln(S_{d,ie}/S_{d,e})$ ratios together with the predicted mean and standard deviation values of $\ln(S_{d,ie}/S_{d,e})$ ratios for several selected story levels and α -values after the 2-stage regression process. The actual and predicted mean and standard deviation values are almost equal to each other. Separate from these analyses, the residual analyses that would yield the deviations of mean $\ln(S_{d,ie}/S_{d,e})$ estimations from the actual data were also computed. The residual analyses indicated that the standard deviation of the mean $\ln(S_{d,ie}/S_{d,e})$ estimations overlap the $\zeta_{\Omega(\alpha,R,k)}$ expression presented in Equation (5.8). Therefore, Equations (5.7) and (5.8) can be used for the estimation of the mean and standard deviations about mean error for Ω that is described in the probabilistic procedure.

Additional information is given about the performance of regression results in Figures D.1 to D.8 using the fits calculated by Equations (5.7) to (5.12). These figures compare the log-normal Ω distributions computed from the λ_{Ω} and ζ_{Ω} of the nonlinear RHA and the ones calculated by using Equations (5.7) to (5.12). It should be noted that, in Chapter 3, the hypothesis about the log-normality of $S_{d,ie}/S_{d,e}$ (parameter Ω in this chapter) has been verified through various goodness-of-fit checks. Thus, Figures D.1 to D.8 directly compare the log-normal distributions using the log-normal statistics of the observed data from the RHA and the one computed

from Equations (5.7) to (5.12). The performance of the regression results are unexpectedly well for almost all story levels, for all R and α combinations and for with and without pulse ground motions. The following sections also describe the influence of R and α using the probability distribution functions derived using the regression analyses

Table 5.3 Regression constants for model buildings subjected to ground motions that do not contain pulse signals

	ϵ_{11}	ϵ_{12}	ϵ_{13}	ϵ_{21}	ϵ_{22}	ϵ_{23}	ϵ_{31}	ϵ_{32}	ϵ_{33}	ϵ_{41}	ϵ_{42}	ϵ_{43}	ν
3-Story	-3.805	-75.023	263.738	0.196	-2.634	13.559	-0.010	0.163	-0.681	1.038	0.206	0.031	0.090
5-Story	-4.371	-71.034	-44.442	0.135	-2.123	9.387	-0.007	0.118	-0.297	1.261	0.650	0.298	0.080
7-Story	-6.054	-128.149	-1267.995	0.101	-2.085	10.136	-0.006	0.128	-0.472	1.391	0.606	0.230	0.110
9-Story	-6.088	-69.660	-5240.370	0.102	-2.458	11.756	-0.007	0.172	-0.634	1.419	0.707	0.123	0.120

Table 5.4 Regression constants for model buildings subjected to pulse containing ground motions

	ϵ_{11}	ϵ_{12}	ϵ_{13}	ϵ_{21}	ϵ_{22}	ϵ_{23}	ϵ_{31}	ϵ_{32}	ϵ_{33}	ϵ_{41}	ϵ_{42}	ϵ_{43}	ν
3-Story	-2.144	-27.754	130.238	0.361	-3.153	16.117	-0.014	0.149	-0.972	0.974	7.528	1.170	0.090
5-Story	-2.503	-25.314	62.356	0.319	-2.726	11.942	-0.014	0.172	-0.906	0.900	2.744	1.004	0.088
7-Story	-3.248	-56.574	180.264	0.284	-2.931	14.009	-0.013	0.208	-1.067	1.092	1.285	0.717	0.125
9-Story	-3.052	-26.826	149.381	0.253	-2.052	10.688	-0.012	0.168	-0.988	1.332	1.449	1.050	0.097

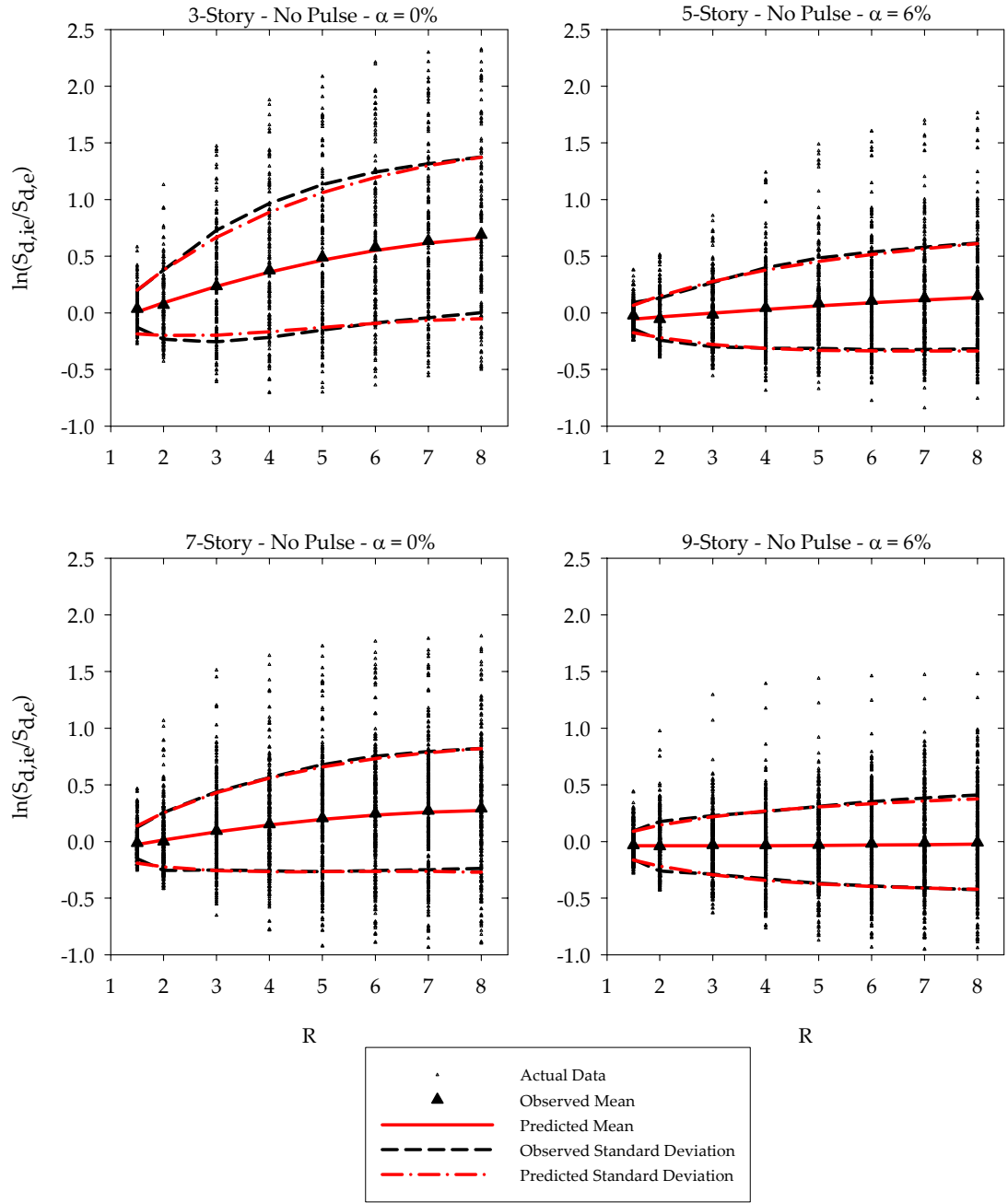


Figure 5.6 Actual and predicted means and standard deviations of $\ln(S_{d,ie}/S_{d,e})$ values for several selected model building and α combinations

5.5.2 Effects of R , α and number of stories on $\Omega = S_{d,ie} / S_{d,e}$

Cumulative distributions of $\Omega = S_{d,ie} / S_{d,e}$ are displayed in Figures 5.7 to 5.14 to describe the influence of R , α and number of stories on the modification factor Ω . The plots are presented for 3-, 5-, 7-, and 9-story model buildings to describe the variations in Ω for records with and without pulse, respectively. The first 4 observations listed below are valid for records without pulse whereas the rest of the observations are done for records with pulse. It should be noted that the observations presented here are parallel to the studies conducted by other investigations (e.g., Ruiz-García and Miranda, 2003; Chopra and Chintanapakdee, 2004) but the probability distributions presented yield a more amplified perspective for the influence of above structural parameters on the behavior of Ω .

1. There is always chance to have Ω values less than 1 for all α , R and number of stories. The chance of having $\Omega \leq 1$ is higher for small R . For example, for 3-story buildings subjected to records without pulse (Figure 5.7), the probability of $\Omega \leq 1$ is 50% when $R = 1.5$. For the same number of stories, this probability is 18% for $\alpha = 0\%$ and 32% for $\alpha = 10\%$ when $R = 6$. These probabilities tend to increase as number of stories increases showing that the short structures are more sensitive to the changes in lateral strength when compared to the relatively taller structures.
2. The variation of α does not influence the probability distribution function of Ω for all buildings when $R \leq 2$. Therefore, the deformation demands on structures with high lateral strength capacities with respect to their elastic strength (i.e., small R – values) would be modified in a similar manner regardless of α .
3. The influence of α on the modifying factor increases considerably for $R \geq 4$ for all stories. This observation is more pronounced for short structures. For

example, the non-exceedance probability of $\Omega \leq 2$ is 64% and 85% for $\alpha = 0\%$ and $\alpha = 10\%$ for 3-story buildings when the normalized lateral strength is 5. The corresponding values are 88% and 99% for 9-story buildings. Thus, given a constant R – value the increase in number of stories implies a lesser influence of α on Ω .

4. The variations in R influence the modifying factor Ω more for short structures when compared to the taller structures. For example, for 3-story structures when $\alpha = 0\%$ is considered (i.e., when the variations in Ω are significant with respect to other α – values) the non-exceedance probabilities for $\Omega \leq 2$ is 64% for $R = 5$ and 50% for $R = 8$. These probabilities are 88% and 83% for 9-story structures.
5. The observations presented in items (1) and (2) are valid for records with pulse.
6. The observation presented in item (3) is also valid for records with pulse with non-exceedance probabilities significantly lower than those presented in item (3). This indicates that the records with pulse would exhibit higher Ω – values with respect to those obtained from records without pulse. For example, when $R = 5$, the non-exceedance probabilities are 31% and 64% for $\alpha = 0\%$ and 10%, respectively in 3-story buildings. For 9-story buildings, these probabilities are 53% and 72% for $\alpha = 0\%$ and 10%, respectively. When compared to the non-exceedance probabilities of records without pulse for the same case as presented in item (3), these probabilities are significantly lower suggesting that the nonlinear deformation demands of records with pulse are more likely to attain higher values with respect to their linear counterparts.
8. Contrary to the observation in item (4) the variations in R (particularly for $R \geq 4$) makes a significant change in the modifying factor Ω for both taller and short structures when subjected to records with pulse.

9. The probability distributions of Ω for short structures with relatively lower strength capacities (i.e., $R \geq 4$) show significantly lower probability of non-exceedance indicating that such structures when subjected to pulse-dominant records, would have significantly large nonlinear deformation demands. The variations in R is drastic similar to the observation stated in item (4). For example, for 3-story structures when $\alpha = 0\%$ is considered the non-exceedance probabilities for $\Omega \leq 2$ is 31% for $R = 5$ and 15% for $R = 8$.

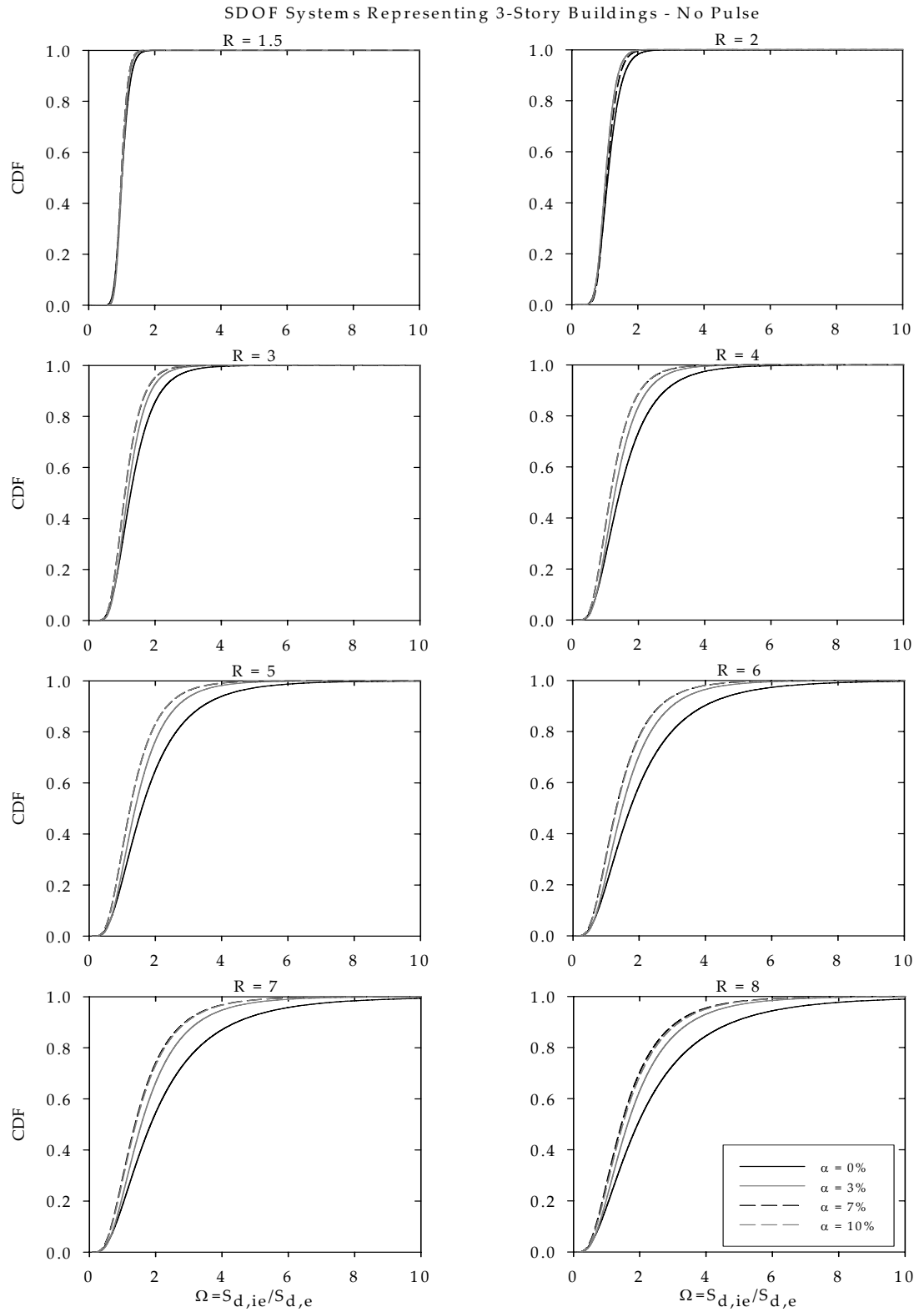


Figure 5.7 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 3-story buildings subjected to ground-motion records without pulse

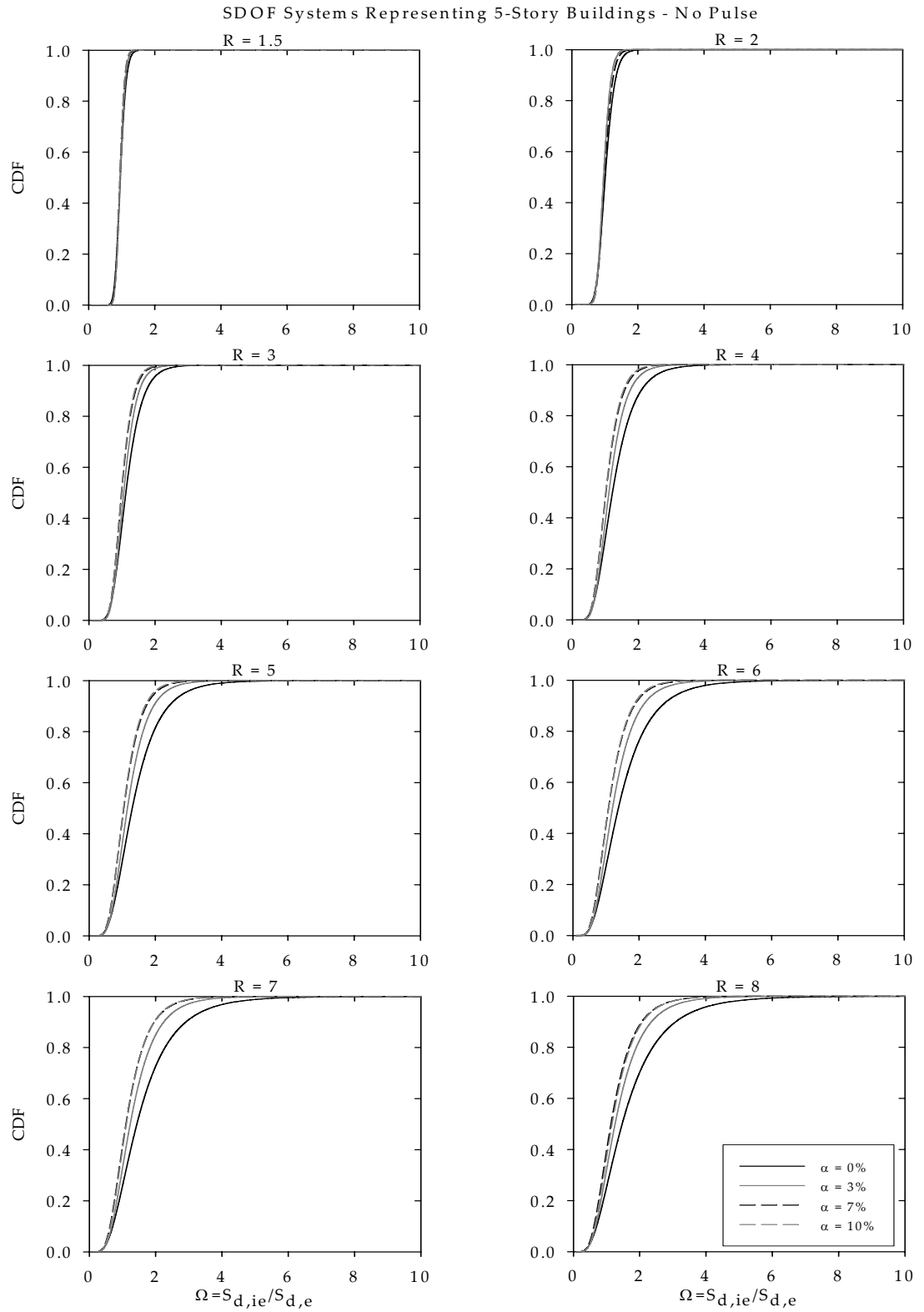


Figure 5.8 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 5-story buildings subjected to ground-motion records without pulse

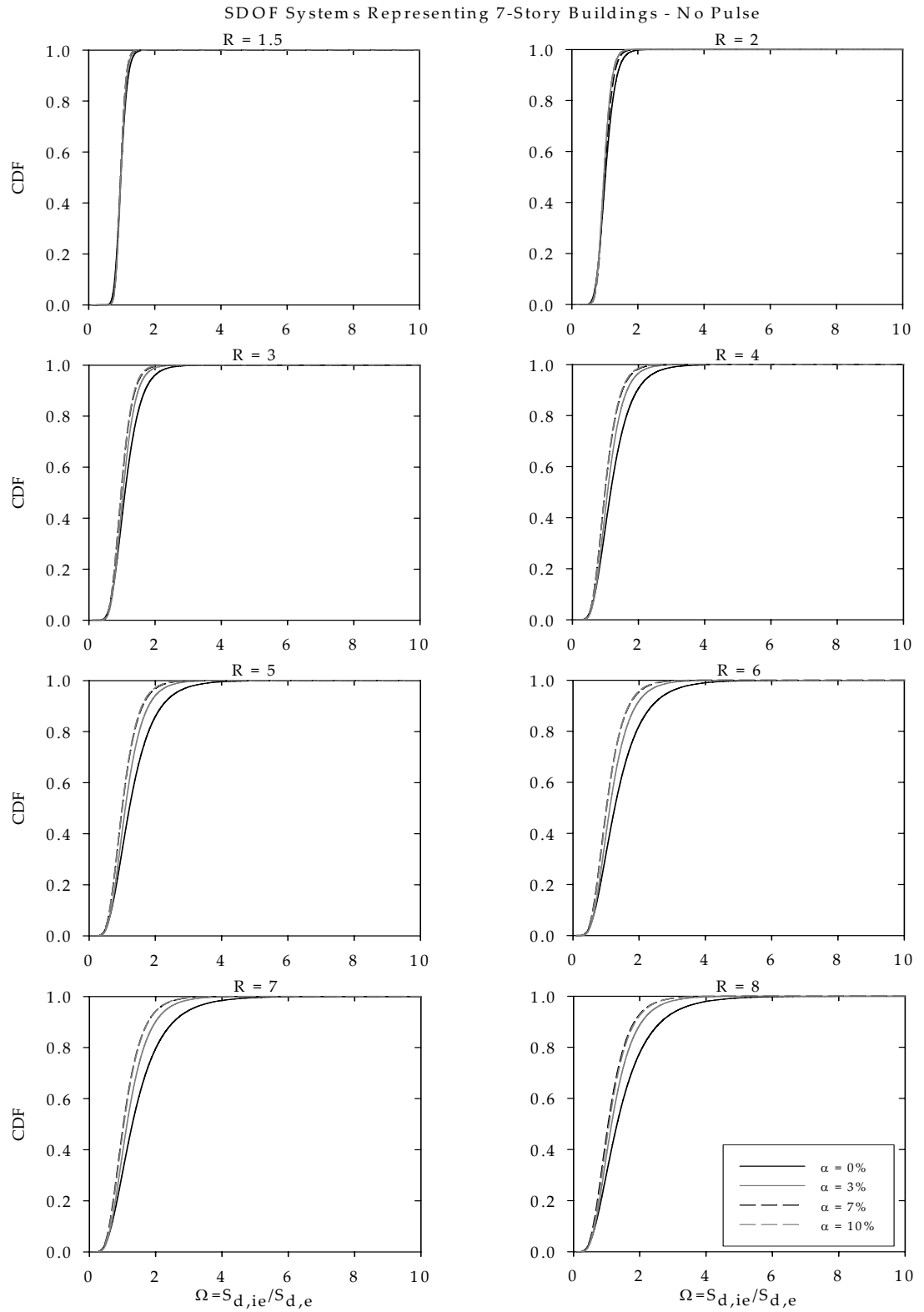


Figure 5.9 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 7-story buildings subjected to ground-motion records without pulse

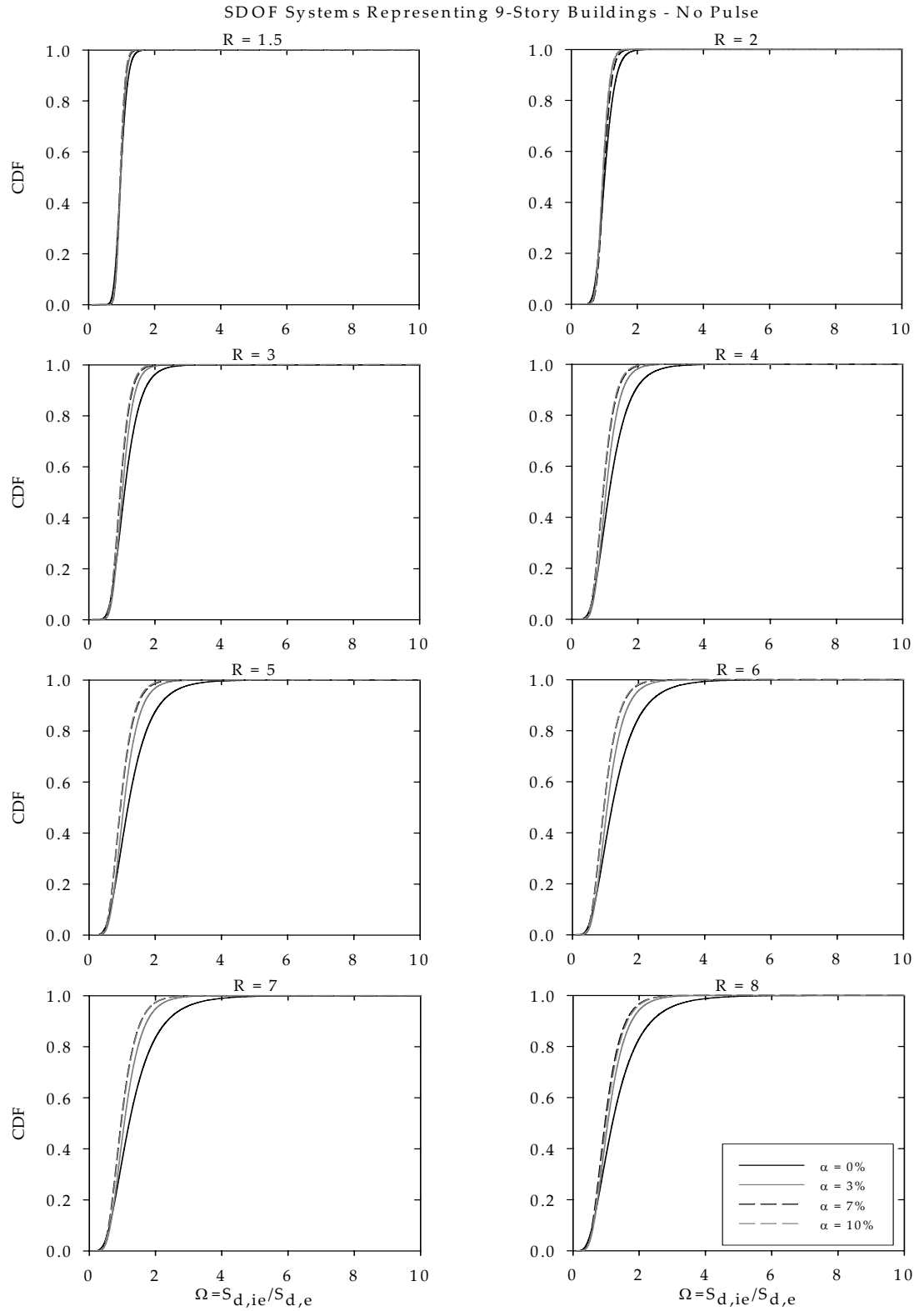


Figure 5.10 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 9-story buildings subjected to ground-motion records without pulse

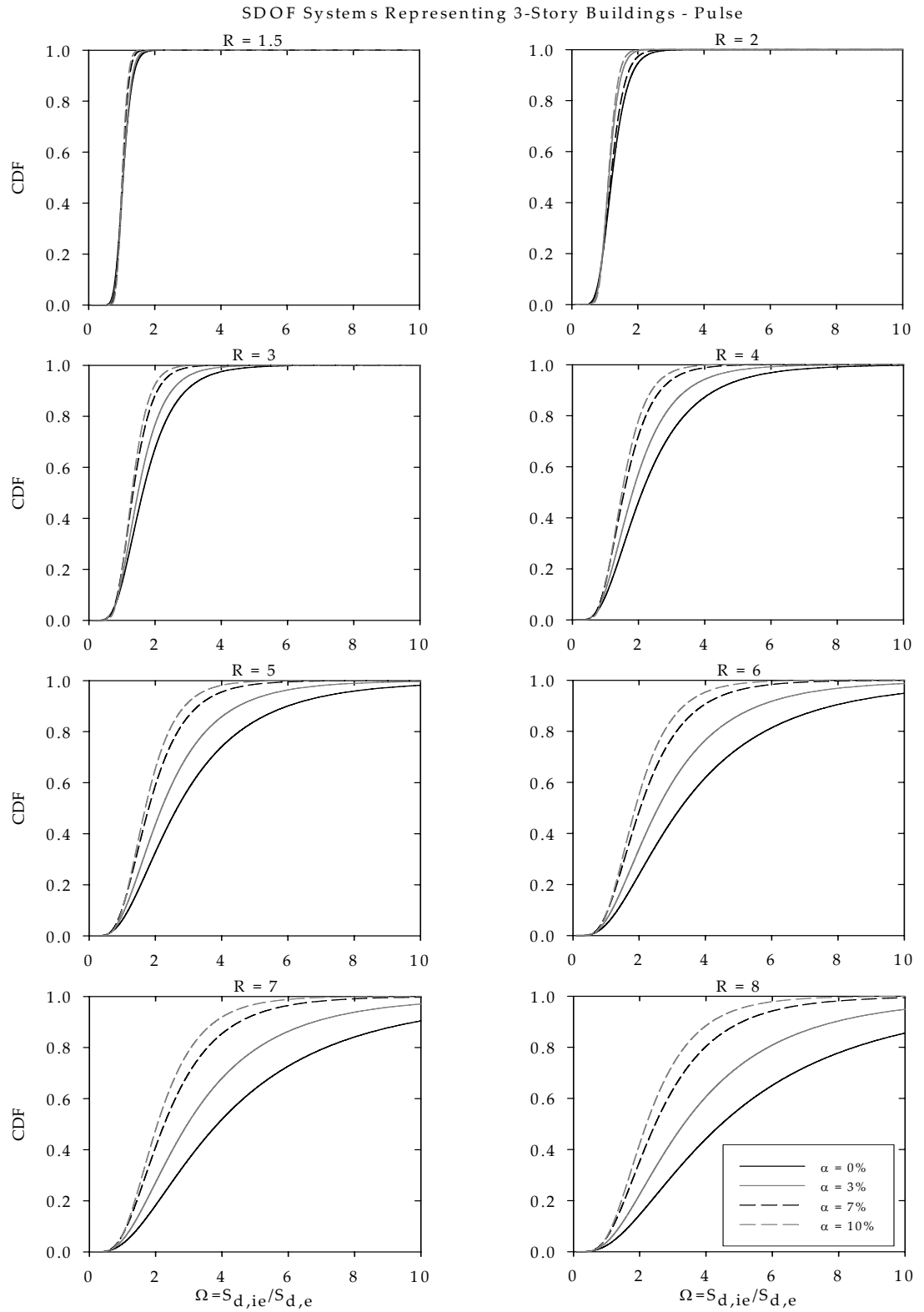


Figure 5.11 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 3-story buildings subjected to ground-motion records with pulse

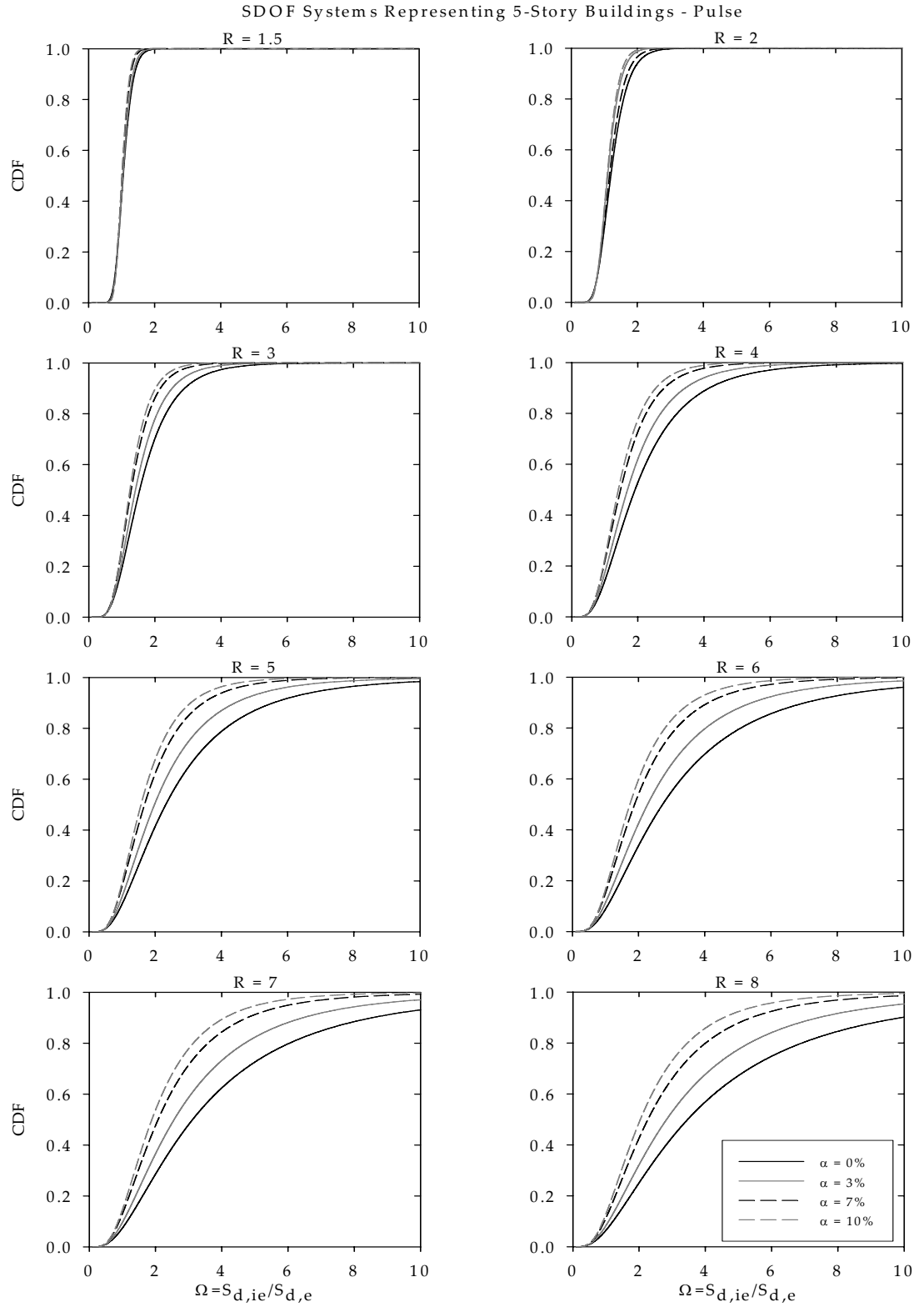


Figure 5.12 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 5-story buildings subjected to ground-motion records with pulse

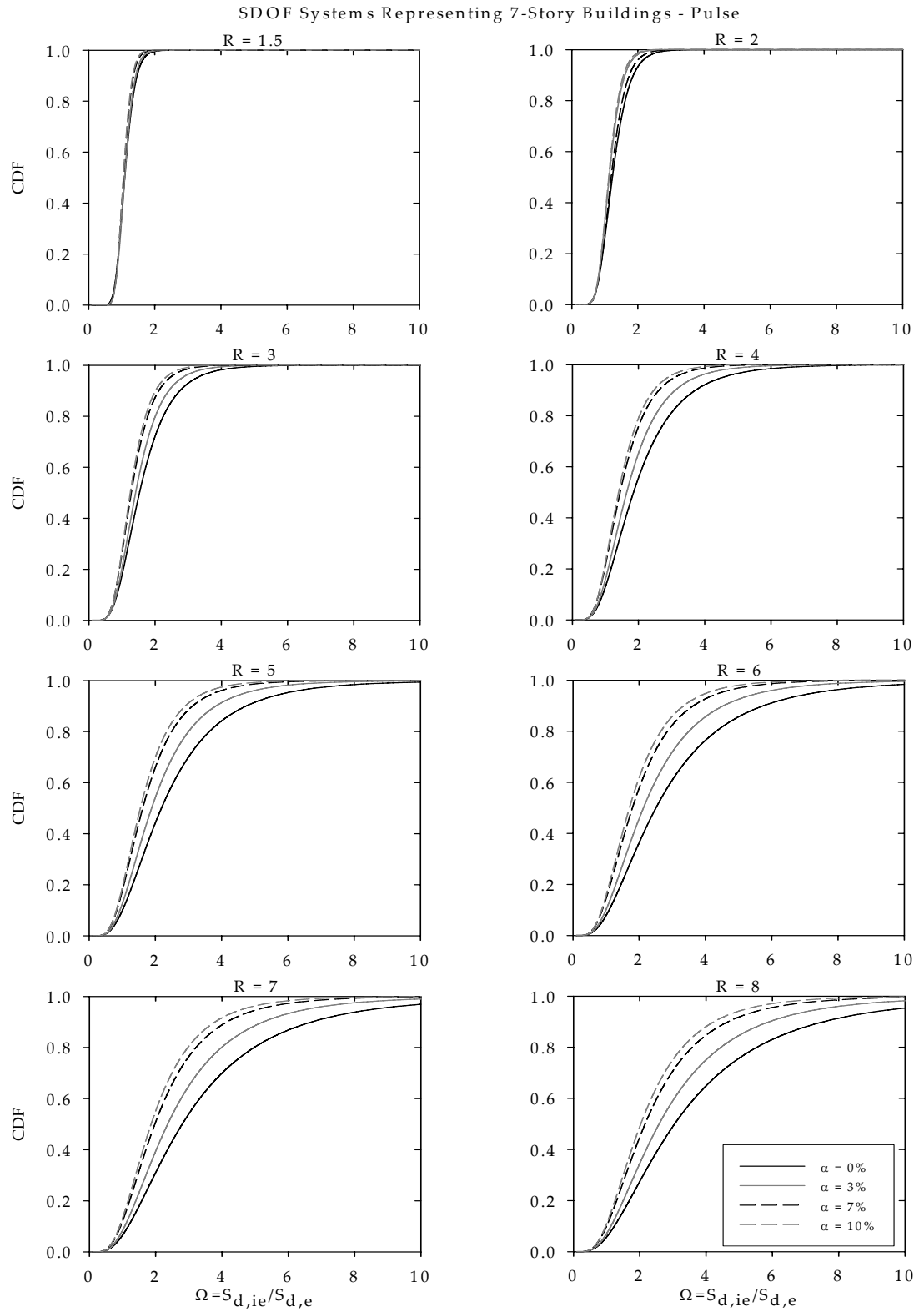


Figure 5.13 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 7-story buildings subjected to ground motion-records with pulse

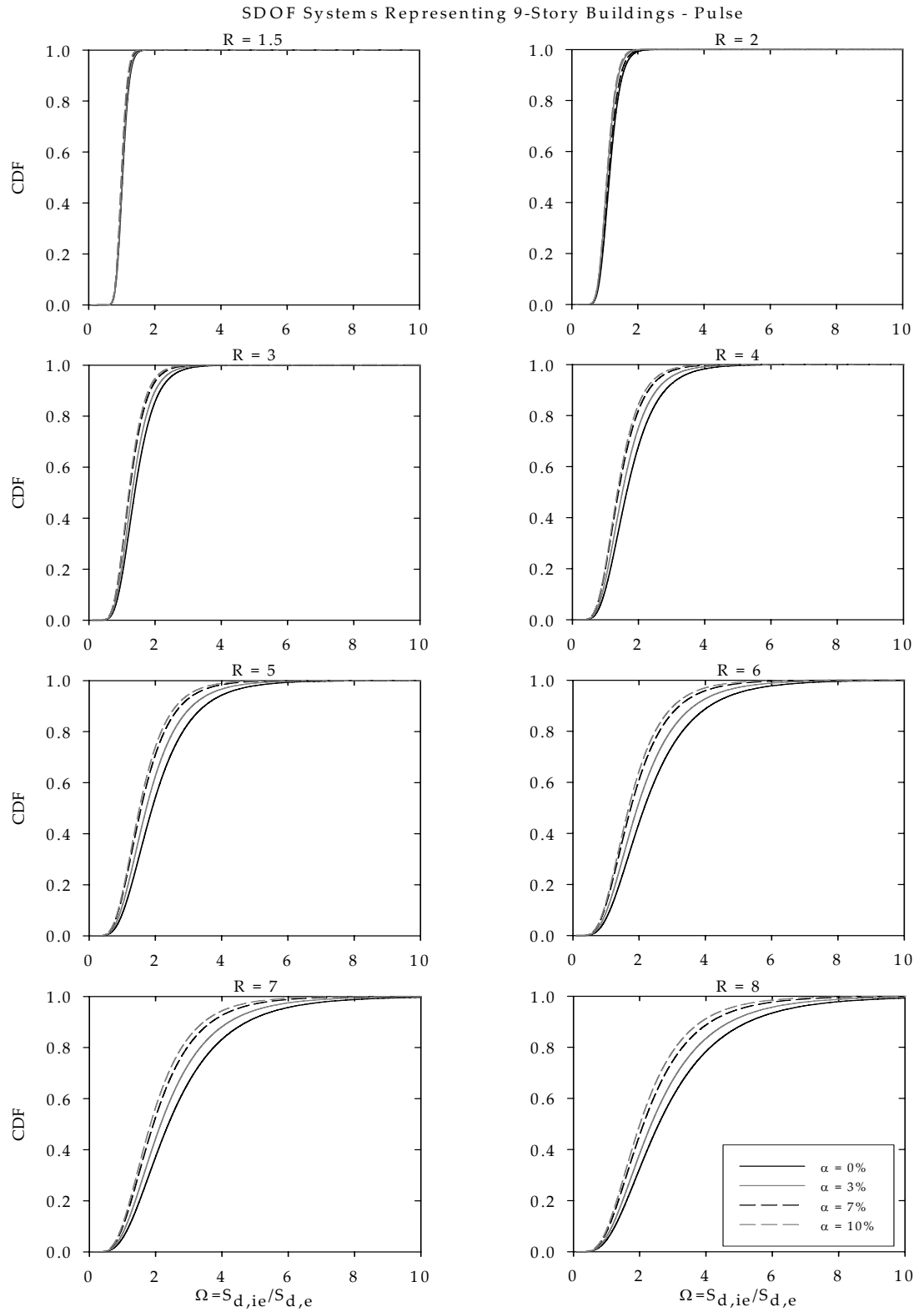


Figure 5.14 Effects of α and R on $\Omega = S_{d,ie} / S_{d,e}$ for 9-story buildings subjected to ground-motion records with pulse

5.5.3 Modified form of Akkar et al. (2005a) for the estimation of $MIDR_{ie}$ in a probabilistic manner

The modification of the proposed procedure by Akkar et al. (2005a) is based on the presumption that the inelastic $MIDR$ can be estimated fairly well if accurate inelastic spectral displacements are provided (Section 5.4). The elastic $MIDR$ expression presented by Akkar et al. (2005a) can be rewritten in the following form to estimate the inelastic $MIDR$:

$$MIDR_{ie}(T_1, \xi, \rho, \alpha, R) = \gamma_1(\rho, T_1) \gamma_2(\rho, T_1) 1.27 \sin\left(\frac{\pi h}{2H}\right) \frac{S_{d,ie}(T_1, \xi, \alpha, R)}{h} \quad (5.13)$$

If the inelastic spectral displacement is estimated using the probabilistic Ω factor presented in this section:

$$MIDR_{ie}(T_1, \xi, \rho, \alpha, R, k) = \gamma_1(\rho, T_1) \gamma_2(\rho, T_1) 1.27 \sin\left(\frac{\pi h}{2H}\right) \frac{\Omega(\alpha, R, k) S_{d,e}(T_1, \xi)}{h} \quad (5.14)$$

The above expression can be simplified further as presented in Equation (5.15) since Equation (5.14) indicates that except the modified spectral displacement the rest of the parameters do not vary for inelastic behavior. This assumption is gross but it seems to be valid for the ground motions and buildings investigated in this study (Section 5.4).

$$MIDR_{ie}(T_1, \xi, \rho, \alpha, R, k) = \Omega(\alpha, R, k) MIDR_e(T_1, \xi, \rho) \quad (5.15)$$

The final form presented in Equation (5.15) emphasizes that the inelastic $MIDR$ can be estimated from the elastic $MIDR$ using the modification factor, Ω , derived for inelastic spectral displacement in Equation (5.7). This is also consistent with the

statistics presented in Chapter 3 since the probability distributions of $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$ reveal a very close match. Note that this assumption is strictly confined to the structures dominated by the first-mode behavior.

The median $MIDR_{ie}$ variation and uncertainty associated due to the contributions of Ω and $MIDR_e$ can be estimated according to the procedure explained below.

As stated in the previous sections, Ω and $MIDR_e$ estimations by Akkar et al. (2005a) fit well to log-normal distribution. The logarithm of a log-normally distributed random variable is normally distributed (Ang and Tang, 1975). Thus, taking the logarithm of both sides in Equation (5.15) one can compute the mean (i.e., $\lambda_{MIDR_{ie}}$) and the variance (i.e., $\zeta_{MIDR_{ie}}^2$) of $MIDR_{ie}$ as shown in Equations (5.17) and (5.18) provided that Ω and $MIDR_e$ are statistically independent. In Figure 5.15, the correlation between $MIDR_e$ and Ω is displayed for all building groups. It is seen that for almost all cases the coefficient of correlation is small suggesting that these two variables can be accepted as statistically independent.

$$\ln[MIDR_{ie}] = \ln[\Omega] + \ln[MIDR_e] \quad (5.16)$$

$$\lambda_{MIDR_{ie}} = \lambda_{\Omega} + \lambda_{MIDR_e} \quad (5.17)$$

$$\zeta_{MIDR_{ie}}^2 = \zeta_{\Omega}^2 + \zeta_{MIDR_e}^2 \quad (5.18)$$

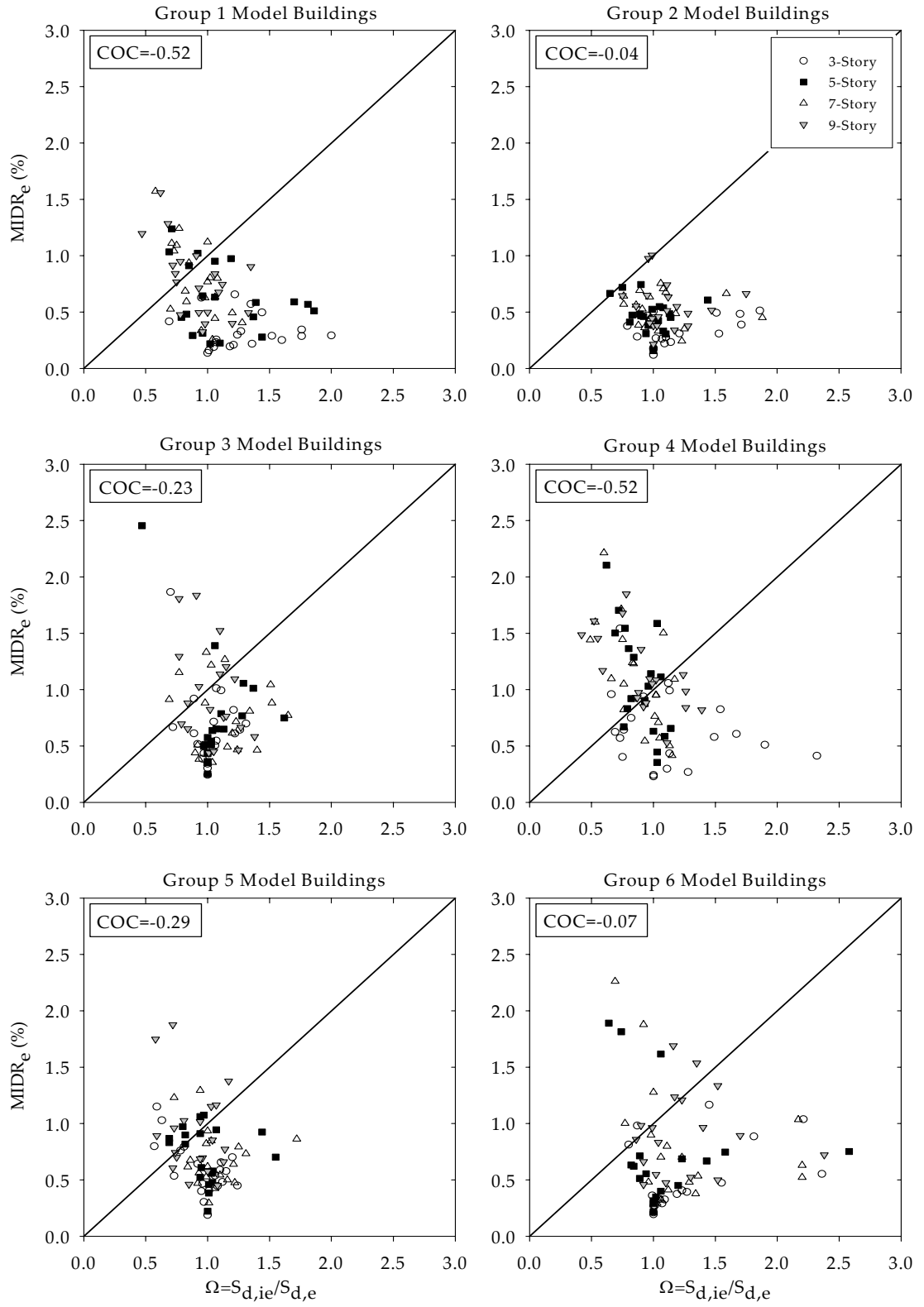


Figure 5.15 $MIDR_e$ versus Ω plots for the model buildings

The parameters λ_{Ω} and λ_{MIDR_e} represent the log-normal means of Ω and $MIDR_e$. These parameters are computed by using Equation (5.7) and (5.1), respectively. Similarly, ζ_{Ω}^2 and $\zeta_{MIDR_e}^2$ represent the log-normal variances of Ω and $MIDR_e$, respectively. The variance $\zeta_{MIDR_e}^2$ is computed from the log-normal statistics of Akkar et al. (2005a) presented in Figure 5.3. The variance ζ_{Ω}^2 is calculated by using Equation (5.8).

The log-normal median value of $MIDR_{ie}$ can be obtained by computing the exponents of each side of Equation (5.17) and the standard deviation of $MIDR_{ie}$ can be obtained by taking the square root of $\zeta_{MIDR_{ie}}^2$ term.

5.6 An illustrative example to demonstrate the probabilistic estimation of $MIDR_{ie}$ using the proposed procedure

This section presents the $MIDR_{ie}(T_1, \xi, \rho, \alpha, R, k)$ estimations for frame-type building behavior for $R = 2$ and $R = 4$ (the R -range covered by the model buildings used in the study). The post-yield stiffness ratio is chosen as $\alpha = 5\%$ for the illustrative case study and ρ is assumed as 0.5 that is of typical for moment-resisting frames. The inelastic $MIDR$ estimations are done for buildings of 3 to 9 number of stories that are subjected to ground motions without pulse. This is the interval of number of stories investigated in this study.

The median variation of $MIDR_{ie}$ is computed using the expressions presented in Equation (5.17). The uncertainty on the median $MIDR_{ie}$ estimation is considered using the standard deviation of each contributing term to $MIDR_{ie}$ (Equation (5.18)).

The elastic displacement spectrum that accounts for the variation of elastic spectral displacements as a function of vibration period is given in Figure 5.16. It is the log-normal median spectrum of Group A and B ground motions that represent the near-fault records without pulse in this study.

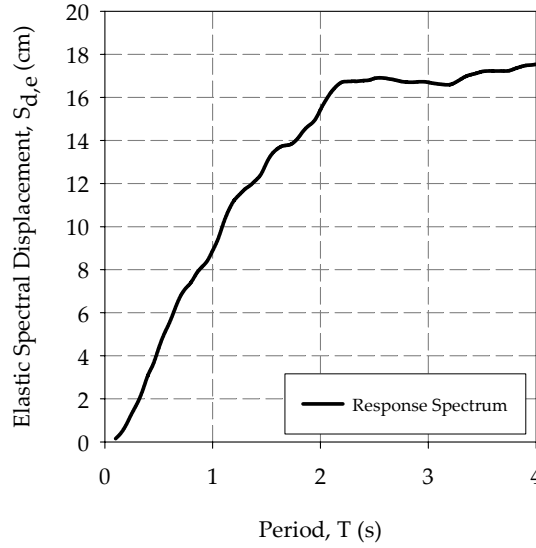


Figure 5.16 Log-normal median elastic displacement spectrum for no-pulse records

The median variation of $MIDR_e$ that is computed by the original expression derived in Akkar et al. (2005a) is given in Figure 5.17. The elastic $MIDR$ estimations for each story is computed by using the predefined fundamental period ranges described in the derivation of the probabilistic approach (i.e., $T_1 = 0.3 - 0.55$ s for 3-story buildings, $T_1 = 0.5 - 0.8$ s for 5-story buildings, $T_1 = 0.75 - 1.05$ s for 7-story buildings and $T_1 = 0.9 - 1.3$ s for 9-story buildings). The upper and lower bounds for the elastic $MIDR$ estimations are presented by the solid lines. The estimations for short structures seem to have relatively large scatter and reveal large values that is expected as the original expression for $MIDR_e$ tends to overestimate the

maximum interstory drift demands for short structures as discussed by Akkar et al. (2005a). The uncertainty due to this overestimation is considered in the inelastic $MIDR$ estimations as discussed in the following paragraphs.

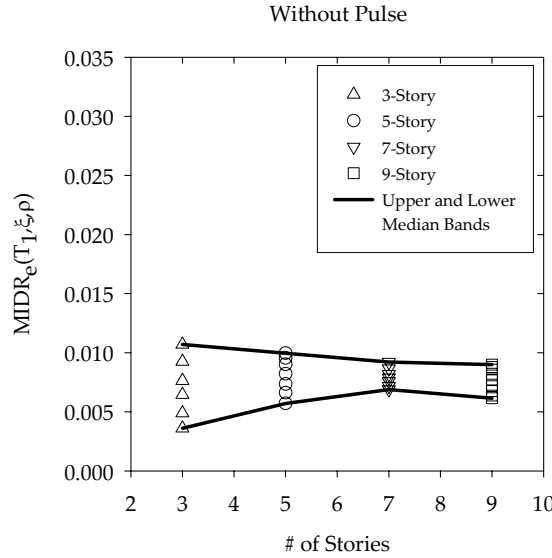


Figure 5.17 Median values of $MIDR_e(T_1, \xi, \rho)$ for 3-, 5-, 7- and 9-story buildings subjected to ground motions without pulse

The median elastic $MIDR$ values are modified for the median $MIDR_{ie}$ using the Ω modifying factor distributions via Equation (5.7) for $R = 2$ and $R = 4$ for a post-yield stiffness ratio of 5%. The results are presented in Figure 5.18 as scatter plots including the upper- and lower-bound median curves described by solid lines. The uncertainty associated with the inelastic $MIDR$ predictions due to the performance of the empirical equations employed and random features of ground motions are also considered by plotting the 16-percentile and 84-percentile envelop curves using Equation (5.18). Thus, the results presented in this figure accounts for all sources of uncertainty (epistemic and aleatory) that one must consider for a reasonable

judgment of structural damage related studies. The actual $MIDR_{ie}$ scatters that are calculated from the nonlinear RHA are also superimposed on these plots in order to see how the proposed procedure envelop the overall variation of $MIDR_{ie}$. Note that these actual $MIDR_{ie}$ values were also presented in Figure 5.4 (the left most upper panel). The plots in Figure 5.18 indicate that the upper and lower median bands as well as the 16-percentile and 84-percentile curves envelop the actual variation of data fairly well. Note that some portion of the actual data is still outside these curves. This can be expected as the data represent the overall random variation of $MIDR_{ie}$ values computed in this study. In other words the actual $MIDR_{ie}$ values presented are not only confined to $R = 2$ and $R = 4$ with a post-yield stiffness ratio of $\alpha = 5\%$. They represent a dispersive variation of R – and α – values. Note that $MIDR_{ie}$ values are overestimated for 3-story structures particularly for $R = 4$. This can be attributed to the sensitivity of short-period structures to the changes in lateral strength capacity as well as the tendency of Akkar et al. (2005a) predictions towards safe estimations, for small period structures. The overall picture revealed by the inelastic $MIDR$ estimations suggests that an upper limit of 1.5% interstory drift ratio bounds most of the uncertainties associated with inelastic $MIDR$ estimations for buildings of 5- to 9-stories when they are subjected to near-fault records without pulse. This limitation is valid for well-behaved buildings (i.e., structures that do not experience severe degradation during cyclic excursions) with relatively higher strength capacity.

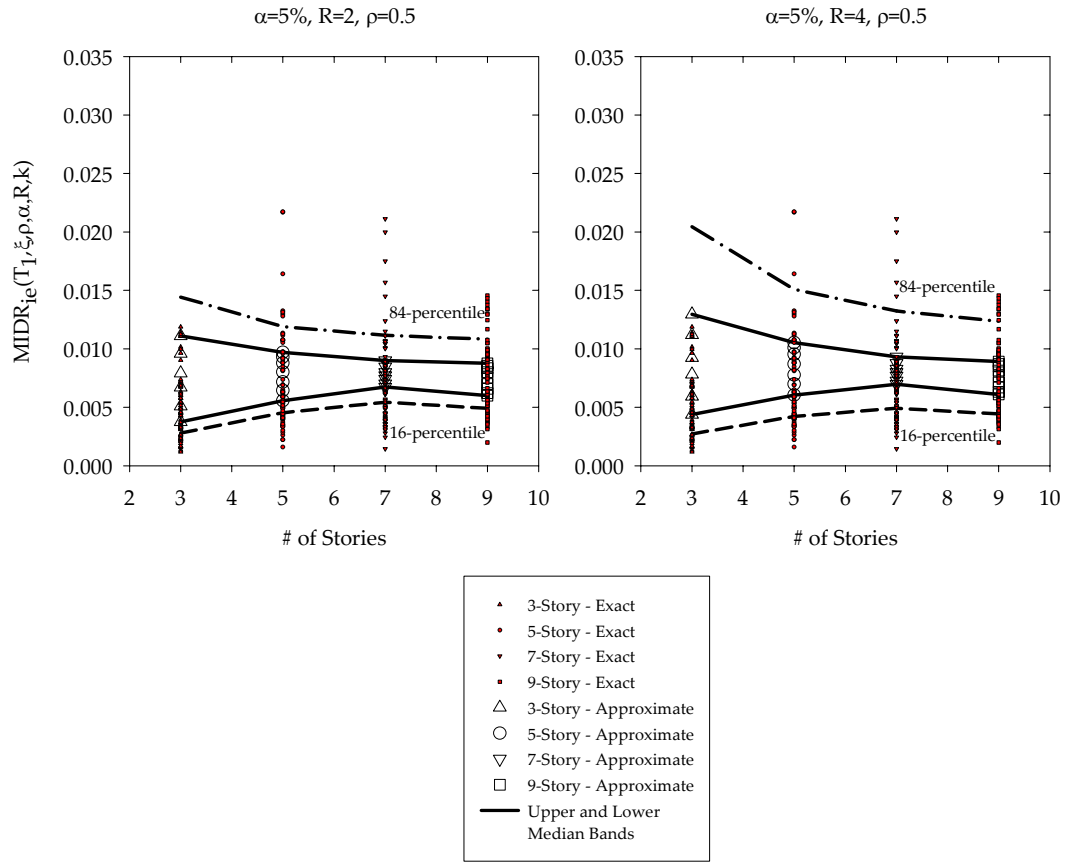


Figure 5.18 Median values of $MIDR_{ie}(T_1, \xi, \rho, \alpha, R, k)$ together with the 16- and 84-percentile bands and actual $MIDR_{ie}$ scatters

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 General

The main subject of this study was to propose a probabilistic method to estimate the inelastic *MIDR* that is an important parameter for the assessment of the structural damage in frame buildings under earthquakes. This method considers the uncertainties associated with important structural parameters and ground motions that are important for structural damage assessment.

The secondary objective of this study was to describe the nonlinear deformation demands on frame buildings in a statistical manner. The relationships of nonlinear deformation demands with lateral strength capacity, post-yield stiffness ratio and important features of near-fault ground motions were investigated in detail. The statistical results derived from this objective served for the probabilistic procedure presented in the study.

As part of this thesis, recently improved approximate procedures that are presented in the FEMA-440 document (ATC, 2005) for estimating global deformation demands were verified by using the model buildings in order to point out some deficiencies of the NSPs.

6.2 Summary and conclusions of Chapter 2

In Chapter 2, detailed information was given about the selection and grouping of ground motions and modeling and design of buildings. The important seismological features of the chosen ground motions, their relations with each other and with some important model building features were also presented.

The study made use of 78 near-fault ground-motion records (40 records do not contain dominant pulse signals whereas 38 records contain dominant pulse waveforms) and 24 model frames to achieve these objectives. The ground motions were divided into 4 subgroups (i.e., A, B, C and D) considering the *PGV* ranges and whether they contain pulse signals or not. Ground-motion records with and without pulse signals were assembled in different groups in order to investigate the prominent features of pulse signals on structural behavior. Groups A and B, each having 20 records, were assembled from records without pulse and they have *PGV* ranges of 40-60 cm/s and 20-40 cm/s, respectively. Groups C and D have 20 and 18 ground motions, respectively, and they were assembled from records with dominant pulse signals. Group C ground motions have a *PGV* range between 40-60 cm/s similar to Group A, whereas Group D ground motions have *PGV* values greater than 60 cm/s. These ground-motion records were used to design 6 building groups (i.e., Groups 1-6). Each group consists of 4 model buildings with 3-, 5-, 7- and 9-story levels. Groups 1 and 4 buildings were designed for the median design spectrum of Group A records. The difference between these building groups is that the fundamental periods of Group 4 models are larger than those of Group 1 models (i.e., Group 1 models are relatively rigid than Group 4 models). Groups 2 and 5 buildings were designed for the median design spectrum of Group B records. Similar to Groups 1 and 4 buildings, Group 5 buildings represent a more flexible behavior with respect to Group 2 building models. Groups 3 and 6 building models were designed for the median design spectra of Group C and D records,

respectively. Group 6 building models reveal larger fundamental-mode periods with respect to those of Group 3 models. The fundamental period range covered in this study simulate the upper and lower bound empirical relationships proposed by Chopra and Goel (2000a). The most important observations obtained from this chapter are listed below.

1. *PGV* and magnitude values of the ground motions are directly proportional with each other, whereas *PGV* and distance values are inversely proportional.
2. The direct proportionality between *PGV* and magnitude is not seen for the other important ground-motion intensity parameter; *PGA*. Ground-motion groups having larger magnitude values yield lower *PGA* values. This can be attributed to the soil nonlinearity effects.
3. The corner period values are consistent with the level of *PGV* presented for each group. Larger *PGV* yields larger T_c value.
4. The ground motions with high *PGV* values (or higher magnitude values) are more likely to have rich long-period *PSA* values.
5. The chances of fundamental periods getting closer to the pulse periods of the records in this study is considerably small. This suggests that the nonlinear behavior of the model buildings will be directly influenced by the first mode that is also indicated by other researchers.
6. Pushover analyses of the model buildings that were based on inverted triangle loading (i.e., first-mode dominant building behavior) fortified. Comparison of pushover curves with the absolute maximum roof displacement versus base

shear plots obtained from RHA revealed a fairly good agreement indicating that the model buildings are predominantly deform under first-mode influence.

6.3 Summary and conclusions of Chapter 3

The results of RHA for building models and corresponding equivalent SDOF systems whose calculations were explained in detail in Chapter 2 were reported in Chapter 3 in a statistical manner. The median and different percentile variations of lateral displacement profiles, interstory drift demands as well as the associated dispersions along the building height both in the elastic and the inelastic ranges were presented. The relationships between normalized strength capacity (γ), *MIDR* and *MRDR* of building models were investigated and information on the normalized lateral strengths of corresponding equivalent SDOF systems were given. The correlations between the deformation demands computed from the building models and corresponding equivalent SDOF systems were also presented. This chapter also evaluated the current approach presented in the approximate NSPs for transforming the SDOF deformation demands to corresponding MDOF deformation demands. The most important observations obtained from this chapter are listed below.

Observations related to the elastic and inelastic lateral displacement profiles:

1. Median inelastic lateral displacements of 7- and 9-story models that are subjected to ground motions without pulse are less than their elastic counterparts. For the same number of stories, the model buildings which are subjected to pulse-dominant records show higher median inelastic lateral displacements with respect to their elastic counterparts for almost all cases. This can be attributed to the fact that the fundamental periods of these buildings are

still less than the pulse periods of the chosen pulse-dominant records that result in larger deformation demands in the inelastic range which is an observation that is also highlighted by the previous research.

2. The short-period and stiff nonlinear systems are more vulnerable to near-fault ground motions whether they are dominated by pulse or not. The median nonlinear deformation demands on short-period structures are generally higher than the corresponding elastic demands. For short period structures subjected to records with pulse this observation can be explained as discussed in item (1): regardless of the fundamental period value if it is less than the corresponding pulse period, the inelastic deformations are amplified.
3. The median nonlinear roof displacement demand is generally less than or equal to its elastic counterpart. The only exception is the Group 6 building models that were subjected to the most severe near-fault ground-motion group both in terms of magnitude, distance and *PGV* (i.e. Group D ground motions).

Observations related to the elastic and inelastic interstory drift ratio profiles:

1. The median elastic *MIDR* migrates towards higher story levels for model buildings with uniformly decreasing lateral stiffness regardless of the near-fault ground-motion type. This observation is particularly true as number of stories increases suggesting the influence of higher modes for these cases in the elastic range.
2. The observation in item (1) does not apply to the building models with uniform lateral stiffness where the median elastic *MIDR* mostly occurs in the lower portion of the building height.

3. The variations in interstory drift ratios along the building height are more predictable for nonlinear behavior. The median interstory drift ratio pattern resembles a first-mode dominant pattern for structures behaving in the nonlinear range. For most cases, $MIDR_{ie}$ is located at the lower stories regardless of the lateral stiffness variation or near-fault ground-motion type.
4. The median $MIDR_{ie}$ is mostly greater than the median $MIDR_e$ for building models subjected to near-fault records with pulse. The explanations given in items (1) and (2) on pages 168 and 169, respectively, are also valid for this case.

Observations related to the dispersions on elastic and inelastic story displacement profiles and interstory drift ratio profiles:

1. In general the elastic and inelastic COV for lateral displacements and interstory drift ratios follow each other at lower stories.
2. The COV of interstory drift ratios at the roof level is significantly lower than the corresponding COV values at the lower portion of buildings. This is particularly true for nonlinear behavior.
3. In general, the COV of lateral displacements tends to decrease towards roof particularly for nonlinear behavior.
4. When items (2) and (3) are evaluated together, one can conclude that the inelastic roof displacement demand parameter draws a less dispersive character with respect to the interstory drift demands (in particular $MIDR_{ie}$) as $MIDR_{ie}$ is mostly located in the lower portion of the buildings.

5. In general, the COV of inelastic lateral displacement and interstory drift ratios for structures with uniformly decreasing lateral stiffness are larger (if not equal) than the corresponding elastic COVs. This observation is particularly valid at the lower story levels for the ground-motion groups used in this study.
6. For structures with uniform lateral stiffness in most cases, the COVs for inelastic lateral displacements and interstory drift ratios are equal or less than the corresponding COVs of elastic behavior.

Observations related to the roof displacement of MDOF systems and equivalent SDOF systems in both elastic and inelastic ranges and comparisons with the FEMA-356 document (ASCE, 2000) for transforming SDOF demands to MDOF demands are listed below.

1. The median $\Delta_{roof,e} / S_{d,e}$ are generally in good agreement with the $\Gamma_1 \phi_1^{roof}$ and the alternatively tabulated constants of the FEMA-356 document (ASCE, 2000) except for the 3-story tabulated constants that display a slightly lower bound.
2. The median $\Delta_{roof,ie} / S_{d,ie}$ ratios reveal that both $\Gamma_1 \phi_1^{roof}$ and the alternatively tabulated constants that depend on number of stories mostly constitute an upper bound for relating the equivalent SDOF displacements to roof displacements.
3. Items (1) and (2) yield that the median $\Delta_{roof,ie} / S_{d,ie}$ ratios are less than median $\Delta_{roof,e} / S_{d,e}$ ratios.

Observations on the relationships between normalized strength capacity (γ), *MIDR* and *MRDR* of building models:

1. For almost all building models, increase in normalized strength capacity, γ , implies a fair decrease in nonlinear roof displacements. The exceptional cases are some of the stiff buildings that show an increase in nonlinear deformation demands with increasing γ . This might suggest the sensitivity of stiff systems to the changes in lateral strength capacity. The other exceptional case is the Group 6 building models that are subjected to the most severe near-fault ground-motion records with dominant pulse signals in this study.
2. The nonlinear interstory drift ratio demands display a decreasing trend with increasing γ similar to nonlinear roof drift ratio. In particular $MIDR_{ie} / MIDR_e$ tends to be larger than 1 (or approximately follows a value of 1) for ground motions with pulse.
3. $MIDR_{ie} / MIDR_e$ ratios are slightly larger than the corresponding $MRDR_{ie} / MRDR_e$ ratios.

Observations on the normalized lateral strength, R , of equivalent SDOF systems:

1. Log-normal distribution resembles good fits to the observed R – distributions.
2. Increase in number of stories implies an increase in R – range.
3. For uniform lateral stiffness models the variations of R follow a very close pattern for 5-, 7- and 9-story building models.

Observations on the deformation demands computed from the MDOF systems and corresponding equivalent SDOF systems:

1. Log-normal distribution have a fairly well match with the actual distributions of $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$ and the level of match increases as number of stories increase.
2. There is a fairly well correlation between inelastic to elastic SDOF and MDOF deformation demand ratios. This observation suggests that the variations in inelastic to elastic MDOF deformation demand can be extracted from the corresponding SDOF deformation ratios for structures primarily deforming under the first-mode influence.
3. $S_{d,ie} / S_{d,e}$, $MRDR_{ie} / MRDR_e$ and $MIDR_{ie} / MIDR_e$ ratios attain larger values for building models subjected to Group D ground motions that is the most severe ground-motion group in terms of magnitude, distance and PGV .

6.4 Summary and conclusions of Chapter 4

The improved DCM and CSM presented in the FEMA-440 document (ATC, 2005) were evaluated in Chapter 4. The results were presented in a statistical manner. The most important observations obtained in this chapter are summarized below.

1. $S_{d,ie} / S_{d,e}$ ratios show a clear dependence on earthquake intensity measures (PGA and PGV) and post-yield stiffness ratio, α .
2. Inelastic deformation demands decrease as α increases for all ground-motion groups.
3. For buildings that have elastoplastic behavior and subjected to ground motions without pulse, C_1 may describe the median $S_{d,ie} / S_{d,e}$ variation fairly well or it

may underestimate it. However, as α takes non-zero values C_1 starts to overestimate the median $S_{d,ie} / S_{d,e}$ variations. This is the result of the observation stated in item (2).

4. For buildings subjected to ground-motions that are dominated by pulse signals, C_1 underestimates the $S_{d,ie} / S_{d,e}$ ratio regardless of the variation in α .
5. For all ground-motion bins, the median variations of $S_{d,ie} / S_{d,e}$ do not show significant variations for $\alpha = 6\%$ and $\alpha = 10\%$.
6. The flip-flop behavior (non-convergency) that occurs frequently in CSM can be handled by applying the bisection method.
7. The improved DCM results more conservative estimations with respect to the improved CSM for the maximum displacements of equivalent SDOF systems and *MRDR*. In almost all cases, the *MRDR* estimations are 10% larger with respect to those of the maximum equivalent SDOF system displacements.
8. The improved NSPs tend to underestimate the *MIDR* particularly for increasing periods. The coefficient method draws a more erratic pattern in the median *MIDR* estimations with respect to the improved CSM.
9. There is a close resemblance between *R* – distributions and the equivalent maximum SDOF displacement and *MRDR* error statistics for improved DCM. The increase in median *R* generally results in an increase in the median error (=approximate/exact) for these demand parameters implying the sensitivity of improved DCM performance to the normalized lateral strength (mostly valid for without pulse cases). This is not observed for the improved CSM whose predictions are based on the individual ground motions whereas improved

DCM employs generic R – dependent empirical modifying coefficients representing the general tendency of a suite of ground motions.

6.5 Summary and conclusions of Chapter 5

This chapter presented a probabilistic procedure for estimating $MIDR_{ie}$ by modifying the corresponding elastic deformation demand by incorporating the variations in R , α and number of stories in a probabilistic manner. The proposed procedure also accounts for the influence of pulse but it could not be fully evaluated due to lack of data that would yield reliable statistical results. The most important observations obtained in this chapter are summarized below.

1. The exact $MIDR_{ie}$ values obtained by nonlinear RHA and approximate $MIDR_{ie}$ computed from Akkar et al. (2005a) method (by replacing $S_{d,e}$ with $S_{d,ie}$) are very similar to each other. However, $MIDR_{ie}$ values obtained by Medina and Krawinkler (2005) study display departures from the exact $MIDR_{ie}$ associated with significant dispersion. This is because of the fact that Medina and Krawinkler (2005) study does not consider the particular variation of R and α in calculation of $MIDR_{ie}$.
2. The proposed procedure is verified using the nonlinear RHA results computed within the context of this study. It is observed that this new procedure predicts the $MIDR_{ie}$ fairly well for 5-, 7- and 9-story buildings. For 3-story buildings it overestimates the $MIDR_{ie}$ particularly for increasing R – values. The reason of this overestimation is thought to be related to the sensitivity of short-period structures to the changes in lateral strength capacities as well as the tendency of

$MIDR_e$ predictions by Akkar et al. (2005a) procedure towards safe estimations for short-period structures.

6.6 Suggested future research

The possible continuation of this study can address the following issues in a broader sense.

1. The probabilistic procedure that is the final product of this study can be improved further for degrading systems.
2. A calibration function can be adapted to the proposed probabilistic procedure to acquire better accuracy for the $MIDR_{ie}$ estimation of short structures.
3. A more systematic study would address the verification of the proposed procedure for research with pulse dominant signals.

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APPENDIX A

RELEVANT FIGURES OF CHAPTER 2

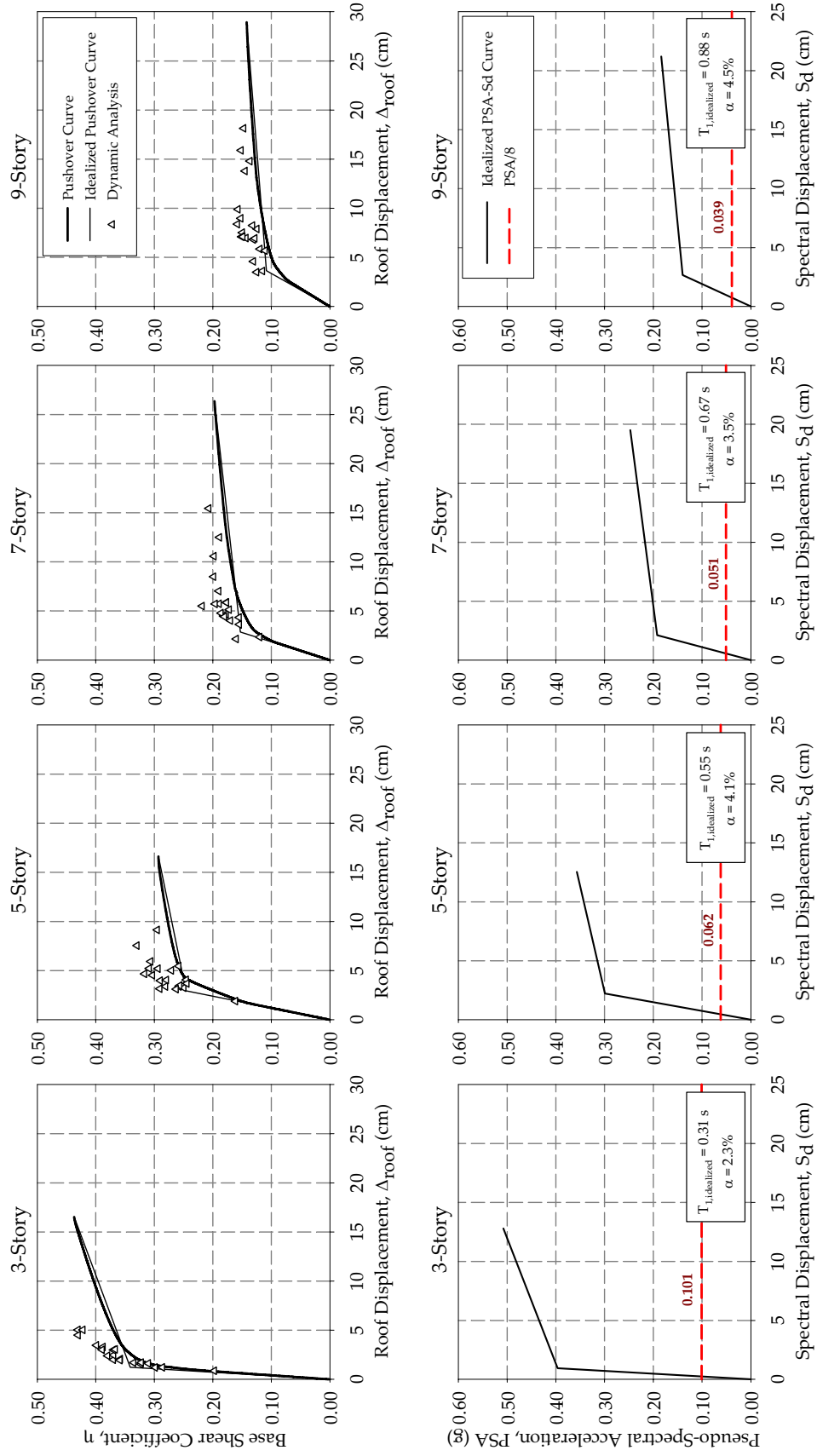


Figure A.1 Pushover curves and ADRS of Group 2 model buildings

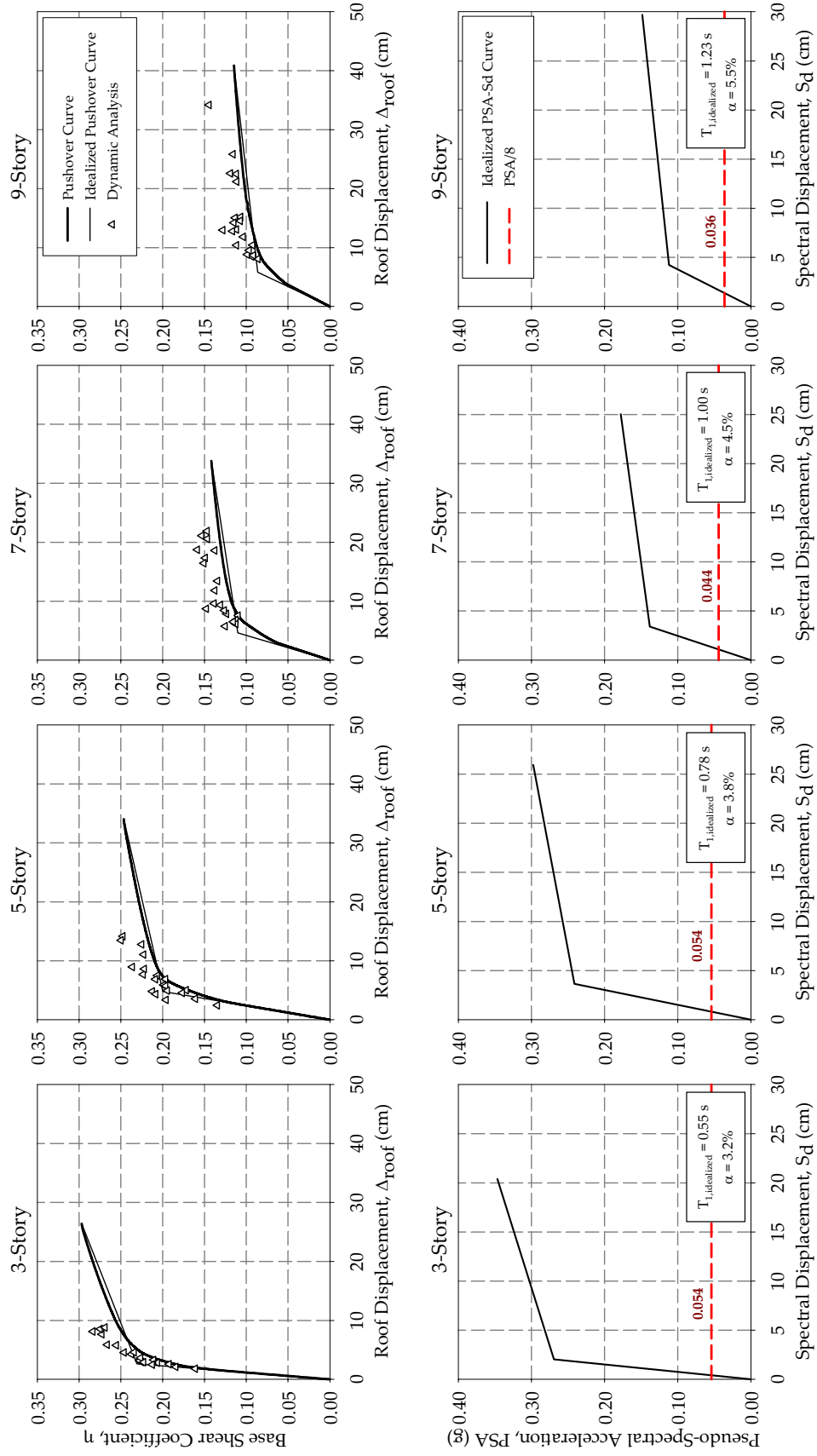


Figure A.2 Pushover curves and ADRS of Group 3 model buildings

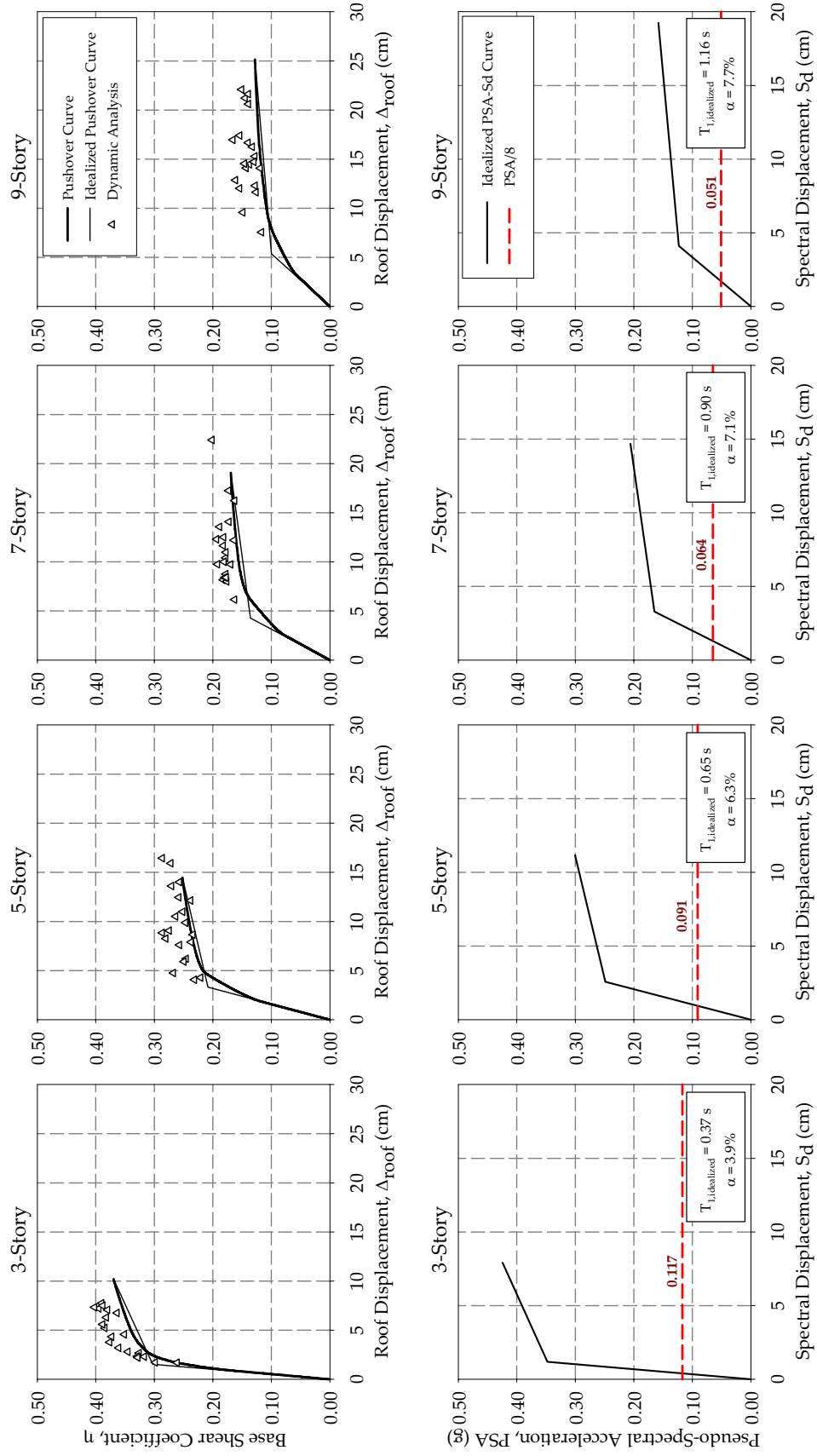


Figure A.3 Pushover curves and ADRS of Group 4 model buildings

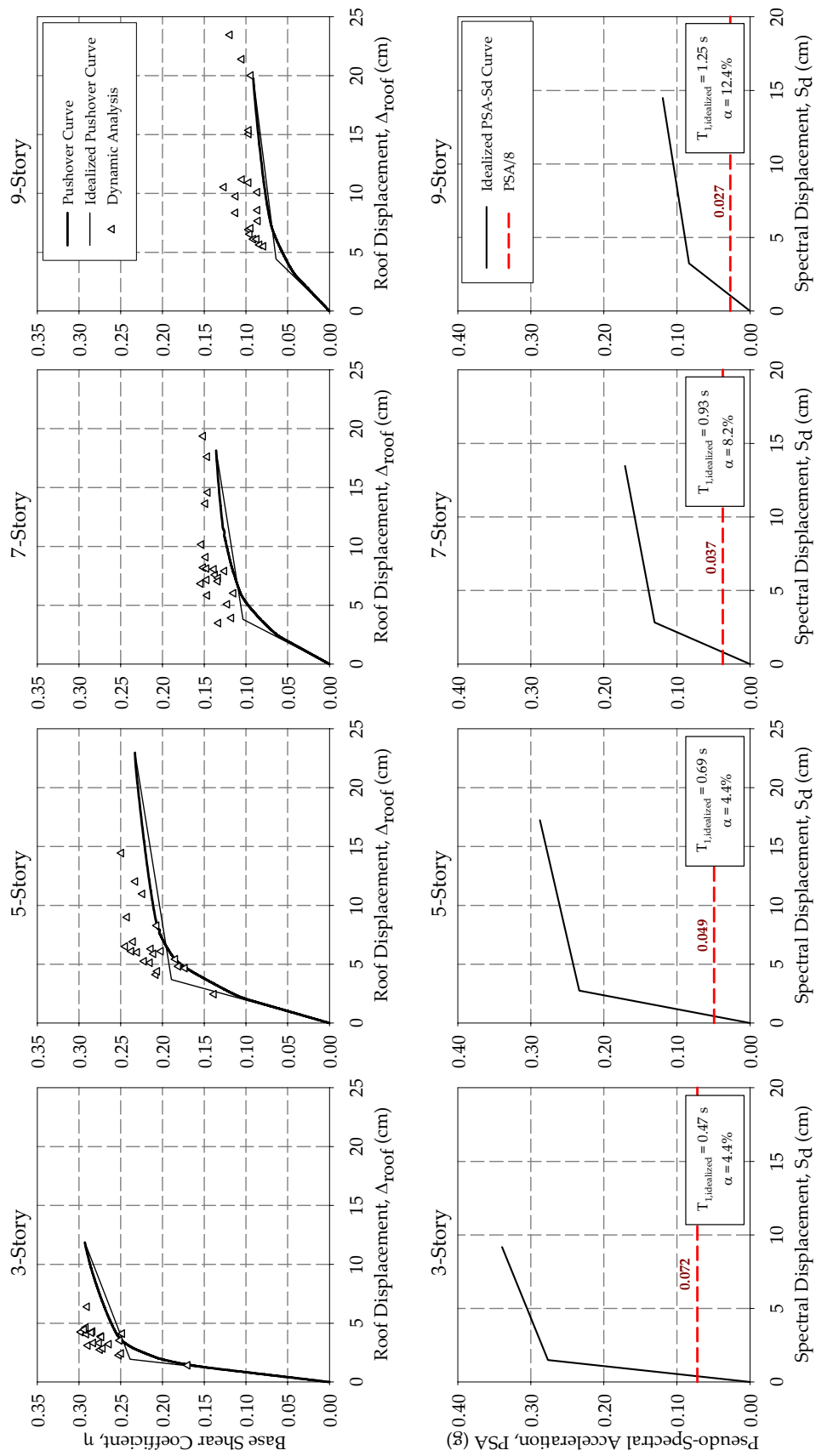


Figure A.4 Pushover curves and ADRS of Group 5 model buildings

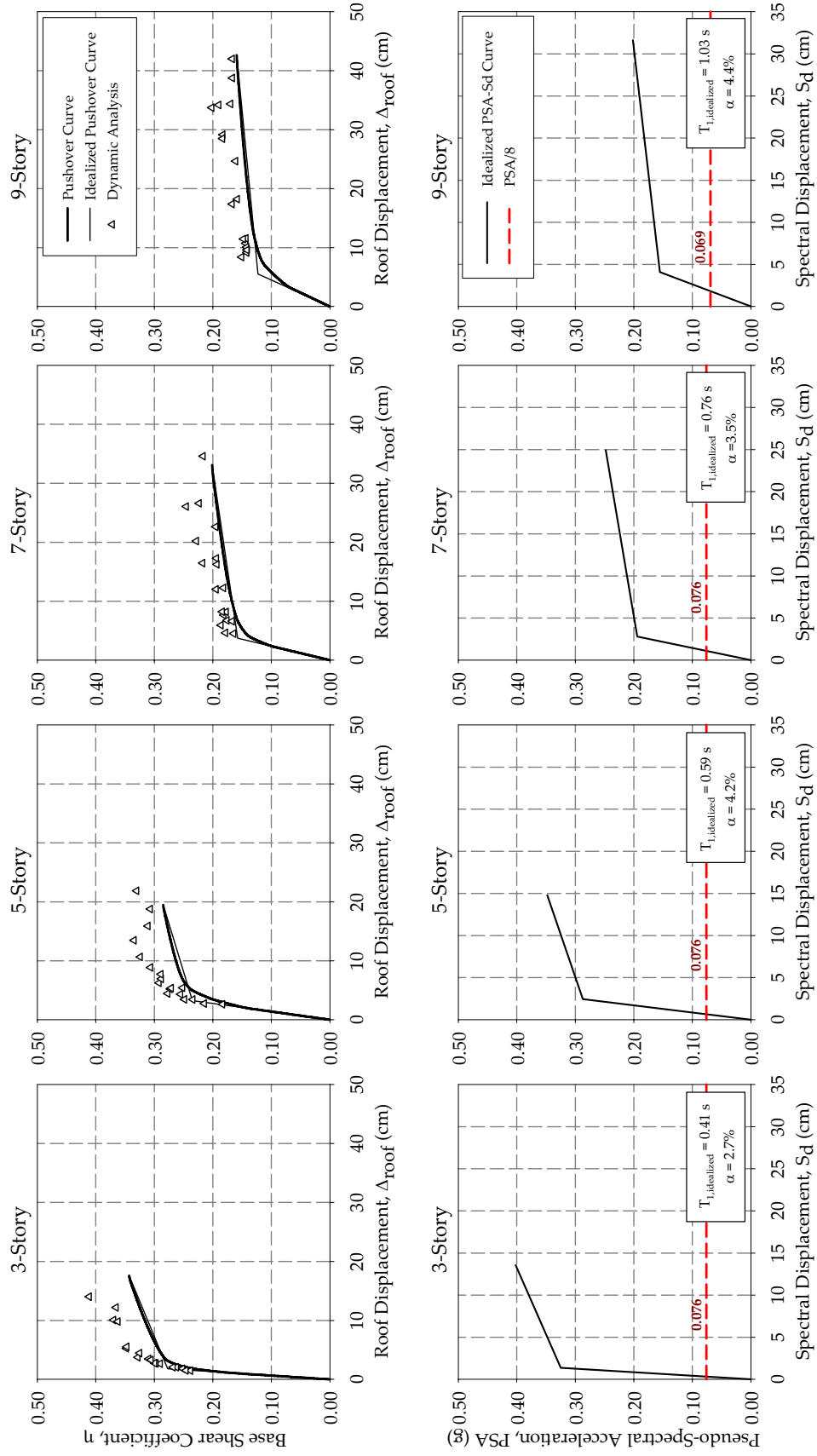


Figure A.5 Pushover curves and ADRS of Group 6 model buildings

APPENDIX B

RELEVANT TABLES AND FIGURES OF CHAPTER 3

Table B.1 Relationships between the maximum elastic and inelastic roof and spectral displacements, Group 1

Record #	3-Story ($T_1=0.27$ s, $T_{1,idealized}=0.27$ s)				5-Story ($T_1=0.45$ s, $T_{1,idealized}=0.44$ s)				7-Story ($T_1=0.63$ s, $T_{1,idealized}=0.64$ s)				9-Story ($T_1=0.82$ s, $T_{1,idealized}=0.82$ s)			
	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_1 \phi_1^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_1 \phi_1^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_1 \phi_1^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_1 \phi_1^{roof}$	FEMA
1	1.30	1.26	1.29	1.20	1.36	1.30	1.30	1.30	1.33	1.12	1.30	1.30	1.34	1.21	1.31	1.30
2	1.19	1.14	1.29	1.20	1.31	1.24	1.30	1.30	1.26	1.00	1.30	1.30	1.31	1.25	1.31	1.30
3	1.41	1.23	1.29	1.20	1.41	1.15	1.30	1.30	1.37	1.14	1.30	1.30	1.27	1.11	1.31	1.30
4	1.24	1.15	1.29	1.20	1.41	1.44	1.30	1.30	1.29	1.20	1.30	1.30	1.22	1.42	1.31	1.30
5	1.24	1.24	1.29	1.20	1.37	0.99	1.30	1.30	1.18	1.42	1.30	1.30	1.33	1.20	1.31	1.30
6	1.26	1.11	1.29	1.20	1.36	1.04	1.30	1.30	1.41	1.37	1.30	1.30	1.41	1.32	1.31	1.30
7	1.30	1.19	1.29	1.20	1.36	1.19	1.30	1.30	1.16	0.99	1.30	1.30	1.37	1.19	1.31	1.30
8	1.25	0.91	1.29	1.20	1.27	1.32	1.30	1.30	1.36	0.95	1.30	1.30	1.31	1.09	1.31	1.30
9	1.27	1.14	1.29	1.20	1.24	0.89	1.30	1.30	1.37	1.27	1.30	1.30	1.29	1.18	1.31	1.30
10	1.29	1.20	1.29	1.20	1.36	1.40	1.30	1.30	1.35	1.30	1.30	1.30	1.42	1.00	1.31	1.30
11	1.38	1.11	1.29	1.20	1.29	1.14	1.30	1.30	1.27	1.34	1.30	1.30	1.33	1.39	1.31	1.30
12	1.24	1.41	1.29	1.20	1.36	1.21	1.30	1.30	1.26	1.11	1.30	1.30	1.35	1.12	1.31	1.30
13	1.29	1.04	1.29	1.20	1.44	1.17	1.30	1.30	1.19	1.15	1.30	1.30	1.26	0.98	1.31	1.30
14	1.27	1.21	1.29	1.20	1.37	0.81	1.30	1.30	1.29	1.08	1.30	1.30	1.39	1.24	1.31	1.30
15	1.26	1.36	1.29	1.20	1.39	1.23	1.30	1.30	1.26	1.18	1.30	1.30	1.34	1.25	1.31	1.30
16	1.28	0.87	1.29	1.20	1.36	1.03	1.30	1.30	1.40	1.24	1.30	1.30	1.31	1.27	1.31	1.30
17	1.26	1.54	1.29	1.20	1.36	1.18	1.30	1.30	1.35	1.14	1.30	1.30	1.25	1.24	1.31	1.30
18	1.23	1.02	1.29	1.20	1.41	1.27	1.30	1.30	1.29	1.35	1.30	1.30	1.27	1.25	1.31	1.30
19	1.27	0.86	1.29	1.20	1.36	1.29	1.30	1.30	1.31	1.28	1.30	1.30	1.35	1.13	1.31	1.30
20	1.32	1.35	1.29	1.20	1.26	0.99	1.30	1.30	1.39	1.19	1.30	1.30	1.31	1.12	1.31	1.30
16-percentile	1.23	1.00			1.30	1.00			1.23	1.06			1.27	1.09		
Median	1.28	1.16			1.35	1.15			1.30	1.18			1.32	1.19		
84-percentile	1.33	1.34			1.41	1.33			1.38	1.32			1.37	1.31		
Standard Deviation	0.04	0.15			0.04	0.14			0.06	0.11			0.04	0.09		

Table B.2 Relationships between the maximum elastic and inelastic roof and spectral displacements, Group 2

Record #	3-Story ($T_1=0.33$ s, $T_{1,idealized}=0.31$ s)				5-Story ($T_1=0.55$ s, $T_{1,idealized}=0.55$ s)				7-Story ($T_1=0.68$ s, $T_{1,idealized}=0.67$ s)				9-Story ($T_1=0.92$ s, $T_{1,idealized}=0.88$ s)			
	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i,e}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i,e}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i,e}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i,e}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA
1	1.48	1.42	1.29	1.20	1.26	0.95	1.32	1.30	1.56	1.19	1.35	1.30	1.61	1.07	1.36	1.30
2	1.76	1.67	1.29	1.20	1.40	1.19	1.32	1.30	1.76	1.06	1.35	1.30	1.32	1.22	1.36	1.30
3	1.55	1.18	1.29	1.20	1.30	1.12	1.32	1.30	1.37	1.07	1.35	1.30	1.43	1.06	1.36	1.30
4	1.15	1.15	1.29	1.20	1.33	1.33	1.32	1.30	1.32	1.27	1.35	1.30	1.71	1.38	1.36	1.30
5	1.95	1.90	1.29	1.20	1.25	0.76	1.32	1.30	1.59	1.22	1.35	1.30	1.94	1.23	1.36	1.30
6	1.52	1.29	1.29	1.20	1.35	1.36	1.32	1.30	1.41	0.85	1.35	1.30	1.73	1.20	1.36	1.30
7	1.01	0.84	1.29	1.20	1.38	1.37	1.32	1.30	1.37	0.96	1.35	1.30	1.28	1.27	1.36	1.30
8	1.56	1.53	1.29	1.20	1.28	0.95	1.32	1.30	1.38	1.04	1.35	1.30	1.20	0.97	1.36	1.30
9	1.48	1.61	1.29	1.20	1.42	0.94	1.32	1.30	1.41	1.10	1.35	1.30	1.59	1.29	1.36	1.30
10	1.44	1.30	1.29	1.20	1.34	1.39	1.32	1.30	1.32	1.12	1.35	1.30	1.39	1.07	1.36	1.30
11	1.26	1.09	1.29	1.20	1.45	1.31	1.32	1.30	1.47	1.28	1.35	1.30	1.63	1.50	1.36	1.30
12	1.37	1.24	1.29	1.20	1.40	1.36	1.32	1.30	1.49	1.03	1.35	1.30	1.41	1.30	1.36	1.30
13	1.64	1.22	1.29	1.20	1.32	1.22	1.32	1.30	1.46	1.19	1.35	1.30	1.58	1.32	1.36	1.30
14	1.40	1.28	1.29	1.20	1.38	1.14	1.32	1.30	1.47	0.93	1.35	1.30	1.54	1.44	1.36	1.30
15	1.61	1.14	1.29	1.20	1.35	0.98	1.32	1.30	1.36	0.79	1.35	1.30	1.49	1.11	1.36	1.30
16	1.55	0.99	1.29	1.20	1.47	1.10	1.32	1.30	1.42	1.19	1.35	1.30	1.46	0.96	1.36	1.30
17	1.57	1.36	1.29	1.20	1.31	1.23	1.32	1.30	1.35	1.68	1.35	1.30	1.53	1.15	1.36	1.30
18	1.51	1.25	1.29	1.20	1.32	1.19	1.32	1.30	1.45	1.29	1.35	1.30	1.37	1.27	1.36	1.30
19	1.32	1.06	1.29	1.20	1.28	1.46	1.32	1.30	1.43	0.90	1.35	1.30	1.54	1.25	1.36	1.30
20	1.56	1.56	1.29	1.20	1.25	1.25	1.32	1.30	1.38	1.14	1.35	1.30	1.44	1.13	1.36	1.30
16-percentile	1.28	1.06			1.28	1.00			1.33	0.92			1.34	1.06		
Median	1.47	1.28			1.34	1.17			1.43	1.10			1.50	1.20		
84-percentile	1.69	1.55			1.41	1.36			1.54	1.31			1.68	1.35		
Standard Deviation	0.14	0.19			0.05	0.16			0.07	0.17			0.11	0.12		

Table B.3 Relationships between the maximum elastic and inelastic roof and spectral displacements, Group 3

Record #	3-Story ($T_1=0.56$ s, $T_{1,idealized}=0.55$ s)				5-Story ($T_1=0.78$ s, $T_{1,idealized}=0.78$ s)				7-Story ($T_1=1.05$ s, $T_{1,idealized}=1.00$ s)				9-Story ($T_1=1.29$ s, $T_{1,idealized}=1.23$ s)			
	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA
1	1.29	1.32	1.29	1.20	1.33	1.31	1.31	1.30	1.64	1.00	1.35	1.30	1.81	1.26	1.38	1.30
2	1.29	1.17	1.29	1.20	1.30	1.18	1.31	1.30	1.71	1.62	1.35	1.30	1.40	1.33	1.38	1.30
3	1.26	1.05	1.29	1.20	1.29	1.20	1.31	1.30	1.40	1.29	1.35	1.30	1.45	1.60	1.38	1.30
4	1.37	1.44	1.29	1.20	1.30	1.11	1.31	1.30	1.51	1.37	1.35	1.30	1.75	1.05	1.38	1.30
5	1.35	1.15	1.29	1.20	1.29	1.19	1.31	1.30	1.32	1.07	1.35	1.30	1.48	1.41	1.38	1.30
6	1.30	1.32	1.29	1.20	1.23	1.25	1.31	1.30	1.38	1.36	1.35	1.30	1.55	1.24	1.38	1.30
7	1.31	1.20	1.29	1.20	1.33	1.08	1.31	1.30	1.63	1.77	1.35	1.30	1.85	1.37	1.38	1.30
8	1.33	0.95	1.29	1.20	1.31	1.29	1.31	1.30	1.42	1.17	1.35	1.30	1.58	1.18	1.38	1.30
9	1.29	1.23	1.29	1.20	1.34	1.21	1.31	1.30	1.28	1.08	1.35	1.30	1.43	1.10	1.38	1.30
10	1.37	1.11	1.29	1.20	1.30	1.05	1.31	1.30	1.68	1.49	1.35	1.30	1.49	1.34	1.38	1.30
11	1.28	1.05	1.29	1.20	1.25	1.32	1.31	1.30	1.47	1.53	1.35	1.30	1.49	1.55	1.38	1.30
12	1.27	1.10	1.29	1.20	1.45	1.05	1.31	1.30	1.49	1.19	1.35	1.30	1.44	1.27	1.38	1.30
13	1.32	1.14	1.29	1.20	1.33	1.21	1.31	1.30	1.64	1.18	1.35	1.30	1.34	1.16	1.38	1.30
14	1.36	1.03	1.29	1.20	1.32	1.04	1.31	1.30	1.50	1.15	1.35	1.30	1.49	1.28	1.38	1.30
15	1.39	1.17	1.29	1.20	1.33	1.34	1.31	1.30	1.55	1.47	1.35	1.30	1.46	1.69	1.38	1.30
16	1.31	1.21	1.29	1.20	1.29	1.15	1.31	1.30	1.41	2.11	1.35	1.30	1.56	0.74	1.38	1.30
17	1.30	1.30	1.29	1.20	1.29	1.16	1.31	1.30	1.49	1.22	1.35	1.30	1.48	1.13	1.38	1.30
18	1.28	1.17	1.29	1.20	1.28	0.94	1.31	1.30	1.47	1.30	1.35	1.30	1.73	1.06	1.38	1.30
19	1.30	1.41	1.29	1.20	1.34	1.09	1.31	1.30	1.42	1.40	1.35	1.30	1.59	1.61	1.38	1.30
20	1.31	1.39	1.29	1.20	1.37	0.88	1.31	1.30	1.76	1.14	1.35	1.30	1.58	1.22	1.38	1.30
16-percentile	1.28	1.06			1.27	1.03			1.38	1.09			1.41	1.06		
Median	1.31	1.19			1.31	1.15			1.50	1.32			1.54	1.26		
84-percentile	1.35	1.33			1.36	1.28			1.64	1.61			1.69	1.50		
Standard Deviation	0.03	0.11			0.03	0.11			0.09	0.20			0.09	0.17		

Table B.4 Relationships between the maximum elastic and inelastic roof and spectral displacements, Group 4

Record #	3-Story ($T_1=0.38$ s, $T_{1,idealized}=0.37$ s)				5-Story ($T_1=0.64$ s, $T_{1,idealized}=0.65$ s)				7-Story ($T_1=0.90$ s, $T_{1,idealized}=0.90$ s)				9-Story ($T_1=1.17$ s, $T_{1,idealized}=1.16$ s)			
	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA
1	1.40	1.38	1.29	1.20	1.31	1.20	1.30	1.30	1.34	1.14	1.30	1.30	1.33	0.92	1.31	1.30
2	1.41	1.30	1.29	1.20	1.29	1.10	1.30	1.30	1.32	1.22	1.30	1.30	1.31	1.26	1.31	1.30
3	1.46	1.45	1.29	1.20	1.33	1.11	1.30	1.30	1.25	1.05	1.30	1.30	1.30	1.27	1.31	1.30
4	1.24	1.27	1.29	1.20	1.23	1.05	1.30	1.30	1.34	1.25	1.30	1.30	1.36	1.07	1.31	1.30
5	1.38	1.39	1.29	1.20	1.11	1.64	1.30	1.30	1.31	1.30	1.30	1.30	1.35	1.24	1.31	1.30
6	1.51	1.22	1.29	1.20	1.43	1.38	1.30	1.30	1.60	1.32	1.30	1.30	1.43	0.79	1.31	1.30
7	1.38	1.18	1.29	1.20	1.12	1.04	1.30	1.30	1.30	1.15	1.30	1.30	1.31	1.22	1.31	1.30
8	1.54	1.19	1.29	1.20	1.38	1.07	1.30	1.30	1.33	1.17	1.30	1.30	1.48	1.66	1.31	1.30
9	1.36	1.08	1.29	1.20	1.42	1.45	1.30	1.30	1.29	1.40	1.30	1.30	1.31	1.38	1.31	1.30
10	1.30	1.07	1.29	1.20	1.43	1.16	1.30	1.30	1.36	1.50	1.30	1.30	1.53	1.06	1.31	1.30
11	1.43	1.35	1.29	1.20	1.24	1.34	1.30	1.30	1.30	1.71	1.30	1.30	1.40	1.21	1.31	1.30
12	1.27	1.23	1.29	1.20	1.25	1.45	1.30	1.30	1.37	1.09	1.30	1.30	1.42	1.20	1.31	1.30
13	1.29	1.30	1.29	1.20	1.14	1.35	1.30	1.30	1.37	1.08	1.30	1.30	1.42	1.21	1.31	1.30
14	1.39	1.05	1.29	1.20	1.29	1.17	1.30	1.30	1.36	1.14	1.30	1.30	1.54	1.18	1.31	1.30
15	1.38	1.35	1.29	1.20	1.24	1.12	1.30	1.30	1.31	0.86	1.30	1.30	1.32	1.66	1.31	1.30
16	1.32	1.19	1.29	1.20	1.44	1.16	1.30	1.30	1.24	1.31	1.30	1.30	1.34	1.10	1.31	1.30
17	1.38	1.26	1.29	1.20	1.34	1.16	1.30	1.30	1.30	1.20	1.30	1.30	1.35	1.31	1.31	1.30
18	1.47	1.42	1.29	1.20	1.29	1.30	1.30	1.30	1.32	1.17	1.30	1.30	1.36	1.03	1.31	1.30
19	1.40	1.20	1.29	1.20	1.34	1.73	1.30	1.30	1.35	1.26	1.30	1.30	1.46	1.28	1.31	1.30
20	1.47	1.00	1.29	1.20	1.42	1.30	1.30	1.30	1.28	1.42	1.30	1.30	1.37	1.10	1.31	1.30
16-percentile	1.31	1.12			1.20	1.07			1.26	1.06			1.31	1.00		
Median	1.39	1.24			1.30	1.25			1.33	1.22			1.38	1.19		
84-percentile	1.47	1.37			1.40	1.45			1.40	1.42			1.46	1.41		
Standard Deviation	0.06	0.10			0.08	0.15			0.05	0.15			0.05	0.17		

Table B.5 Relationships between the maximum elastic and inelastic roof and spectral displacements, Group 5

Record #	3-Story ($T_1=0.47$ s, $T_{1,idealized}=0.47$ s)				5-Story ($T_1=0.71$ s, $T_{1,idealized}=0.69$ s)				7-Story ($T_1=0.96$ s, $T_{1,idealized}=0.93$ s)				9-Story ($T_1=1.31$ s, $T_{1,idealized}=1.25$ s)			
	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i,e}$	$\Gamma_{\phi_1}^{roof}$	FEMA
1	1.34	1.05	1.29	1.20	1.64	1.12	1.42	1.30	1.55	1.02	1.35	1.30	1.61	1.33	1.36	1.30
2	1.33	1.48	1.29	1.20	1.54	1.12	1.42	1.30	1.42	1.38	1.35	1.30	1.61	1.11	1.36	1.30
3	1.39	1.19	1.29	1.20	1.22	1.05	1.42	1.30	1.42	1.22	1.35	1.30	1.71	1.49	1.36	1.30
4	1.33	1.34	1.29	1.20	1.37	1.35	1.42	1.30	1.52	1.19	1.35	1.30	1.56	1.26	1.36	1.30
5	1.30	1.28	1.29	1.20	1.54	1.37	1.42	1.30	2.01	1.20	1.35	1.30	1.76	1.67	1.36	1.30
6	1.36	1.39	1.29	1.20	1.45	1.37	1.42	1.30	1.34	1.27	1.35	1.30	1.25	1.25	1.36	1.30
7	1.29	1.34	1.29	1.20	1.41	1.27	1.42	1.30	1.43	1.22	1.35	1.30	1.14	1.30	1.36	1.30
8	1.34	1.09	1.29	1.20	1.22	0.92	1.42	1.30	1.36	1.01	1.35	1.30	1.57	0.79	1.36	1.30
9	1.33	1.07	1.29	1.20	1.43	1.59	1.42	1.30	1.40	1.47	1.35	1.30	1.34	1.14	1.36	1.30
10	1.32	1.11	1.29	1.20	1.36	1.24	1.42	1.30	1.33	1.08	1.35	1.30	1.57	1.12	1.36	1.30
11	1.34	1.22	1.29	1.20	1.47	1.37	1.42	1.30	1.59	1.39	1.35	1.30	1.50	1.29	1.36	1.30
12	1.27	1.13	1.29	1.20	1.58	1.19	1.42	1.30	1.53	1.16	1.35	1.30	1.52	1.56	1.36	1.30
13	1.34	1.21	1.29	1.20	1.47	1.30	1.42	1.30	1.55	1.24	1.35	1.30	1.31	0.99	1.36	1.30
14	1.25	1.15	1.29	1.20	1.43	1.12	1.42	1.30	1.45	1.38	1.35	1.30	1.54	1.21	1.36	1.30
15	1.35	1.16	1.29	1.20	1.31	0.99	1.42	1.30	1.58	1.22	1.35	1.30	1.72	1.23	1.36	1.30
16	1.30	1.07	1.29	1.20	1.28	1.35	1.42	1.30	1.46	0.95	1.35	1.30	1.93	1.39	1.36	1.30
17	1.30	1.20	1.29	1.20	1.44	1.61	1.42	1.30	1.46	1.37	1.35	1.30	1.37	1.42	1.36	1.30
18	1.31	1.33	1.29	1.20	1.45	1.26	1.42	1.30	1.24	1.24	1.35	1.30	1.60	1.33	1.36	1.30
19	1.32	1.14	1.29	1.20	1.51	1.02	1.42	1.30	1.54	1.45	1.35	1.30	1.50	1.20	1.36	1.30
20	1.31	1.18	1.29	1.20	1.35	1.31	1.42	1.30	1.28	1.17	1.35	1.30	1.32	1.34	1.36	1.30
16-percentile	1.29	1.09			1.31	1.06			1.31	1.09			1.33	1.08		
Median	1.32	1.20			1.42	1.23			1.46	1.22			1.51	1.26		
84-percentile	1.35	1.32			1.54	1.43			1.63	1.38			1.71	1.46		
Standard Deviation	0.02	0.10			0.08	0.15			0.11	0.12			0.12	0.15		

Table B.6 Relationships between the maximum elastic and inelastic roof and spectral displacements, Group 6

Record #	3-Story ($T_1=0.41$ s, $T_{1,idealized}=0.41$ s)				5-Story ($T_1=0.59$ s, $T_{1,idealized}=0.59$ s)				7-Story ($T_1=0.76$ s, $T_{1,idealized}=0.76$ s)				9-Story ($T_1=1.05$ s, $T_{1,idealized}=1.03$ s)			
	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA	$\Delta_{roof,e}/S_{d,e}$	$\Delta_{roof,i}/S_{d,i}$	$\Gamma_{\phi_1}^{roof}$	FEMA
1	1.30	1.27	1.30	1.20	1.37	1.42	1.32	1.30	1.35	1.40	1.33	1.30	1.39	1.11	1.35	1.30
2	1.30	1.19	1.30	1.20	1.31	1.14	1.32	1.30	1.29	0.93	1.33	1.30	1.39	1.26	1.35	1.30
3	1.30	1.68	1.30	1.20	1.34	1.24	1.32	1.30	1.36	0.80	1.33	1.30	1.41	0.77	1.35	1.30
4	1.30	1.34	1.30	1.20	1.30	1.21	1.32	1.30	1.34	1.46	1.33	1.30	1.39	1.16	1.35	1.30
5	1.32	1.37	1.30	1.20	1.36	1.20	1.32	1.30	1.36	0.86	1.33	1.30	1.41	0.85	1.35	1.30
6	1.31	1.18	1.30	1.20	1.28	1.13	1.32	1.30	1.31	1.07	1.33	1.30	1.38	1.06	1.35	1.30
7	1.34	1.00	1.30	1.20	1.35	1.07	1.32	1.30	1.33	1.19	1.33	1.30	1.51	1.18	1.35	1.30
8	1.29	1.32	1.30	1.20	1.30	1.08	1.32	1.30	1.25	1.40	1.33	1.30	1.22	1.38	1.35	1.30
9	1.34	1.37	1.30	1.20	1.43	1.37	1.32	1.30	1.30	1.24	1.33	1.30	1.36	1.44	1.35	1.30
10	1.31	1.38	1.30	1.20	1.30	1.32	1.32	1.30	1.41	1.22	1.33	1.30	1.43	1.25	1.35	1.30
11	1.28	1.31	1.30	1.20	1.31	1.29	1.32	1.30	1.31	1.99	1.33	1.30	1.55	1.62	1.35	1.30
12	1.32	1.11	1.30	1.20	1.28	1.12	1.32	1.30	1.33	1.20	1.33	1.30	1.37	1.40	1.35	1.30
13	1.29	1.35	1.30	1.20	1.31	1.26	1.32	1.30	1.30	0.94	1.33	1.30	1.27	1.15	1.35	1.30
14	1.27	1.32	1.30	1.20	1.29	1.31	1.32	1.30	1.36	1.21	1.33	1.30	1.43	1.19	1.35	1.30
15	1.32	1.07	1.30	1.20	1.30	1.27	1.32	1.30	1.33	1.50	1.33	1.30	1.46	1.90	1.35	1.30
16	1.39	1.14	1.30	1.20	1.39	1.37	1.32	1.30	1.30	1.42	1.33	1.30	1.46	1.21	1.35	1.30
17	1.29	1.33	1.30	1.20	1.28	1.36	1.32	1.30	1.28	1.37	1.33	1.30	1.35	1.33	1.35	1.30
18	1.30	1.21	1.30	1.20	1.29	1.09	1.32	1.30	1.24	1.73	1.33	1.30	1.52	1.30	1.35	1.30
16-percentile	1.28	1.12			1.28	1.13			1.28	0.98			1.33	1.00		
Median	1.31	1.26			1.32	1.23			1.32	1.24			1.40	1.23		
84-percentile	1.33	1.42			1.36	1.35			1.36	1.57			1.49	1.50		
Standard Deviation	0.02	0.12			0.03	0.09			0.03	0.23			0.06	0.20		

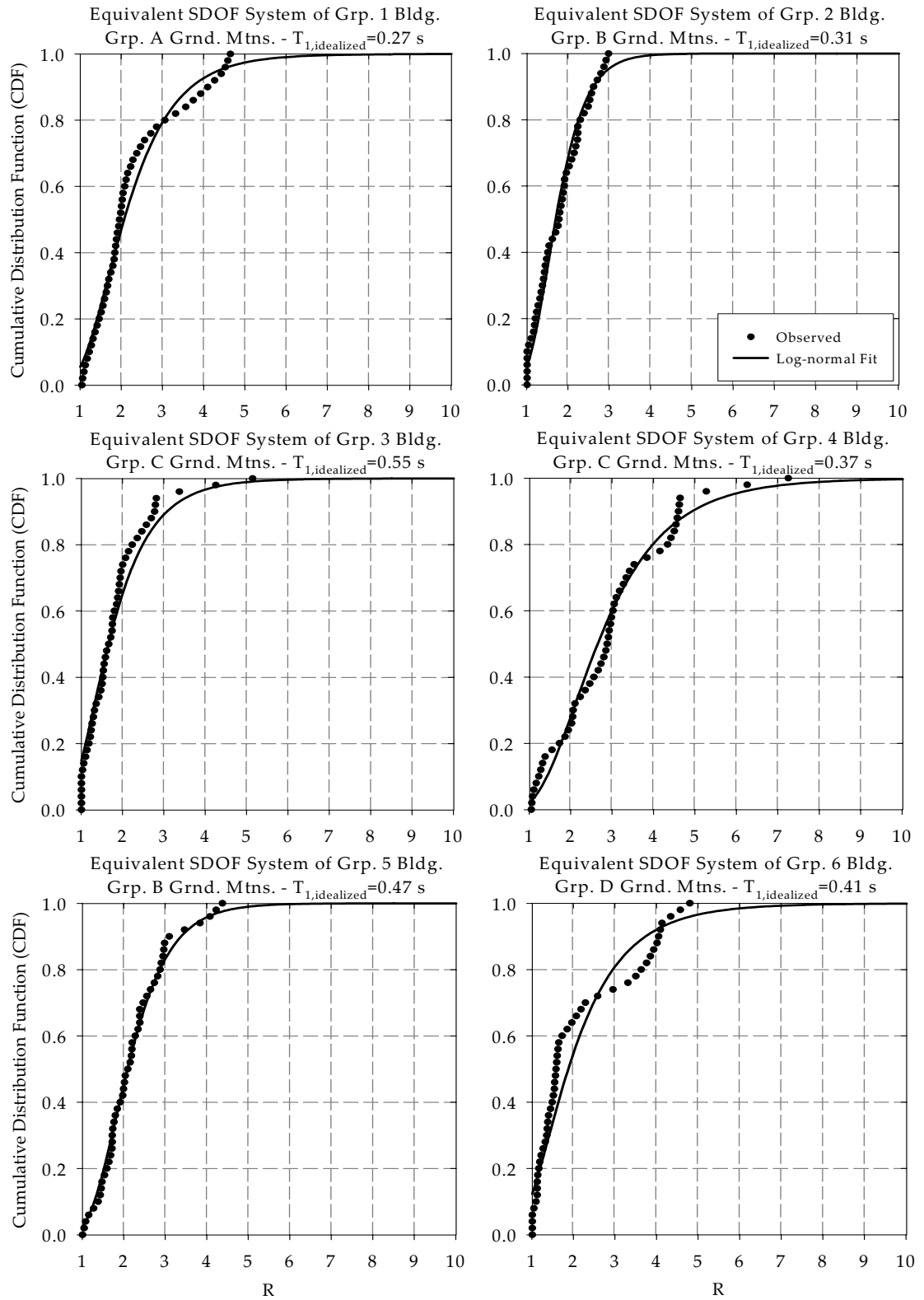


Figure B.1 R – distributions for 3-story model buildings

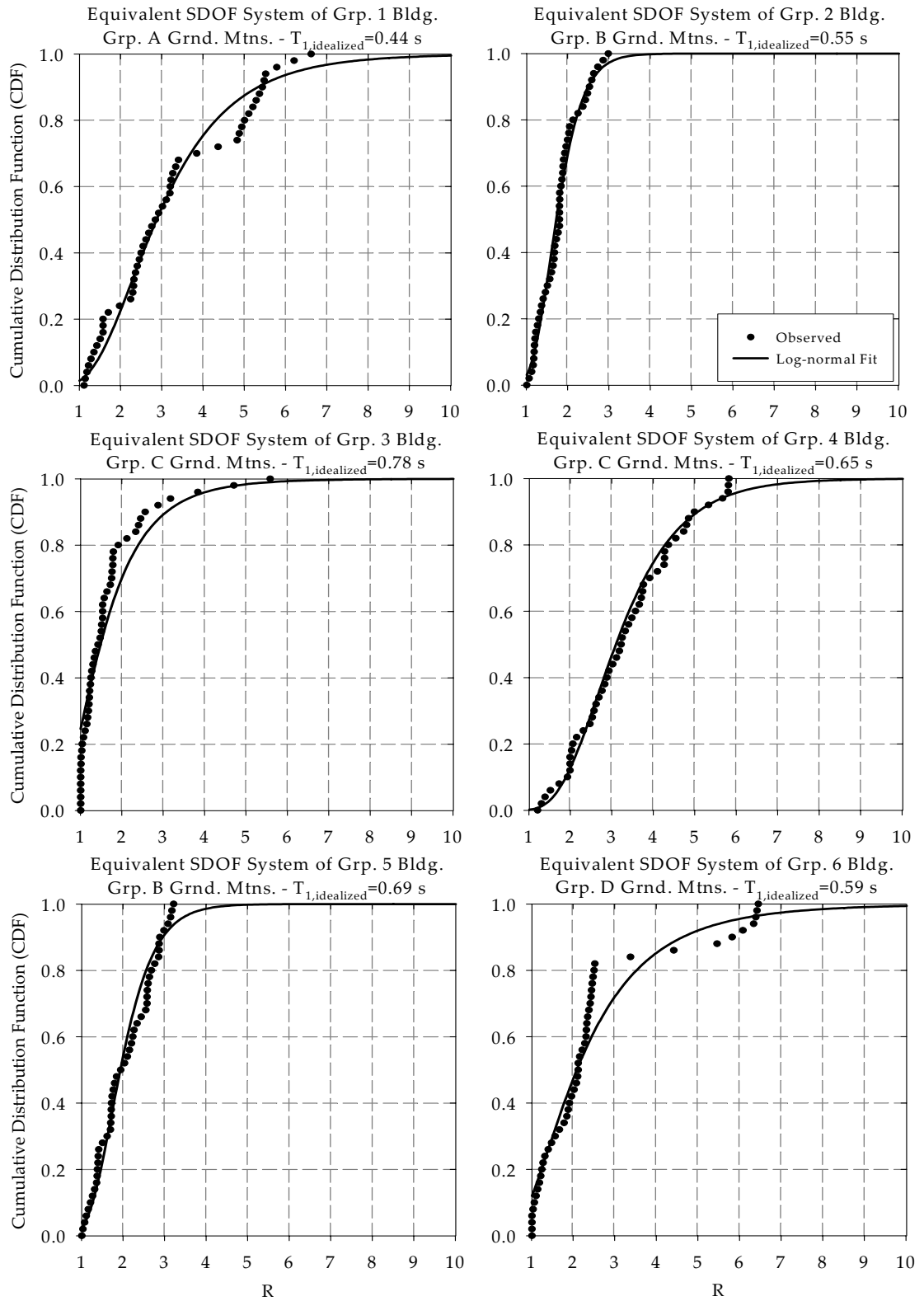


Figure B.2 R – distributions for 5-story model buildings

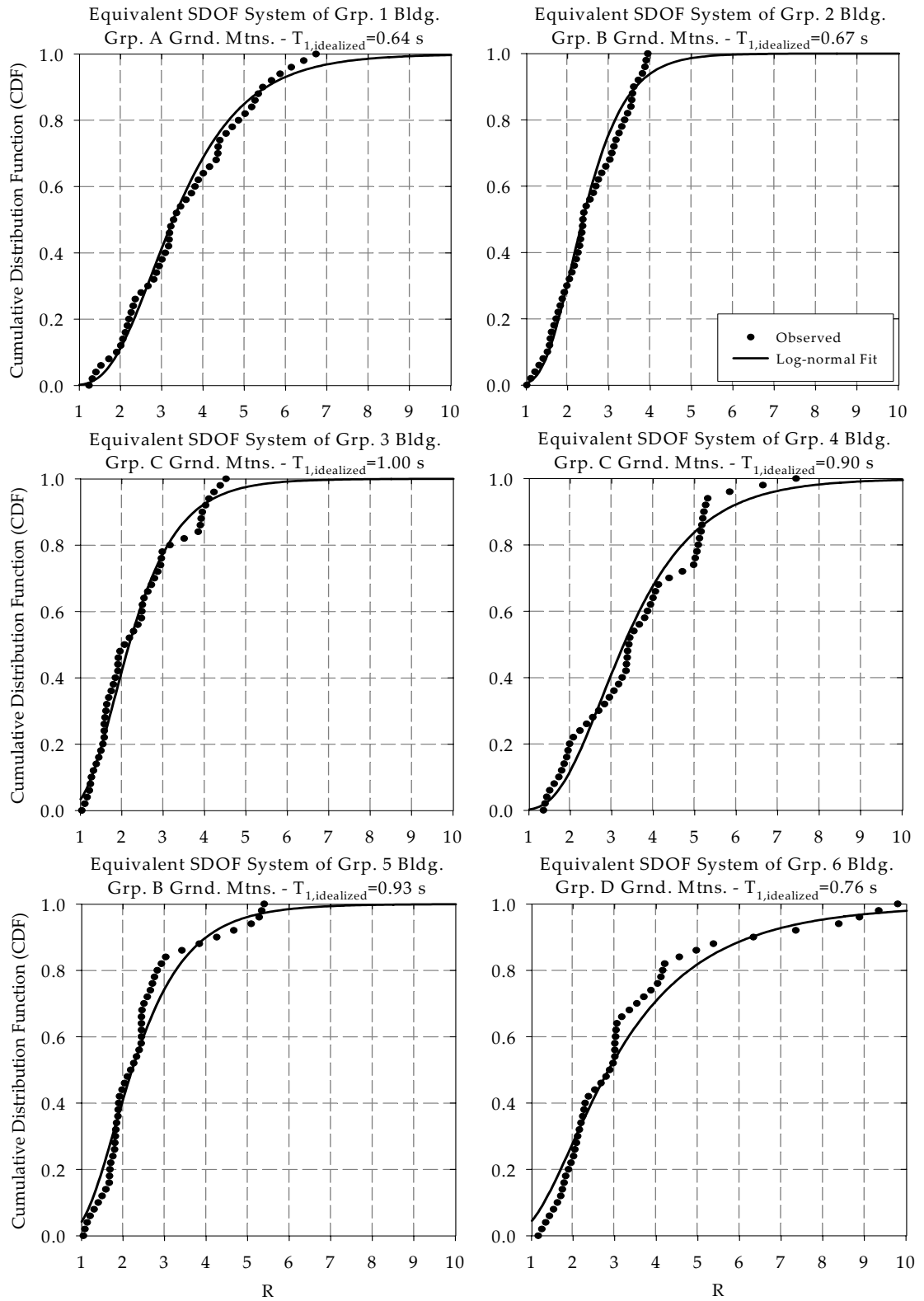


Figure B.3 R – distributions for 7-story model buildings

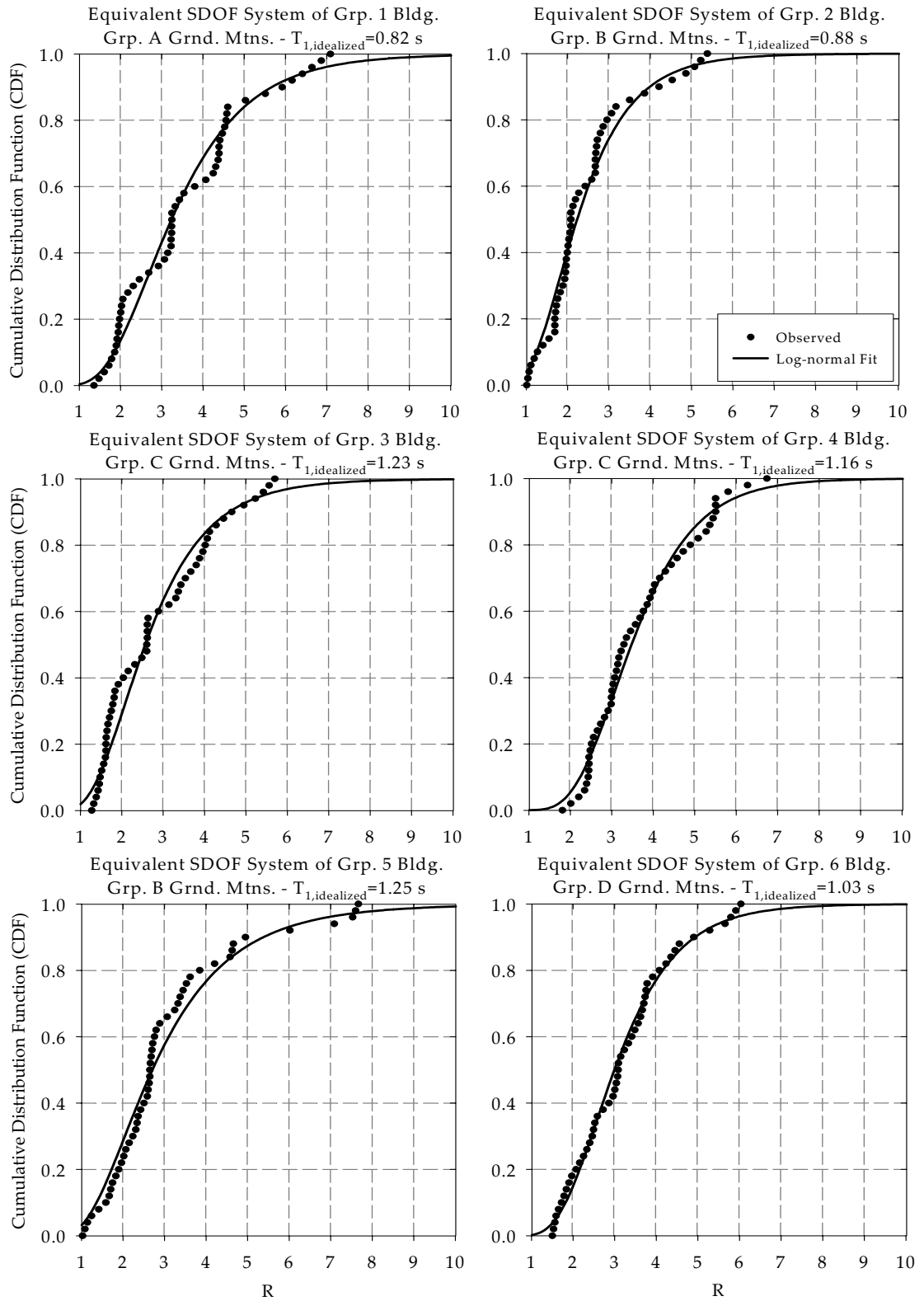


Figure B.4 R – distributions for 9-story model buildings

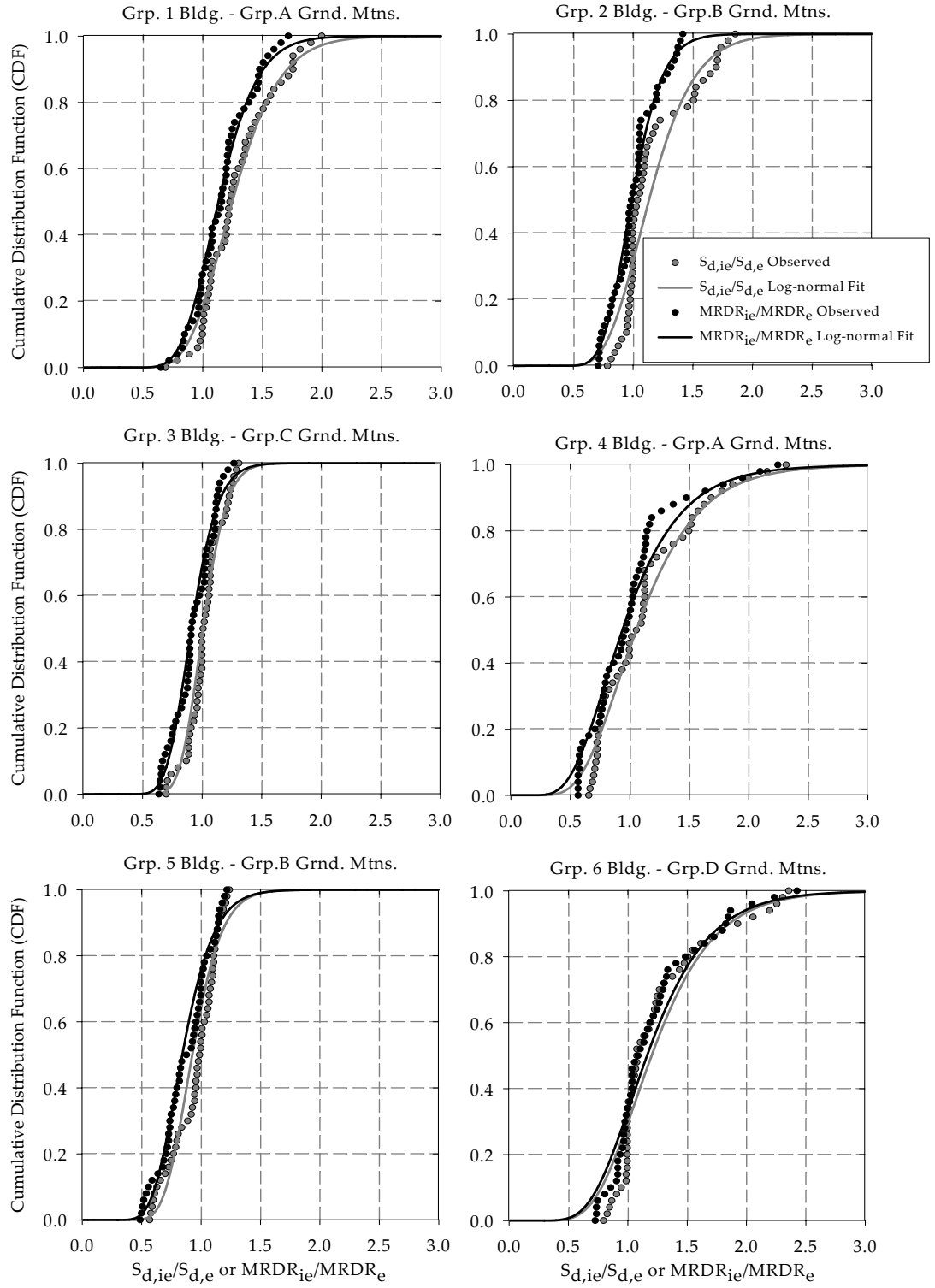


Figure B.5 Distributions of $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ ratios of 3-story model buildings

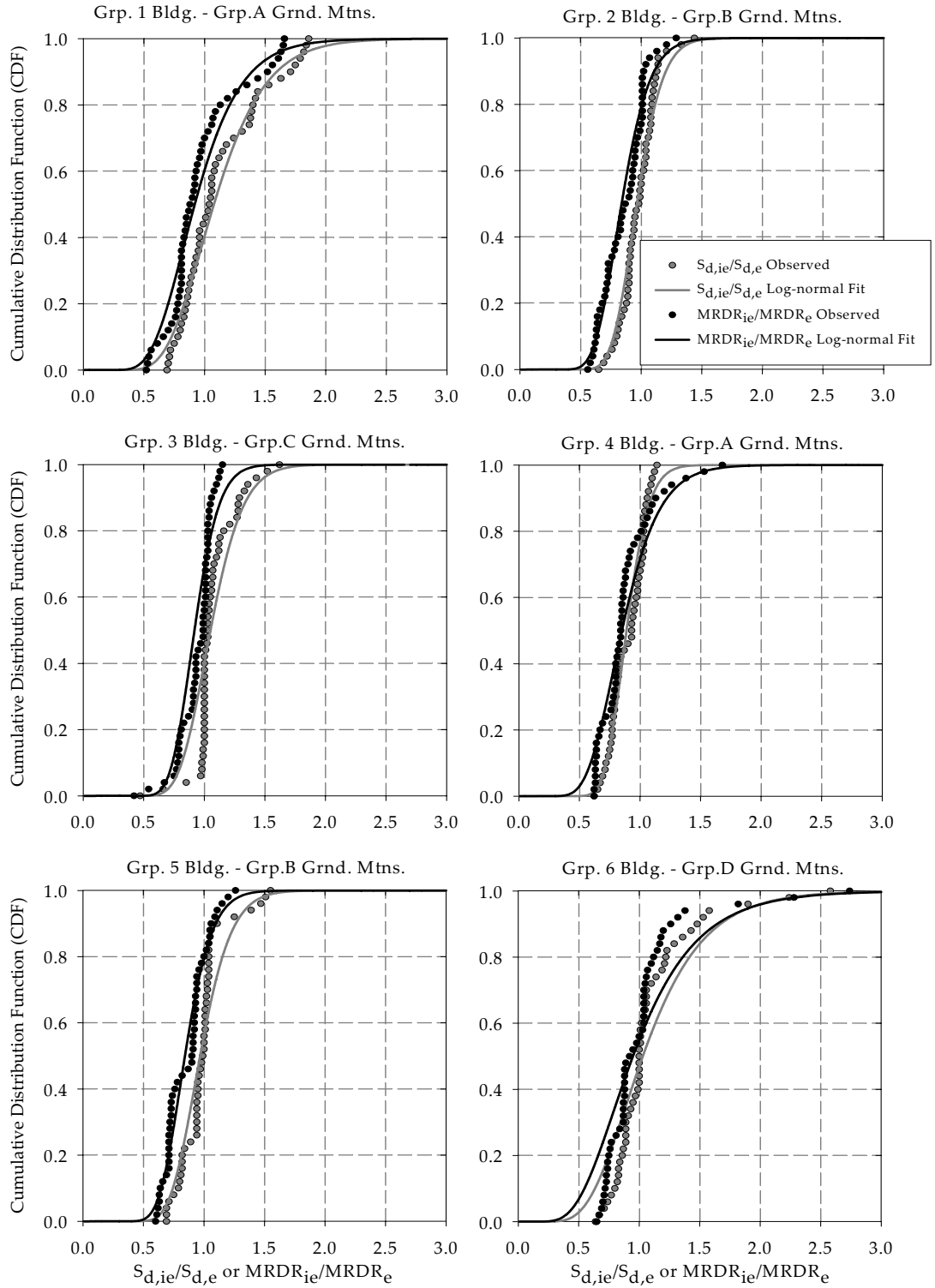


Figure B.6 Distributions of $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ ratios of 5-story model buildings

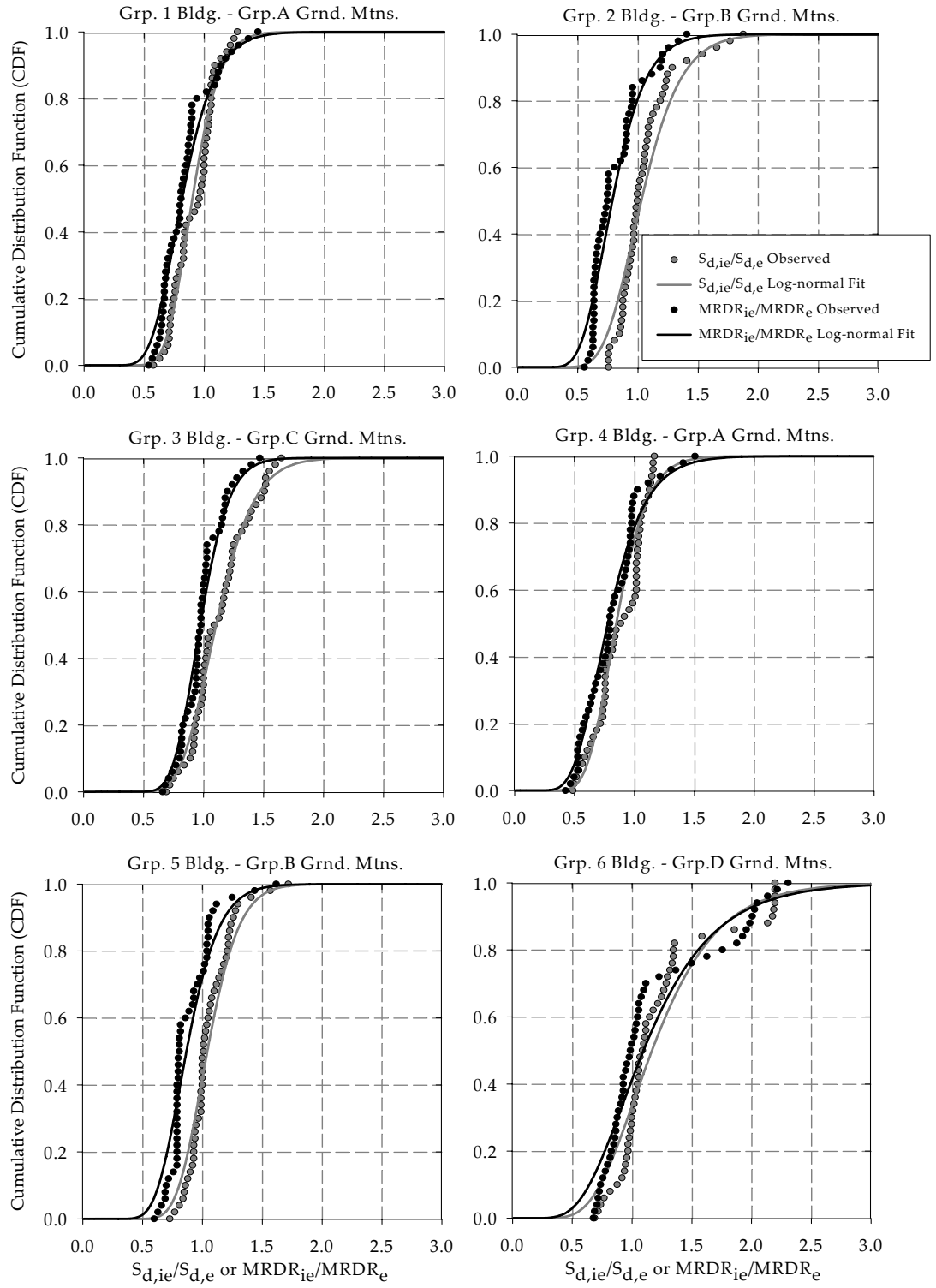


Figure B.7 Distributions of $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ ratios of 7-story model buildings

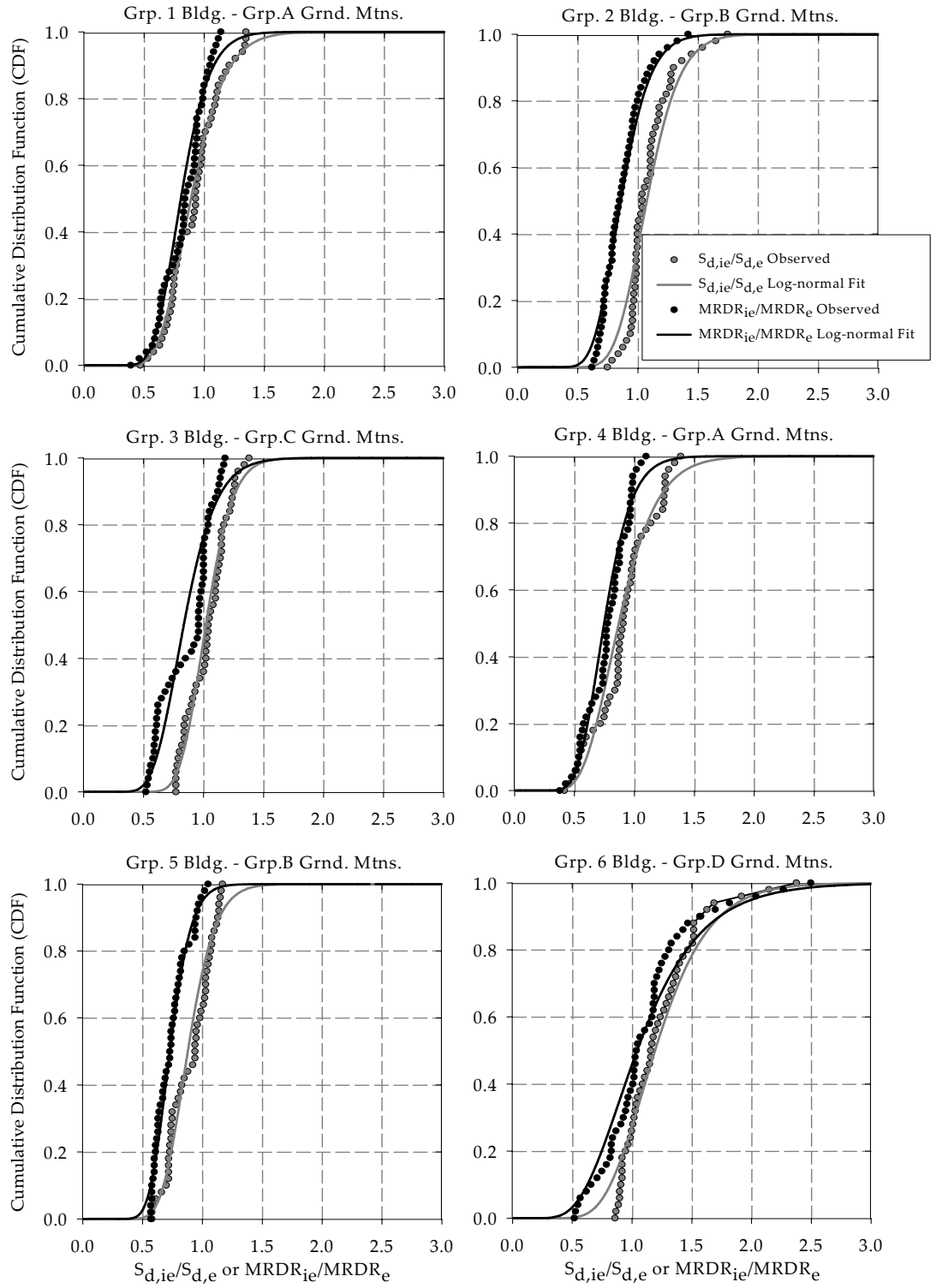


Figure B.8 Distributions of $S_{d,ie} / S_{d,e}$ and $MRDR_{ie} / MRDR_e$ ratios of 9-story model buildings

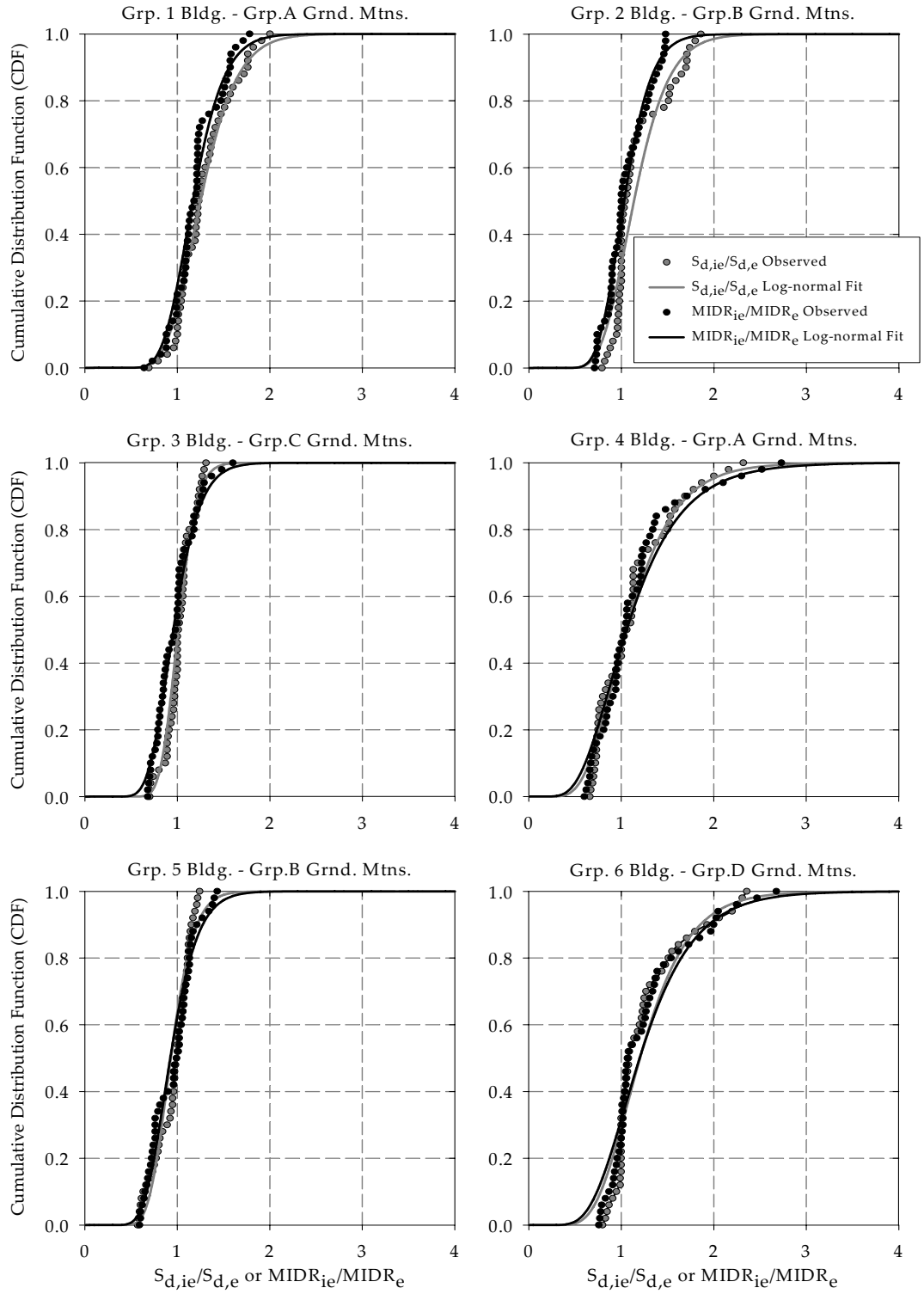


Figure B.9 Distributions of $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$ ratios of 3-story model buildings

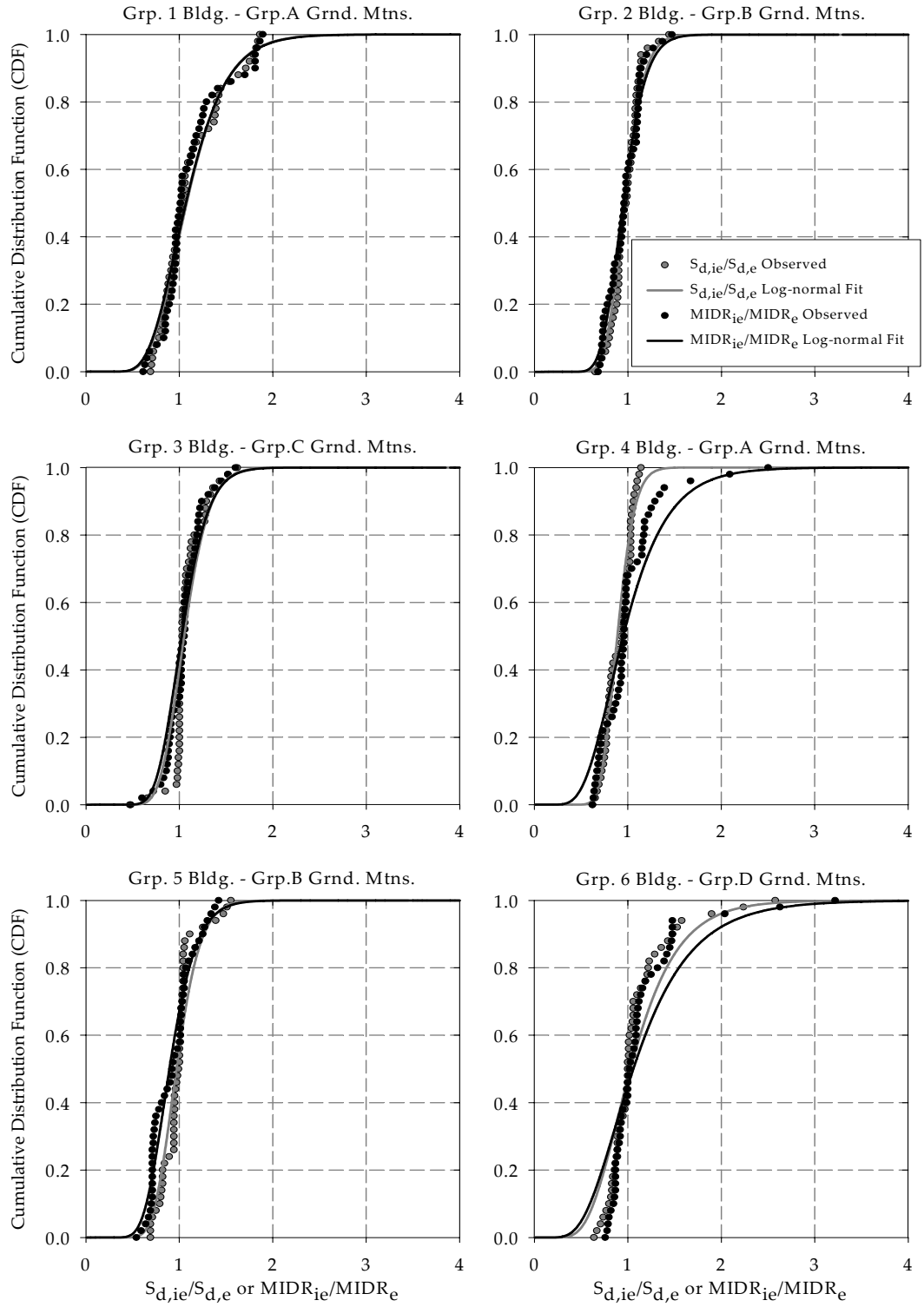


Figure B.10 Distributions of $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$ ratios of 5-story model buildings

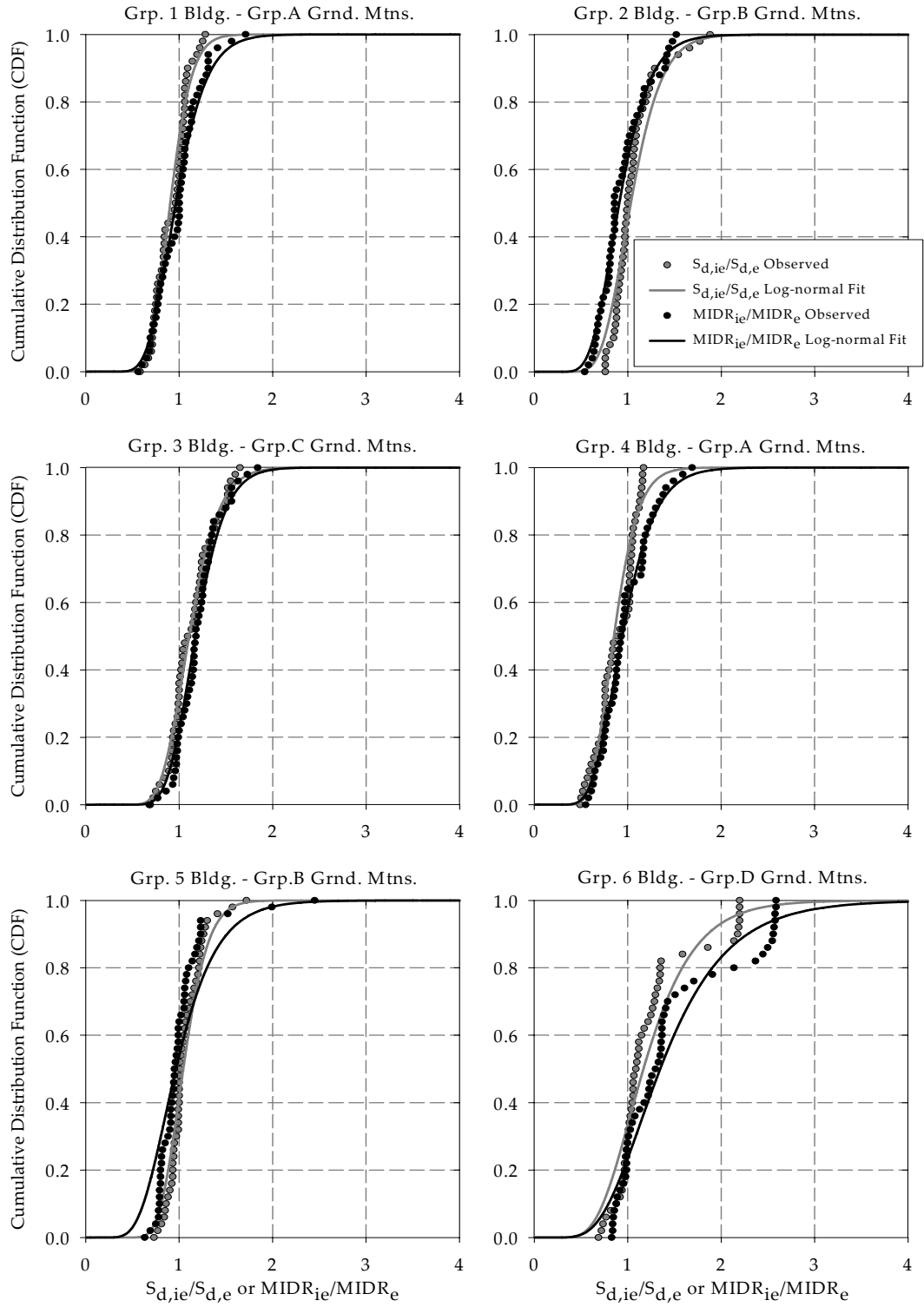


Figure B.11 Distributions of $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$ ratios of 7-story model buildings

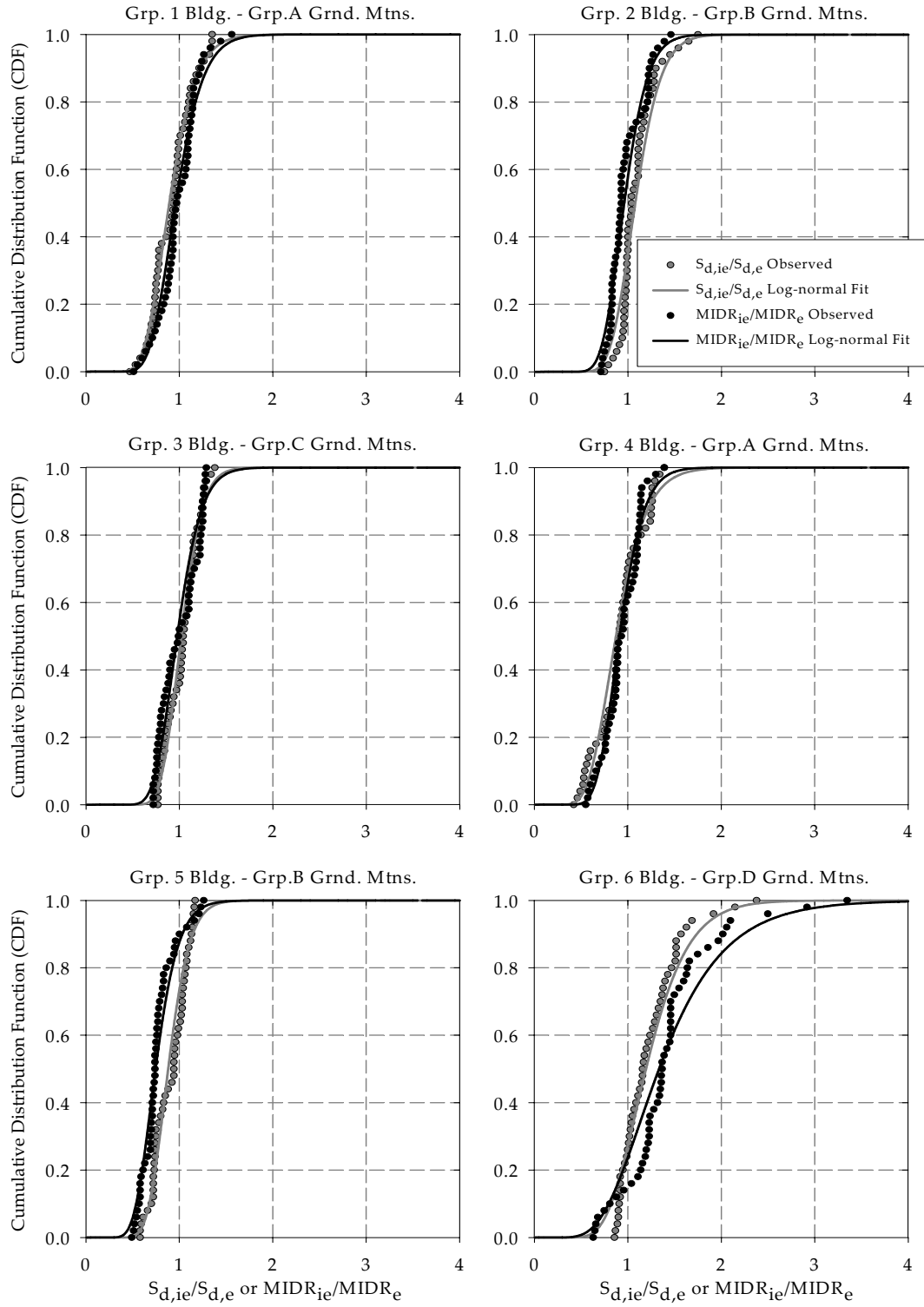


Figure B.12 Distributions of $S_{d,ie} / S_{d,e}$ and $MIDR_{ie} / MIDR_e$ ratios of 9-story model buildings

APPENDIX C

RELEVANT FIGURES OF CHAPTER 4

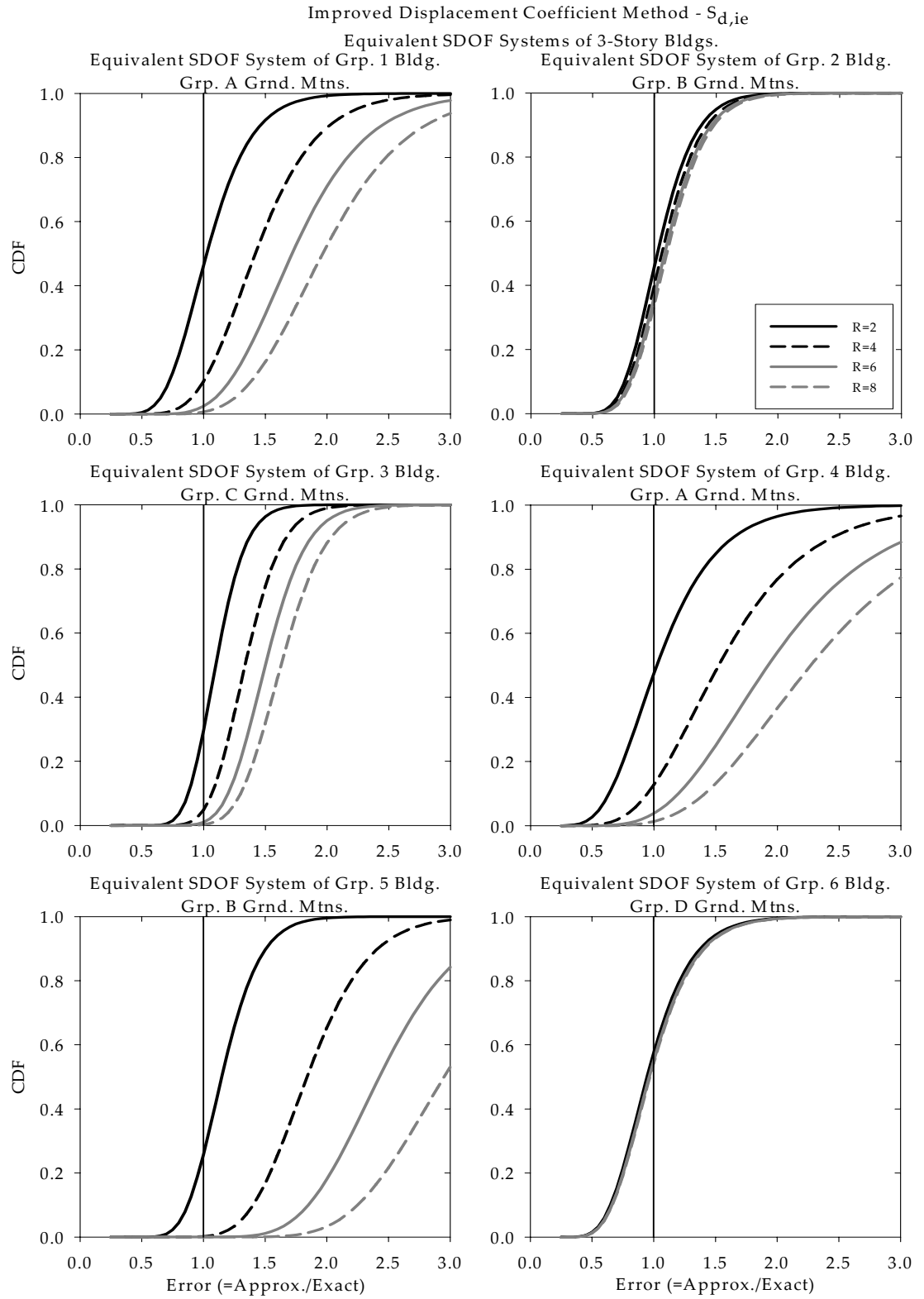


Figure C.1 Error distributions of equivalent SDOF systems for 3-story model buildings of improved DCM

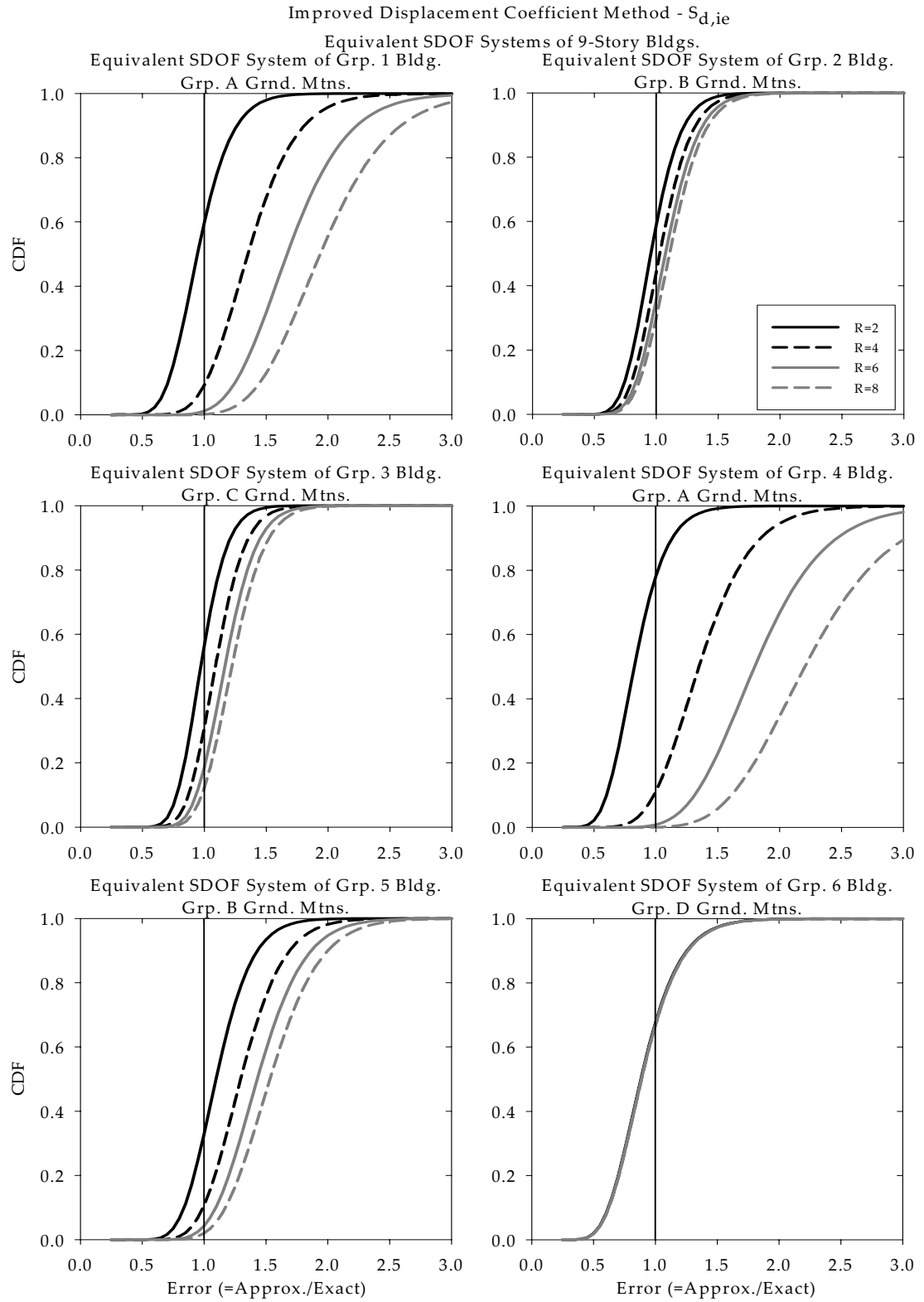


Figure C.2 Error distributions of equivalent SDOF systems for 9-story model buildings of improved DCM

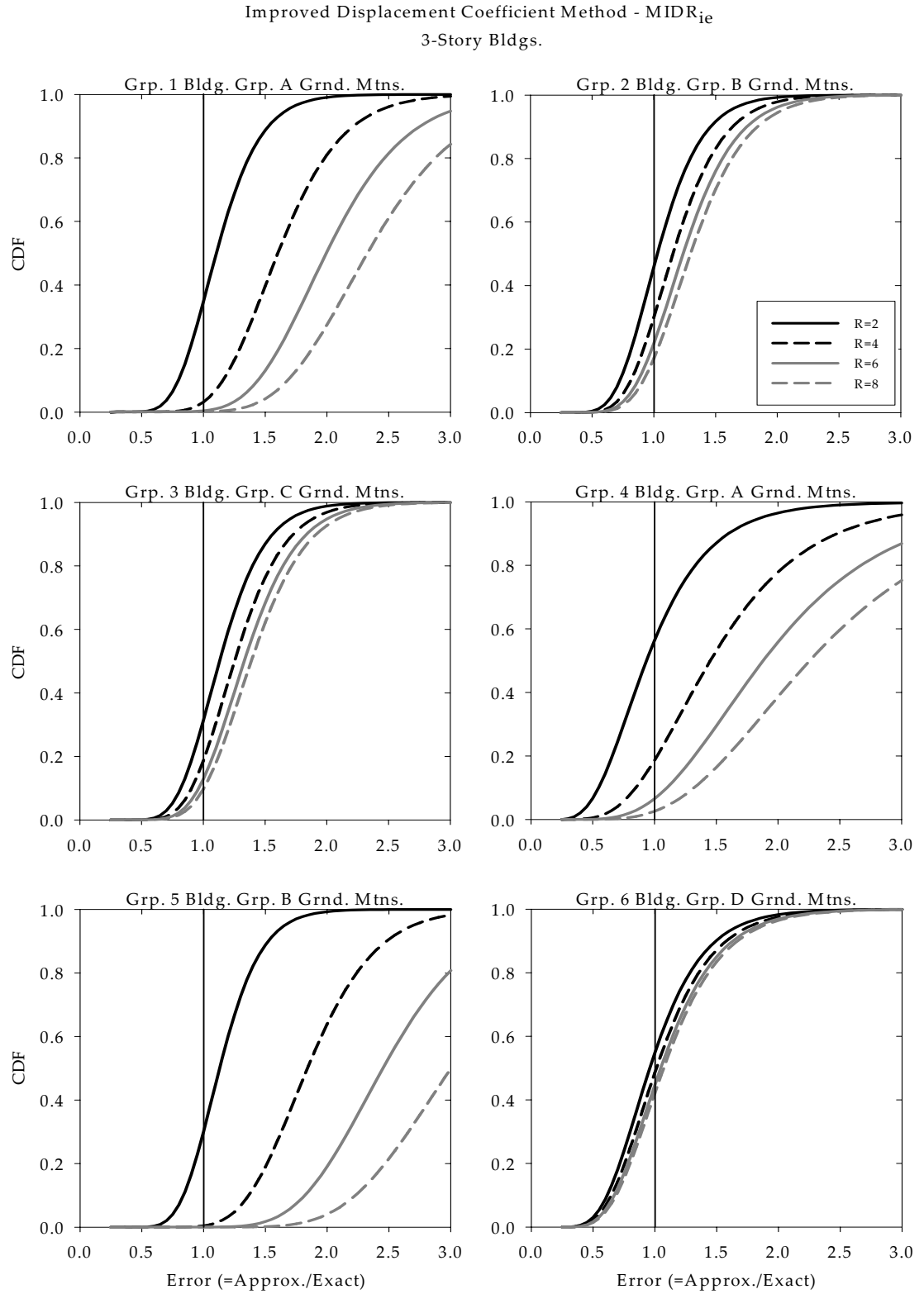


Figure C.3 Error distributions of $MIDR_{ie}$ for 3-story model buildings of improved DCM

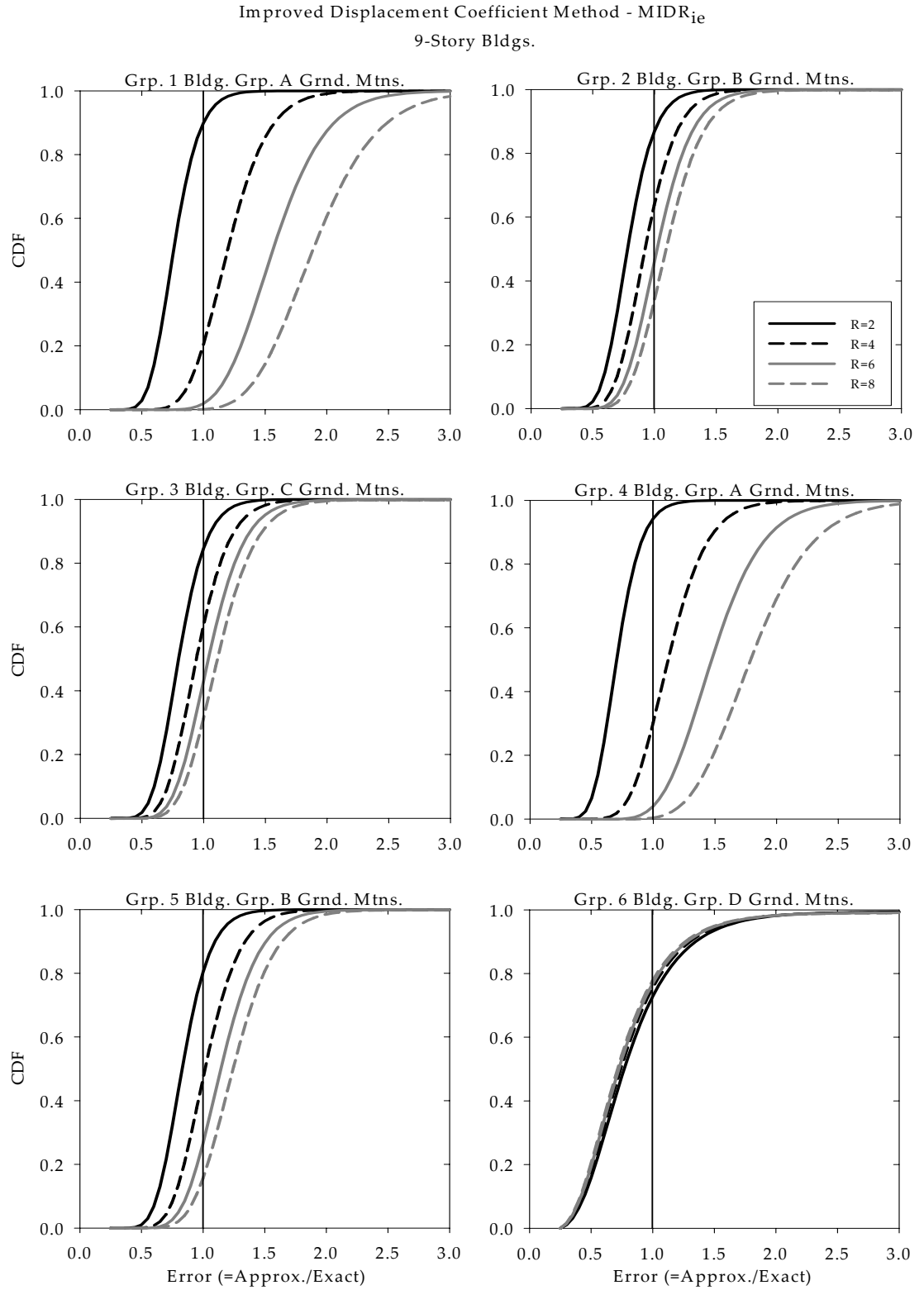


Figure C.4 Error distributions of $MIDR_{ie}$ for 9-story model buildings of improved DCM

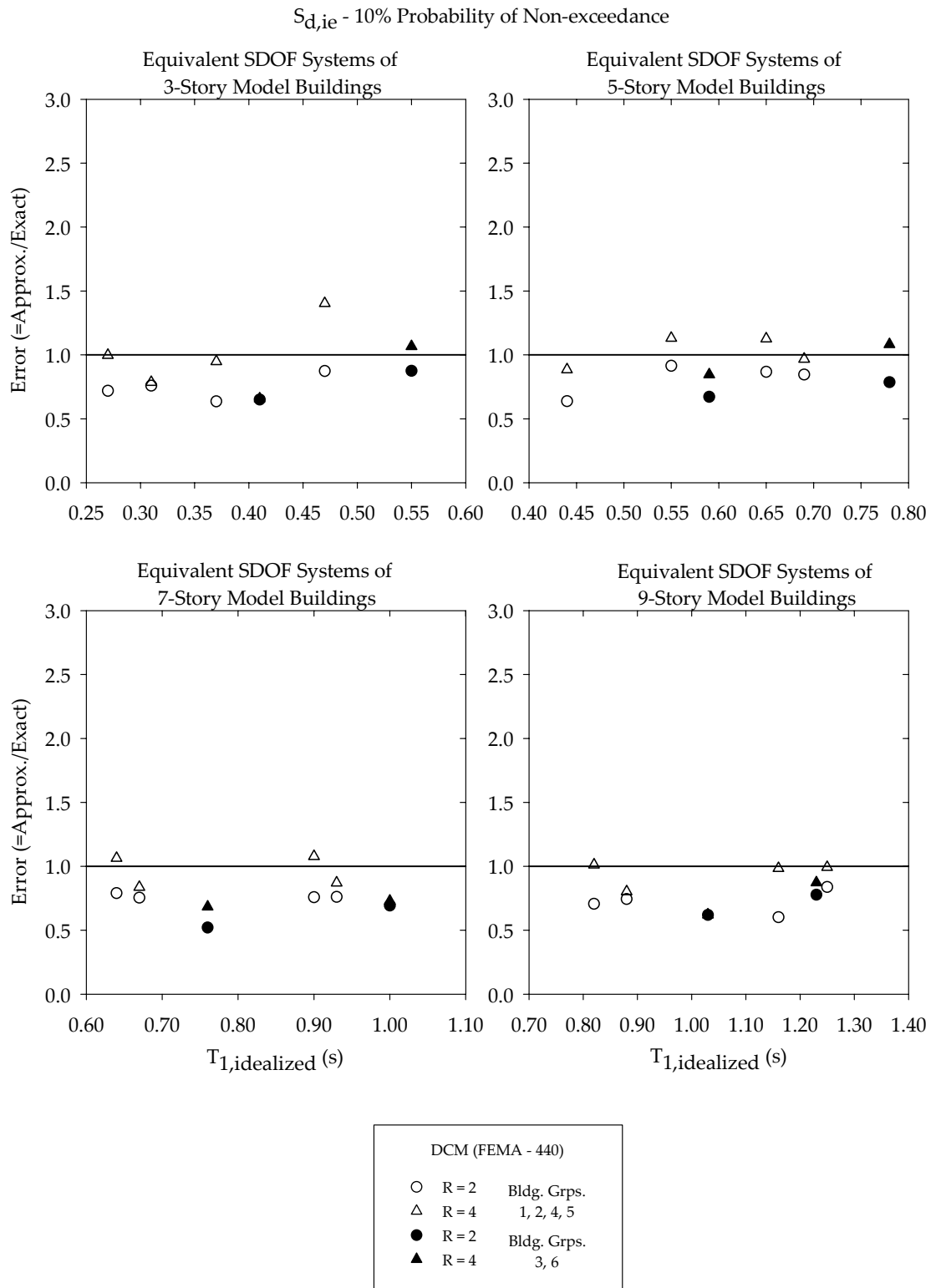


Figure C.5 Error statistics for 10% probability of non-exceedance for improved DCM conditional to R for $S_{d,ie}$

MRDR_{ie} - 10% Probability of Non-exceedance

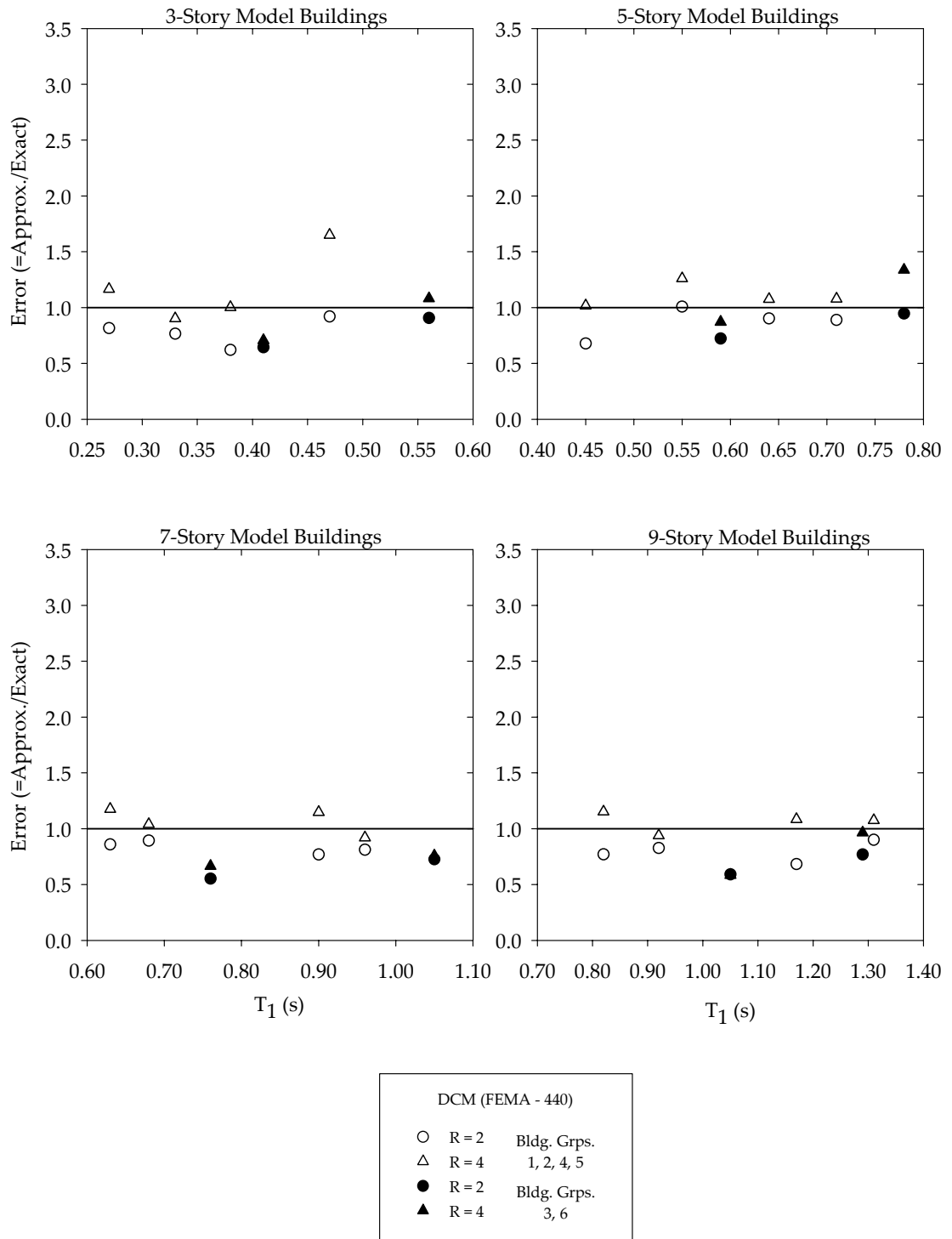


Figure C.6 Error statistics for 10% probability of non-exceedance for improved DCM conditional to R for $MRDR_{ie}$

MIDR_{ie} - 10% Probability of Nonexceedance

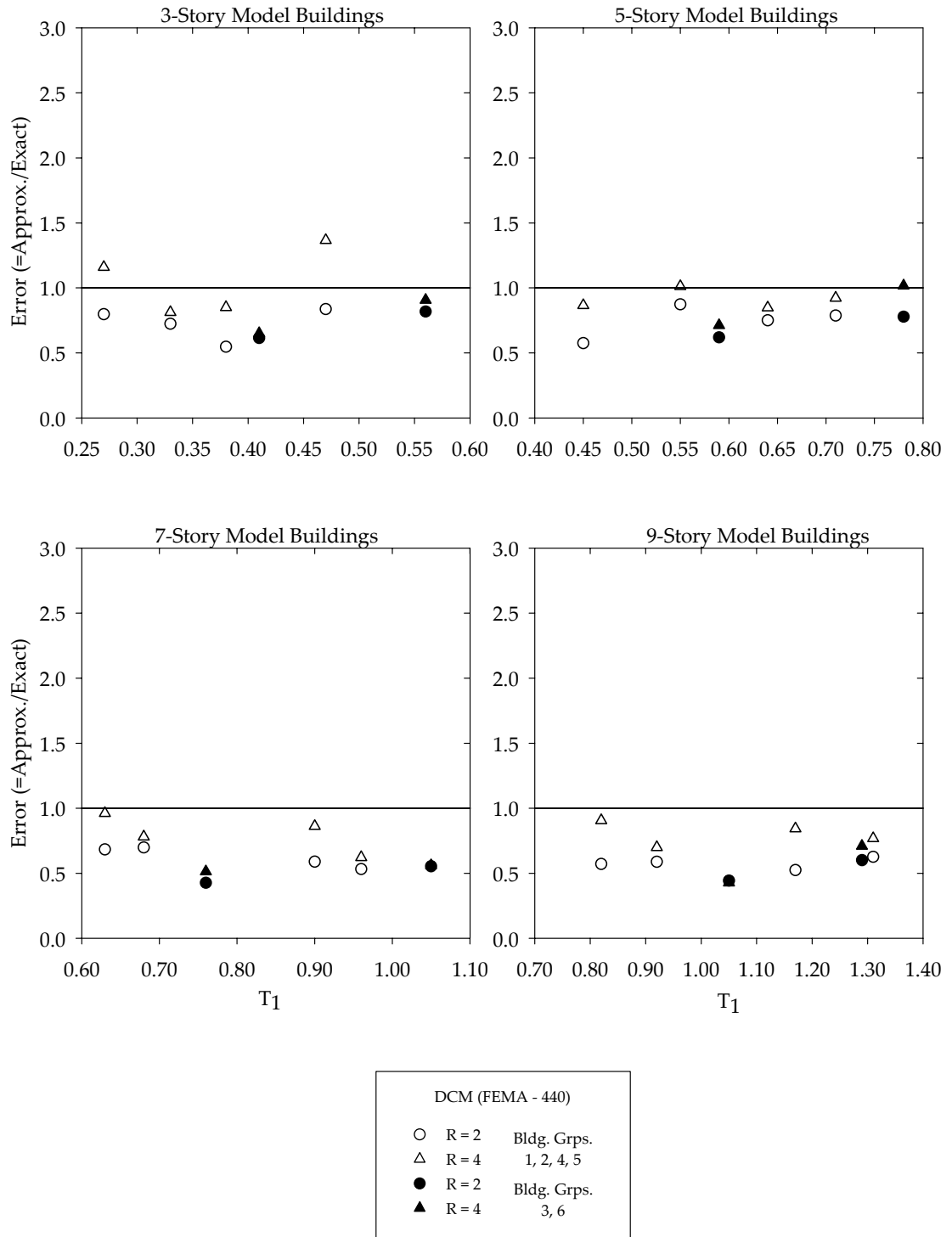


Figure C.7 Error statistics for 10% probability of non-exceedance for improved DCM conditional to R for $MIDR_{ie}$

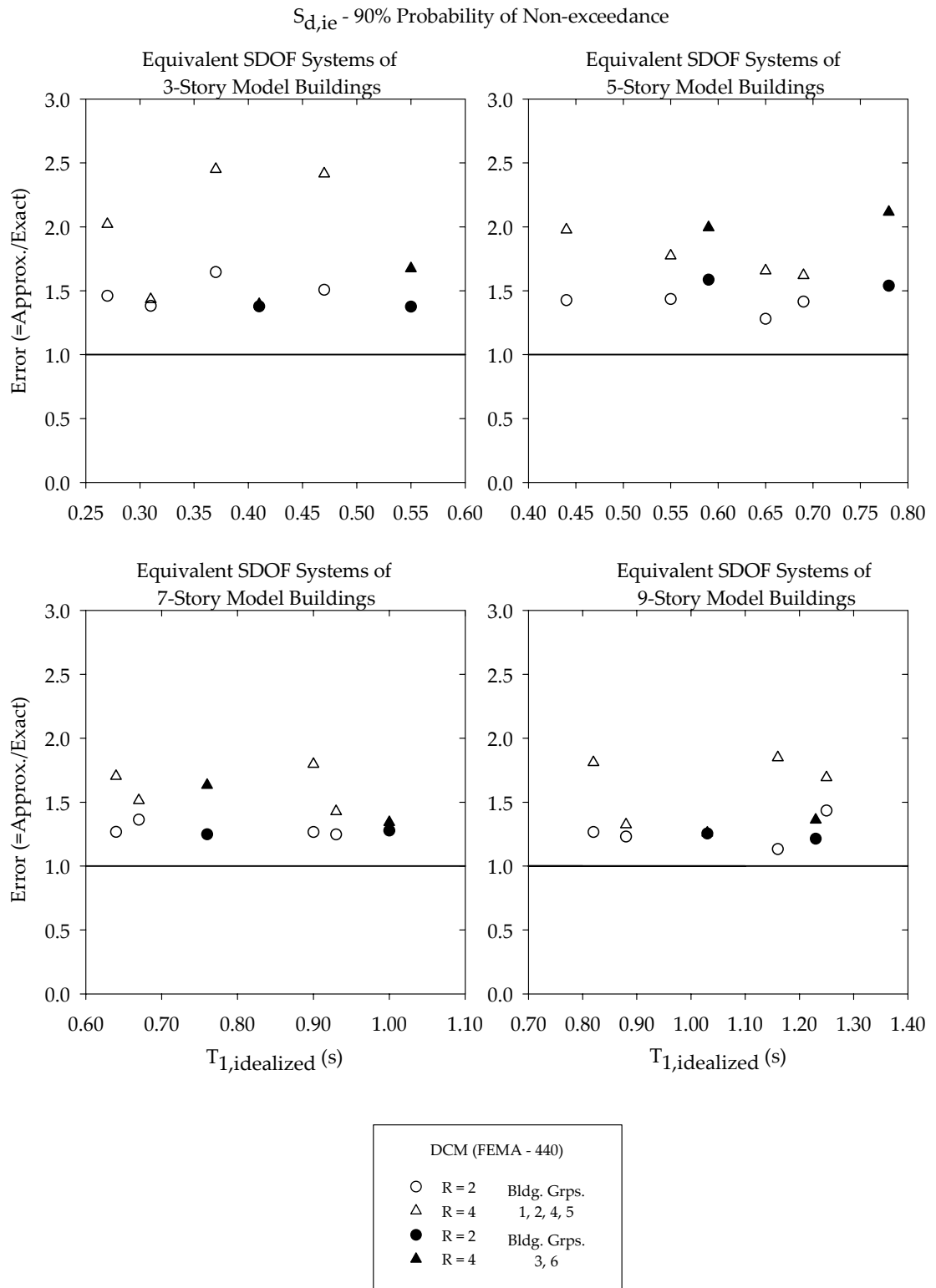


Figure C.8 Error statistics for 90% probability of non-exceedance for improved DCM conditional to R for $S_{d,ie}$

MRDR_{ie} - 90% Probability of Non-exceedance

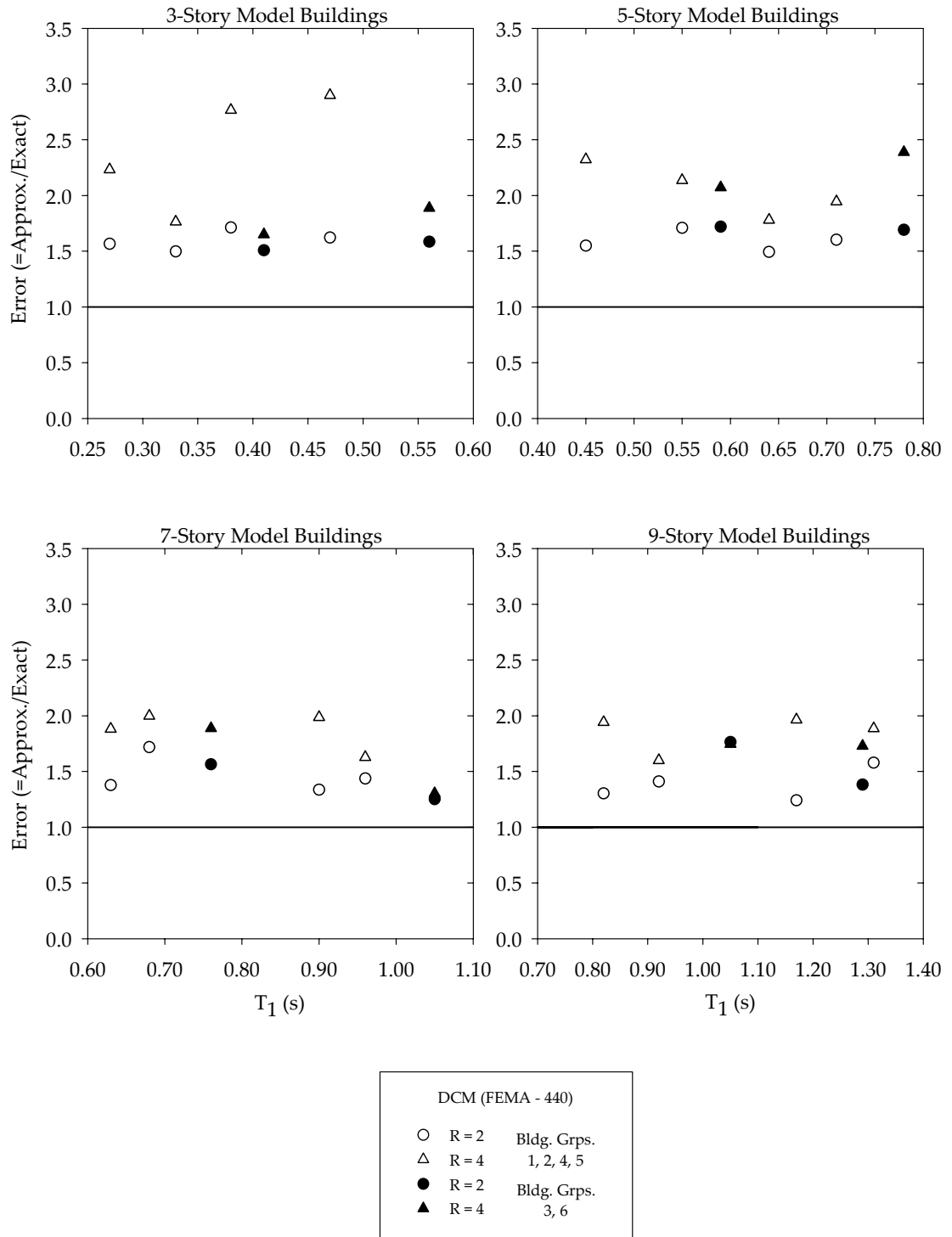


Figure C.9 Error statistics for 90% probability of non-exceedance for improved DCM conditional to R for $MRDR_{ie}$

MIDR_{ie} - 90% Probability of Non-exceedance

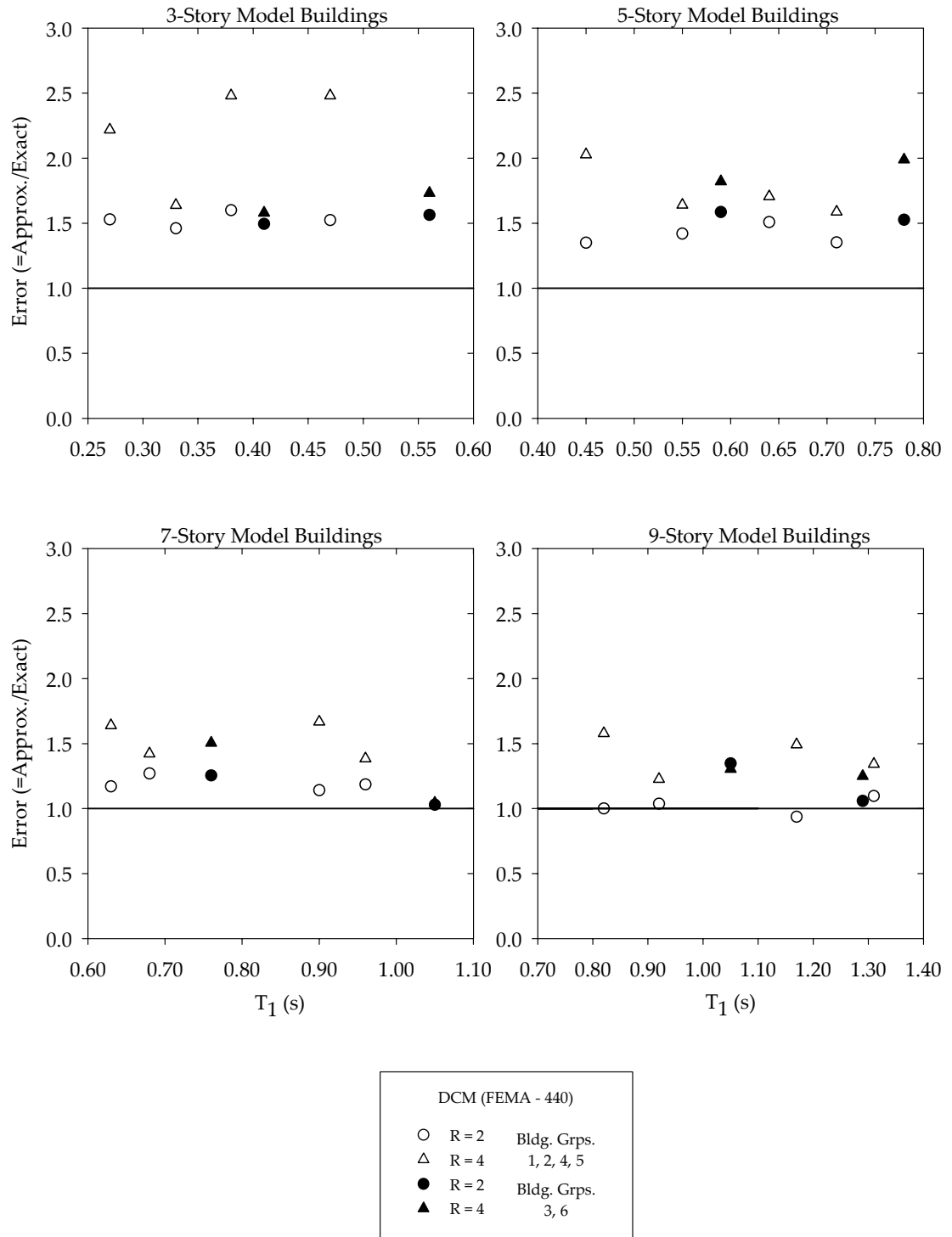


Figure C.10 Error statistics for 90% probability of non-exceedance for improved DCM conditional to R for MIDR_{ie}

APPENDIX D

RELEVANT FIGURES OF CHAPTER 5

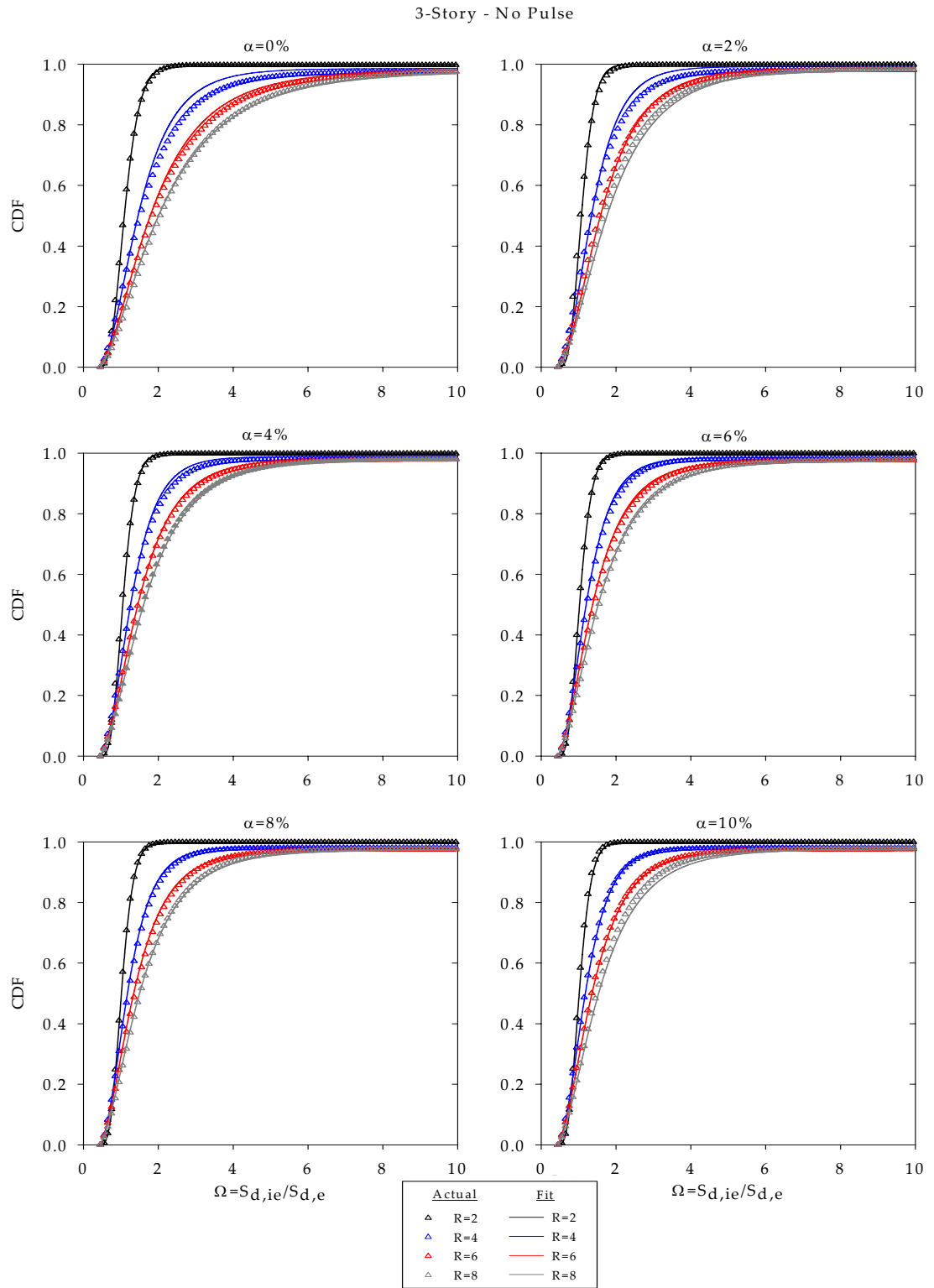


Figure D.1 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 3-story buildings-no pulse

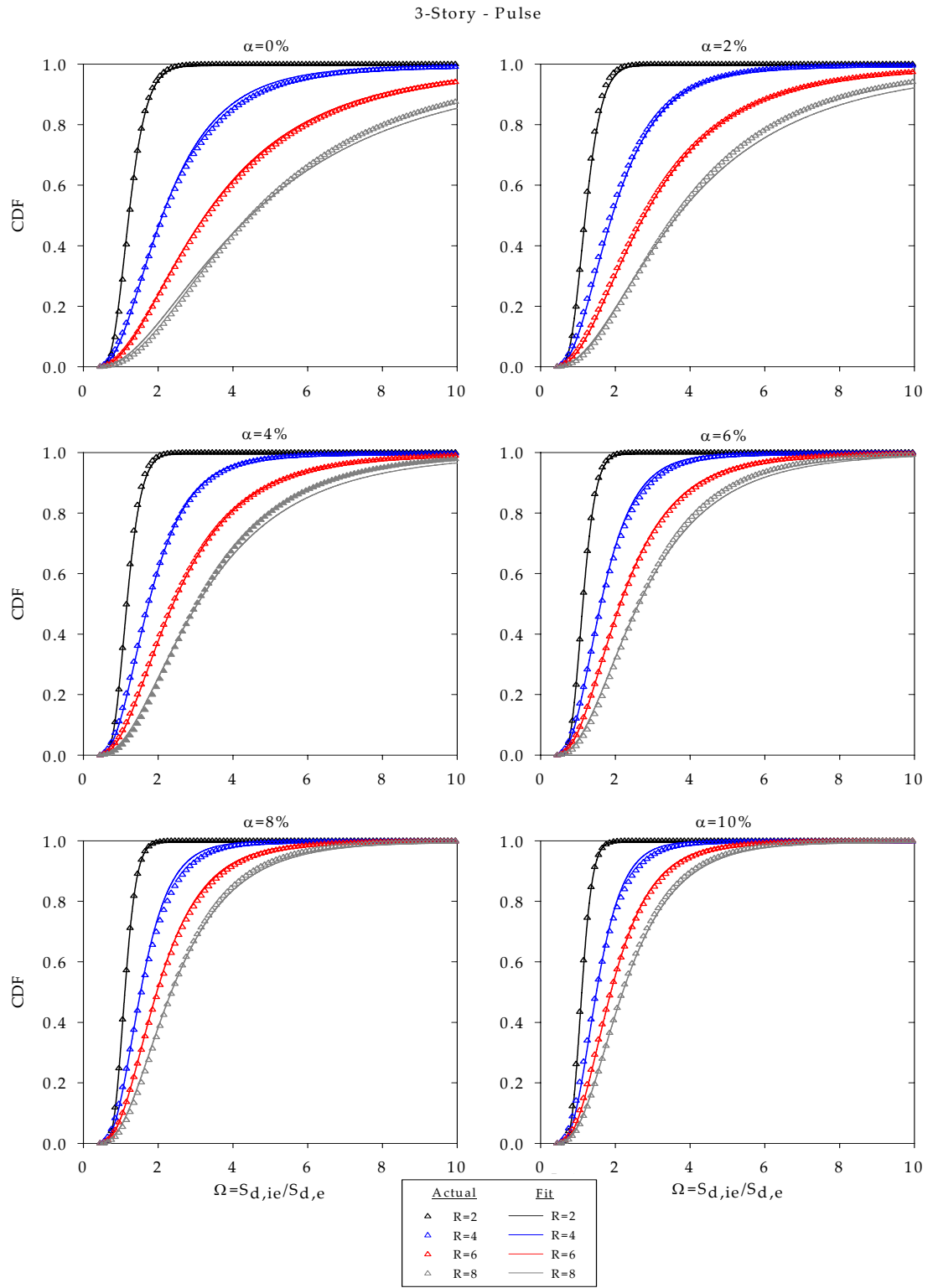


Figure D.2 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 3-story buildings-with pulse

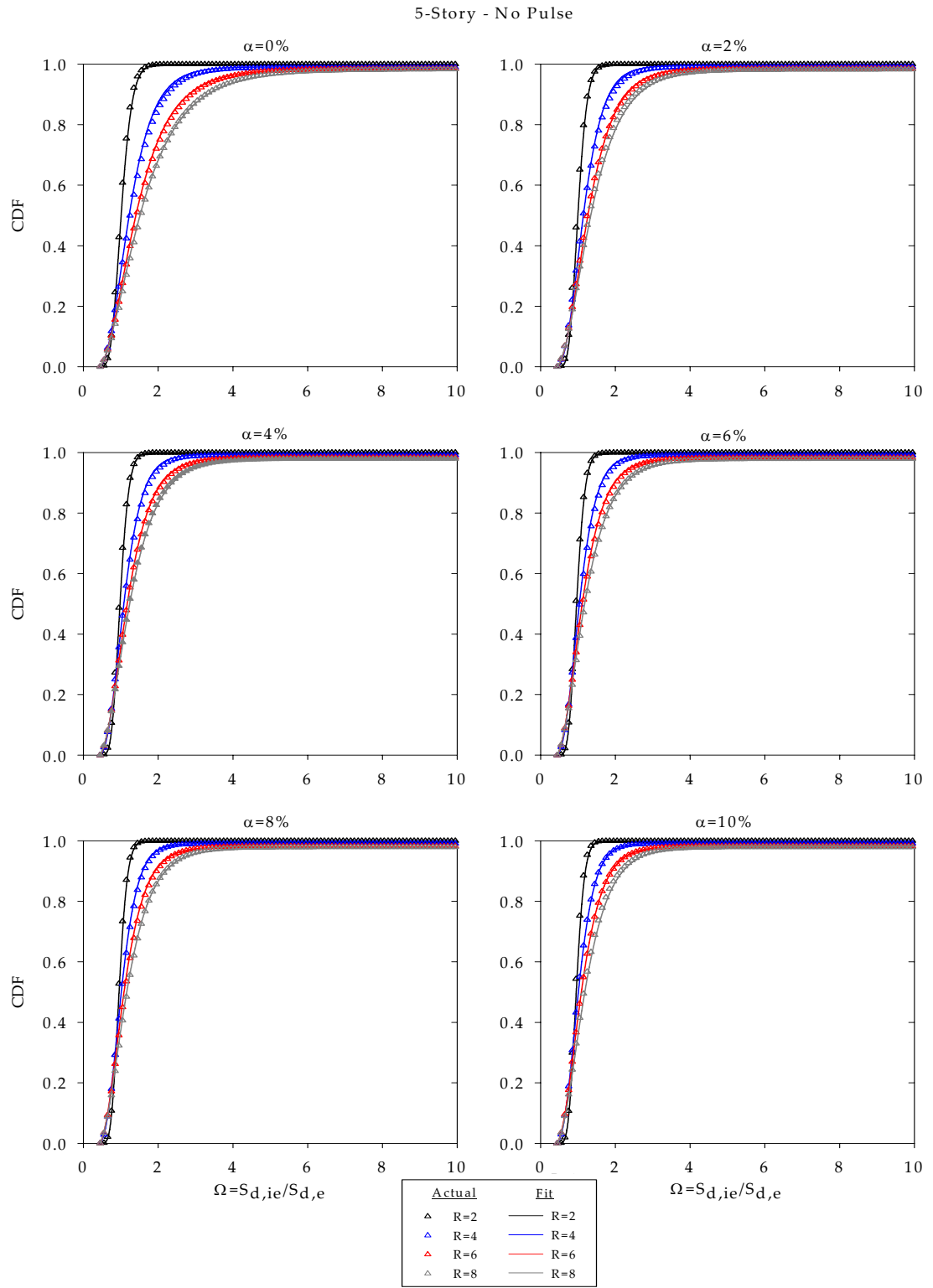


Figure D.3 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 5-story buildings-no pulse

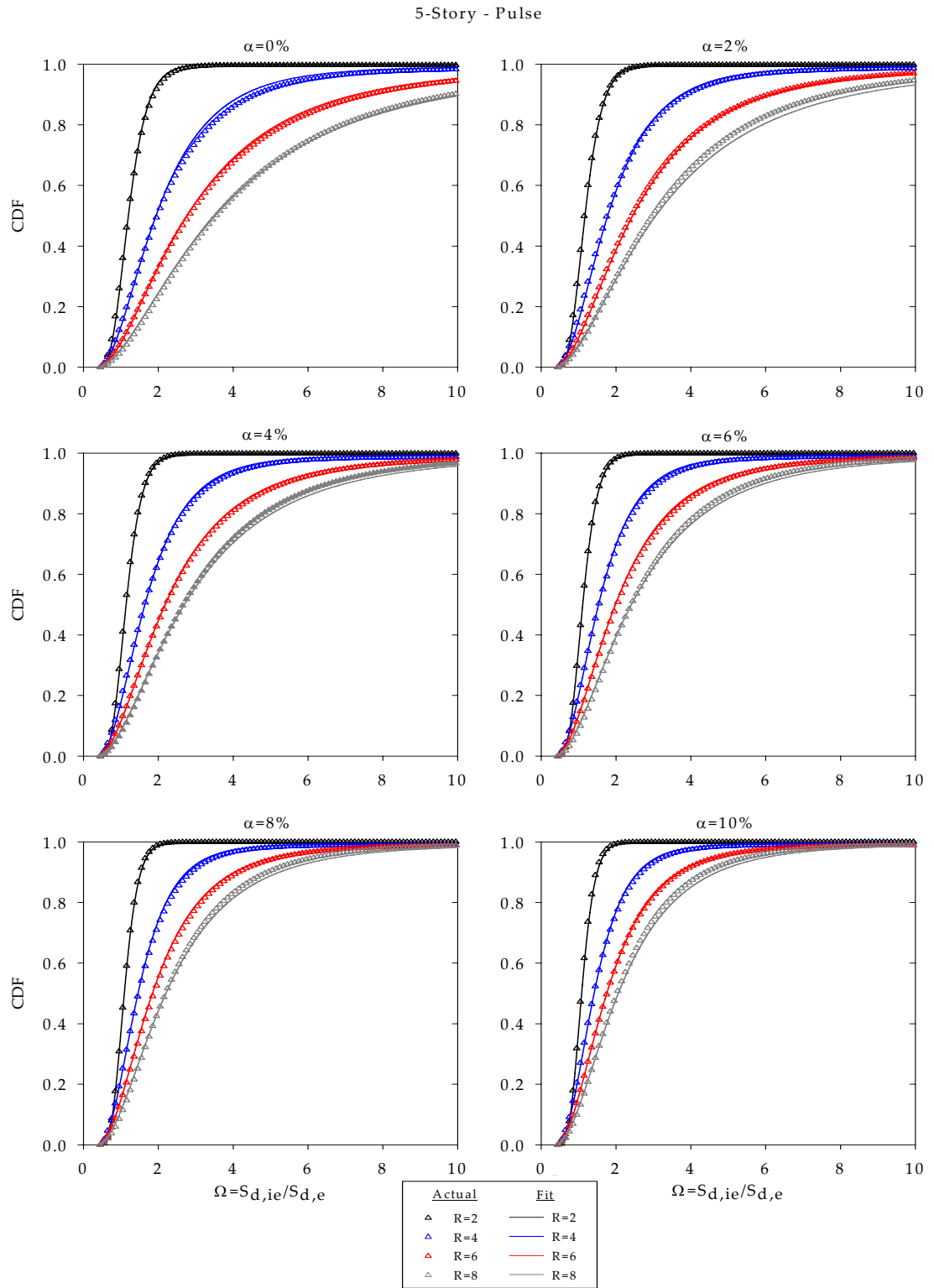


Figure D.4 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 5-story buildings-with pulse

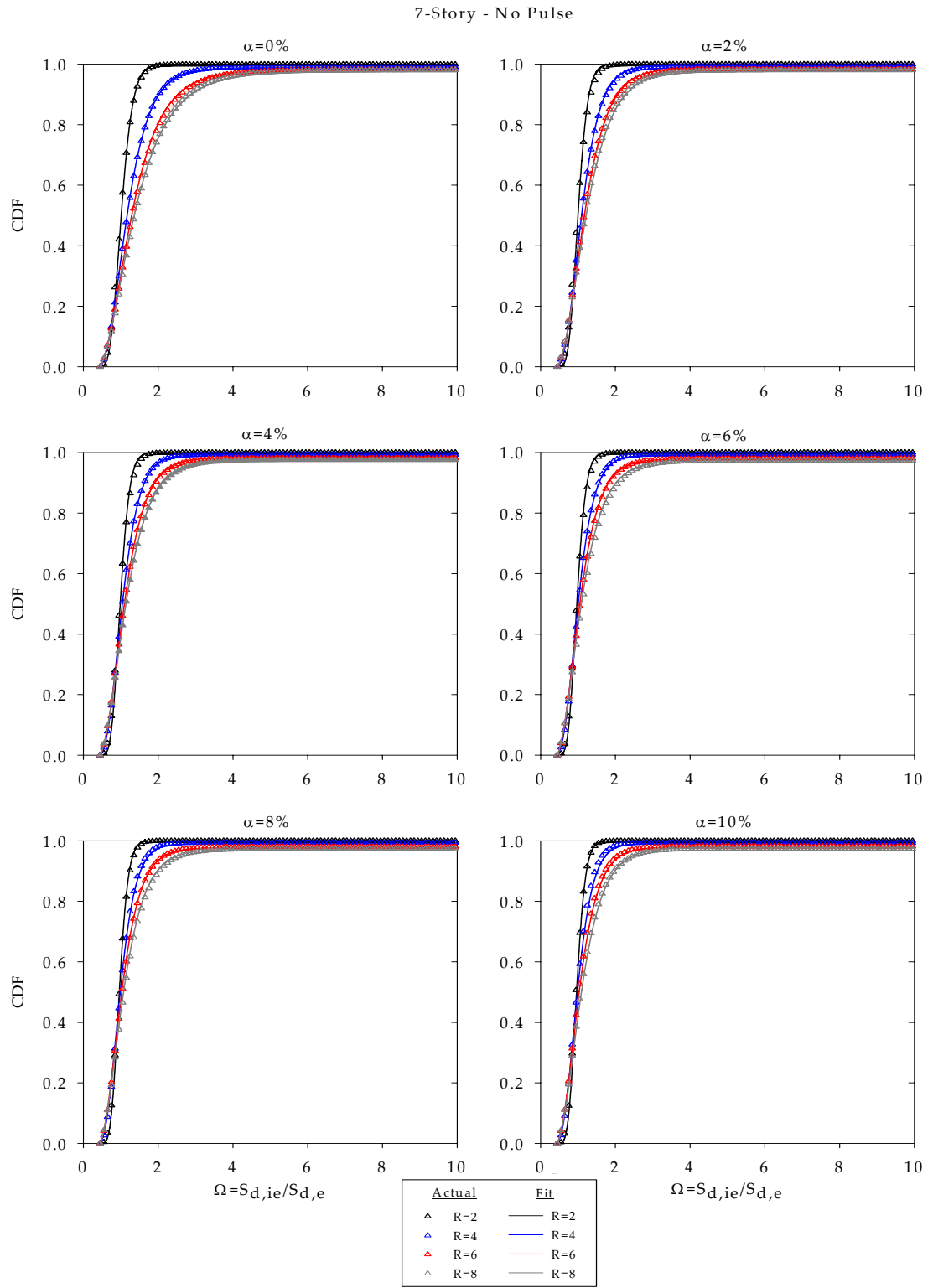


Figure D.5 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 7-story buildings-no pulse

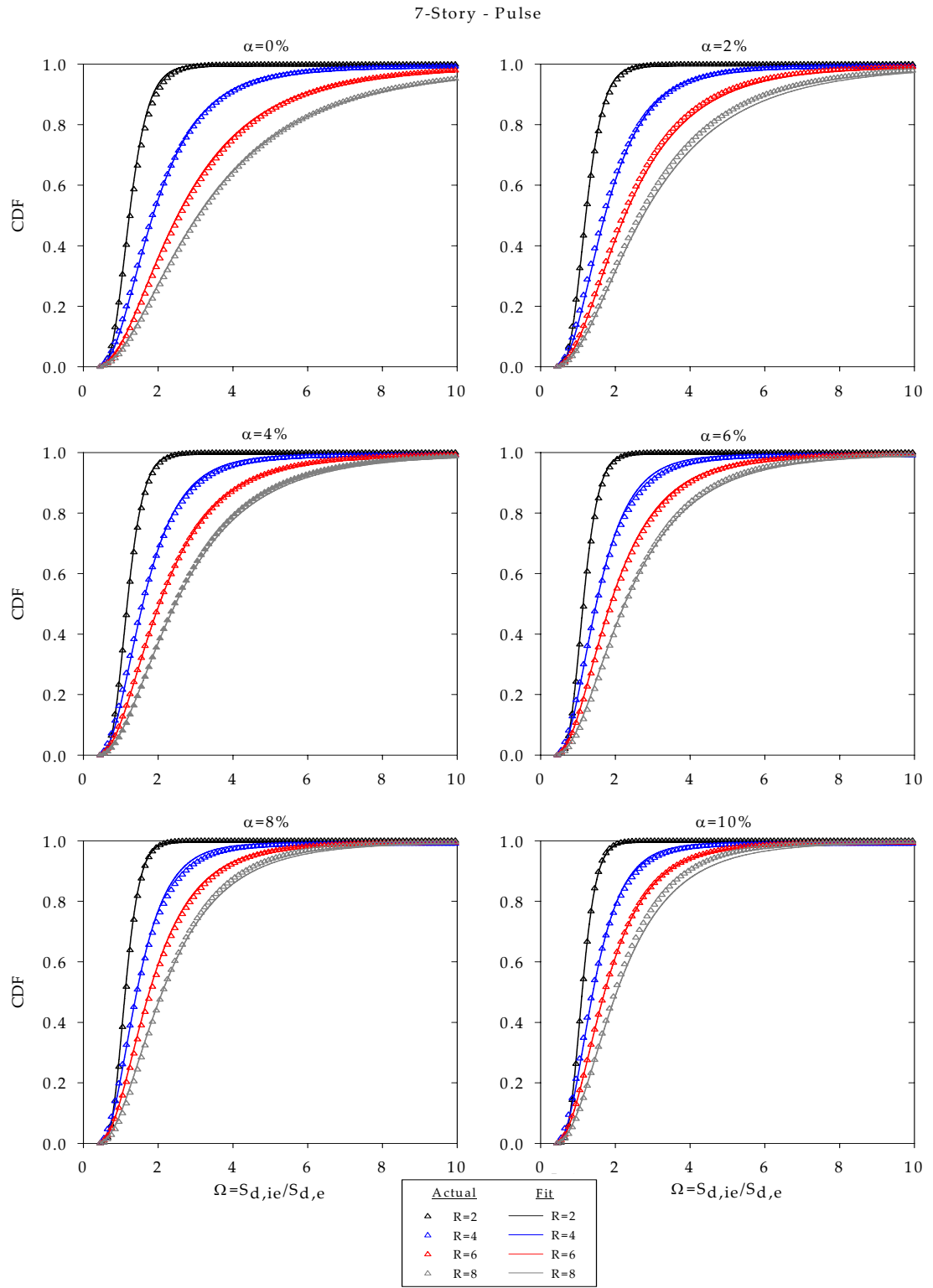


Figure D.6 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 7-story buildings-with pulse

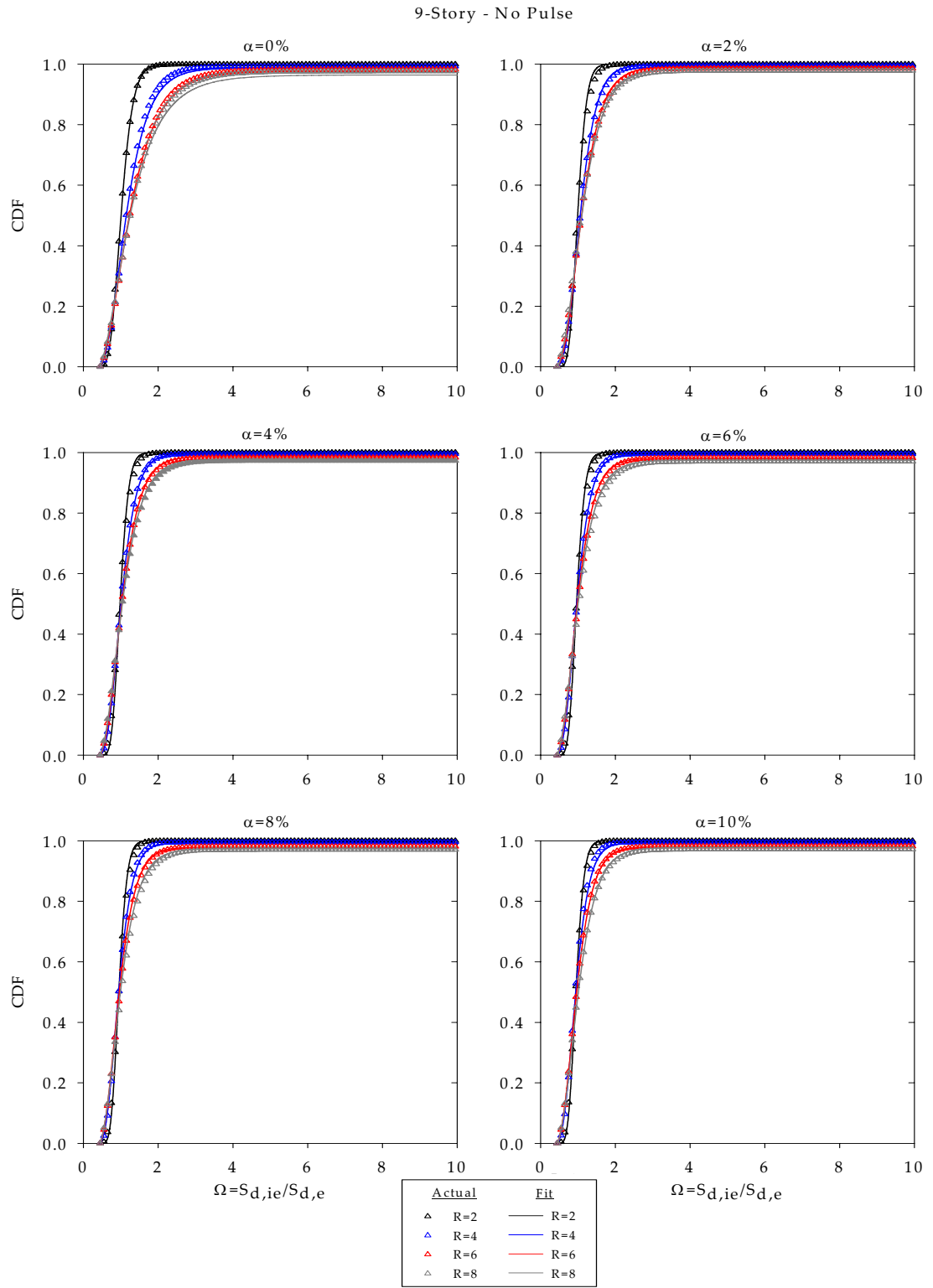


Figure D.7 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 9-story buildings-no pulse

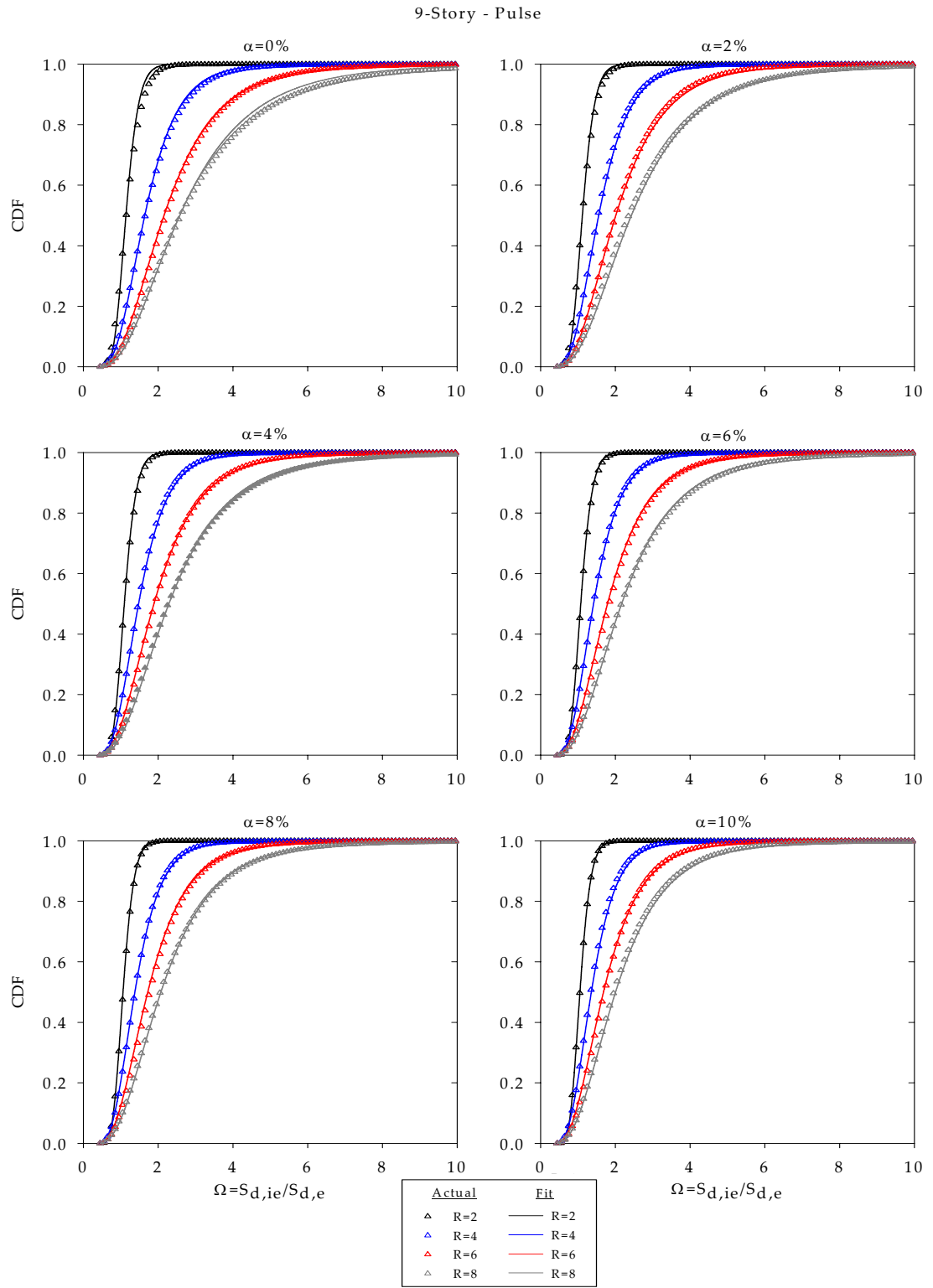


Figure D.8 Comparisons of log-normal distribution functions of Ω from the regression coefficients and the actual statistics of nonlinear RHA: 9-story buildings-with pulse