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&
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MASTER THESIS

**SEISMIC LOAD PATTERN FOR QUASISTATIC NONLINEAR ANALYSIS
(Case Study for RC Frame Residential Buildings in Antakya, Turkey)**

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DEDICATION

To my mother

All that I am my mother made me.

John Quincy Adams (1767-1848)

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LIST OF SYMBOLS & IDs

ADRS	Acceleration-Displacement Response Spectrum
br	Brace
CP	Control Point
D	Roof Displacement
D1, D2, D3	Epicentral Distance Range 1, 2, 3
d_e	Epicentral Distance in kilometres
DOF	Degree of Freedom
EC8	Eurocode 8
LP	Load Pattern
LS	Life Safety
M	Mass Center
M1,M2,M3,M4	Surface Magnitude Range 1, 2, 3, 4
MDS	Magnitude Distance Soil
MI	Masonry Infill
M_s	Surface Magnitude
NL	Nonlinear
PH	Plastic Hinge
PO	Pushover
POA	Pushover Analysis
RC	Reinforced Concrete
RS	Response Spectrum
S	Stiffness Center
S_a	Spectral Acceleration
SA,SB,SC,SD	Soil class A, B, C, D
S_d	Spectral Displacement
SDOF	Single Degree of Freedom
SMDN	Strong Motion Datascape Navigator
SRSS	Square Root of Sum of Squares
STR1,STR2,STR3	Structure 1, 2, 3 without MI Walls
STR1,STR2,STR3-infill	Structure 1, 2, 3 with MI Walls
TH	Time History
V	Base Shear Force
Institutions/Standards	
ATC	American Technology Council
EDAC	Earthquake Damage Analysis Center
FEMA	Federal Emergency Management Agency

CHAPTER 1 – INTRODUCTION

1.1 Motivation

During the course of last decade or so, there has been a significant surge of interest in developing processes and procedures that incorporate performance based concepts in design and evaluation of structural system, the findings and observations presented raise important issues that point to the need for additional research and the development and validation of methodologies that minimize uncertainty and maximize confidence. Beyond question, apart from many advantages, these methodologies have some shortcomings which need to be overcome.

In close collaboration with local partners, Earthquake Damage Analysis Center (EDAC) of Bauhaus-Universität Weimar currently runs a project on Seismic Assessment and Mitigation in the city of Antakya, capital of the Turkish province Hatay [1]. The main objective of the project, called Seismic Risk Assessment and Mitigation in the Antakya-Maraş Region on the basis of Microzonation, Vulnerability and Preparedness Studies, abbreviated as SERAMAR, is to utilize current tools for earthquake risk mitigation within an environment where research entities from the European research area, local universities in Turkey and professional associations as well as local governments enable the establishment of a unique partnership that would serve as a model of similar future endeavours. In order to reach this goal, a thorough Microzonation of the urban area in the forefront of a damaging seismic event has to be conducted including the identification of the vulnerability of the local building stock, and social preparedness.

1.2 Tasks and Objectives

The primary focus of this study is to provide an analytical approach to develop the generalized seismic load patterns which the structures experience during different earthquakes at their highest demand levels and develop general forcing function which can be used as load patterns for seismic performance analysis of structures.

The layouts of typical multi-storeyed RC buildings need to be elaborated and transformed into mechanical models using ETABS software tool. The scatter of buildings' response is required to be examined for site-specific tri-component strong motion records. Performance characteristics, damage progression and critical ground motion levels have to be predicted to derive the appropriate load pattern at the time of the highest demand.

A second emphasis has been given towards the performance evaluation of the existing structures presented herein. In this context, the need of retrofitting has to be discussed taking into account the interaction with masonry infill walls and seismic action types.

Tasks are given in detail as follows:

Task 1: Modelling and Linear analysis of RC buildings in the study area

- Modelling of 3, 5 and 7 storey RC frame structures (with and without infills);
- Modification of structural input parameters to calibrate the model; and
- Check of the response in both horizontal directions.

Task 2: Prediction of seismic load pattern for different sets of strong motion data

- Elaboration of three component strong motion recordings;
- Classification of action types (magnitude, distance, subsoil conditions) and selection of representative records;
- Evaluation and visualization of time dependent response of linear time history analysis; and
- Identification of different load pattern at time of the highest demand; repetition for different action types.

Task 3: Prediction of the nonlinear response

- Detailing of reinforcement in columns and beams, and modelling for nonlinear analysis; and
- Determination of capacity curves for the predicted load pattern (vertical and horizontal distributions)

Task 4: Comparison between linear and nonlinear response

- Time history analysis for a 3-storey frame with RAM PERFORM (fibre element model) with ground motion from task 2;
- Identification of changes in load pattern in case of damage response induced by increasing load factors of the horizontal strong motion components; and
- Discussion of consequences for the load model for Pushover analysis.

Task 5: Need and efficiency of rehabilitation measure

- Prediction of the damage progression by conventional PO analysis approach and by the refined approach with the realistic load assumption;
- Identification of critical structural zones; and
- Proposals for measures of strengthening.

1.3 Organization and Contents

This document is organised into one volume. The main body is divided into seven chapters. Fig. 1 shows the main constituents of all the chapters. A virtual second part of this report consists of six annexure which contain supporting material.

Chapter 2 presents an overview of the data available and their selection methodology for this study.

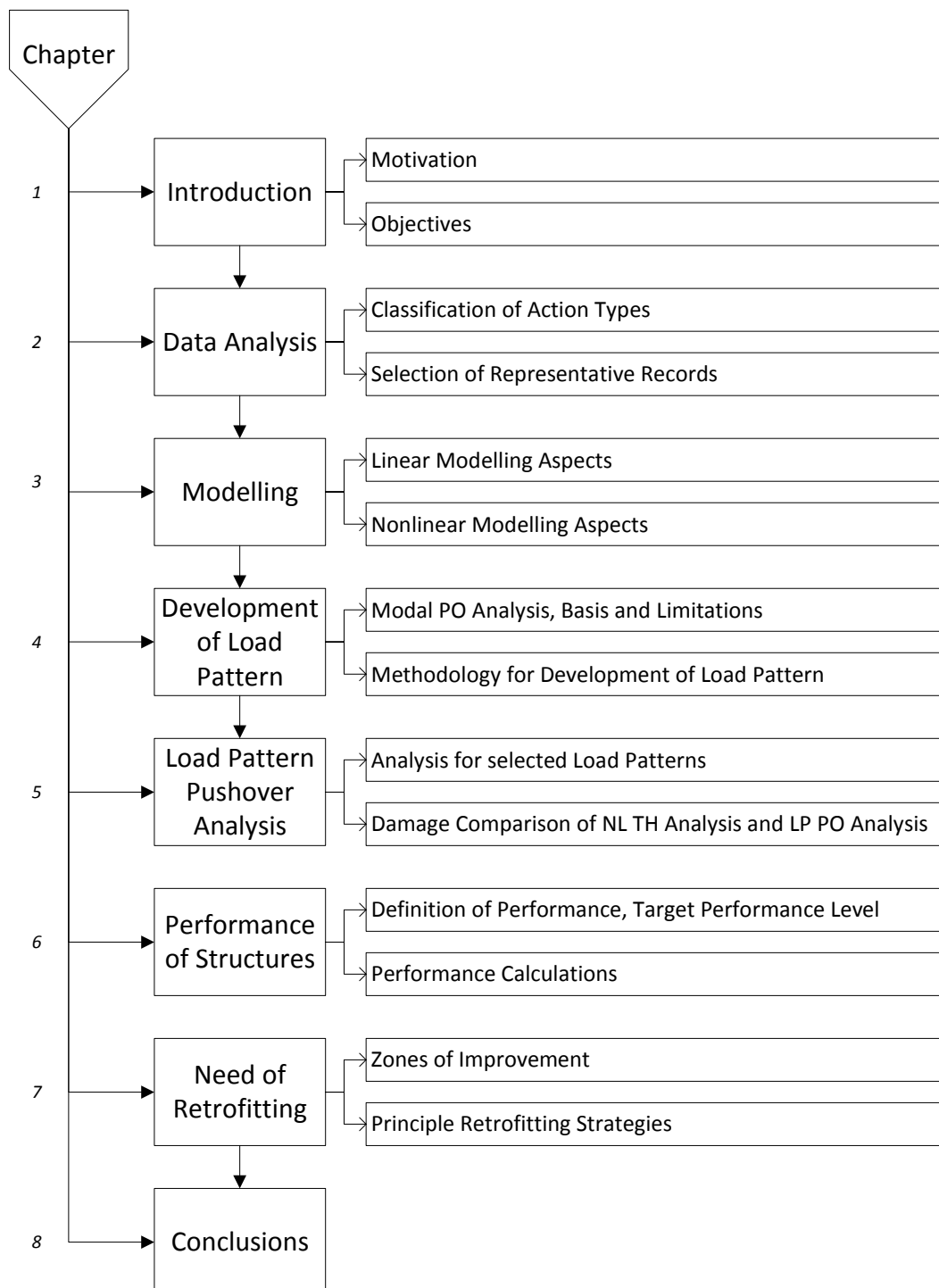


Fig. 1: Organization and Contents

Chapter 3 features the linear and nonlinear modelling details of the three structures discussed.

Chapter 4 gives an overview of conventional modal pushover analysis and its limitations. Methodology for obtaining the displacement based load patterns is also presented.

Chapter 5 comprises of nonlinear static pushover analyses of all the structures, and these have been carried out for the selected load patterns. Comparison of nonlinear time history results and load pattern based pushover analysis results is also given.

Chapter 6 discusses the performance of the structure for different load cases in view of the local hazard level and target performance limits and find the need if the structures need to be retrofit.

Chapter 7 carries some principle retrofitting strategies and highlights the needs of the structures which underperformed.

Finally, chapter 8 concludes the main body of this document with conclusion.

CHAPTER 2 – SEISMIC ACTIONS AND DATA ANALYSIS

For the purpose of developing load patterns for seismic response of structures, the earthquake data compiled by Ambraseys et al. (2004) [2] in the form of a CD ROM have been used. This CD ROM contains corrected acceleration, velocity and displacement time histories of 462 triaxial strong-motion records from 110 earthquakes and 261 stations in Europe and the Middle East. In addition, it also contains linear elastic response spectra, inelastic (elastoplastic) constant strength spectra and inelastic (elastoplastic) constant ductility spectra. The associated earthquake, station and waveform parameters of these 462 records are also present on the CD ROM so that full use of the data can be made. All of these data can be viewed and analyzed using the Strong-Motion Datascape Navigator (SMDN) software provided on this CD ROM. SMDN allows to export the time histories, spectra and the associated parameters of records to ASCII files in a standard format. All the data are free-field recordings.

2.1 Classification of Action Types

In order to classify the earthquake data in dependence with the magnitude and distance and soil types, surface magnitude M_s and epicentral distance d_e are taken as reference entities. Selection of surface magnitude as reference entity than moment magnitude or some other also gives the chance to compare the action types in the form of response spectra with EC8 spectra.

2.1.1 Magnitude-Distance based Action Types

Magnitude-wise, the seismic events have been divided into following four different ranges:

- M1, representing surface magnitude M_s of range 3.2-4.5;
- M2, representing surface magnitude M_s of range 4.5-5.5;
- M3, representing surface magnitude M_s of range 5.5-6.5; and
- M4, representing surface magnitude M_s of range 6.5-7.8,

whereas distance-wise, divided into following three ranges:

- near-field D1 (d_e 0-20 km) event;
- transition-field D2 (d_e 21-40 km); and
- far-field D3 (d_e 41-200 km) events.

The usual assumption that the earthquakes distant more than 200 km are structurally unimportant has been employed. Thus, out of total of 462 events, 444 events lie within the epicentral range of 200 km.

2.1.2 Site-Class based Action Types

All the records have already been classified according to the site conditions as per Eurocode 8 (EC8). The data corresponding to only first four soil types (SA to SD) are present in this CD ROM.

Fig. 2.1 shows the distribution of strong motion data used in this study. Solid dots show the events selected for detailed analysis (see section 5.1). Fig. 2.2 shows the distribution of data across countries.

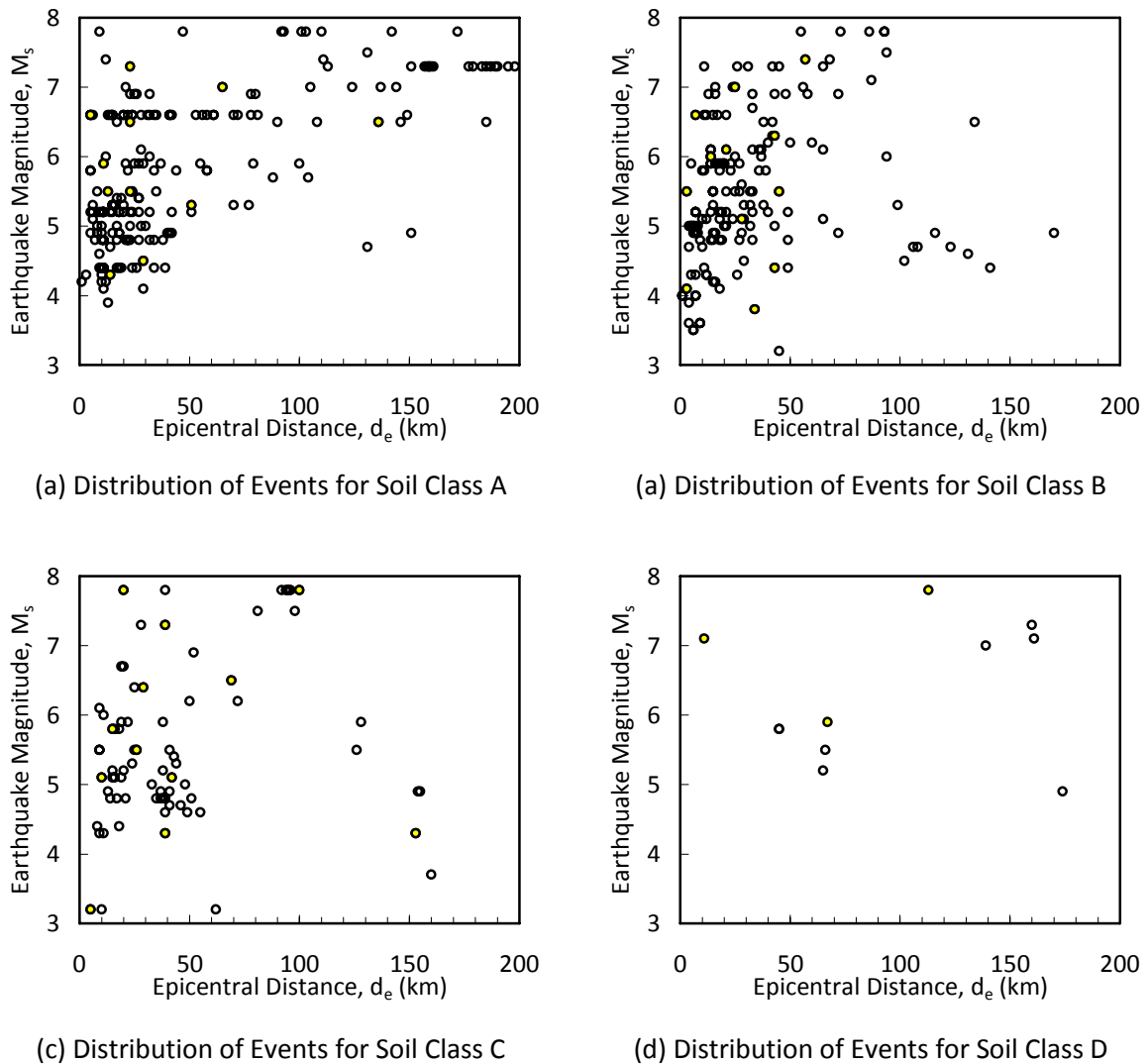


Fig. 2.1: Distribution of Strong Motion Data used in this study [2]

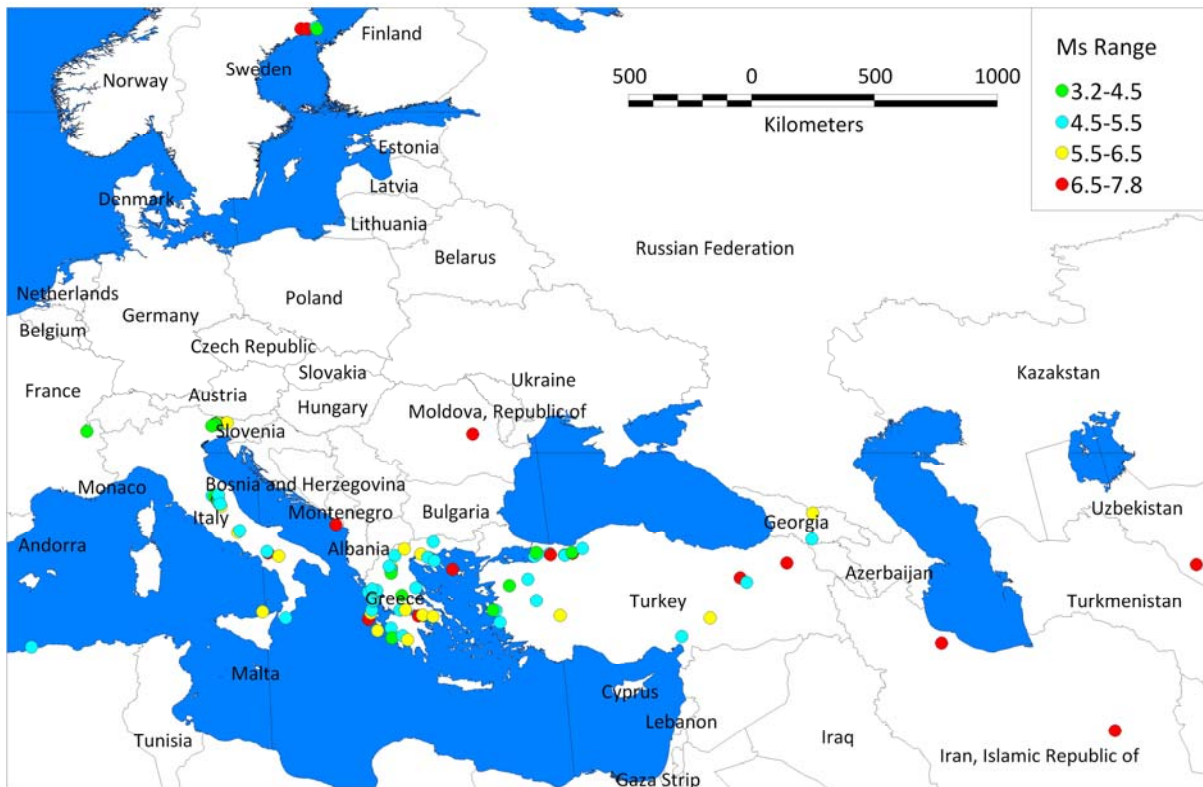


Fig. 2.2: Spatial Distribution of Earthquakes across Countries

Fig. 2.3 gives a perspective view of all the 444 events categorized according to the magnitude-, soil- and distance class scheme presented above and also gives a pictorial representation of the relative frequency of events.

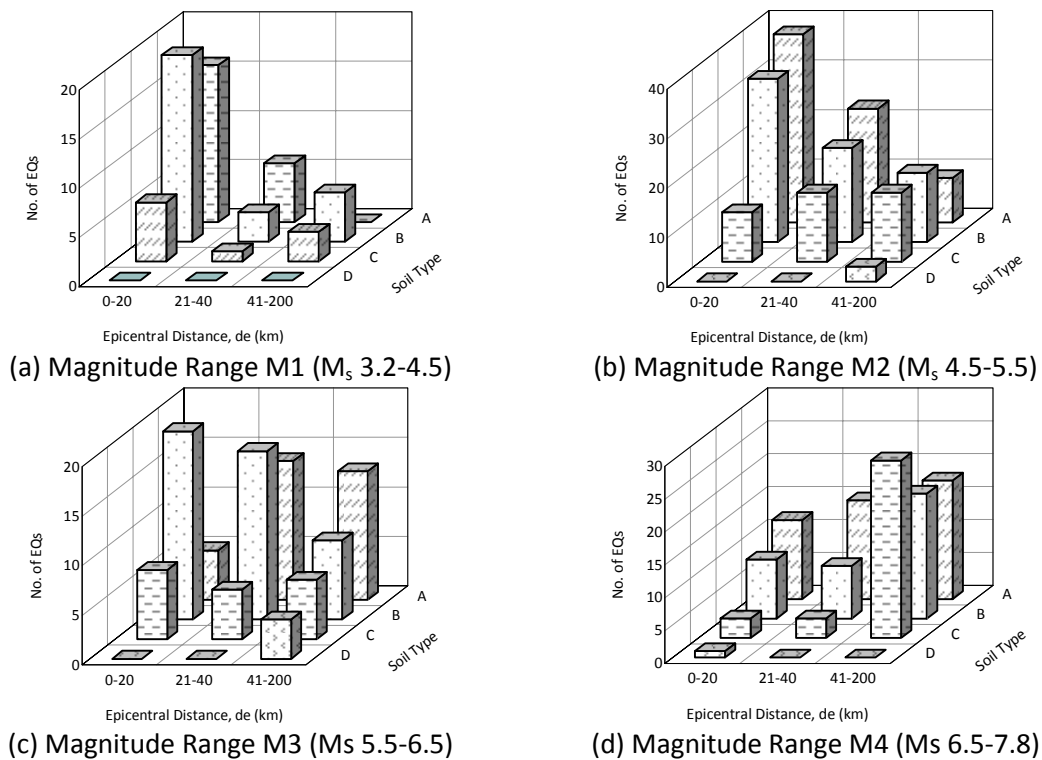


Fig. 2.3: 444 Earthquakes classified according to Magnitude, Soil type and Distance class

2.2 Selection of Representative Records

To select a representative record for each magnitude-distance-soil (MDS) combination for the development of seismic load pattern, elastic acceleration response spectra corresponding to 5% damping ratio have been drawn for all the records. Thereafter, for each MDS combination, an acceleration response spectrum according to EC8 [3] has been approximated. Records matching closely to these approximated response spectra have been chosen as representative record for the development of seismic load pattern. Fig. 2.4 shows this selection approach.

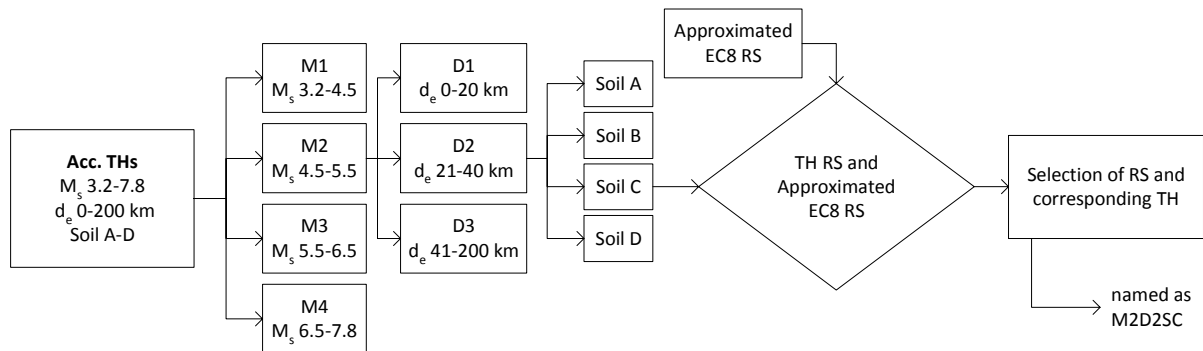


Fig. 2.4: Process of Selection of Representative Time History Records

Following the said approach, Fig. 2.5 represents acceleration response spectra derived from time histories and approximated EC8 spectra for one earthquake only. Response spectra for all other MDS combination can be seen in Annexure A. Details regarding recording stations, depth, waveform ID etc. for all the events can also be seen in Annexure A. Legend of each graph shows the selected record and EC8 approximated spectrum with PGA. Names of all the selected records and IDs assigned to them are tabulated in Table 2.1.

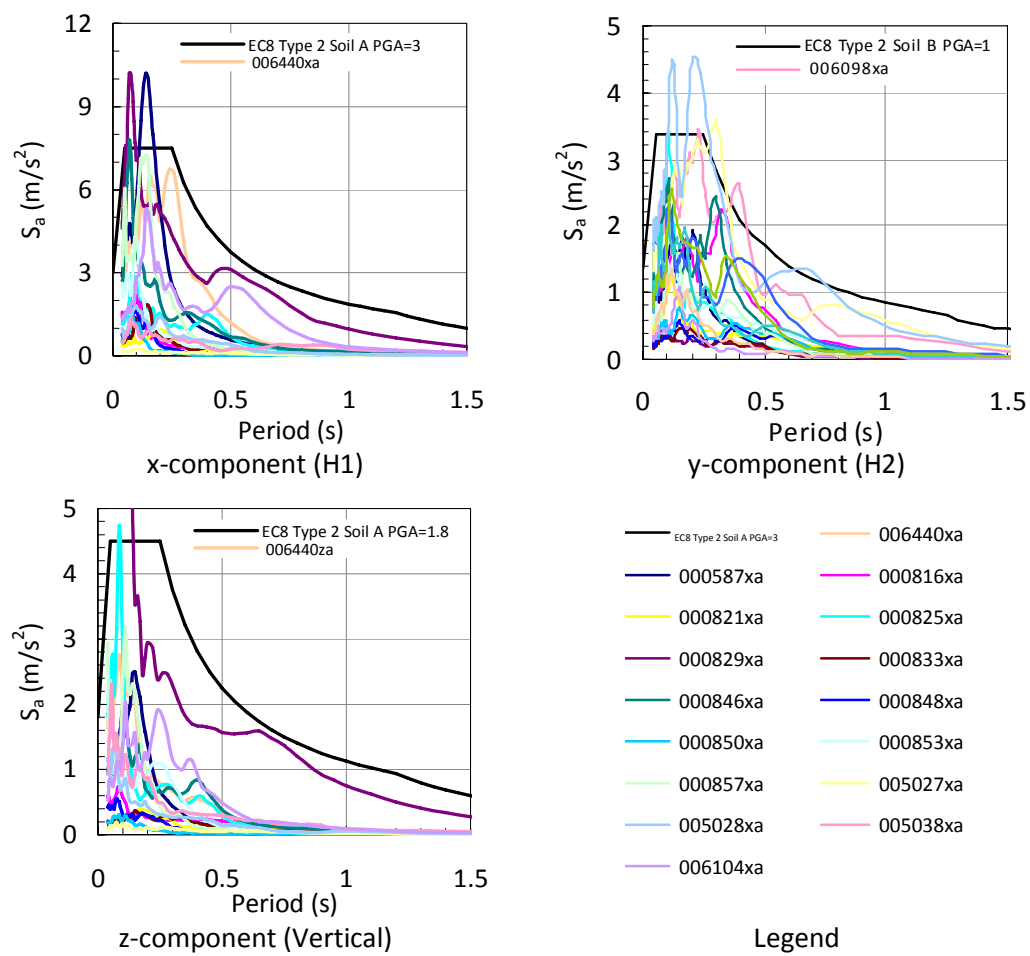


Fig. 2.5: Time History Response Spectra and Approximated EC8 Response Spectra

(complete set of figures can be found in Annexure A)

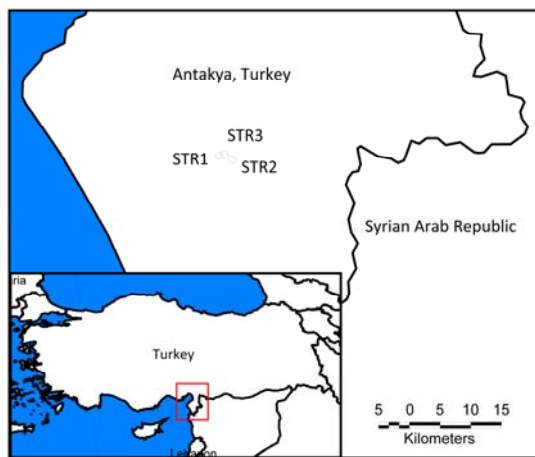
Table 2.1: Names of Representative Acceleration Time Histories

Magnitude Range	Ep. Distance d_e (Km)	Soil Type	Time History IDs	Waveform ID in CD ROM
M1 (M_s 3.2-4.5)	0-20 (D1)	A	M1D1SA	6440
		B	M1D1SB	6098
		C	M1D1SC	2916
		D	-	-
	21-40 (D2)	A	M1D2SA	6153
		B	M1D2SB	7060
		C	M1D2SC	784
		D	-	-
	41-200 (D3)	A	-	-
		B	M1D3SB	5029
		C	M1D3SC	6994
		D	-	-
M2 (M_s 4.5-5.5)	0-20 (D1)	A	M2D1SA	593
		B	M2D1SB	591
		C	M2D1SC	558
		D	-	-
	21-40 (D2)	A	M2D2SA	763
		B	M2D2SB	336
		C	M2D2SC	1415
		D	-	-
	41-200 (D3)	A	M2D3SA	495
		B	M2D3SB	538
		C	M2D3SC	337
		D	M2D3SD	613
M3 (M_s 5.5-6.5)	0-20 (D1)	A	M3D1SA	594
		B	M3D1SB	146
		C	M3D1SC	42
		D	-	-
	21-40 (D2)	A	M3D2SA	55
		B	M3D2SB	126
		C	M3D2SC	175
		D	-	-
	41-200 (D3)	A	M3D3SA	5820
		B	M3D3SB	6142
		C	M3D3SC	5837
		D	M3D3SD	375
M4 (M_s 6.5-7.8)	0-20 (D1)	A	M4D1SA	6349
		B	M4D1SB	6263
		C	M4D1SC	1257
		D	M4D2SD	74
	21-40 (D2)	A	M4D2SA	6500
		B	M4D2SB	196
		C	M4D2SC	1560
		D	-	-
	41-200 (D3)	A	M4D3SA	200
		B	M4D3SB	187
		C	M4D3SC	1226
		D	-	-

- indicates that no data are available

CHAPTER 3 – MODELLING OF STRUCTURES

This chapter introduces the structures used in this study, their structural system and modelling aspects for both linear and nonlinear analyses. There are discussed three existing structures, named STR1, STR2 and STR3, located in Antakya, capital of the Turkish province Hatay. Fig. 3.1 shows the location of these buildings and their photos. All the data related to structures have been collected by the members of German TaskForce coming from Earthquake Damage Analysis Center of Bauhaus-Universität Weimar [4]. It is worth mentioning here that STR1 is a 3-storey residential building and the picture has been taken during its construction phase.



(a) Location of structures



(b) Building STR1 (under construction)



(c) Building STR2



(d) Building STR3

Fig. 3.1: Location of the Structures

3.1 Structural Systems of Buildings

All the three buildings are reinforced concrete structures. Frame action constitutes the main load bearing system. Hollow bricks are used for partition walls. All the masonry infill (MI) walls are non-load bearing walls. RC Frames are designed to sustain the gravity loads and, in case of earthquakes, also the horizontal loads. Fig. 3.2 shows the typical structural views giving an idea of member sizes and their spans and storey heights.

3.1.1 Structural System of Building STR1

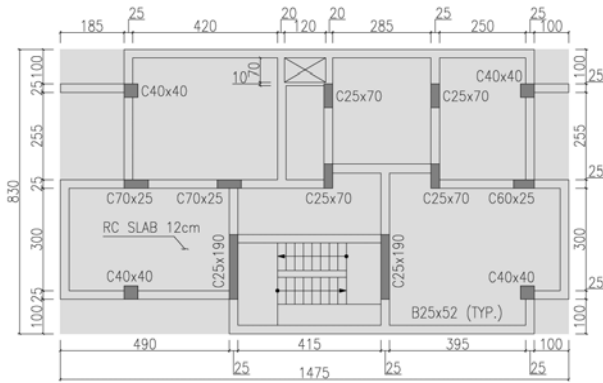
STR1 is a 3-storey reinforced concrete residential house. Each storey has a uniform height of 2.9m. Gravity structural system consists of RC frame action. Floor slabs, 12cm thick, are built from prefabricated RC elements. Diaphragmatic action along with RC frame action provides lateral load resisting system. Ground plans of all the storeys are similar. Column grid remains unchanged over the total height of the building. Brick walls are inserted in all floors, creating small rooms. All MI walls have a thickness of 10cm. Staircase is eccentrically positioned in the central part of the building.

3.1.2 Structural System of Building STR2

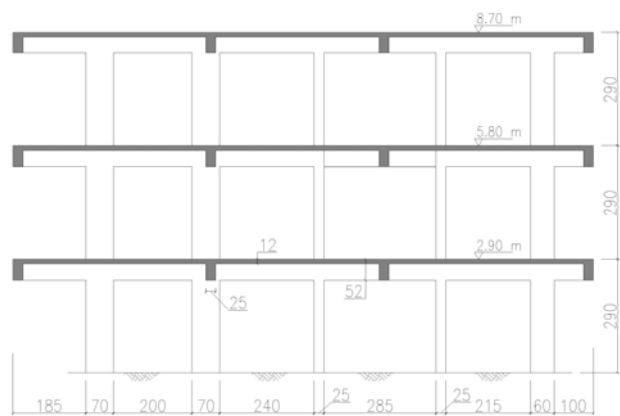
STR2 is a 5-storey reinforced concrete commercial building. Each storey has a different storey height. Main structural system consists of RC frames. Floor slabs, 14cm thick, are cast in-place. Ground floor is used for parking purposes; thus making it act as soft storey. Non-load bearing masonry walls are 15cm thick. Two staircases are present; dog-legged, the main staircase, and single-flight staircase. Notable thing is the irregularities across plans which give rise to non-uniform mass distribution and will lead to high mass-stiffness eccentricity. All the storey levels do not have floor slabs. So for the lumped mass technique, it is possible to treat it as a 4-storey building.

3.1.3 Structural System of Building STR3

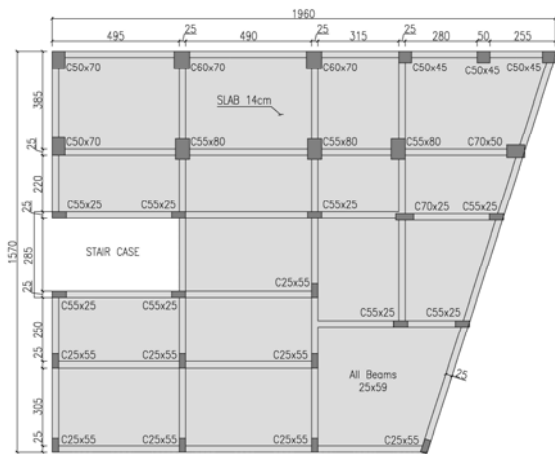
STR3 is a 7-storey reinforced concrete residential apartments building. Among seven storeys, there is present 1-storey basement of height 2.21m. First storey has a height of 3.74m and all other storeys have a common height of 3.06m. Floor slabs are cast in-place of thickness 16cm. Stair- and elevator cases are situated at middle-left, both on the same side, causing a non-uniform mass distribution. Gravity structural system consists of RC frames. Diaphragmatic action of floors, RC frames and RC elevator core furnish as the lateral load resisting system.



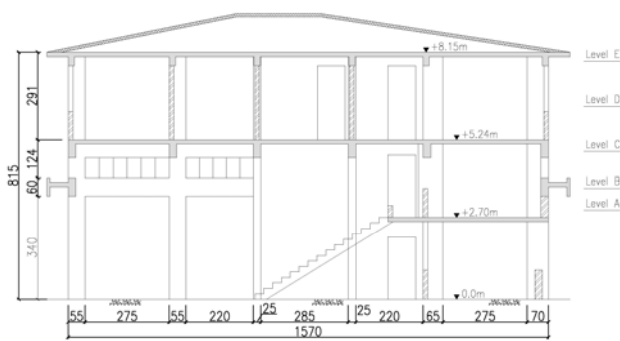
(a) Typical Plan-STR1



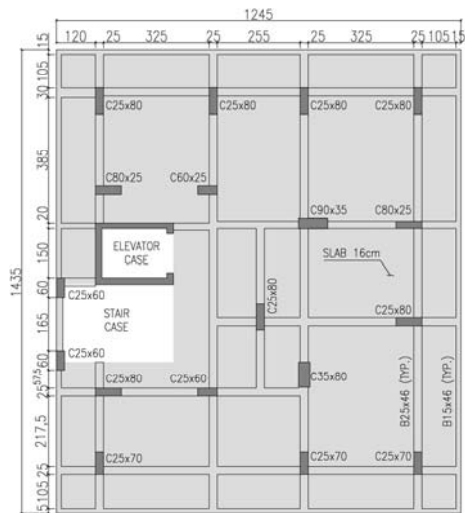
(b) Typical Elevation-STR1



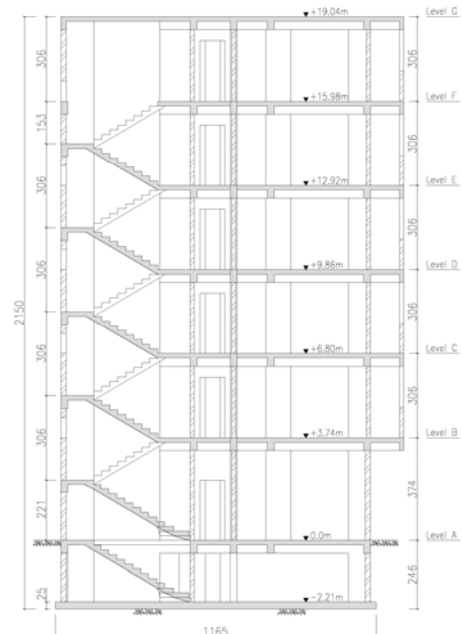
(c) Typical Plan-STR2



(d) Typical Elevation-STR2



(e) Typical Plan-STR3



(f) Typical Elevation-STR3

Fig. 3.2: Typical Plans of three buildings

(All dimensions in cm, levels in m)

3.2 Software Tool Used

For linear-static and nonlinear-static analyses, state-of-the-art structural analysis tool ETABS has been used. ETABS v8.26 is used for linear time history analysis, whereas ETABS v9.0.9 is used for nonlinear static analysis as this version gives an additional benefit over v8.26, of taking masonry infill braces as compression-only elements in nonlinear static analysis.

3.3 Linear Modelling of Structures

3.3.1 Material Properties

Following material properties of reinforced concrete, masonry and reinforcing steel are used in all the calculations.

Table 3.1: Material Properties

Parameter		Concrete	Masonry
Young's Modulus, E^*	kN/m^2	1.375e7	1.75e6
Weight Density	kN/m^3	25	9
Mass Density	t/m^3	2.55	0.918
f_c'	kN/m^2	20 800	6 400
Poisson Ratio, ν	-	0.2	0.1
f_y (steel)	kN/m^2	420 000	-

* 50% reduced values are given (see section 3.3.3)

3.3.2 Load Assumptions and Seismic Weight Contribution

Typical values, used in Turkey, of dead and live loads for buildings have been considered and tabulated in [Table 3.2](#). Seismic weight consists of one times all the dead loads including self weight, floor load and load of masonry infills, plus 30% live load in accordance with EC8.

Table 3.2: Dead and Live Loads

Load Type	Description	Characteristic Value kN/m^2	Seismic Weight Contribution
Dead	on roof	1	1
	on floors and stairs	2	
Live	on roof	1	0.3
	on floors and stairs	1.5	

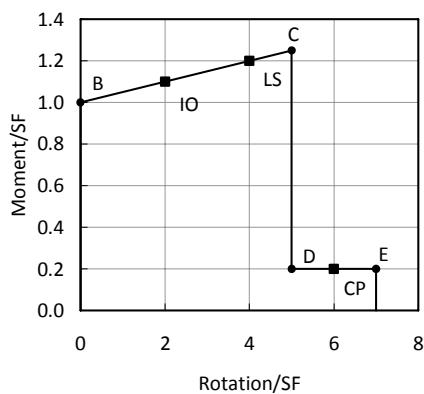
3.3.3 Stiffness Calibration of Structures

As buildings have not been instrumented, explicit calibration, therefore, is not possible. Nevertheless, an erudite treatment of vulnerability analysis of the buildings of the same region by Abrahamczyk et al. (2004) [5] shows that for slightly damaged RC frame systems with and without infills, there's a strong stiffness reduction of approximately 50% which also agrees with the recommended value of in EC8 for cracked sections. Therefore, all the analyses are conducted using half the modulus of elasticity.

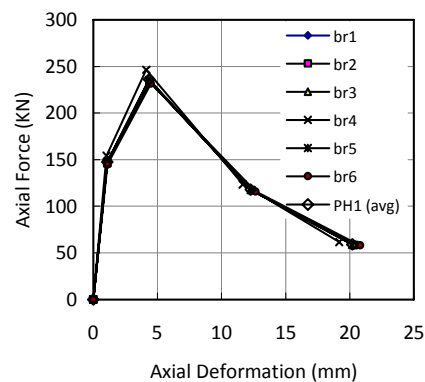
3.4 Nonlinear Modelling of Structural and Non-Structural Components

3.4.1 Material Laws for RC Sections and Masonry Infill Walls

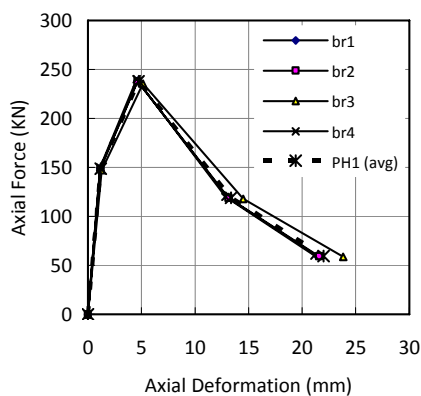
Following scaled moment-rotation relation (Fig. 3.3a) is used for RC cross sections. Masonry infill walls have been taken as compression-only bracing elements, their stiffness is calculated using FEMA-306 [6] and their axial force-deformation relations have been calculated as proposed by Fajfar et al. (2001) [7]. Fig. 3.3b-d shows these relationships. MI Walls, as an instance, of STR1 are categorized into six cross sections for bracing elements. Force-deformation relations for all these cross sections have been found similar to each other (Fig. 3.3b). Therefore, an average of these has been used. Similarly, such material laws are derived for MI walls of structures STR2 and STR3, and averaged where deemed appropriate. Shown are the derived relations but all of them have been used as ideal-plastic laws to cater for software limitation.



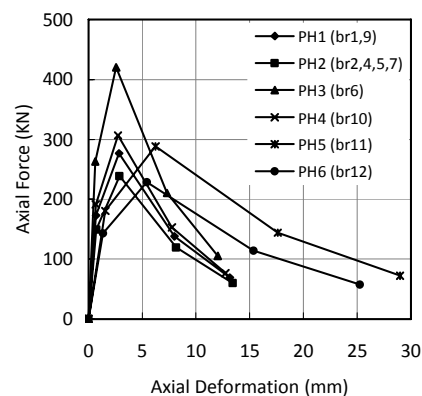
(a) Material Law for RC Sections



(b) Material Law for braces of STR1



(c) Material Law for braces of STR2



(d) Material Law for braces of STR3

Fig. 3.3: Material Laws for RC Sections and Masonry Braces

3.4.1.1 Overview of Reinforcement

Due to lack of information about the reinforcement of beams, all of them have been considered as elastic in nonlinear analysis. Fig. 3.4 gives in detail the reinforcement used in columns. It is evident that all the structures are lightly reinforced with bulky concrete cross sections.

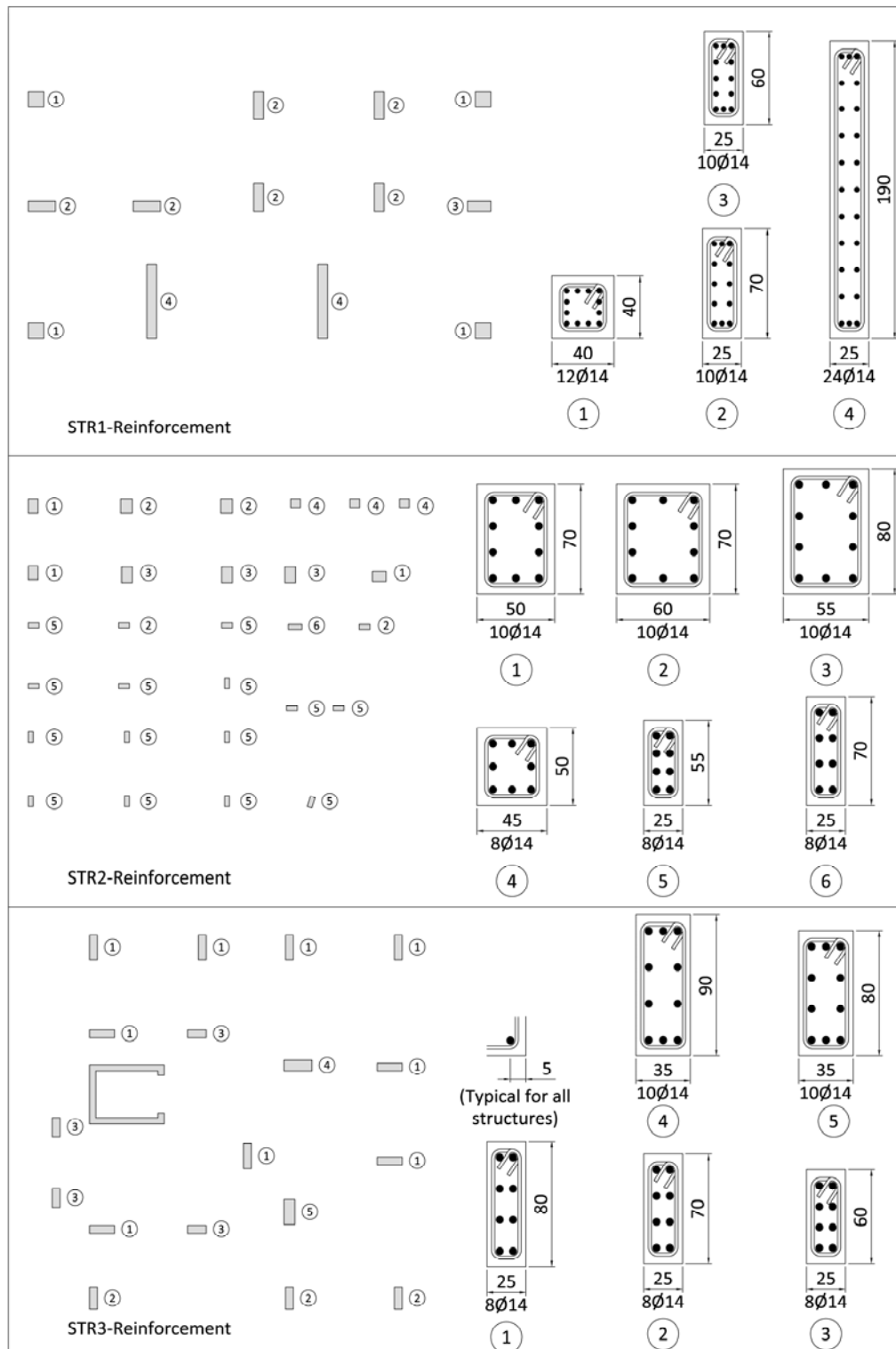
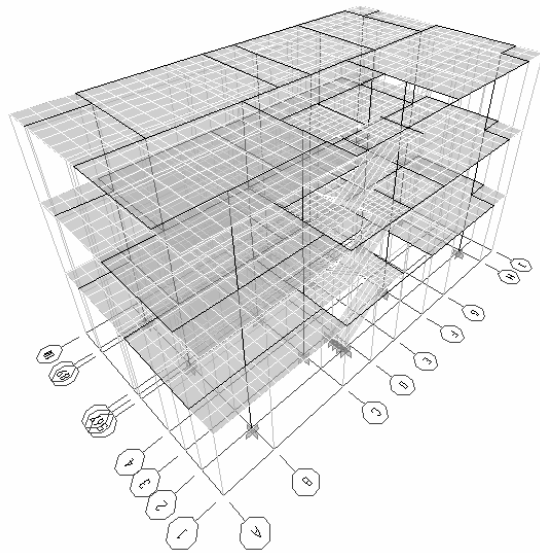


Fig. 3.4: Column Reinforcement for three structures

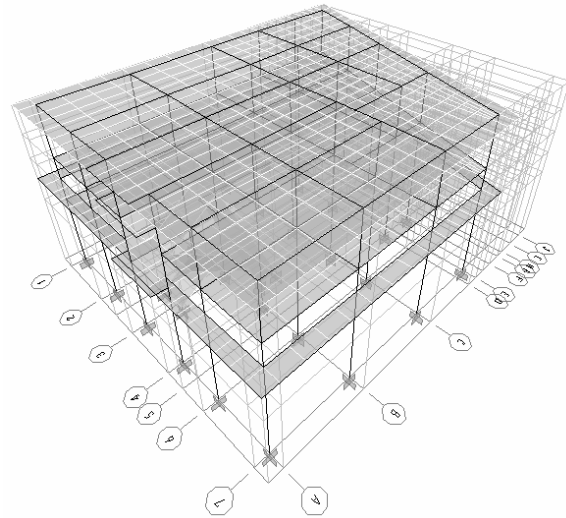
3.4.2 Geometrical Nonlinear Effects

Due allowance of 2nd order effects ($P-\Delta$ effects) has been made in nonlinear analysis to take into account the geometrically nonlinear behaviour of the structures.

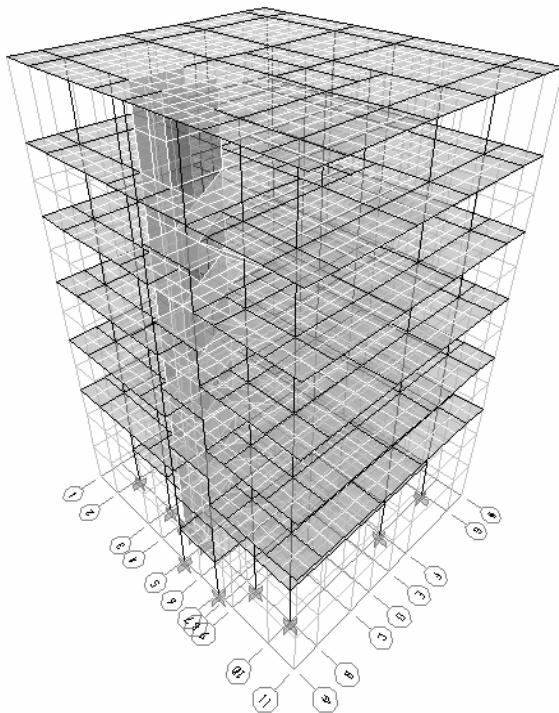
Fig. 3.5 shows all the structures in the modelled form.



(a) Model of STR1



(b) Model of STR2



(c) Model of STR3

Fig. 3.5: Structures as modelled

CHAPTER 4 – LINEAR TIME HISTORY ANALYSES AND DEVELOPMENT OF SEISMIC LOAD PATTERNS

4.1 Basis of Modal Pushover Analysis

Pushover analysis for any forcing function requires the static nonlinear analysis of a model which allows monitoring the progressive yielding of the structure, and establishing capacity curve; a plot of base shear versus roof displacement. The structure is pushed with a lateral loading shape that follows the fundamental shape of the pre-yielding state of the building, while vertical earthquake loading is ignored.

The idea behind nonlinear static pushover methods is to characterize the nature of seismic loads acting on the structure. The response of the structure to any dynamic load is considered to be the combination of dynamic modes of vibration of the system [8]. Hence, it is conventional to perform modal analysis.

4.1.1 Limitations of Modal Pushover Analysis

Modal pushover analysis procedure is generally considered more realistic in assessing the seismic vulnerability of structures than force-based methods. Although the procedure is displacement-based and treats nonlinearity in a more explicit manner, the procedure suffers from several fundamental deficiencies. An exhaustive review of modal pushover analysis can be seen as done by Simon Kim et al. (1999) [9].

One of the deficiencies is the forcing function. Although there are various ways to specify forcing function, time-honoured capacity spectrum method uses the first mode shape forcing function to push the model. First mode shape of a 3D structure is predominantly 2-dimensional which is mainly suitable for 2D structures only or to the 3D structures which can be simplified into 2D representations. Therefore, modal pushover analysis tends to portray the 2D image of the structural behaviour. For a 3D structure, it is also difficult to determine the critical direction in which a structure must be pushed. Clearly, 3D interaction effect should be included in order to account for torsional irregularity.

Also, mode shapes are representation of the free body motion of a structure and entirely depend on the mass distribution and stiffness characteristics of the structure. This is how mode shapes do not represent the effect of the soil on which the structure is built and magnitude-distance dependence of an earthquake wave.

4.2 EC8 Recommended Load Patterns

There are recommended in EC8 two vertical distributions of the lateral loads as follows [3]:

- a uniform pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration); and

- a modal pattern, proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis.

and it is recommended to at least consider both of these one by one for nonlinear static analysis.

4.3 Development of 3D Displacement-based Load Patterns

4.3.1 Method and Use of Tri-Component Earthquake Records for Load Patterns

The purpose behind using the real-time three-component strong motion records is to get a load pattern at highest seismic demand level for each of the corresponding representative record. One way to get it is to see the response of the structure in a time history analysis. It can be realized that the structural response may or may not be maximum corresponding to maximum acceleration in any of the three TH-components. Therefore, the response must be seen for different successive narrow-range time windows but this still is a very cumbersome approach. Alternatively, another way has been devised in the following section.

4.3.2 Criterion for Obtaining the Maximum Displacement Response

A displacement based approach has been used to develop the load patterns. Tri-component acceleration time history analyses have been performed and following the criterion outlined below, 3D load patterns are developed. Fig. 4.1 represents the idealized roof plan in its deformed and undeformed shapes. Columns shown represent the corner columns of an actual structure. Each column point is named as Control Point (CP).

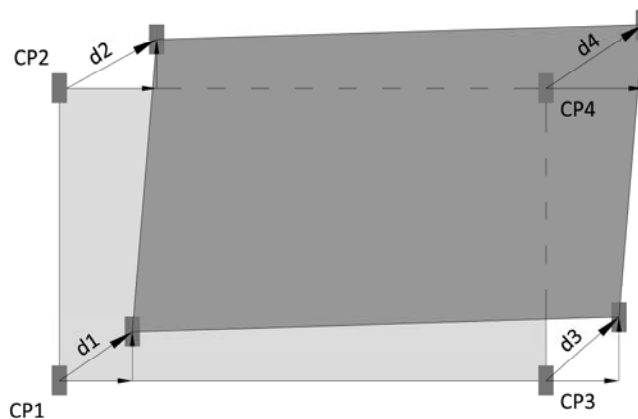


Fig. 4.1: Idealized Roof Displaced Shape with Control Points

For a time history analysis, each CP has a corresponding time history response as shown below: (z-component ignored)

Time	CP1		CP2		CP3		CP4	
t	X	Y	X	Y	X	Y	X	Y
t_0	x_{01}	y_{01}	x_{02}	y_{02}	x_{03}	y_{03}	x_{04}	y_{04}
t_1	x_{11}	y_{11}	x_{12}	y_{12}	x_{13}	y_{13}	x_{14}	y_{14}
:	:	:	:	:	:	:	:	:
t_n	x_{n1}	y_{n1}	x_{n2}	y_{n2}	x_{n3}	y_{n3}	x_{n4}	y_{n4}

For each CP,

- get $x_{\max}$ and $x_{\min}$, and corresponding y-displacements and time values,
- get $y_{\max}$ and $y_{\min}$, and corresponding x-displacements and time values,
- calculate Square Root of Sum of Squares (SRSS) for each displacement value,
- get maximum SRSS value and corresponding time value as t_{SRSS} .

The above criterion is tabulated in [Table 4.1](#).

Table 4.1: Maximum and Minimum Time History Displacements

X	Y	Time	SRSS(X,Y)
$x_{\max}$	y at $x_{\max}$	$t_{x,\max}$	$\text{SRSS}(x_{\max}, y \text{ at } x_{\max})$
$x_{\min}$	y at $x_{\min}$	$t_{x,\min}$	$\text{SRSS}(x_{\min}, y \text{ at } x_{\min})$
x at $y_{\max}$	$y_{\max}$	$t_{y,\max}$	$\text{SRSS}(x \text{ at } y_{\max}, y_{\max})$
x at $y_{\min}$	$y_{\min}$	$t_{y,\min}$	$\text{SRSS}(x \text{ at } y_{\min}, y_{\min})$
		t_{SRSS}	$d = \text{Max}(\text{SRSS}(X,Y))$

Repeating the same procedure for other three CPs gives three more SRSS displacement values and their corresponding time values as follows:

CP1		CP2		CP3		CP4	
t_{SRSS1}	d_1	t_{SRSS2}	d_2	t_{SRSS3}	d_3	t_{SRSS4}	d_4

Time corresponding to the maximum d-value is chosen as the time point of maximum response of the structure. Displacements of other points on other storey points below roof are taken for the same t-value and load pattern based on deformed shape of the structure is developed.

The idea of taking into account the SRSS value to estimate the maximum value of each action effect on the structure due to two horizontal components of seismic action has also been recommended by EC8 [3].

4.3.3 Application of Load Pattern to Model

As has been mentioned in the previous section 4.3.2 and explained by Fig. 4.1 that the load patterns have been developed by considering the displacement response of four corner columns of the structure. Due to rigid diaphragmatic action of concrete slab, loads are assumed to vary linearly at diaphragm (storey) level. Thus loads for the column points in between the corner column are linearly interpolated and applied as point loads to the column points at each storey level (Fig. 4.2).

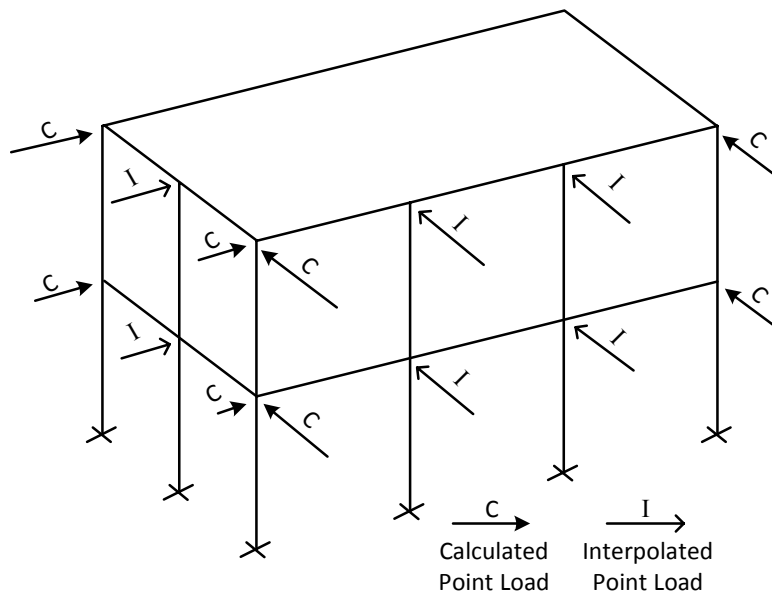


Fig. 4.2: Application of Load Pattern to Model

4.3.4 Verification of the Criterion

In order to investigate the reliability of above-mentioned criterion, four examples are presented; two for each STR1 and STR2. One example is presented in this text and other three are given in Annexure B. For each of these structures, both bare frame and frame with infills have been considered. Different time windows of time history analysis results are considered to verify the maximum response.

Fig. 4.4 shows the response of STR1 with & without infills, giving the displacement and time values corresponding to the maximum response for each time-window (Fig. 4.3) and the values selected considering the said criterion. For each time-window, time value at maximum acceleration and SRSS displacement values are given. “Chosen” shows the time step extracted following the above-mentioned criterion. It can be seen that response for the chosen time step is maximum. Examples presented in Annexure B further reinforce the viability of the criterion.

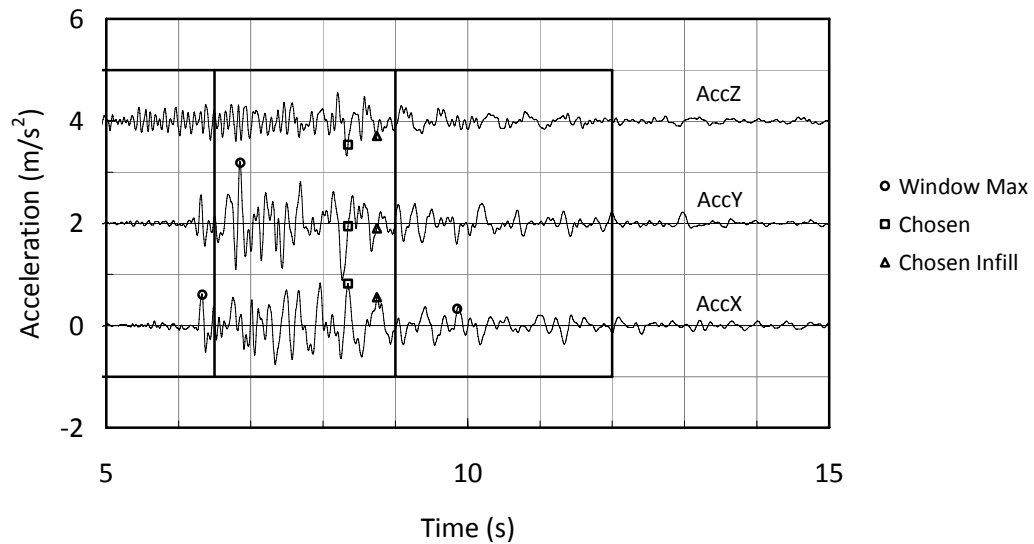


Fig. 4.3: Time History M1D1SB with Pre-Event, Event and Post-Event Windows

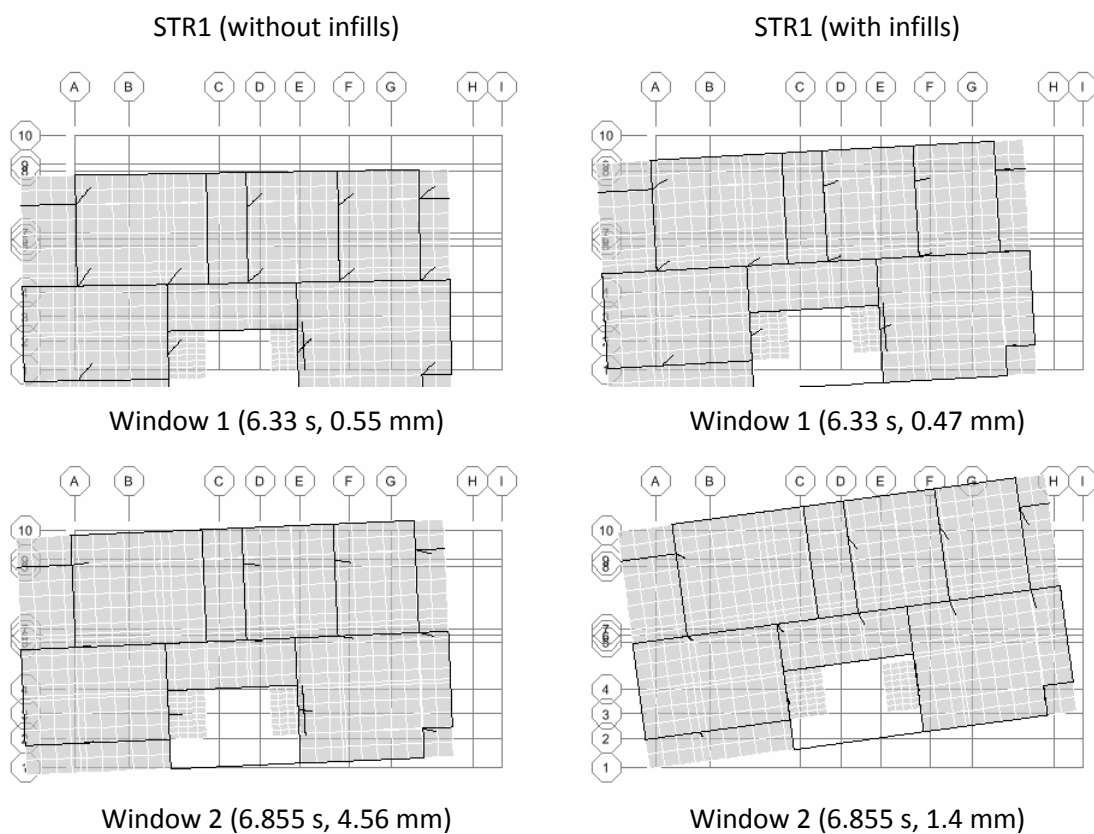


Fig. 4.4 Time History Response of Building STR1 for different time windows
Cont'd

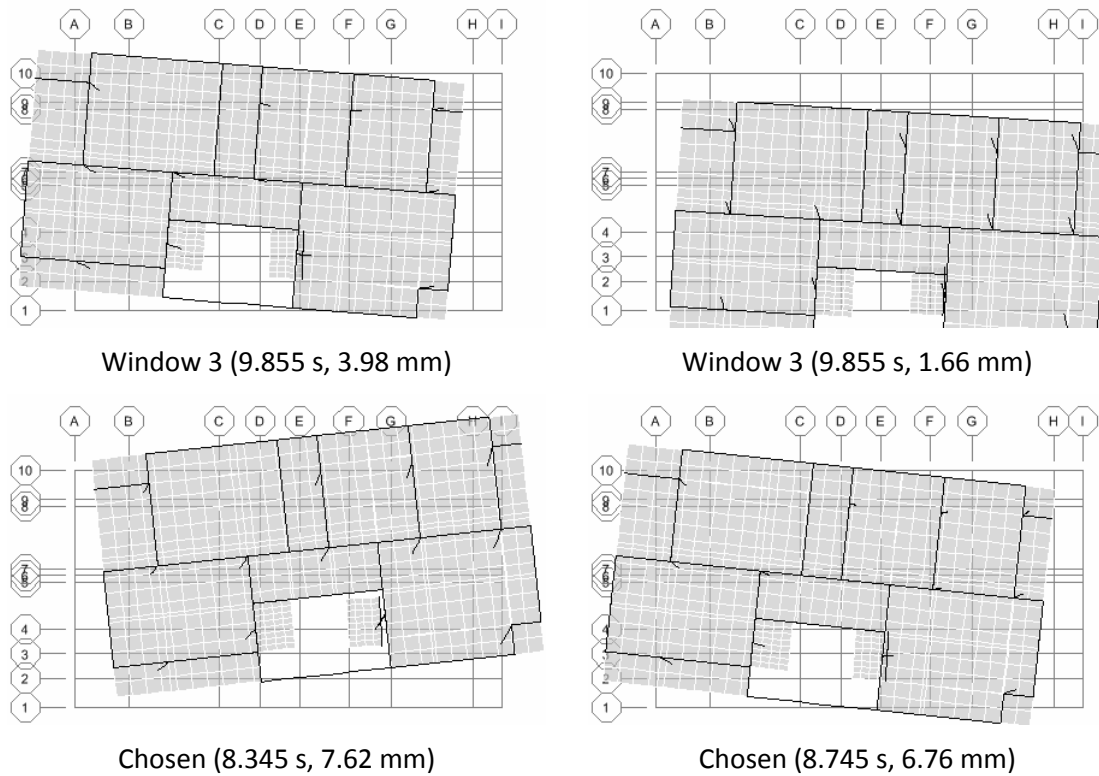


Fig. 4.4: Time History Response of Building STR1 for different time windows

4.3.5 Linear Time History Analyses and Developed Load Patterns

38 acceleration time history records representative of 444 earthquakes given in [Table 2.1](#) have been used in order to capture the structural response at highest seismic demand level of the corresponding earthquake.

All the earthquakes are scattered across different countries and each recording has its own reference frame of axes, therefore, it is not feasible for so many records to match the orientation of recorded response with the actual orientation of the structures. That's why the assumption has been employed that each component (x, y or z) of recorded response acts in the positive direction of its corresponding axis (X, Y or Z) of the modelled structure.

As a first step to developing the forcing function, tri-component linear time history analyses for the representative records are performed taking 5% damping ratio. [Fig. 4.5](#) shows the resulting load patterns for STR1 (without infills) for magnitude range M1 covering all the near-field, transition, and far-field earthquakes for all the available site classes. Other load patterns have been presented in [Annexure C](#).

Load patterns for STR1, in general, resemble with the first mode shape. In plan, only few torsion-producing load shapes can be seen and this is in agreement with the small mass-stiffness eccentricity ([Fig. 4.6](#)).

Load patterns for STR2, though resemble with the first mode shape along the height, contain lot of shapes which are torsion-dominant. Mass stiffness eccentricity is also found very high for STR2 ([Fig. 4.6](#)).

STR3, considerably taller than STR1 and STR2, regular in shape and structural properties, exhibits patterns matching with both mode shapes 1 & 2 both in plan and elevation (Fig. 4.6). Thus, taller structures or relatively more flexible structures will tend to deform in shapes which are more complex than the fundamental mode shape.

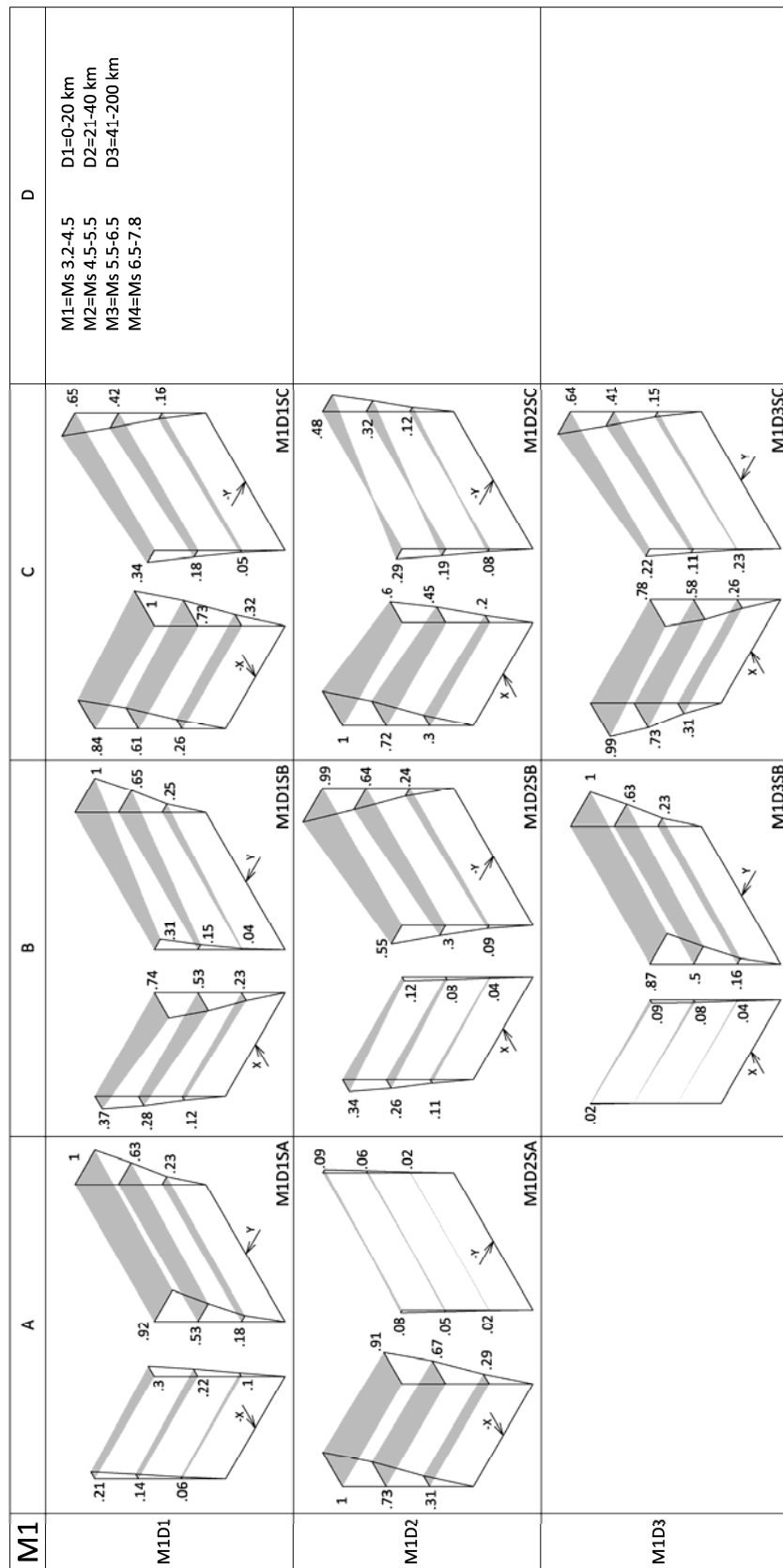


Fig. 4.5: Load Patterns for STR1 (Magnitude Range M1)

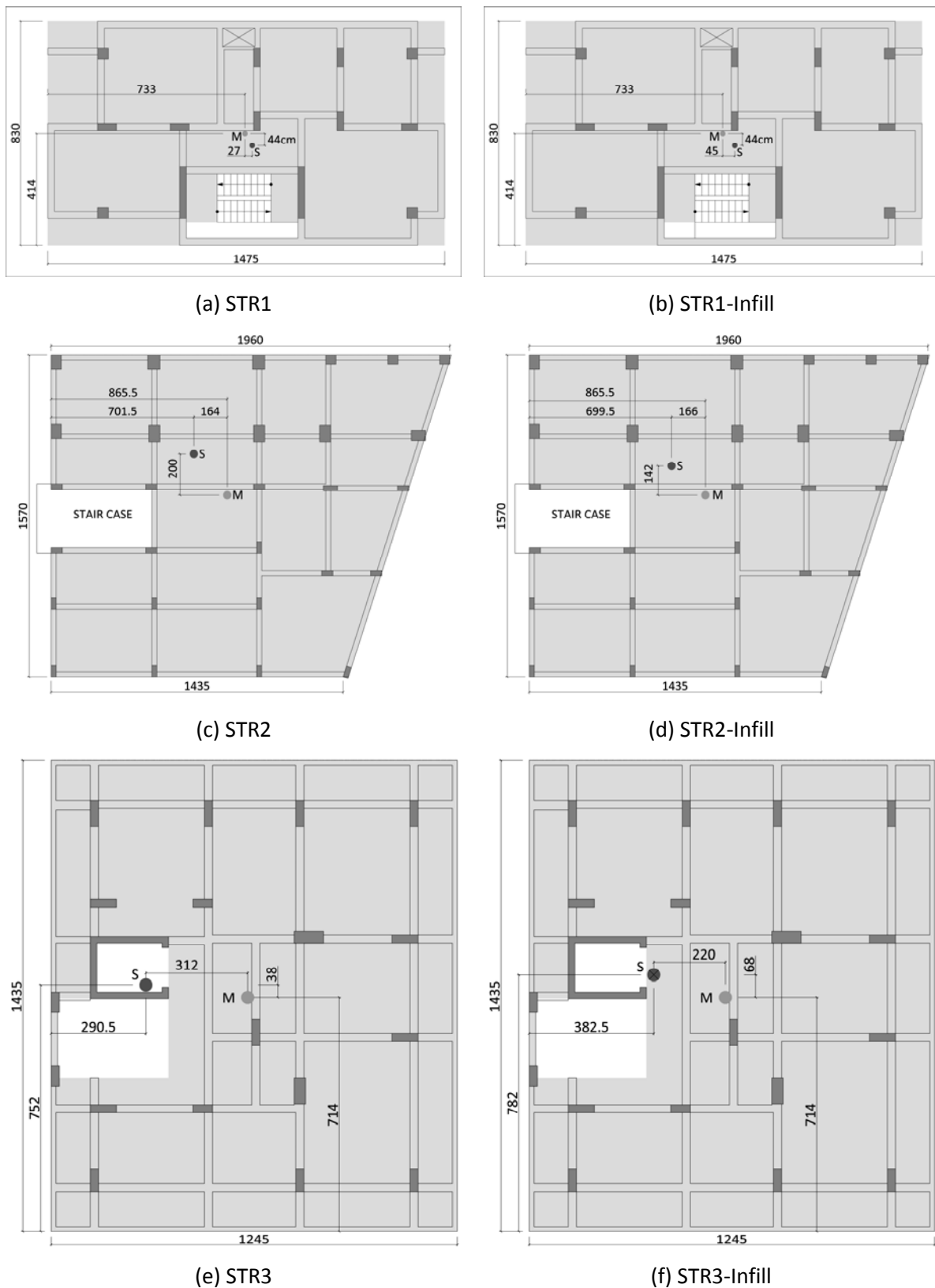


Fig. 4.6: Mass and Stiffness Centres of Structures

(All dimensions in cm)

CHAPTER 5 – PUSHOVER ANALYSES FOR SELECTED LOAD PATTERNS

5.1 Selected Load Pattern Cases

Table 5.1 contains all the analysis cases used to perform pushover analysis. Analysis cases used for pushover analysis are marked grey.

Table 5.1: Analysis Cases for Pushover Analysis

(a)					(b) Analysis Cases for STR1				
M1	Soil A	Soil B	Soil C	Soil D	M2	Soil A	Soil B	Soil C	Soil D
D1	M1D1SA	M1D1SB	M1D1SC	-	D1	M2D1SA	M2D1SB	M2D1SC	-
D2	M1D2SA	M1D2SB	M1D2SC	-	D2	M2D2SA	M2D2SB	M2D2SC	-
D3	-	M1D3SB	M1D3SC	-	D3	M2D3SA	M2D3SB	M2D3SC	M2D3SD

(c) Analysis Cases for STR2					(d) Analysis Cases for STR3				
M3	Soil A	Soil B	Soil C	Soil D	M4	Soil A	Soil B	Soil C	Soil D
D1	M3D1SA	M3D1SB	M3D1SC	-	D1	M4D1SA	M4D1SB	M4D1SC	M4D1SD
D2	M3D2SA	M3D2SB	M3D2SC	-	D2	M4D2SA	M4D2SB	M4D2SC	-
D3	M3D3SA	M3D3SB	M3D3SC	M3D3SD	D3	M4D3SA	M4D3SB	M4D3SC	-

5.1.1 Characteristics of Selected Load Pattern Cases

Analysis cases have been chosen in a way to reflect the representation of different magnitudes, distance ranges and soil classes. For example, analysis cases for each structures represent two soil classes and three distance ranges.

5.2 Analyses Results and Relative Damage Index

Capacity curves of STR1 for the analysis cases given in Table 5.1b are shown in Fig. 5.1. As global response shown by capacity curves, out of four, three cases show identical response in the beginning but, however, differ in the ultimate strength as they cause damage to the structure.

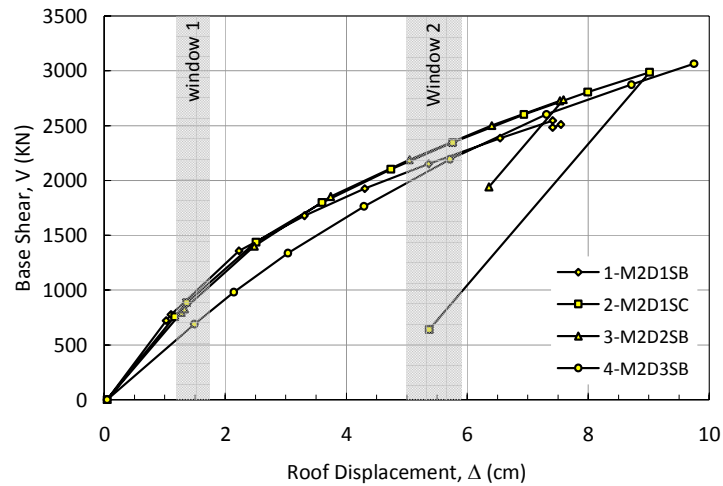


Fig. 5.1: Capacity Curves of STR1

At three different stages of the nonlinear behaviour, the damage distribution in the structure is shown in Fig. 5.3 to Fig. 5.5. Location of reference elevations can be seen in plan in Fig. 5.2. Capacity curves and damage distribution for other structures taking their respective analysis cases (Table 5.1) are given in Annexure D. By comparing all the cases for each structure individually, a relative damage index is produced which indicates the relative damage potential considering the shape of the load pattern both in plan and elevation for all load patterns for the structures with and without infills. Table 5.2, Table 5.3 and Table 5.4 show respectively damage index for building STR1, STR2 and STR3.

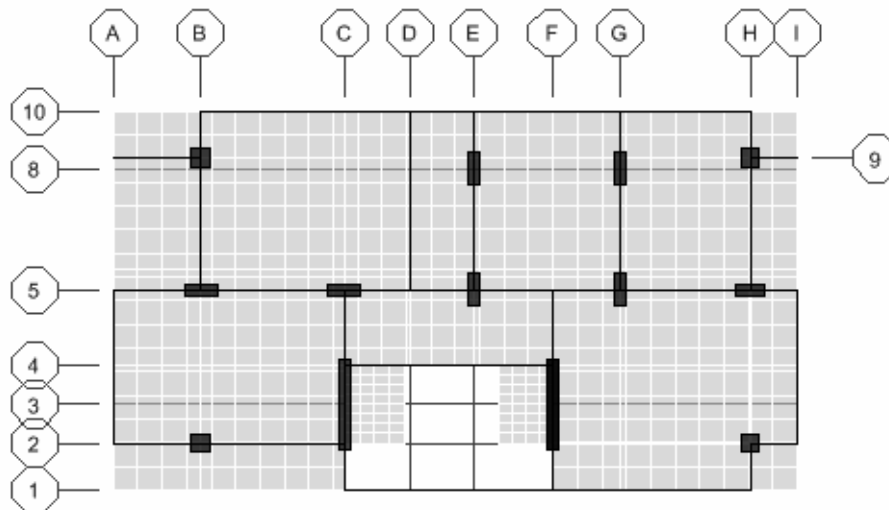


Fig. 5.2: Reference Plan and Elevation Lines of STR1

Fig. 5.3: Damage Distribution for STR1 (Roof Displacement:1.10-1.50 cm, window 1)

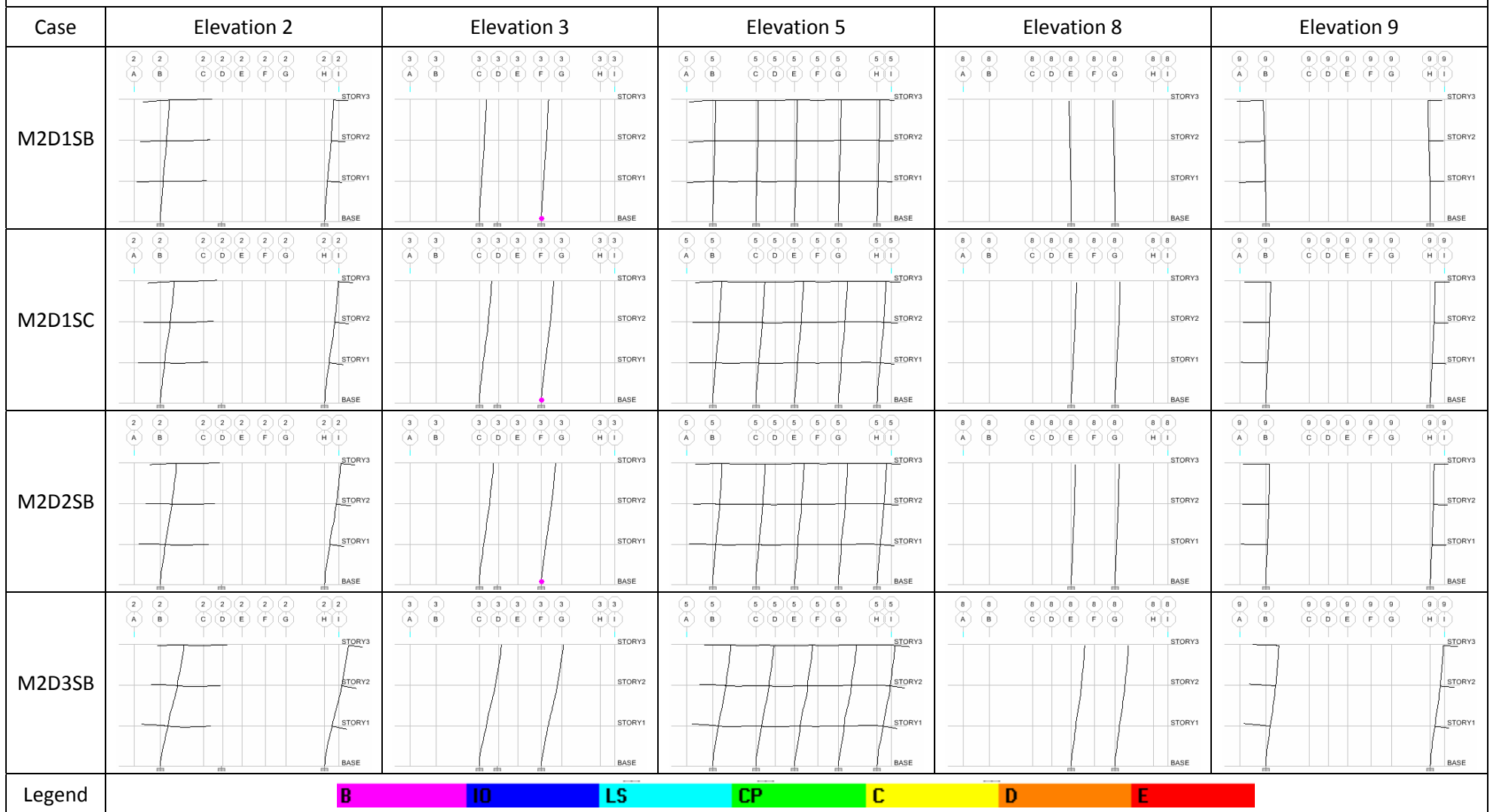


Fig. 5.4: Damage Distribution for STR1 (Roof Displacement:5.05-5.75 cm, window 2)

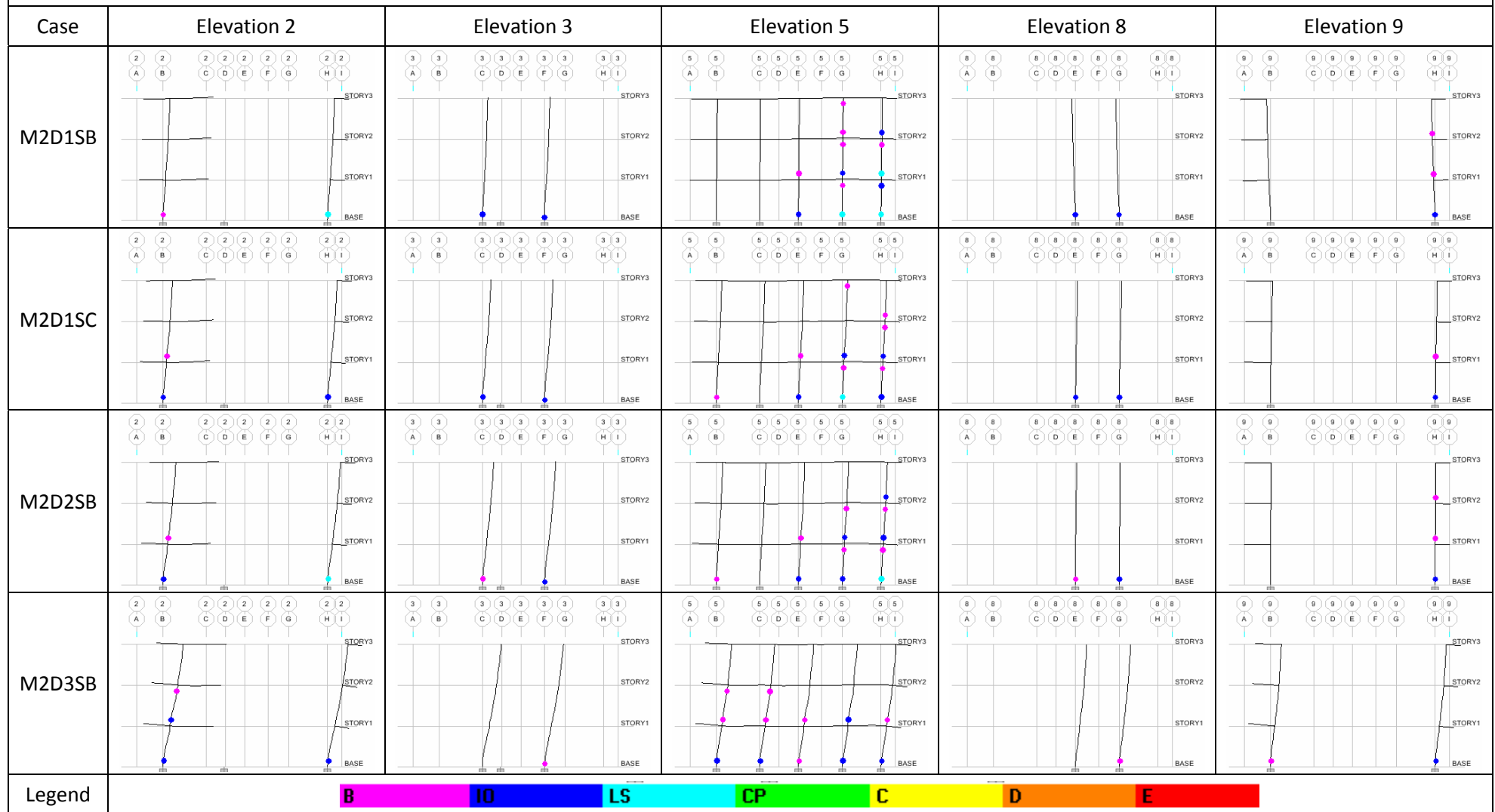


Fig. 5.5: Damage Distribution for STR1 (at ultimate state)

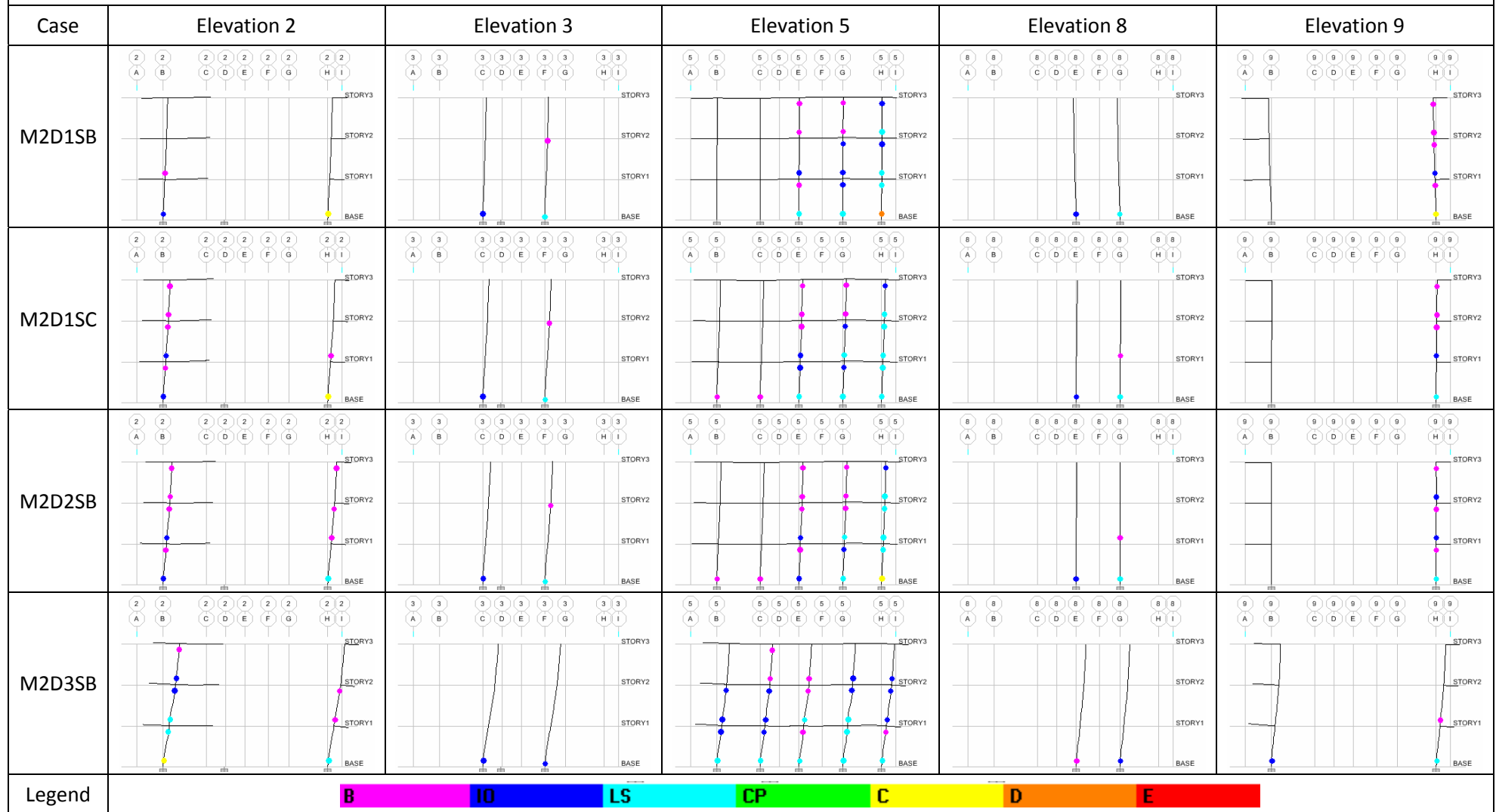


Table 5.2: Relative Damage Index for STR1 for selected load cases

Structure ID	Shape of Load Pattern		Site & Distance Dependent Seismic Action			
	In Plan	In Elev.	M2D1SB	M2D1SC	M2D2SB	M2D3SB
STR1	 or 					
	 or 					
	 or 					
	 or 					
STR1-Infill	 or 					
	 or 					
	 or 					
	 or 					

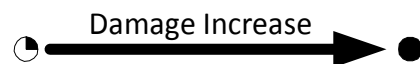
Table 5.3: Relative Damage Index for STR2 for selected load cases

Structure ID	Shape of Load Pattern		Site & Distance Dependent Seismic Action			
	In Plan	In Elev.	M3D1SB	M3D1SC	M3D2SB	M3D3SB
STR2	 or 					
	 or 					
	 or 					
	 or 					
STR2-Infill	 or 					
	 or 					
	 or 					
	 or 					

Table 5.4: Relative Damage Index for STR3 for selected load cases

Structure ID	Shape of Load Pattern		Site & Distance Dependent Seismic Action			
	In Plan	In Elev.	M4D1SA	M4D1SB	M4D2SA	M4D3SA
STR3	 or 					
	 or 					-
	 or 					
	 or 					
STR3-Infill	 or 					
	 or 					
	 or 					
	 or 					

Note: Shapes of load patterns in plan and elevation have been taken from the developed load patterns and then generalized. Damage index are shown for each analysed case against their corresponding load pattern shapes.



5.3 Comparison of NL TH Analysis and LP Pushover Analysis Results

Software tool RAM PERFORM is used to perform the nonlinear time history analysis on the fibre-element model of STR1 both with and without infills. Input acceleration time histories are given in Fig. 5.6. The purpose of this exercise is to check the viability of application of load patterns as forcing function in nonlinear static analysis by comparing the damage obtained from NL TH analysis with that of LP PO analysis.

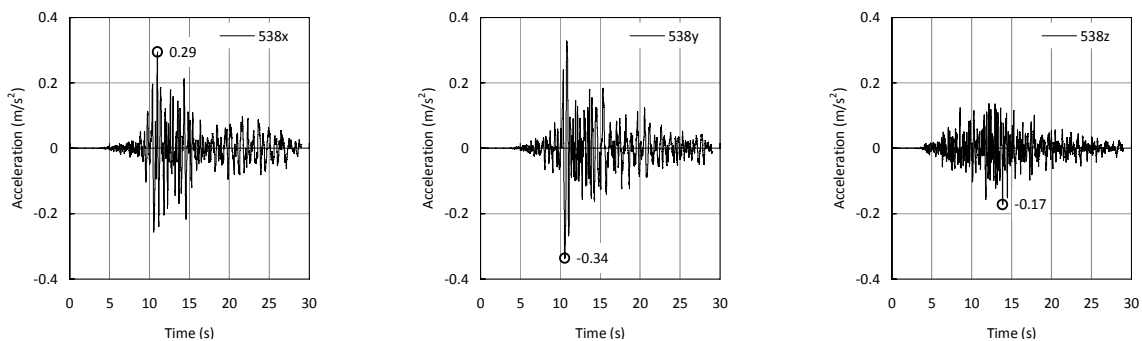
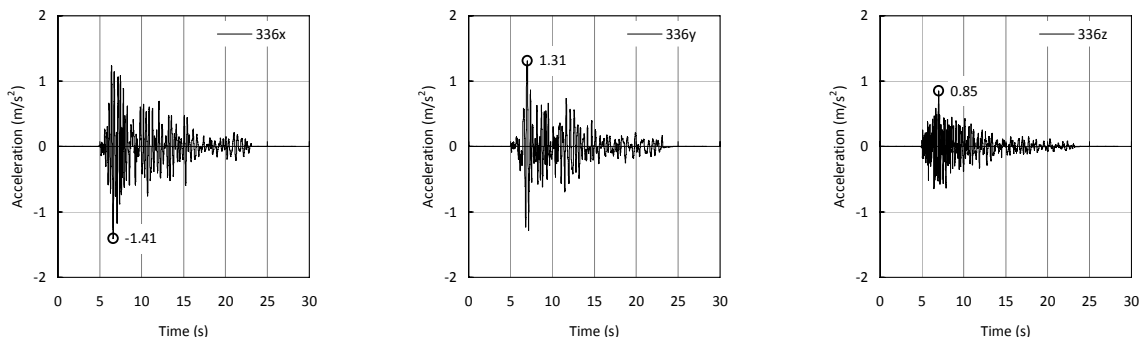
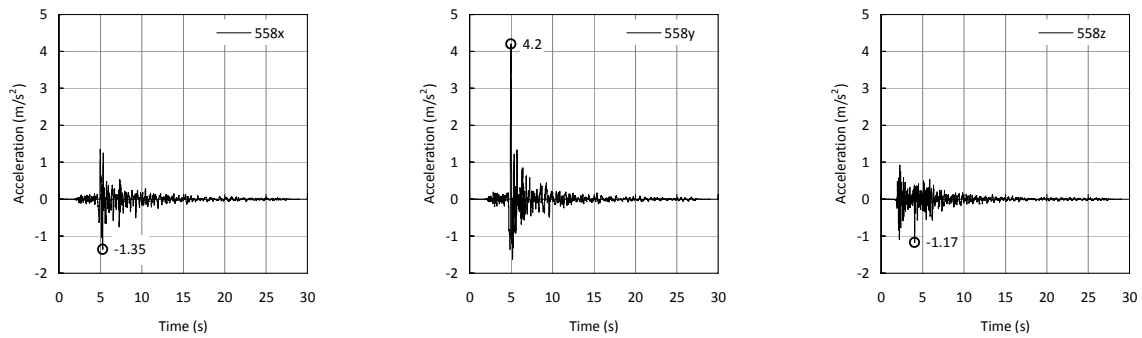
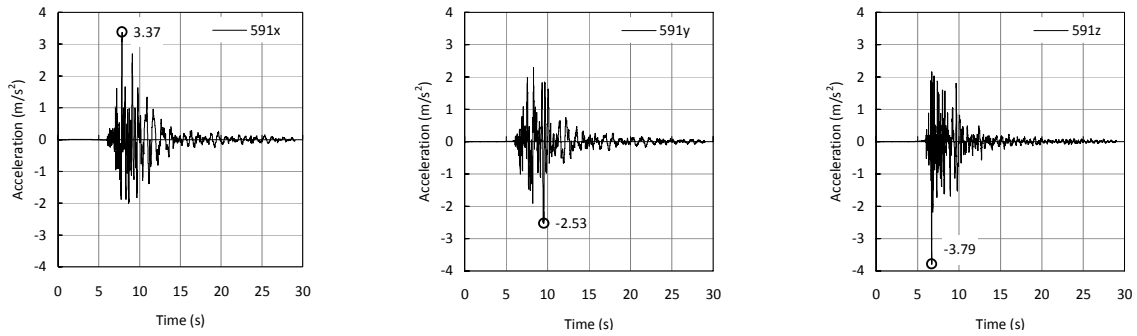


Fig. 5.6: Acceleration Time History Records for NL TH Analysis

5.3.1 Method of Comparison

The procedure of comparing the results is given in Fig. 5.7. Capacity spectrum from LP POA is used as common basis for the damage comparison from both types of analysis. For input time history, performance point (say *a*) is calculated. Spectral displacement at this performance point can be converted to the corresponding roof displacement in LP POA. A point (say *b*) can be located for this roof displacement on the capacity curve obtained from LP POA. The damage has been compared for point *a* and point *b*. Given the damages or damage patterns are nearly similar; the viability of the load pattern is proven.

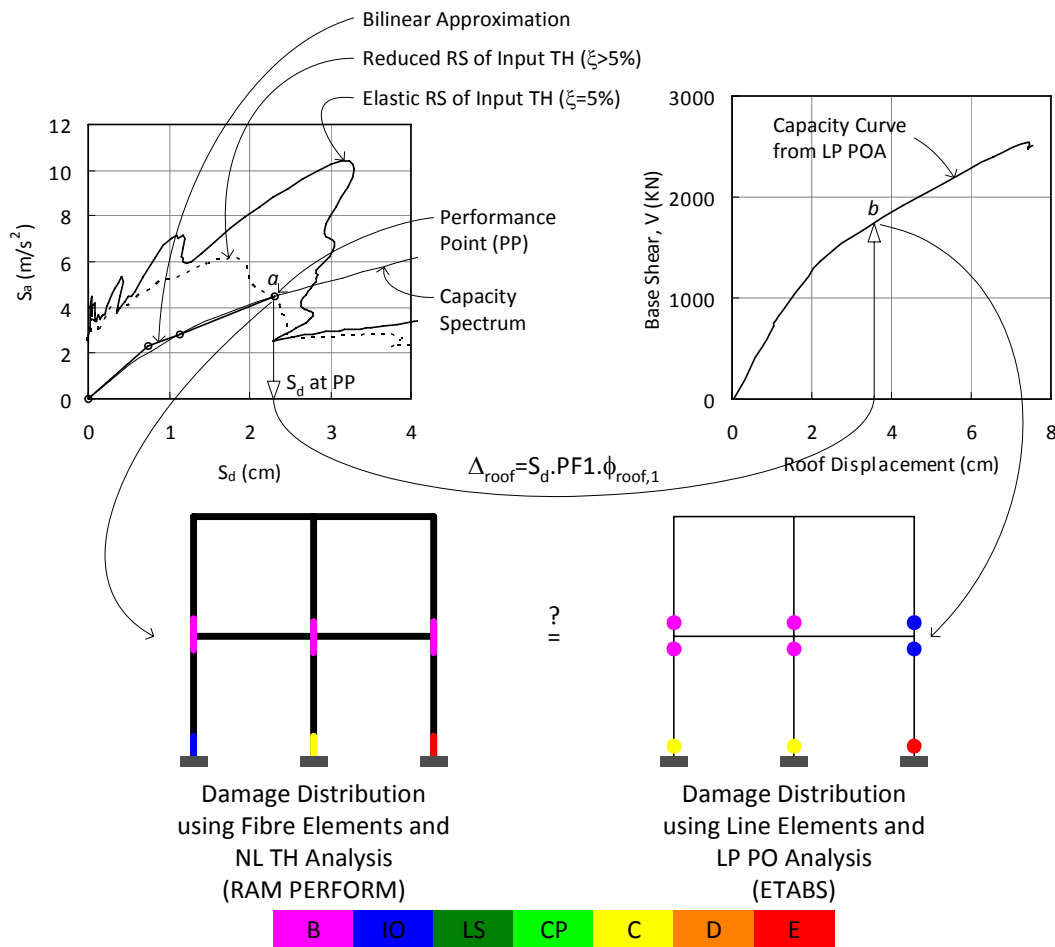
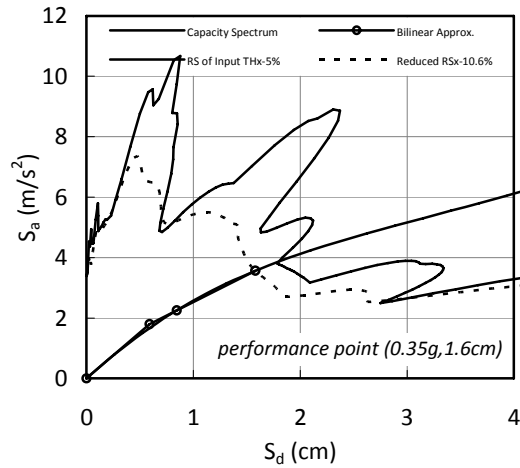


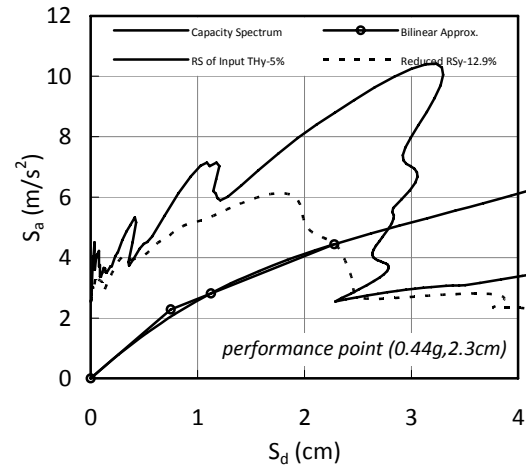
Fig. 5.7: Pictorial Representation of Procedure of Comparison of NLTH and LP PO Analysis

Fig. 5.8 shows the performance calculations for the time histories shown in Fig. 5.6 for STR1 without infills. Each horizontal component of time history gives a different performance point. Therefore, damage distributions for LP POA corresponding to both performance points are shown in Fig. 5.9. Same calculations are also done for STR1 with infills, given in Annexure E.

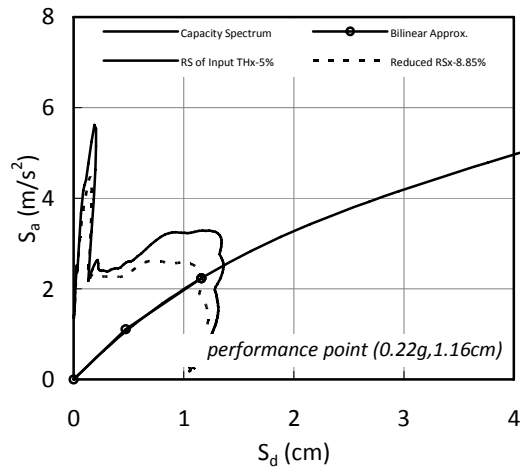
The comparison shows that there is little scatter in the damage and its distribution for two different types of analysis. Especially the difference in damage zones is very narrow for STR1-infill shown in Annexure E. The resemblance in damage zones shows the applicability of the load patterns to be used as forcing functions for nonlinear static analysis.



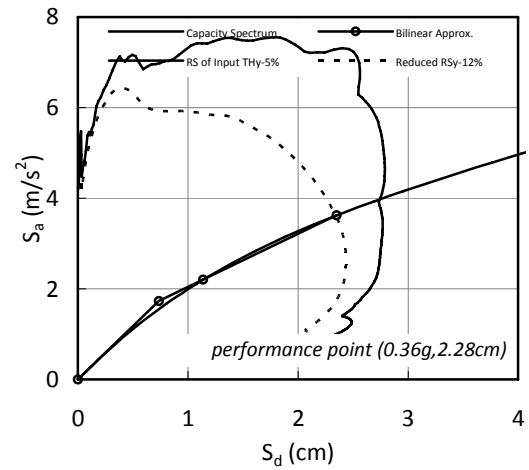
(a) M2D1SB (x-comp)



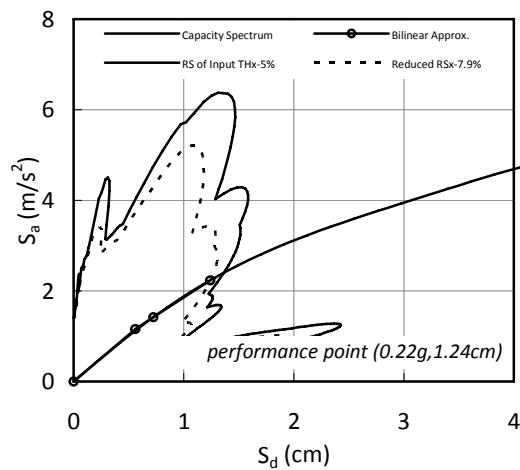
(b) M2D1SB (y-comp)



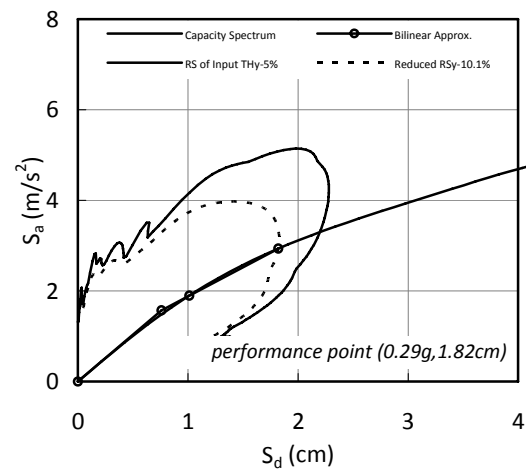
(c) M2D1SC (x-comp)



(d) M2D1SC (y-comp)



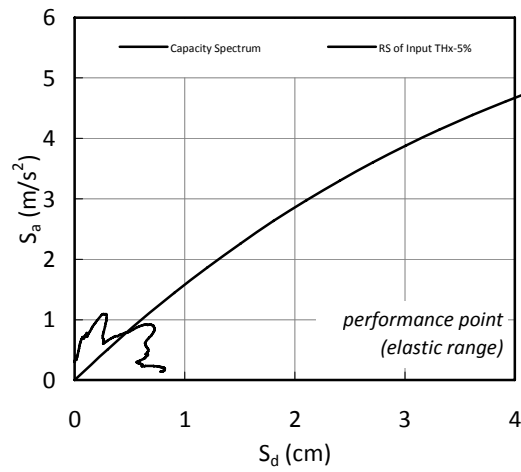
(e) M2D2SB (x-comp)



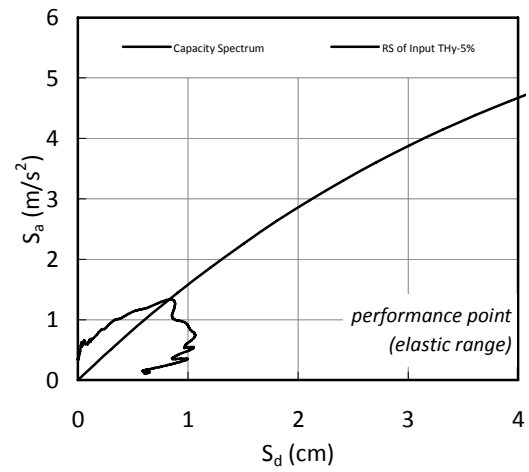
(f) M2D2SB (y-comp)

Fig. 5.8: Performance Calculations for STR1 (without infills)

cont'd



(g) M2D3SB (x-comp)



(h) M2D3SB (y-comp)

Fig. 5.8.: Performance Calculations for STR1 (without infills)

Fig. 5.9a: Comparison of Damage Distribution for NLTH and LP PO Analyses (STR1)

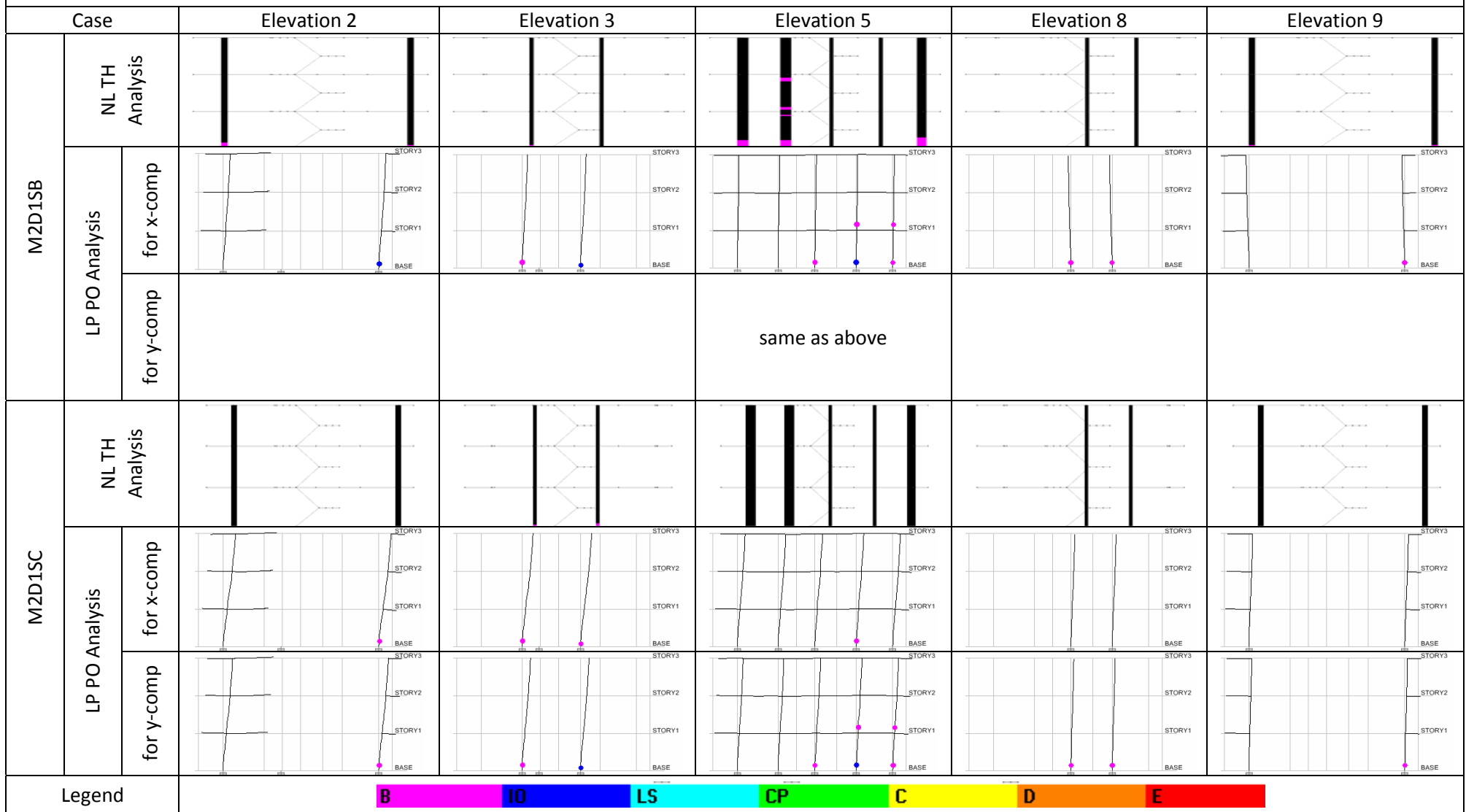
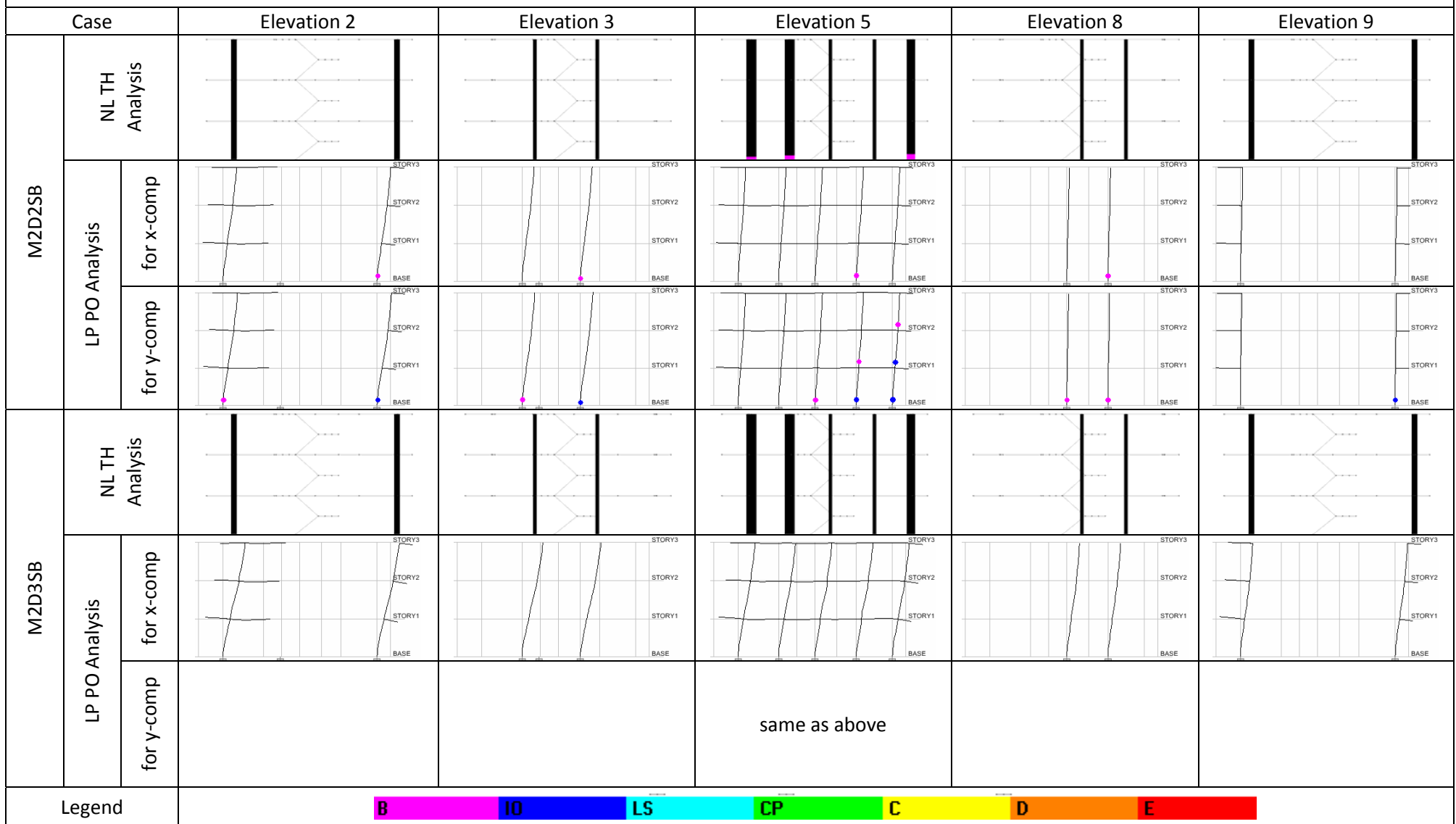


Fig. 5.9b: Comparison of Damage Distribution for NLTH and LP PO Analyses (STR1)

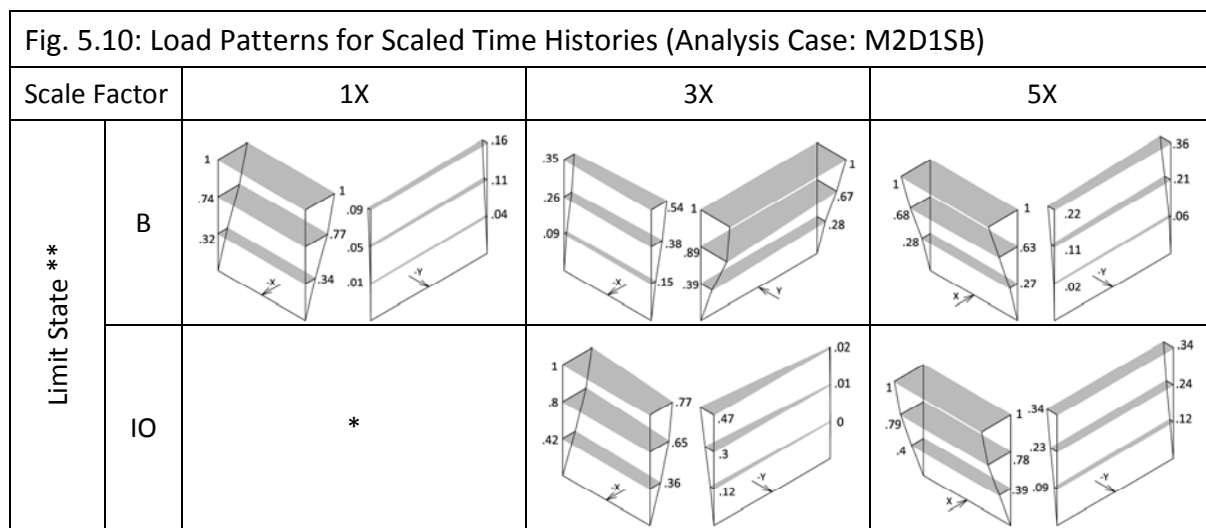


5.4 Nonlinear Scaled Time History Analysis-Changes in Load Pattern

The idea of performing nonlinear time history analysis for the scaled time histories is to observe the changes in load pattern at different nonlinear limit states of the model. Results of this analysis are presented for building STR1 only.

Time history records for the analysis cases of building STR1 ([Table 5.1](#)) have been scaled to 3X and 5X.

[Fig. 5.10](#) presents the load patterns as a result of nonlinear time history analysis for analysis case M2D1SB with each time history record having scale factors of 1, 3 and 5. Results of other analysis cases can be seen in the [Annexure F](#). Load patterns are developed by taking the displacement response at each successive nonlinear limit state that the structure undergoes.



** For definition of limit states, refer to [Fig. 3.3a](#)

* Analysis does not reach this limit state

5.4.1 Effect of Nonlinearity on Load Patterns

[Fig. 5.11](#) shows the comparison of load patterns taken at different nonlinear limit states; applied action being same. It can be seen that as the structure undergoes nonlinear changes, the intensities of loads in the load pattern increase. A similar behaviour is seen in load patterns for other load cases, e.g. for cases STR1-M2D1SB-5X, STR1-M2D1SC-5X and STR1-infill-M2D1SC-3X etc (see [Annexure F](#)).

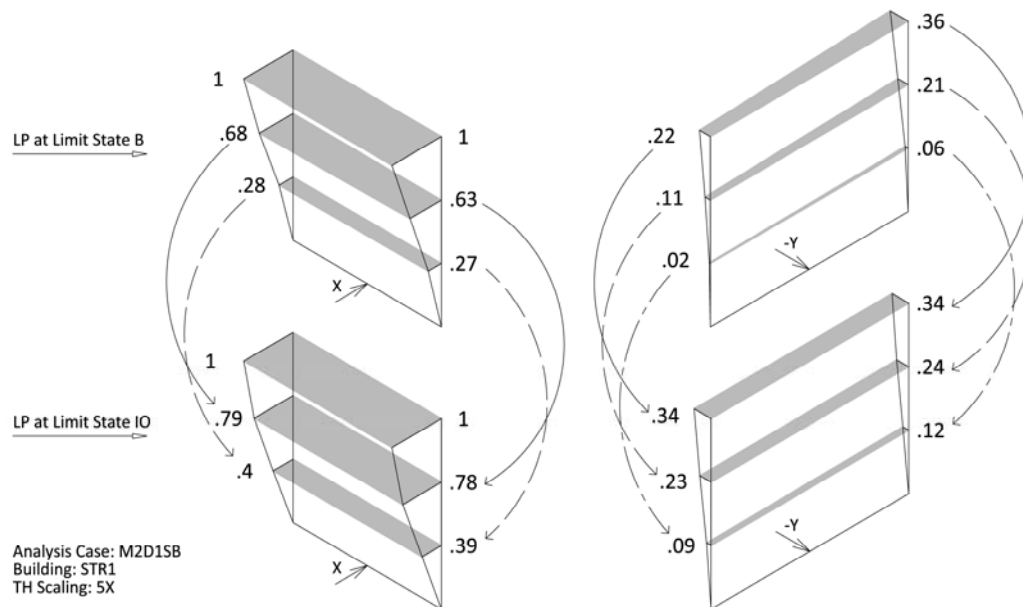


Fig. 5.11: Effect of Nonlinearity on Load Patterns

Fig. 5.12 shows the damage comparison for NL TH analysis; the time history scaled to 3X, with LP PO Analysis for STR1. Damage comparison is made for the first appearance of yielding point (B) and first appearance of immediate occupancy (IO) limit state in the structure. Load patterns applied for the comparison of each limit state are calculated from NL TH analysis and given in Fig. 5.10. As an instance, load pattern 3X-B is applied for yield state damage comparison, and similarly, load pattern 3X-IO is applied for immediate occupancy state damage comparison.

For analysis case M2D1SB, same comparison is made for STR1-infill for limit states B, IO and LS, as shown in Fig. 5.13. Load patterns for case M2D1SB are attached as Annexure F.

Damage distribution is comparable for STR1; damage for NL TH analysis being more conservative. For STR1-infill, the scatter of damage is more in case of NL TH analysis than LP PO analysis. Therefore, it can be concluded that as the complexity of the structure increases, the difference in scatter of damage also increases.

Fig. 5.12: Comparison of Damage Distribution for NL scaled TH and LP PO Analyses (STR1)

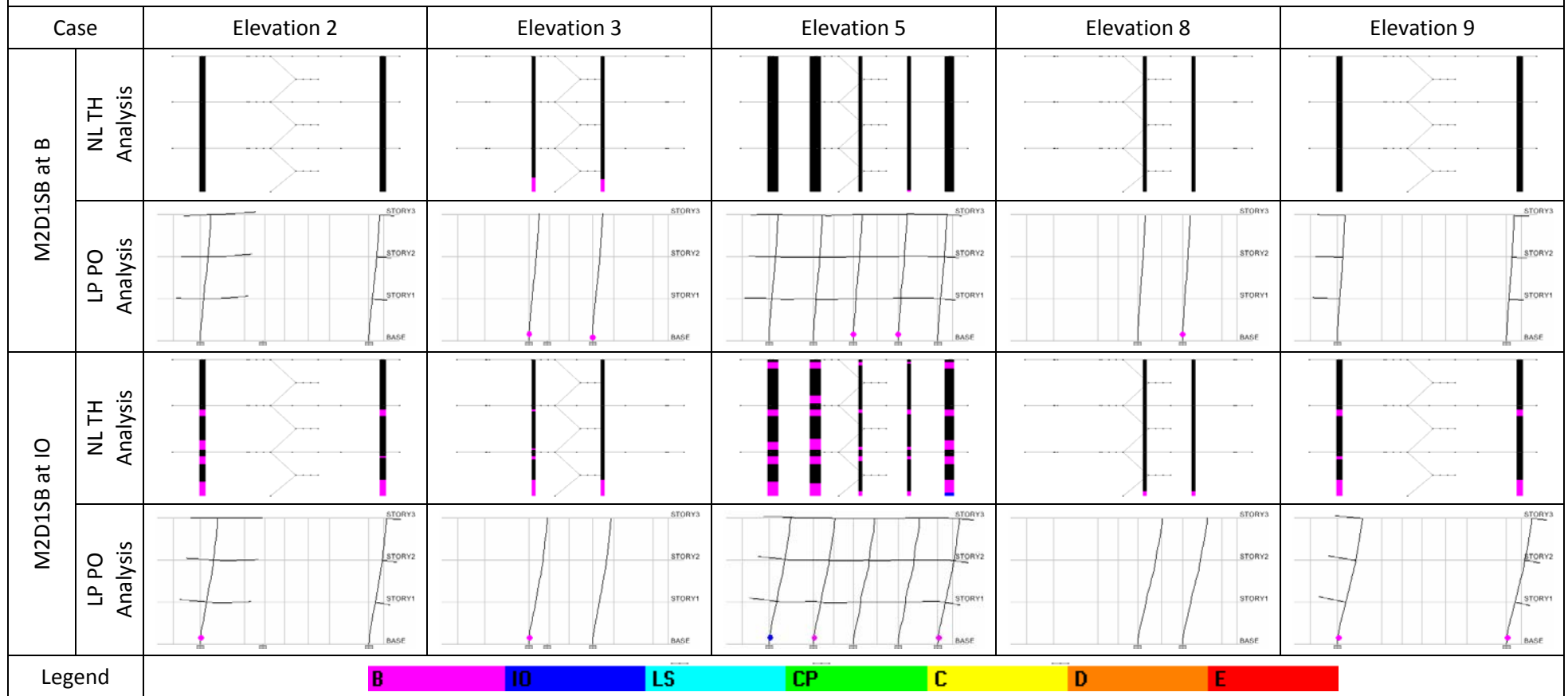
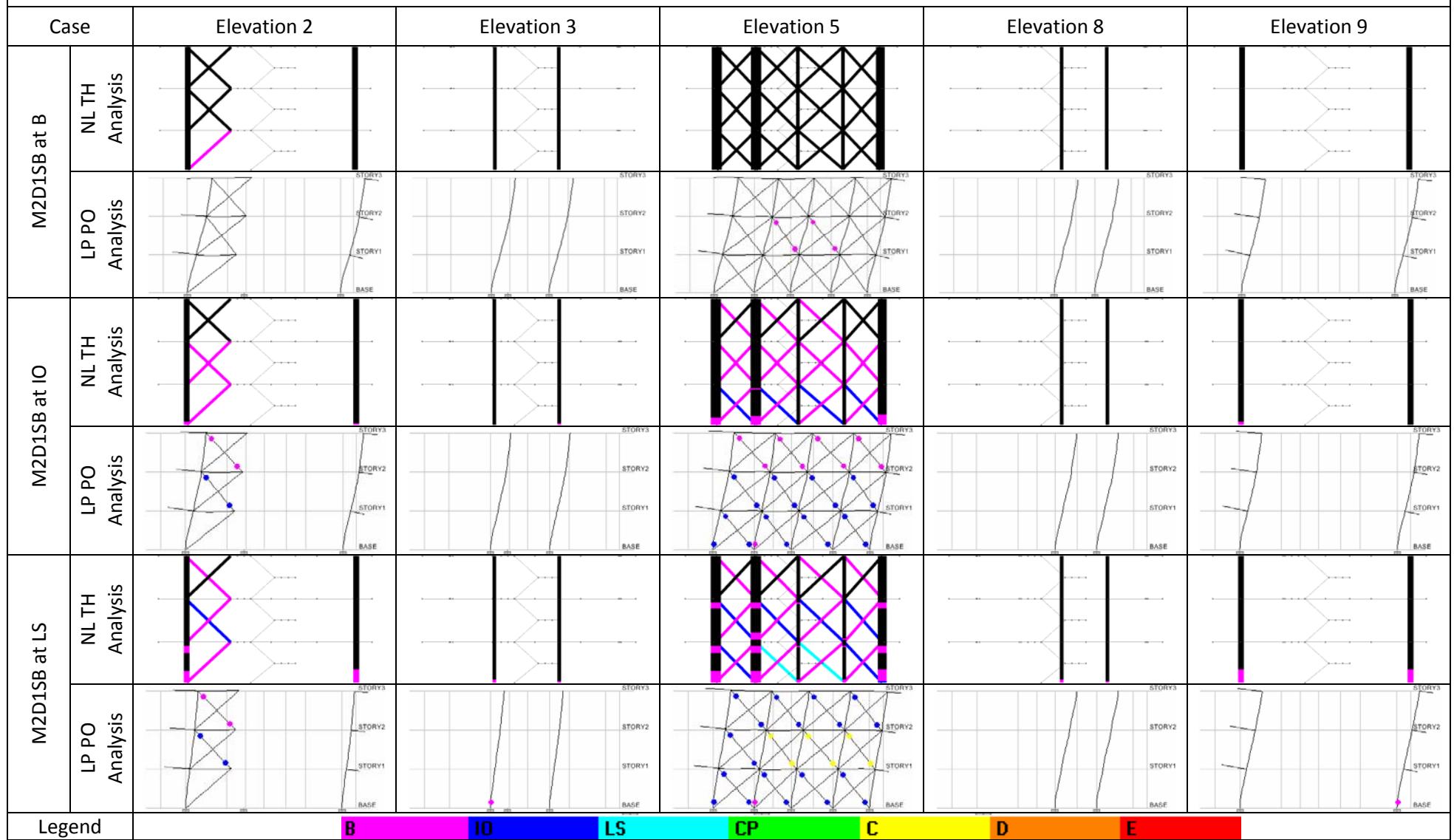


Fig. 5.13: Comparison of Damage Distribution for NL scaled TH and LP PO Analyses (STR1-infill)



CHAPTER 6 – PERFORMANCE OF STRUCTURES

6.1 Description of Performance

A seismic performance objective has two essential parts; a damage state and a level of hazard. Life safety (LS) and immediate occupancy (IO) are descriptors of damage states that do not constitute performance objectives until they are associated with a specific level of hazard. The hazard might be an earthquake or a probability of an intensity of ground motion. Defined in this way, a performance objective represents a specific risk.

6.1.1 Description of Hazard

The structures discussed are located in the city of Antakya in Hatay province of Turkey. Fig. 6.1 shows the seismic zoning map of Turkey [10] with Antakya region as marked. Antakya lies in Zone 1 which has effective peak ground acceleration A_0 of 0.4g and seismically most active among all zones.

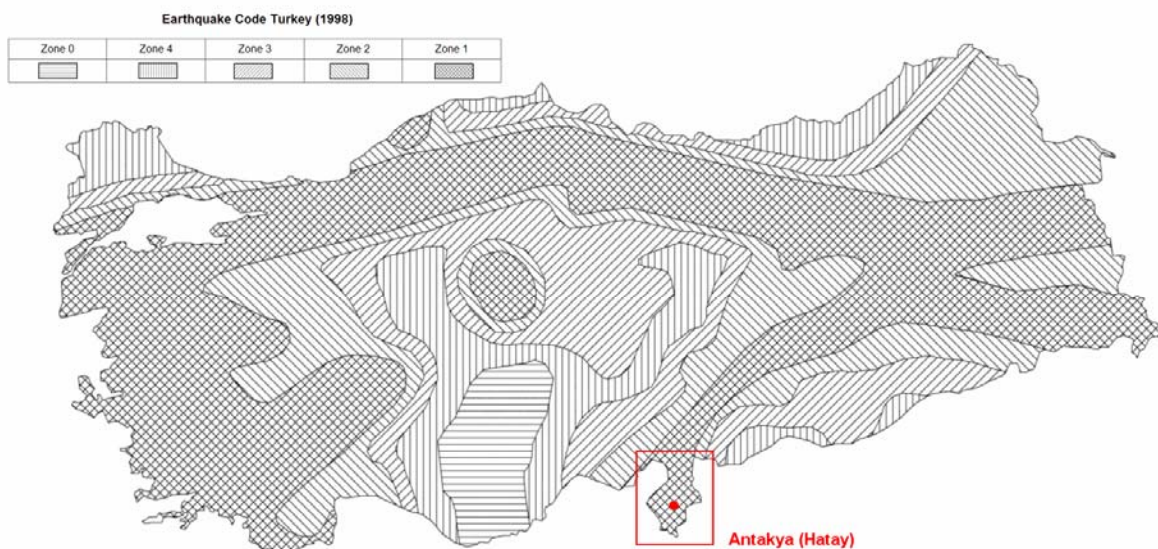


Fig. 6.1: National Seismic Zoning Map of Turkey [10]

5% damped Elastic response spectra for different soil classes according to Turkish National earthquake code have been shown in Fig. 6.2. For performance analysis, it has been assumed that soil class A of EC8 resembles with soil class Z1 of Turkish code, Soil B with Z2, C with Z3 and Soil class D with Z4.

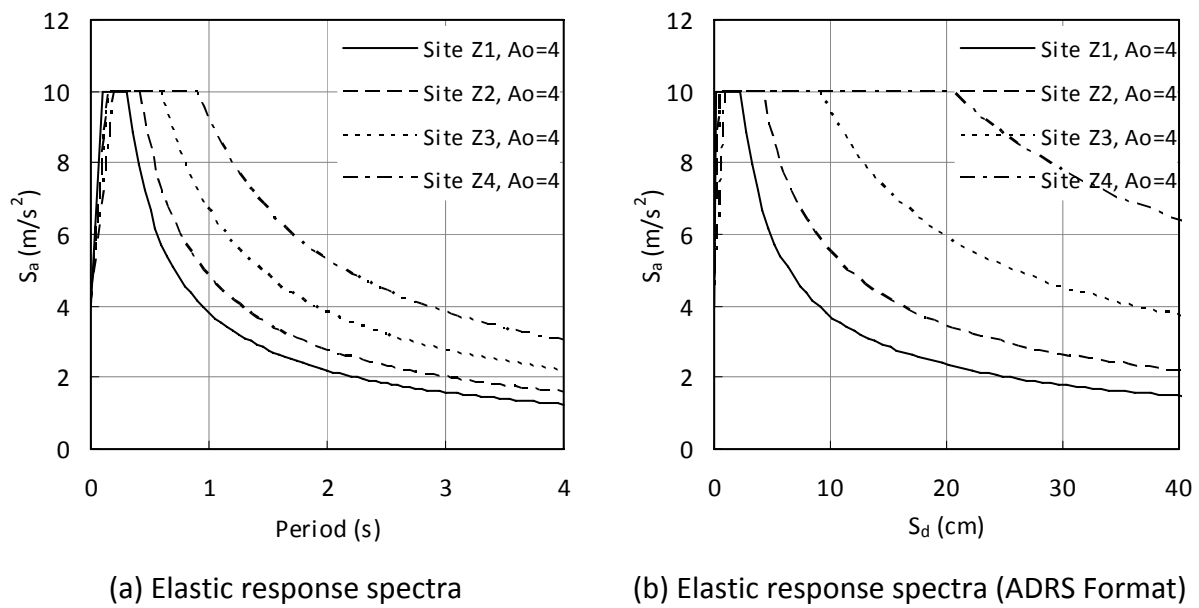


Fig. 6.2: Elastic Response Spectra according to Turkish National Earthquake Code

A_o in m/s^2

6.1.2 Description of Damage

On the response side, damage description contains two types of information [11]: first, the absolute statements that the gravity load capacity of the structure should remain substantially intact, and second, the qualitative or quantitative statements regarding the magnitude of observed damage to structural and non-structural components.

6.1.2.1 Global Building Acceptability Limits

Gravity Loads

The gravity load capacity of the structure must remain intact for acceptable performance at any level. Where an element loses capacity to support gravity loads, the structure must be capable of redistributing its load to other elements of the structure.

Lateral Loads

Some components are subject to degrading over multiple load cycles. If significant number of components degrades, the overall lateral force resistance of the structure may be affected.

Lateral Deformations

Table 6.1 presents deformation limits for various performance levels according to ATC-40 [11].

Table 6.1: Deformation Limits

	Performance Level			
Interstorey Drift Limit	Immediate Occupancy (IO)	Damage Control (DC)	Life Safety (LS)	Structural Stability (SS)
Maximum total drift	0.01	0.01-0.02	0.02	$0.33V_i/P_i$
Maximum inelastic drift	0.005	0.005-0.015	no limit	no limit

6.1.2.2 Element Acceptability Limits

Each element must be checked to determine whether its individual components satisfy acceptability requirements under performance point forces and deformation. Numerical response limits for structural and non-structural elements have already been given in Fig.3.3.

6.2 Target Performance Level

For checking performance of all the structures presented and to find if there is a need of retrofiting against some particular hazard level (here 0.4g), life safety (LS) performance level has been taken as reference for structural elements. This means that apart from the gravity load support and lateral stability, the interstorey drifts should be limited to 2% for global performance (Table 6.1), and rotation/SF of each element limited to 4 for element level performance (Fig. 3.3a).

6.3 Checking Performance

For performance calculations according to the capacity spectrum method, all the structures have been treated as average buildings, which determines the structural behaviour type as Type B. Presented calculations are for short duration shaking.

6.3.1 Performance Analysis of Building STR1

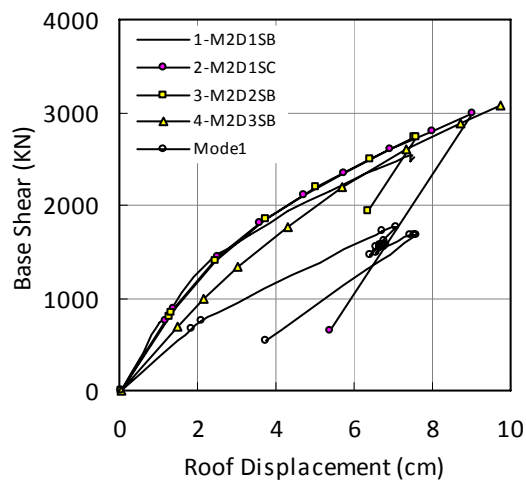
Fig. 6.3 shows the performance calculations according to the capacity spectrum method. Calculations have been performed considering four different load patterns and first mode shape of the structure. Life safety performance limit is also demarcated, considering the allowable interstorey drift and LS limit of plasticity for structural member, whichever reaches first.

Structure shows underperformance for 3 out of 5 load cases. In case of performance against first mode shape, capacity curve even does not reach to the spectral acceleration of 0.4g, which, among other requirements, is necessary for the structure to stand safely against a hazard level of 0.4g.

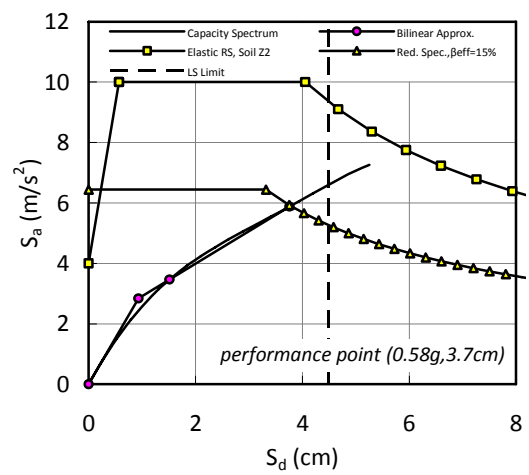
For cases M2D1SC and M2D2SB, structure shows underperformance due to the lack of hysteretic damping required for more spectral reduction. Lacklustre performance of the

structure for case Mode1 is due to dearth of strength. For cases M2D1SB and M2D3SB, near-field and far-field quake respectively for the same site class, structure shows satisfactory performance.

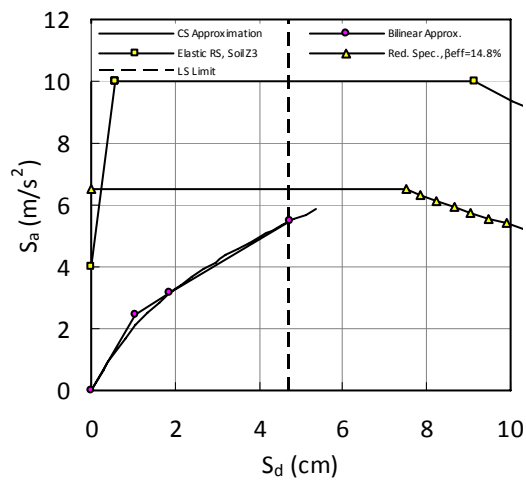
STR1-infill, owing to increased stiffness due to braces, shows a good performance agreement with the target levels (Fig. 6.4). However, performance of the structure against mode 1 is still objectionable.



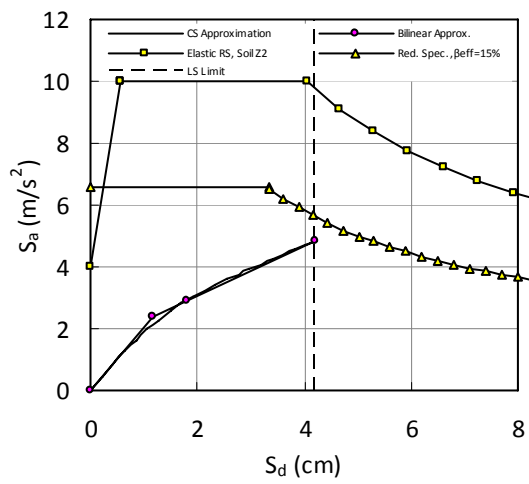
(a) Capacity Curves



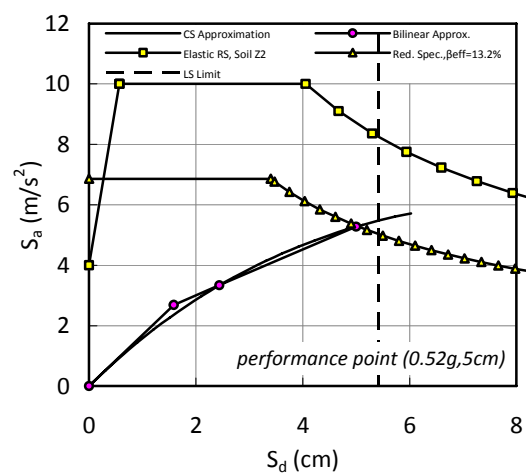
(b) Performance Check for Load Case M2D1SB



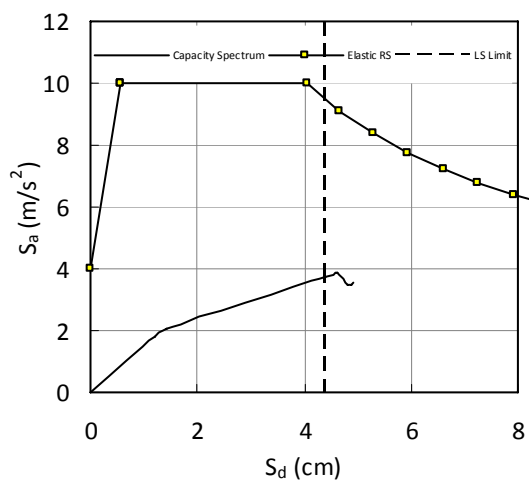
(c) Performance Check for LP Case M2D1SC



(d) Performance Check for LP Case M2D2SB

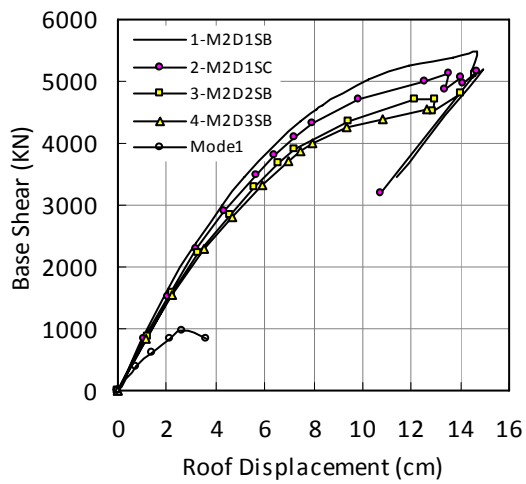


(e) Performance Check for LP Case M2D3SB

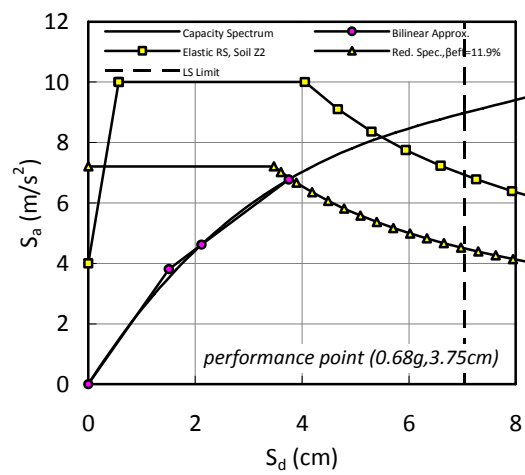


(f) Performance Check for Case Mode1

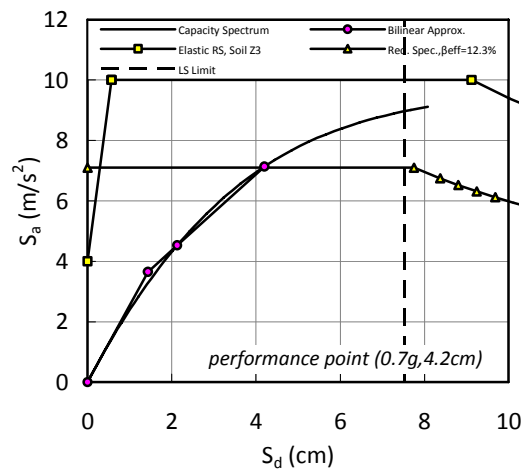
Fig. 6.3: Performance check of STR1 (without infills)



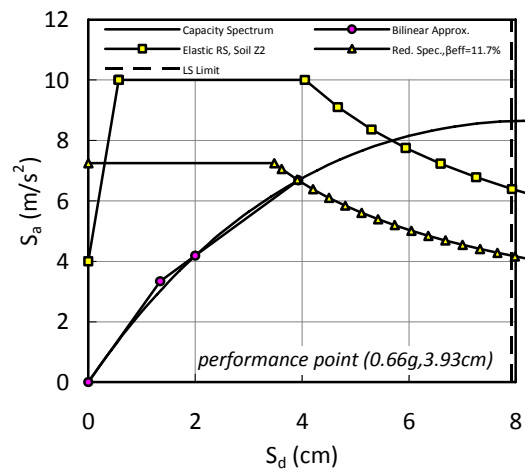
(a) Capacity Curves



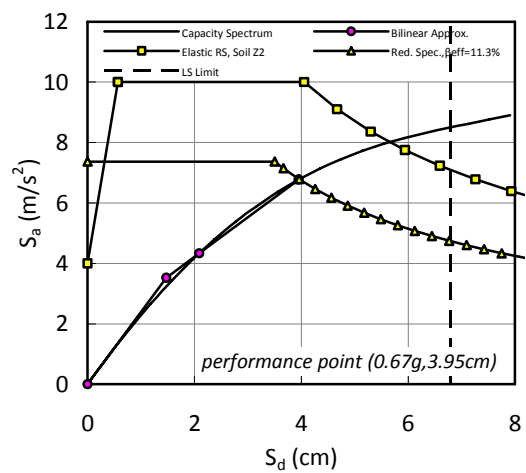
(b) Performance Check for LP Case M2D1SB



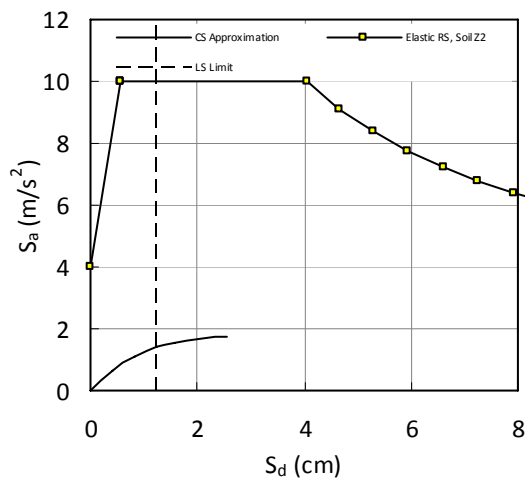
(c) Performance Check for LP Case M2D1SC



(d) Performance Check for LP Case M2D2SB



(e) Performance Check for LP Case M2D3SB



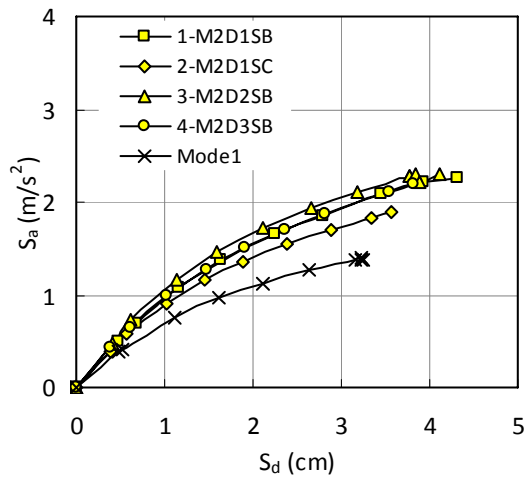
(f) Performance Check for LP Case Mode1

Fig. 6.4: Performance check of STR1-infill

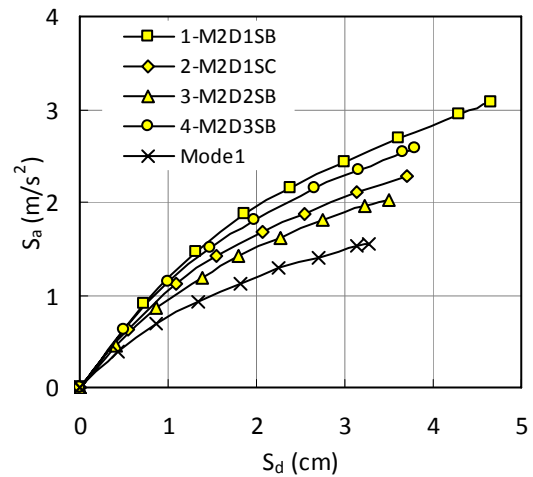
6.3.2 Performance Analysis of Building STR2 and Building STR3

From the previous section, it is evident that before starting the detailed calculations, it should be confirmed that the capacity spectrum crosses the limit of the desired hazard level. Obviously, structures with capacity spectra unable to reach this limit are expected not to withstand that hazard satisfactorily.

Fig. 6.5 covers the capacity spectra of STR2 for different LP cases. Capacity spectra in all the cases are below the hazard limit of 0.4g. Therefore, it can be safely pointed out that the structure is unable to meet seismic demand.



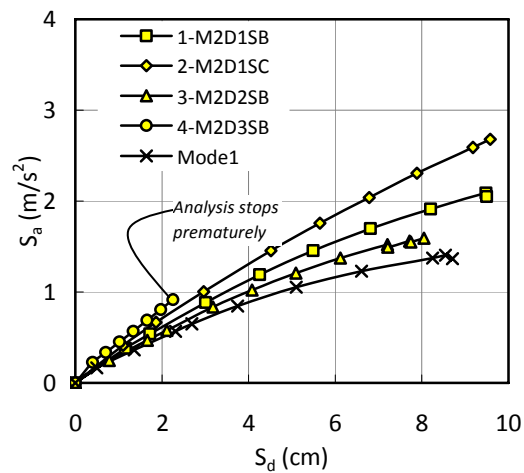
(a) Capacity Spectra of STR2



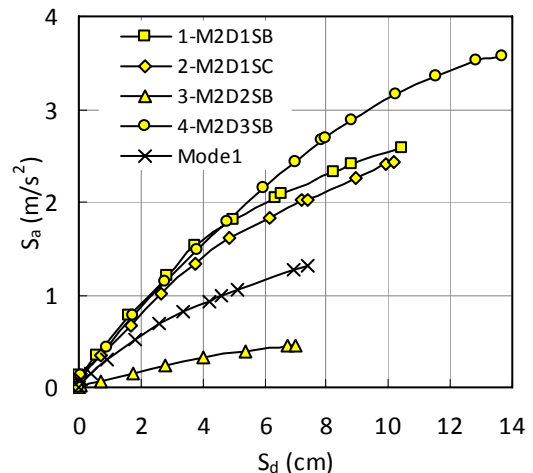
(b) Capacity Spectra of STR2-infill

Fig. 6.5: Capacity Spectra of STR2 and STR2-infill

A similar behaviour of underperformance has been seen for STR3 as shown in Fig. 6.6.



(a) Capacity Spectra of STR3



(b) Capacity Spectra of STR3-infill

Fig. 6.6: Capacity Spectra of STR3 and STR3-infill

CHAPTER 7 – NEED OF RETROFITTING AND RETROFITTING STRATEGIES

A plethora of good references like, for example, ATC-40 and a compendium of research work, is available for retrofitting techniques. The viability of many techniques is also time-proven. Therefore, the purpose of this chapter is not to test the efficacy of any particular retrofit measure but rather to, firstly, assess the need to retrofit the structures and, secondly, present some conceptual retrofitting strategies.

7.1 Performance of STR1

Performance calculations for STR1 (Fig. 6.3) show that the structure does not meet the required performance level. As this is the case without considering infills, performance of the structure can be important just after erection of RC frame and before constructing infill walls, or in case if MI walls are not constructed to interact with structural elements in an earthquake event. For cases M2D1SC and M2D2SB, structure underperforms due to the lack of hysteretic damping required for more spectral reduction. Lacklustre performance of the structure for case Mode1 is due to dearth of strength. However, structure performs well for both near-field (case M2D1SB) and far-field quakes (case M2D3SB). Also, both cases represent the same soil class.

Performance calculations for STR1-infill (Fig. 6.4) show that the structure meets the target performance level. Therefore, for the said performance level, structure does not require any explicit treatment of retrofitting. Also, during an earthquake event, this is the likely behaviour of the structure. Comparison of capacity curves of STR1 and STR1-infill clearly shows the contribution of masonry walls in the form of enhanced stiffness and strength. For this very reason, STR1-infill shows better performance. This comparison is shown in Fig. 7.1 for two analysis cases.

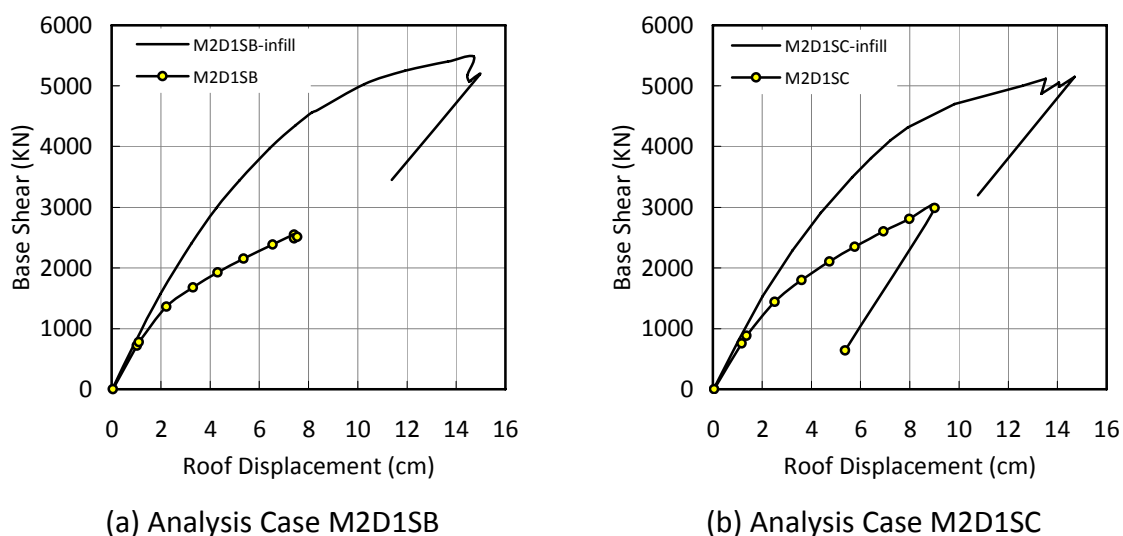


Fig. 7.1: Comparison of Capacity Curves of STR2 and STR2-infill

7.2 Performance and Critical Zones of STR2

Performance calculations for building STR2, both with and without infills (Fig. 6.5) show that the structure does not have enough strength to meet the seismic demand of 0.4g. Serviceability requirements like interstory drift should be checked once the structure under investigation fulfills strength requirements. Therefore, this aspect cannot be checked here.

Fig. 7.2 highlights the columns of STR2-infill which underperformed in each analysis case. As this is an existing building, therefore, structure with infills only has been considered. Critical zones at and beyond life safety (LS) performance level have been marked.

High eccentricity between M and S leads to torsional dominant behaviour of the structure and as a result, corner columns are affected most.

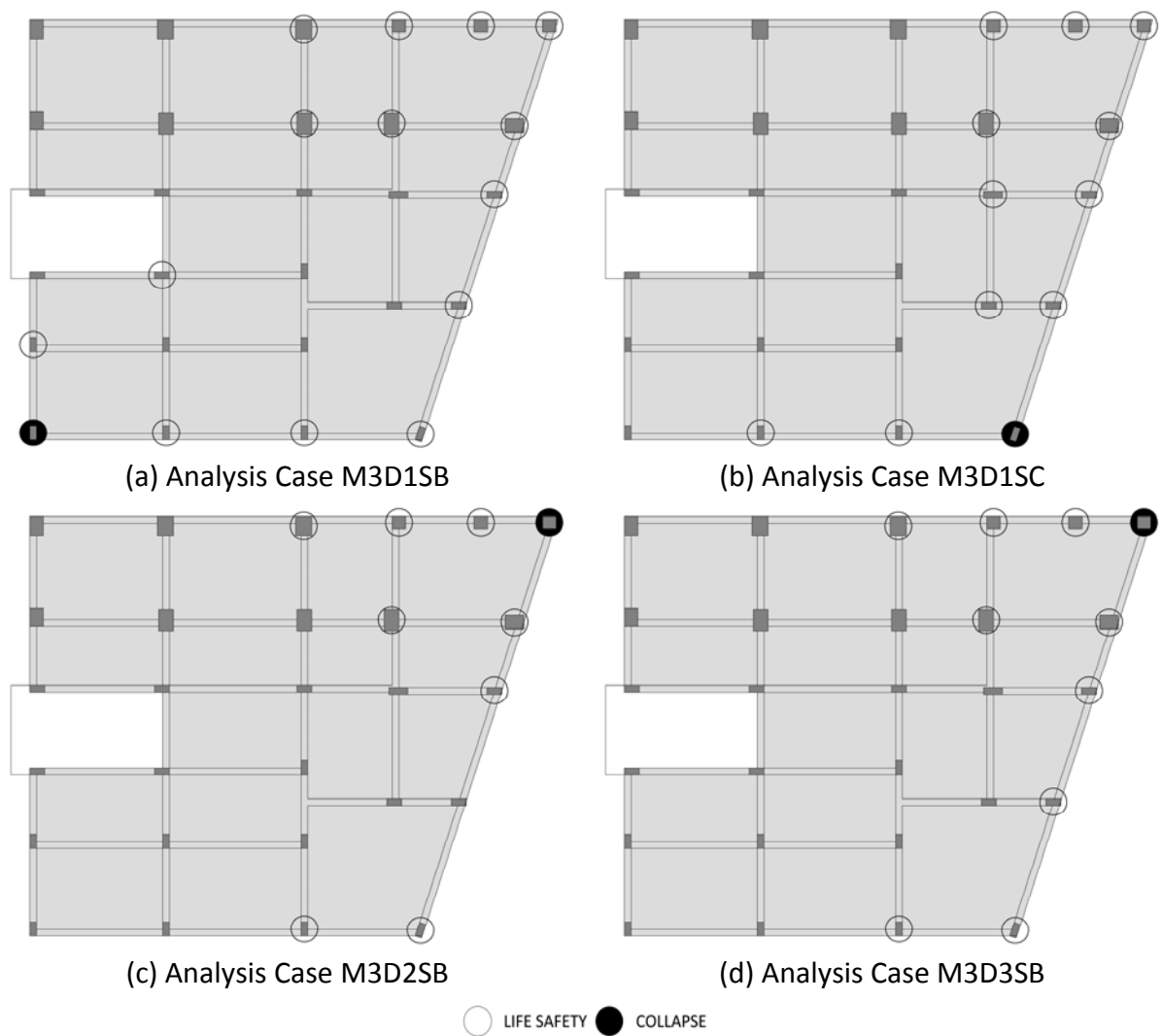


Fig. 7.2: Critical Zones of STR2-infill

7.3 Performance and Critical Zones of STR3

Performance calculations for building STR3, both with and without infills (Fig. 6.6) show that the structure does not have enough strength to meet the seismic demand of 0.4g.

Serviceability requirements like interstory drift should be checked once the structure under investigation fulfills strength requirements. Therefore, this aspect cannot be checked here.

Fig. 7.3 highlights the columns of STR3-infill which underperformed for each analysis case. As this is an existing building, therefore, structure with infills has been considered. Critical zones at and beyond life safety (LS) performance level have been marked.

STR3 consists of columns which vary largely in stiffness and strength about their principal axes. Such a design requires a non-proportional seismic demand from such elements. Irregular column grid further adds insult to the injury. A combination of these effects is reflected in the form of critical columns most of which are located on irregular grid lines. Columns which support staircase are found critical for 2 of 4 cases.

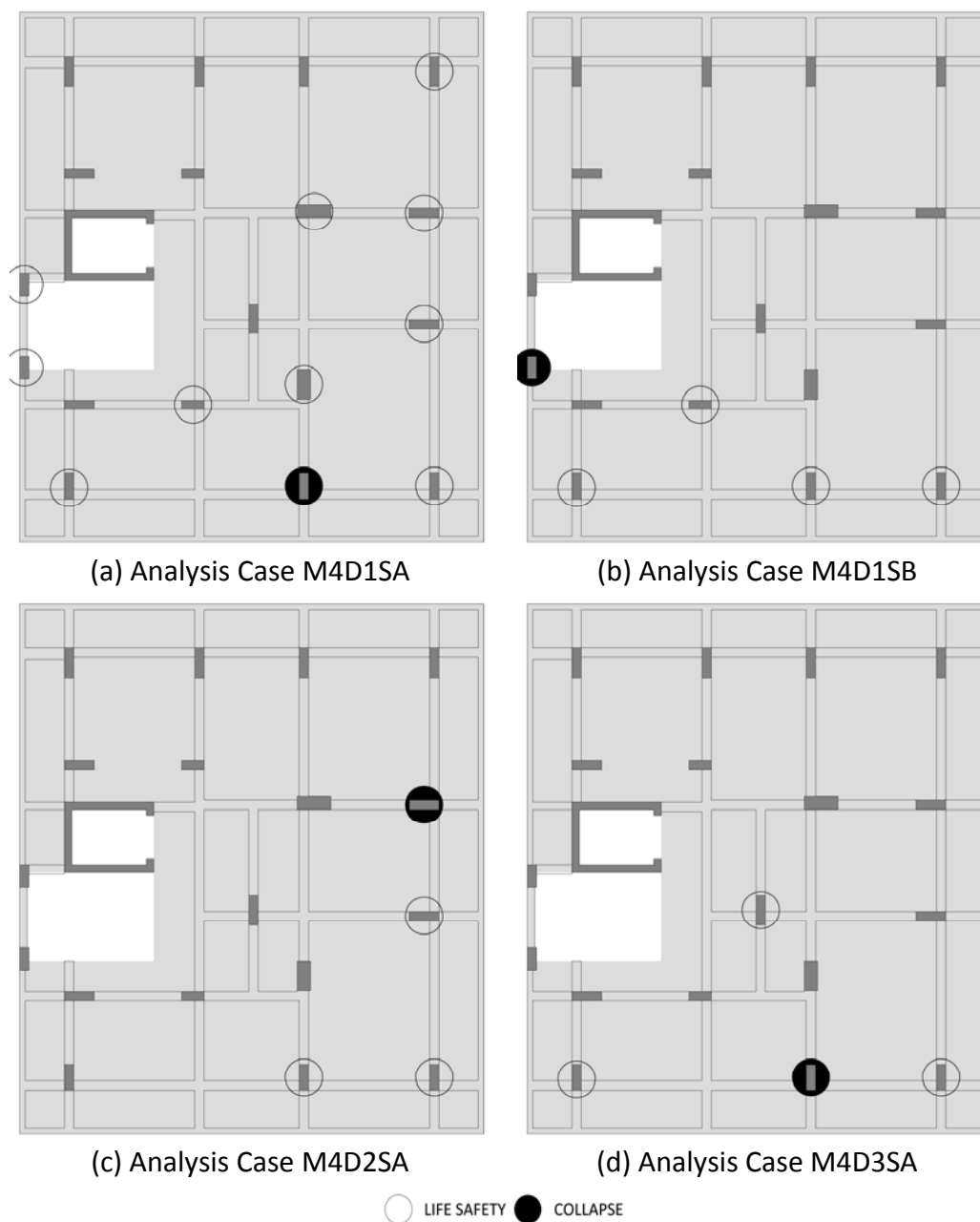


Fig. 7.3: Critical Zones of STR3-infill

7.4 Principle Retrofitting Strategies

There are seven principle retrofitting strategies used to improve either any or a combination of strength, ductility, stiffness and damping of the structure, given in Fig. 7.4 [12]. Explanation of these strategies with typical remedial measures is tabulated in Table 7.1. It is worth mentioning here that there are also a number of other remedial measures available resulting in the same desired effects and these can be employed according to the constraints present. For example, column jacketing improves strength and stiffness.

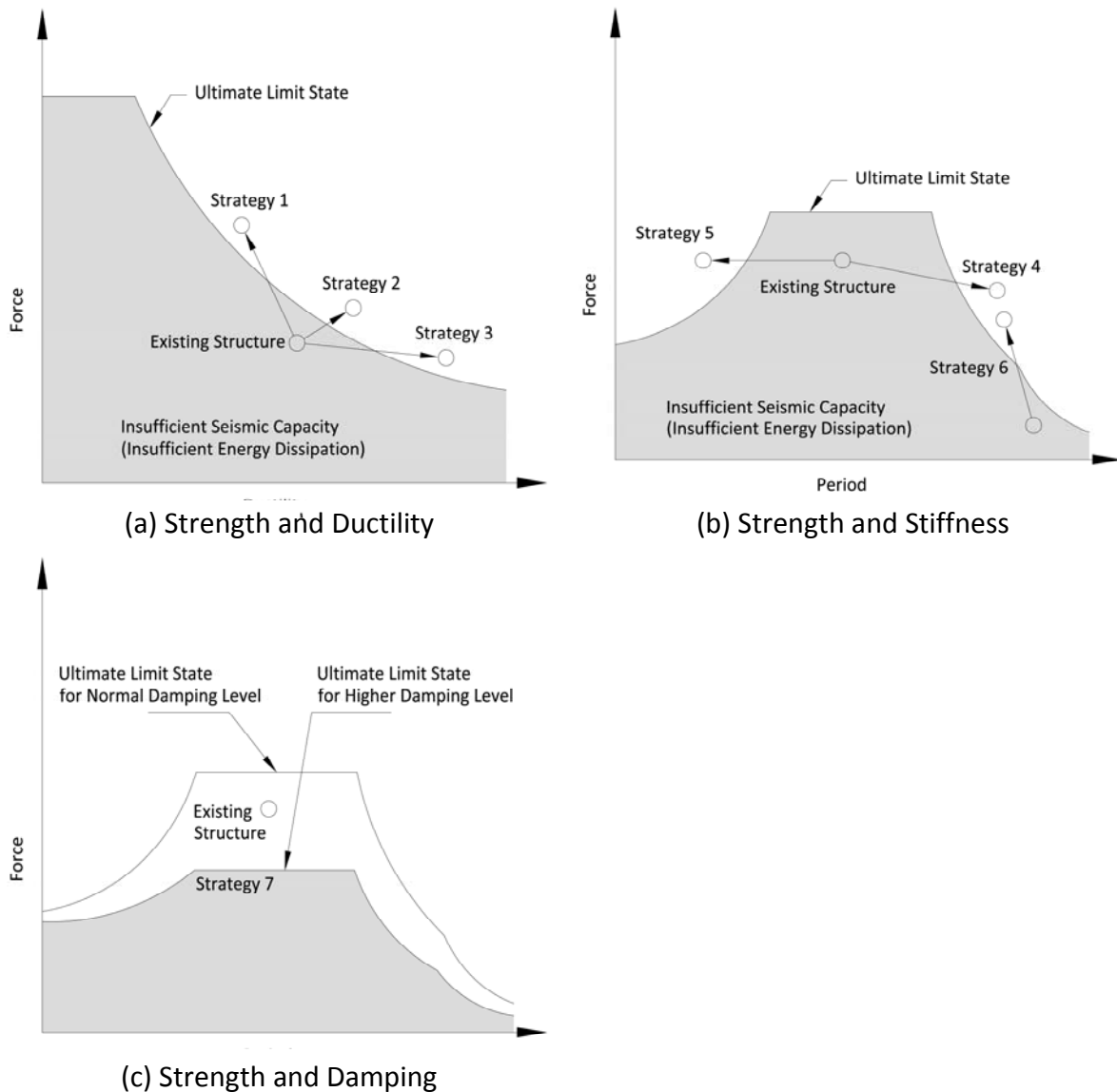


Fig. 7.4: Principle Retrofitting Strategies [12]

Table 7.1: Retrofitting Strategies

Strategy	Desired Effects		Typical Remedial Measure	Recommended for
1	Strength	↑	Insertion of RC walls into RC frame	STR2, STR3
	Ductility	↓		
	Stiffness	-		
	Damping	-		
2	Strength	↑	Wrapping of columns of an RC frame with effect of moving plastic hinge from columns to beams	STR2, STR3
	Ductility	↑		
	Stiffness	-		
	Damping	-		
3	Strength	↓	Infill masonry walls removed from an RC frame	-
	Ductility	↑		
	Stiffness	↓		
	Damping	-		
4	Strength	-	Seismic isolation	-
	Ductility	-		
	Stiffness	↓		
	Damping	-		
5	Strength	-	(of little practical significance)	-
	Ductility	-		
	Stiffness	↑		
	Damping	-		
6	Strength	↑	Additional RC structural walls	STR2, STR3
	Ductility	-		
	Stiffness	↑		
	Damping	-		
7	Strength	-	Seismic isolation	-
	Ductility	-		
	Stiffness	-		
	Damping	↑		

↑ increase ↓ decrease

CHAPTER 8 - CONCLUSIONS

For representative structures of typical building stock in the city Antakya, Turkey, seismic load patterns using displacement based approach have been developed. These load patterns have been developed using a host of real-time earthquake recordings which inherently represent magnitude-distance dependence and local site characteristics. Thus this very important factor of incorporating effects of magnitude, distance and local site characteristics is, in turn, reflected in the different loading shapes and their loading intensities.

These load patterns show a considerable difference with the fundamental mode of vibration of the structures which are conventionally used as forcing function in nonlinear static analysis.

It has been observed that as the flexibility of the structure increases, load patterns tend to show torsion-dominant shapes both in plan and along the height of the structure.

A comparison of results of nonlinear time history analysis preformed on fibre-element model with that of load pattern pushover analysis performed on line-element shows a reasonable agreement in the damage and its progression, and this pinpoints the applicability of the developed load pattern as forcing function for nonlinear static analysis.

The developed load patterns have a shape resembling the shear deformation shape under lateral loads. This is also in consistence with the structural system, i.e., frame action, of each structure. Therefore, care must be exercised while using such load patterns as these would not be applicable for structures with, for example, shear walls as the main lateral-force resistant system deforming predominantly in the bending shape under lateral loads.

As the structures experience nonlinear behaviour, it's been revealed that the intensities of loads in the load pattern increase.

From comparison of capacity curves, it is concluded that the mode 1 as forcing function is more detrimental and underestimates the strength of the structure than load patterns as forcing function. This fact gets further reinforcement from the comparison of results of nonlinear time history analysis with that of load pattern pushover analysis.

Performance analysis for all the structures has been carried out for a predefined target performance level and against the local seismic hazard present in Antakya. Vulnerability to seismic activity has been checked considering bare frame models and frames along with the effect of masonry infill walls.

From performance analysis, it is deduced that building STR1 shows underperformance for few seismic actions but this happens only for the structure without infills. As the structure is already functional with infills, there seems to be no danger to the structure for the local hazard of 0.4g.

Buildings STR2 and STR3, however, are in substantial need of retrofit. Both the structures show considerable reduction in strength as the seismic demand increases. Critical zones of both the structures corresponding to the different action types have been highlighted and conceptual retrofit strategies rather than the discrete remedial measures have been proposed.

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ANNEXURE A

TIME HISTORY RESPONSE SPECTRA AND APPROXIMATED EC8 RESPONSE SPECTRA

This part is an annexure of section [2.2](#) of the main text. It contains the response spectra obtained from acceleration time histories and response spectra approximated according to EC8.

Data are shown in the form of a set of table and figure. Each table contains the details of earthquake, the response spectra of which are shown in the figure followed by it.

Table A1 (a): Earthquake Recording Details for Magnitude Range M1, Distance Range D1

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
7/5/1984	41.666 N	13.820 E	2	4.2	Colfiorito-Casermette	Italy	1	829	A
10/16/1997	43.034 N	12.891 E	3	4.3	Colfiorito-Casermette	Italy	3	825	A
13/10/1997	42.892 N	12.922 E	-	4.4	Hveragerdi-Church	Iceland	9	5028	A
6/10/1997	43.019 N	12.838 E	3	4.3	Nocera	Italy	10	816	A
7/10/1997	43.025 N	12.865 E	2	4.2	Nocera	Italy	10	821	A
16/10/1997	43.034 N	12.891 E	10	4.4	Sellano	Italy	10	846	A
9/16/1977	46.280 N	12.980 E	13	4.1	Kentro-Town-Hall	Greece	11	6104	A
9/16/1978	33.249 N	57.384 E	9	3.9	Nocera-Umbra	Italy	13	587	A
13/05/1995	40.183 N	21.660 E	10	4.4	Nocera	Italy	17	850	A
4/6/1998	64.036 N	21.293 W	-	4.4	Selfoss-Hospital	Iceland	18	5027	A
4/29/1984	43.204 N	12.585 E	10	4.4	Nocera	Italy	19	848	A
3/4/1998	43.200 N	12.790 E	-	4.4	Oseyrarbru	Iceland	11	5038	A
9/15/1976	46.318 N	13.119 E	2	4.2	Nocera	Italy	12	833	A
12/10/1997	42.910 N	12.946 E	7	4.3	LDEOStationNo.C0375VO	Turkey	14	6440	A
11/9/1976	46.286 N	13.160 E	10	4.4	Nocera	Italy	10	857	A
26/09/1997	43.031 N	12.862 E	10	4.4	Nocera	Italy	11	853	A
12/11/1999	40.768 N	31.148 E	-	4.4	Thorlakshofn	Iceland	11	5030	B
3/10/1997	43.033 N	12.838 E	7	4.3	LDEOStationNo.D0531WF	Turkey	12	6441	B
9/15/1976	46.318 N	13.119 E	9	4	Forgaria-Cornio	Italy	1	89	B
4/6/1998	64.036 N	21.293 W	9	4	San-Rocco	Italy	1	93	B
11/23/1980	40.780 N	15.330 E	13	4.1	Karpero-Town-Hall	Greece	3	6098	B
11/6/1976	46.230 N	13.000 E	13	3.6	Forgaria-Cornio	Italy	4	83	B
26/09/1997	43.023 N	12.892 E	9	3.9	Colfiorito	Italy	4	586	B
3/9/1997	43.015 N	12.880 E	7	4.3	LDEOStationNo.C0362CH	Turkey	5	6439	B
1/16/1981	40.850 N	15.280 E	7	3.5	Forgaria-Cornio	Italy	6	75	B
1/16/1981	40.850 N	15.280 E	7	3.5	San-Rocco	Italy	6	76	B
5/18/1976	46.270 N	13.030 E	9	4	Maiano-Piano-Terra	Italy	7	91	B
7/18/1979	39.660 N	28.650 E	9	4	Maiano-Prato	Italy	7	92	B
11/6/1976	46.230 N	13.000 E	7	4.3	LDEOStationNo.C1061	Turkey	7	6437	B
7/11/1999	40.697 N	30.717 E	13	3.6	Maiano-Piano-Terra	Italy	9	85	B
17/06/2000	63.970 N	20.360 W	13	3.6	Maiano-Prato	Italy	9	86	B
9/13/1986	37.100 N	22.190 E	7	4.3	LDEOStationNo.C1060BU	Turkey	12	6436	B
9/15/1976	46.291 N	13.153 E	5	4.2	Maiano-Municipio	Italy	15	66	B
11/9/1976	46.286 N	13.160 E	15	4.2	Forgaria-Cornio	Italy	16	94	B
15/06/1991	42.465 N	44.030 E	13	4.1	Knidi-Forest-Inspe	Greece	18	6107	B
11/23/1980	40.780 N	15.330 E	10	4.4	Nocera	Italy	8	872	C
5/4/1998	43.185 N	12.719 E	10	4.4	Gubbio-Piana	Italy	18	855	C
11/23/1980	40.780 N	15.330 E	10	3.2	Duzce-Meteoroloji-Mudu	Turkey	5	2916	C
11-Apr-73	38.780 N	20.550 E	7	4.3	LDEOStationNo.C1062FI	Turkey	9	6438	C
11/9/1976	46.277 N	13.175 E	10	3.2	Golyaka	Turkey	10	2914	C
9/15/1976	46.291 N	13.153 E	7	4.3	Buia	Italy	11	137	C

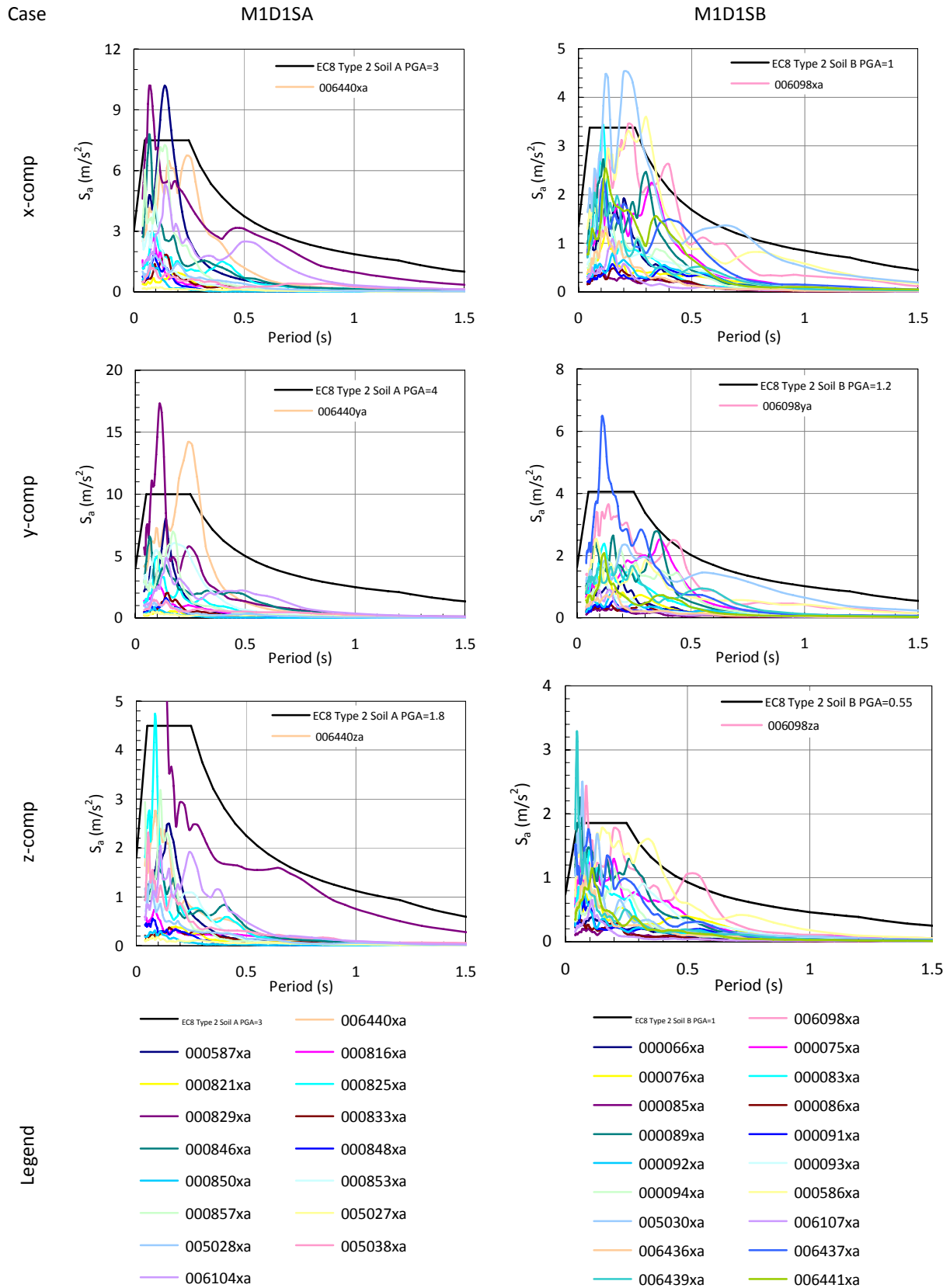


Fig. A1 (a): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

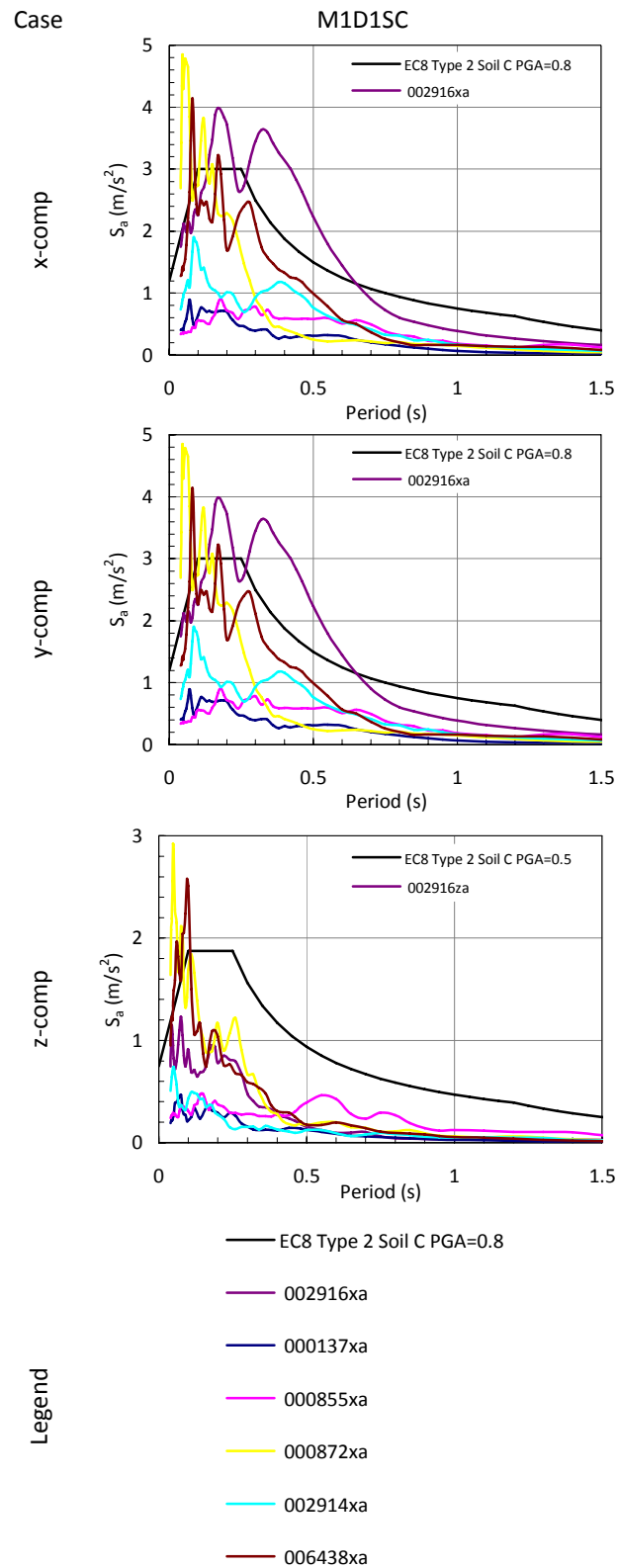


Fig. A1 (a): Acceleration Time History- and Approximated EC8 Response Spectra

Table A1 (b): Earthquake Recording Details for Magnitude Range M1, Distance Range D2

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
12/10/1997	42.910 N	12.946 E	14	4.4	Karpenisi-Prefecture	Greece	24	5822	A
11/23/1980	40.780 N	15.330 E	7	4.4	Kyparrisia-Agriculture	Greece	26	5824	A
5/5/1986	38.020 N	37.790 E	8	4.1	Poligiros-Prefecture	Greece	29	6152	A
3/5/1995	40.580 N	23.650 E	16	4.5	Poligiros-Prefecture	Greece	29	6153	A
9/11/1997	42.854 N	13.000 E	-	4.4	Reykjavik-Hus-Verslunar	Iceland	34	5035	A
3/4/1998	43.200 N	12.790 E	10	4.4	Sellano	Italy	39	859	A
9/19/1979	42.760 N	13.020 E	7	4.3	LDEOStationNo.C1058BV	Turkey	26	6434	B
5/5/1990	40.644 N	15.875 E	10	4.5	Genf-Marziano	Swit	29	1330	B
11/9/1976	46.286 N	13.160 E	5	3.8	Balikesir-Bayindirlik	Turkey	34	7060	B
31/08/1999	40.756 N	29.935 E	3	4.3	Gubbio-Piana	Italy	39	784	C

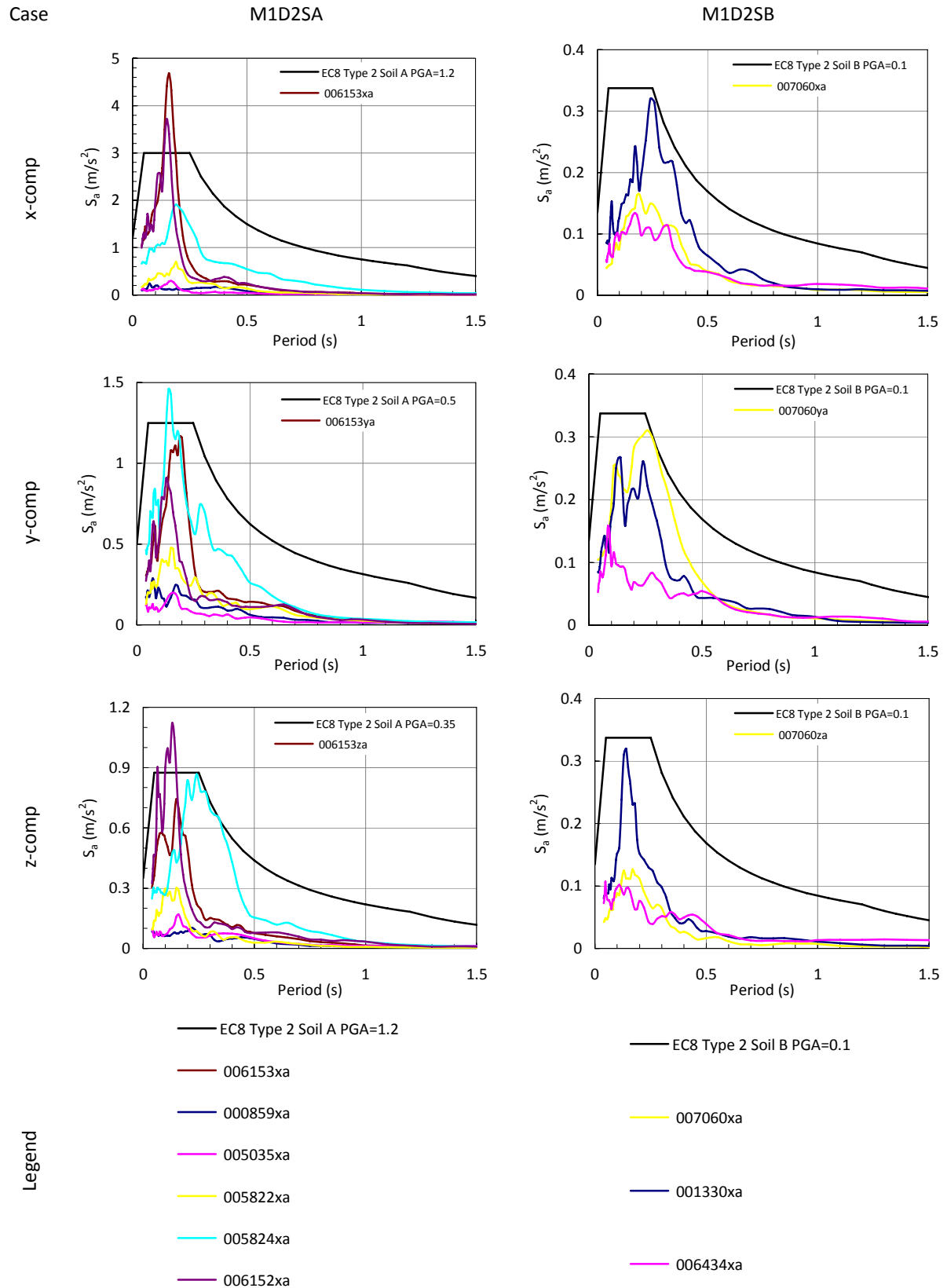


Fig. A1 (b): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

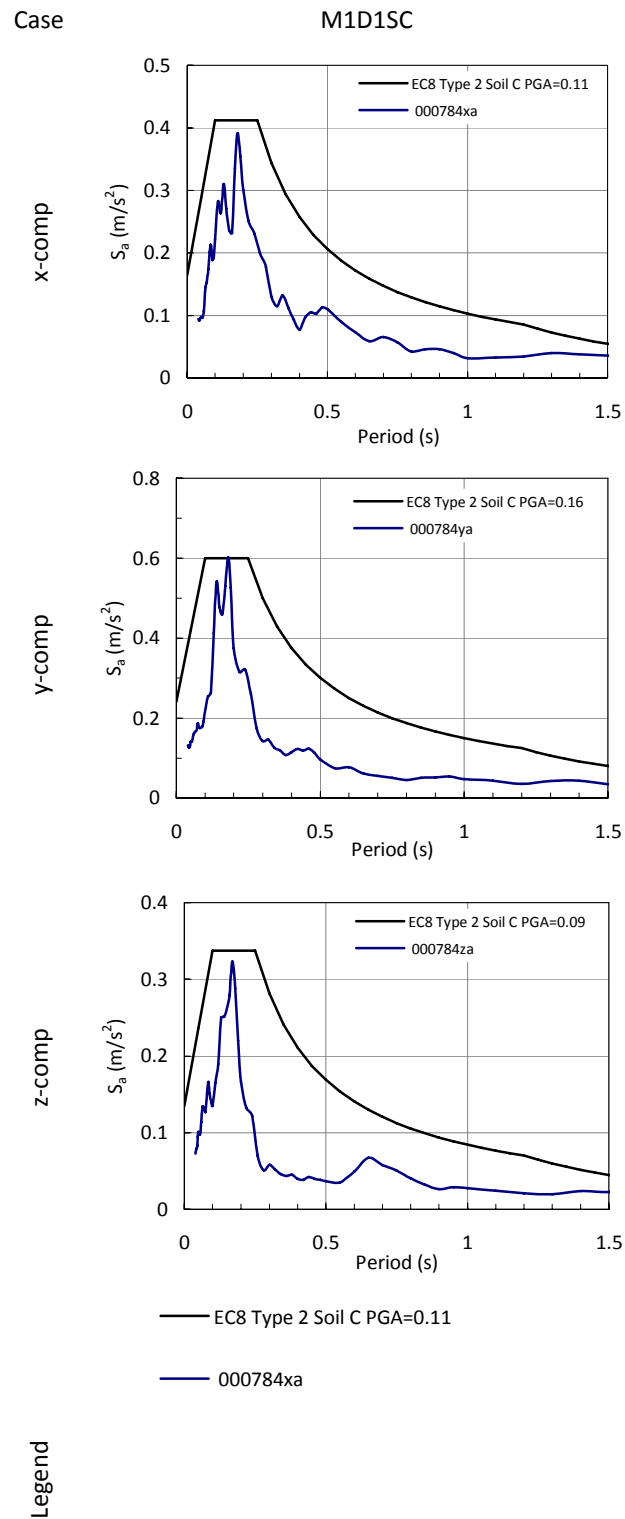


Fig. A1 (b): Acceleration Time History- and Approximated EC8 Response Spectra

Table A1 (c): Earthquake Recording Details for Magnitude Range M1, Distance Range D3

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
12/11/1999	40.768 N	31.148 E	-	4.4	Kaldarholt	Iceland	43	5029	B
4/5/1995	40.590 N	23.600 E	10	3.2	Akyazi	Turkey	45	2915	B
29/08/1999	40.798 N	31.124 E	-	4.4	Hella	Iceland	49	5031	B
17/08/1999	40.702 N	29.987 E	10	4.5	Yverdon-Jordils	Swit	102	1340	B
29/09/1999	40.738 N	29.331 E	12	4.4	AngradoHeroismo,GZCAH	Portuga	141	7338	B
13/06/1993	39.250 N	20.570 E	10	3.2	Adapazari-3	Turkey	62	2918	C
20/06/1990	36.960 N	49.410 E	7	4.3	Fatih-Tomb	Turkey	153	6994	C
26/09/1997	43.023 N	12.892 E	-	3.7	Duzce-MeteorolojiMudur	Turkey	160	4473	C

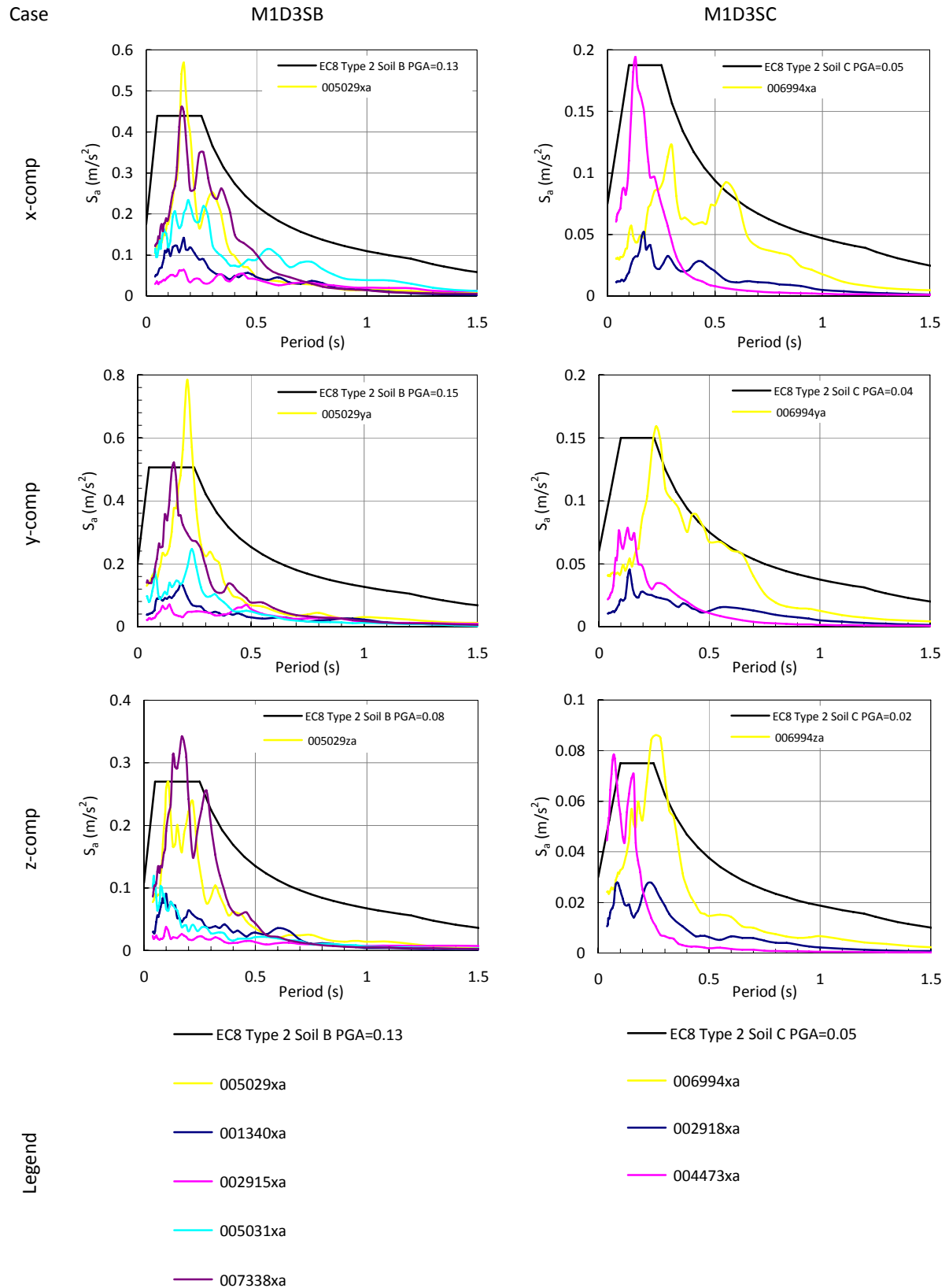


Fig. A1 (c): Acceleration Time History- and Approximated EC8 Response Spectra

Table A2 (a): Earthquake Recording Details for Magnitude Range M2, Distance Range D1

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
6/21/2000	63.970 N	20.710 W	8	5.3	Villetta-Barrea	Italy	6	384	A
11/5/1984	41.732 N	13.921 E	18	5.1	Adra-Refugio-de-la-	Spain	6	877	A
23/12/1993	36.800 N	2.980 W	-	5.2	Hveragerdi-Church	Iceland	6	5079	A
21/06/2000	63.970 N	20.710 W	2	4.8	Norcia	Italy	7	840	A
9/11/1997	42.854 N	13.000 E	5	5	Lioni-Macello	Italy	8	321	A
1/16/1981	40.850 N	15.280 E	6	4.9	Nocera	Italy	8	813	A
3/10/1997	43.033 N	12.838 E	8	5.2	Somplago-Centrale	Italy	9	982	A
9/16/1977	46.280 N	12.980 E	10	4.6	Borgo-Cerreto	Italy	9	654	A
17/08/1999	40.702 N	29.987 E	15	5	Ferruzzano	Italy	10	169	A
11/3/1978	38.020 N	16.030 E	6	4.9	Nocera	Italy	10	770	A
3/10/1997	43.033 N	12.838 E	7	5.2	Nocera	Italy	10	649	A
21/03/1998	42.958 N	12.879 E	6	4.8	Nocera	Italy	10	856	A
3/4/1998	43.200 N	12.790 E	8	5.2	Tolmezzo-Base-Diga	Italy	11	981	A
11/6/1995	39.920 N	21.620 E	2	4.8	Colfiorito-Casermette	Italy	11	823	A
4/10/1997	42.933 N	12.936 E	7	5.2	Nocera	Italy	11	623	A
3/9/1997	43.015 N	12.880 E	7	5.5	Nocera	Italy	13	593	A
26/09/1997	43.023 N	12.892 E	6	4.7	Tolmezzo-Diga-Ambiesta	Italy	14	706	A
21/06/2000	63.970 N	20.710 W	12	4.9	Ouranoupolis-Seismogr	Greece	15	1917	A
8/26/1983	40.450 N	23.920 E	8	5.3	Atina-Pretura-PianoT	Italy	15	385	A
11/5/1984	41.732 N	13.921 E	8	5.3	Atina-PreturaTerrazza	Italy	15	990	A
11/5/1984	41.732 N	13.921 E	-	5.2	Irafoss-Hydroelectric	Iceland	15	5085	A
21/06/2000	63.970 N	20.710 W	8	5.3	Atina	Italy	16	382	A
11/5/1984	41.732 N	13.921 E	9	5.4	Gubbio	Italy	17	359	A
4/29/1984	43.204 N	12.585 E	30	5	Kyparrisia-Agricultu	Greece	17	1900	A
21/03/1998	42.958 N	12.879 E	6	4.8	Cassignano	Italy	17	873	A
3/4/1998	43.200 N	12.790 E	41	4.9	Nocera	Italy	18	849	A
26/03/1998	43.238 N	12.931 E	41	4.9	Nocera	Italy	18	851	A
26/03/1998	43.238 N	12.931 E	-	5.2	Oseyrarbru	Iceland	18	5090	A
4/6/1998	64.036 N	21.293 W	-	5.2	Selfoss-Hospital	Iceland	18	5078	A
13/11/1998	63.954 N	21.346 W	9	5.4	Nocera-Umbra	Italy	19	361	A
6/5/1976	46.292 N	13.253 E	6	4.9	Colfiorito-Casermette	Italy	5	822	A
3/10/1997	43.033 N	12.838 E	7	5.2	Colfiorito-Casermette	Italy	5	651	A
6/10/1997	43.019 N	12.838 E	6	5.2	Borgo-Cerreto	Italy	11	765	A
12/10/1997	42.910 N	12.946 E	6	4.8	Nocera	Italy	11	852	A
17/06/2000	63.970 N	20.360 W	6	5.2	Colfiorito-Casermette	Italy	14	826	A
17/06/2000	63.970 N	20.360 W	28	5.3	Akhalkalaki	Georgia	20	487	A
12/11/1999	40.768 N	31.148 E	3	5.5	Tarcento	Italy	8	707	A
5/4/1998	43.185 N	12.719 E	7	5.2	Assisi-Stallone	Italy	20	670	A
11/6/1976	46.230 N	13.000 E	6	4.7	Forgaria-Cornio	Italy	4	67	B
8/26/1983	40.450 N	23.920 E	6	4.7	Maiano-Municipio	Italy	10	68	B
9/15/1976	46.318 N	13.119 E	3	5.5	Forgaria-Cornio	Italy	15	114	B
1/16/1981	40.850 N	15.280 E	8	5.2	Forgaria-Cornio	Italy	7	159	B
26/03/1998	43.238 N	12.931 E	8	5.2	San-Rocco	Italy	7	160	B
26/09/1997	43.031 N	12.862 E	5	5	Dursunbey-Kandilli-Goz	Turkey	6	239	B
9/6/1976	46.260 N	13.010 E	5	5	Conza-Vetta	Italy	5	317	B
11/5/1976	46.258 N	12.985 E	5	5	Cairano-1	Italy	5	318	B
7/11/1999	40.697 N	30.717 E	5	5	Cairano-3	Italy	6	319	B

Cont'd

Table A2 (a): Earthquake Recording Details for Magnitude Range M2, Distance Range D1

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
11/6/1976	46.230 N	13.000 E	5	5	Cairano-4	Italy	7	320	B
9/16/1977	46.280 N	12.980 E	5	5	Procisa-Nuova	Italy	8	322	B
21/06/2000	63.970 N	20.710 W	28	5.3	Bogdanovka	Georgia	15	488	B
11/23/1980	40.780 N	15.330 E	10	5.1	Aigio-OTE-Building	Greece	18	577	B
9/6/1976	46.260 N	13.010 E	10	5.1	Nafpaktos-OTE-Building	Greece	10	578	B
23/08/2000	40.680 N	30.720 E	7	5.2	Colfiorito	Italy	7	622	B
17/08/1999	40.702 N	29.987 E	6	4.9	Colfiorito	Italy	7	776	B
12/10/1997	42.910 N	12.946 E	6	5.2	Norcia	Italy	18	790	B
13/11/1998	63.954 N	21.346 W	6	5.2	Colfiorito	Italy	14	792	B
5/18/1976	46.270 N	13.030 E	41	4.9	Matelica	Italy	6	867	B
16/12/1990	41.370 N	43.720 E	6	4.8	Matelica	Italy	18	871	B
11/6/1995	39.920 N	21.620 E	5	5	Contrada-Fiumicell	Italy	4	980	B
7/9/1999	38.080 N	23.580 E	26	5	Argostoli-OTE-Building	Greece	20	1862	B
1/16/1981	40.850 N	15.280 E	12	4.9	Ierissos-Police-Station	Greece	8	1882	B
23/08/2000	40.680 N	30.720 E	5	5.1	Vasiliki-Town-Hall	Greece	12	1990	B
13/03/1992	39.720 N	39.630 E	5	4.8	Almiros-Volos-Town-Hall	Greece	14	2006	B
7/9/1999	38.080 N	23.580 E	18	5.2	Drama-Prefecture	Greece	19	2017	B
1/16/1981	40.850 N	15.280 E	9	4.8	Chromio-Community	Greece	9	6050	B
7/9/1999	38.080 N	23.580 E	9	4.8	Grevena-Town-Hall	Greece	19	6070	B
10/5/1976	46.280 N	13.220 E	7	4.9	Grevena-Town-Hall	Greece	15	6072	B
19/05/1995	40.090 N	21.600 E	7	4.9	Karpero-Town-Hall	Greece	16	6093	B
18/11/1997	37.482 N	20.692 E	21	4.8	Vasiliki-Town-Hall	Greece	15	6167	B
17/08/1999	40.702 N	29.987 E	3	5.5	Buia	Italy	9	116	C
23/08/2000	40.680 N	30.720 E	6	4.9	Nocera	Italy	13	771	C
11/6/1995	39.920 N	21.620 E	7	5.2	Nocera	Italy	15	624	C
9/11/1997	42.854 N	13.000 E	5	5.1	Lefkada-Hospital	Greece	15	1911	C
18/11/1997	37.482 N	20.692 E	21	4.8	Lefkada-Hospital	Greece	17	1913	C
29/08/1999	40.798 N	31.124 E	10	5.1	Pyrgos-Agriculture-Bank	Greece	10	558	C
26/03/1993	37.590 N	21.390 E	4	4.8	Yarimca-Petkim	Turkey	14	6958	C
9/15/1976	46.318 N	13.132 E	5	5.1	Lefkada-OTE-Building	Greece	16	581	C
31/08/1999	40.756 N	29.935 E	10	5.1	Patra-OTE	Greece	19	579	C
25/02/1994	38.730 N	20.580 E	7	5.2	Castelnuovo-Assisi	Italy	20	625	C

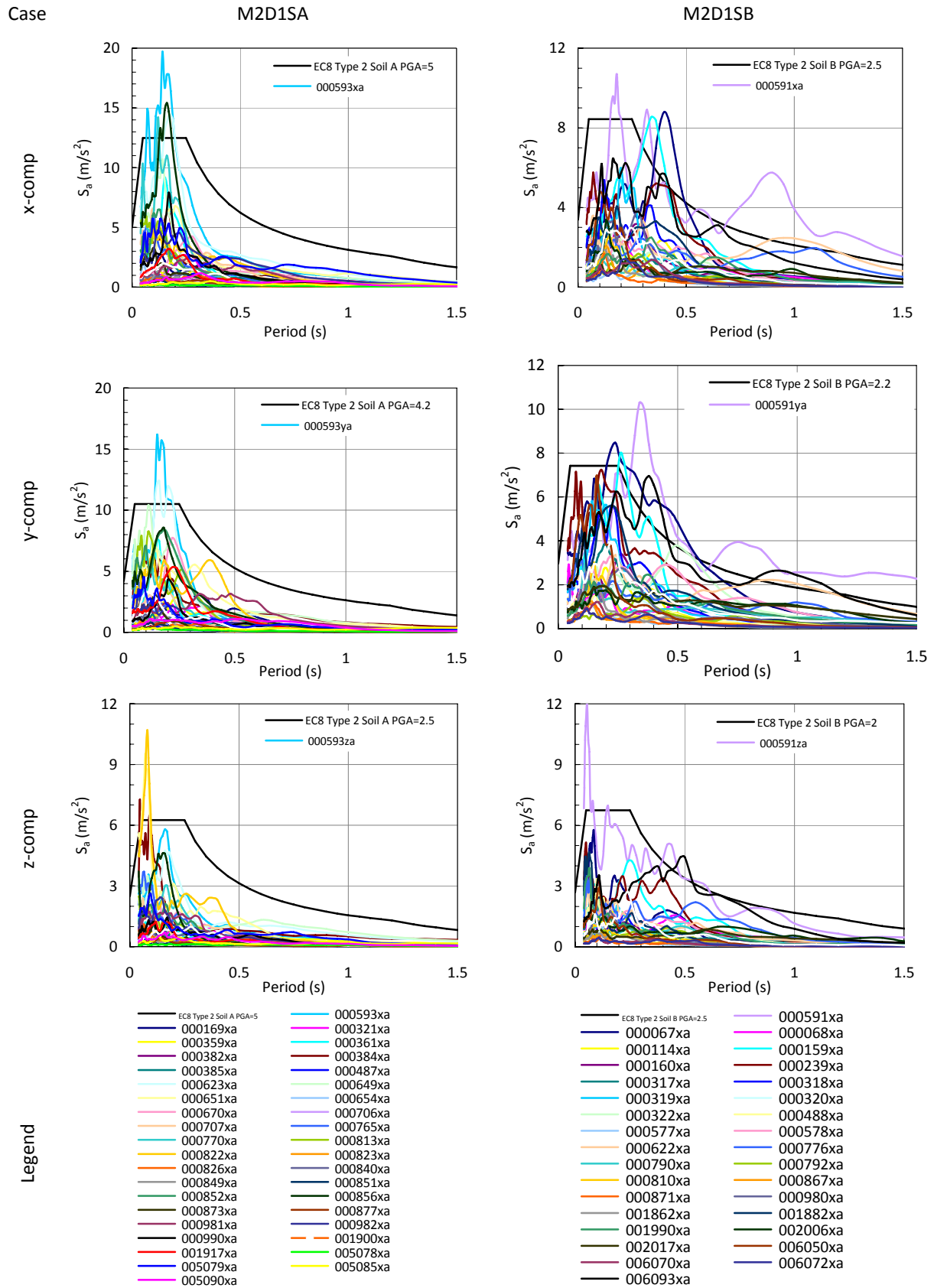


Fig. A2 (a): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

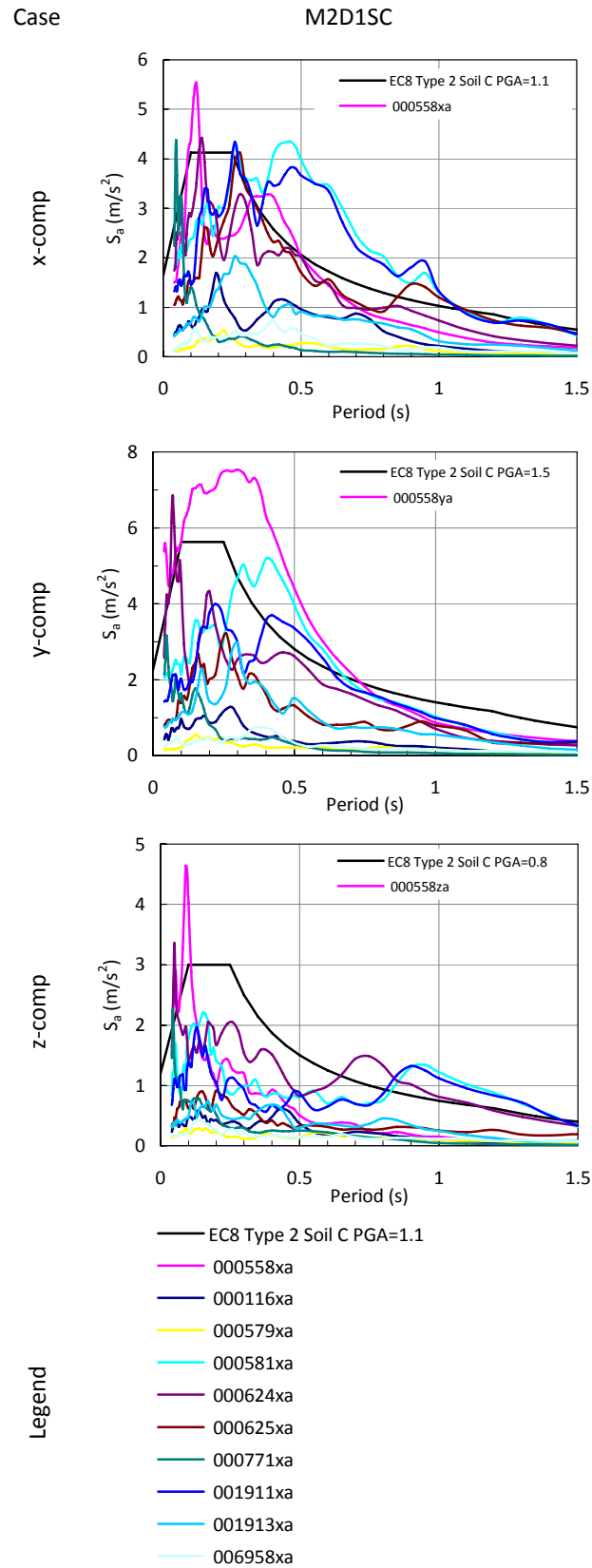


Fig. A2 (a): Acceleration Time History- and Approximated EC8 Response Spectra

Table A2 (b): Earthquake Recording Details for Magnitude Range M2, Distance Range D2

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
26/09/1997	43.031 N	12.862 E	2	4.8	Colfiorito-Casermette	Italy	21	836	A
7/5/1984	41.666 N	13.820 E	22	4.8	Kozani-Prefecture	Greece	22	6124	A
17/07/1995	40.210 N	21.550 E	2	4.8	Nocera	Italy	22	814	A
11/23/1980	40.780 N	15.330 E	26	5	Valsamata-Seismograp	Greece	23	428	A
26/09/1997	43.023 N	12.892 E	2	4.8	Nocera	Italy	23	830	A
4/10/1997	42.933 N	12.936 E	-	5.2	Reykjavik-Heidmork	Iceland	23	5089	A
26/09/1997	43.023 N	12.892 E	6	5.2	Nocera	Italy	24	818	A
26/04/1997	37.181 N	21.385 E	9	4.8	Tolmezzo-Diga-Ambiesta	Italy	27	59	A
7/5/1976	46.245 N	13.269 E	9	5.4	Pietralunga	Italy	27	363	A
4/29/1984	43.204 N	12.585 E	9	5.4	Umbertide	Italy	27	362	A
4/29/1984	43.204 N	12.585 E	-	5.2	Reykjavik-Foldaskoli	Iceland	27	5088	A
9/15/1976	46.291 N	13.153 E	14	5	Poligiros-Prefecture	Greece	28	6154	A
3/5/1995	40.580 N	23.650 E	5	5	Veria-CulturalCenter	Greece	30	5895	A
4/15/1978	38.270 N	14.860 E	2	4.8	Nocera	Italy	34	838	A
26/09/1997	43.031 N	12.862 E	6	4.8	Sellano	Italy	38	858	A
5/4/1998	43.185 N	12.719 E	5	4.9	Andirin-Tufanpassallko	Turkey	40	7083	A
11/23/1980	40.780 N	15.330 E	-	5.2	Reykjavik-Hus-Verslunar	Iceland	32	5087	A
6/10/1997	43.019 N	12.838 E	2	4.8	Nocera	Italy	32	834	A
7/9/1999	38.080 N	23.580 E	3	5.5	Breginj-Fabrika-IGLI	Slov	21	113	B
7/9/1999	38.080 N	23.580 E	5	5	Edessa-Prefecture	Greece	21	5892	B
7/9/1999	38.080 N	23.580 E	7	5.2	Bevagna	Italy	21	620	B
9/7/1984	40.660 N	21.890 E	10	5.1	Amaliada-OTE-Building	Greece	24	559	B
4/15/1979	41.980 N	18.980 E	9	4.8	Kozani-Nurse-School	Greece	27	6110	B
26/03/1993	37.590 N	21.390 E	10	5.1	Preveza-OTE-Building	Greece	28	336	B
7/11/1999	40.697 N	30.717 E	7	4.9	Kozani-Nurse-School	Greece	28	6113	B
15/05/1995	40.080 N	21.650 E	28	5.3	Bavra	Armenia	29	490	B
10/3/1981	39.200 N	20.800 E	5	5.1	Preveza-OTE-Building	Greece	29	582	B
16/12/1990	41.370 N	43.720 E	8	5.3	San-Agapito	Italy	32	381	B
25/02/1994	38.730 N	20.580 E	14	5	Ierissos-PoliceStation	Greece	32	6087	B
15/07/1996	45.939 N	6.089 E	3	5.5	Kobarid-Osn.Skola	Slov	33	112	B
4/5/1995	40.590 N	23.600 E	6	4.8	Bevagna	Italy	33	875	B
8/9/2000	39.395 N	27.667 E	38	5.3	Preveza-OTE-Building	Greece	38	474	B
26/09/1997	43.031 N	12.862 E	7	5.2	Norcia	Italy	33	627	B
3/10/1997	43.033 N	12.838 E	2	4.8	Castelnuovo-Assisi	Italy	37	812	C
4/15/1978	38.270 N	14.860 E	7	5.2	Gubbio-Piana	Italy	38	630	C
4/11/1993	38.340 N	21.910 E	6	4.8	Gubbio-Piana	Italy	21	854	C
2/24/1981	38.099 N	22.842 E	8	5.3	Cassino-SantÆElia	Italy	24	386	C
17/08/1999	40.702 N	29.987 E	10	5.5	Erzincan-Eksisu	Turkey	26	1415	C
11/5/1984	41.732 N	13.921 E	15	5	Pellaro	Italy	33	168	C
2/25/1981	38.135 N	23.050 E	4	4.8	AdapazariBahtiyat	Turkey	35	6942	C
15/03/1992	39.530 N	39.930 E	6	4.9	Gubbio-Piana	Italy	37	783	C
11/3/1978	38.020 N	16.030 E	4	4.8	AdapazariBayindirlikM	Turkey	38	6946	C
26/09/1997	43.031 N	12.862 E	4	4.8	AdapazariA.Babalioglu	Turkey	39	6944	C
31/08/1999	40.756 N	29.935 E	4	4.8	AdapazariKadin-D.Cocuk	Turkey	39	6947	C
7/10/1997	43.025 N	12.865 E	4	4.8	AdapazariToyotasa	Turkey	39	6943	C
17/08/1999	40.702 N	29.987 E	13	4.6	Horta	Portuga	39	7343	C

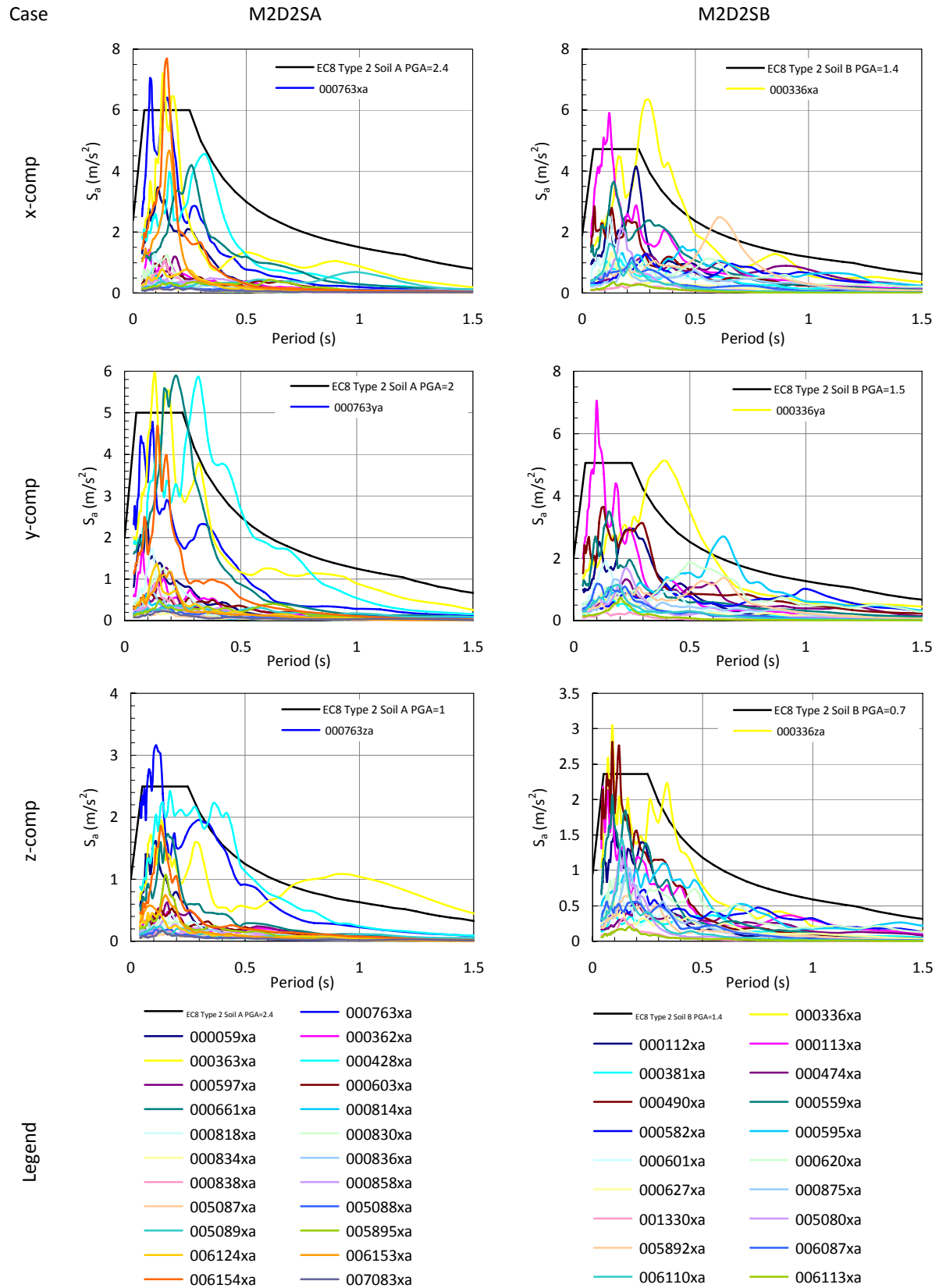


Fig. A2 (b): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

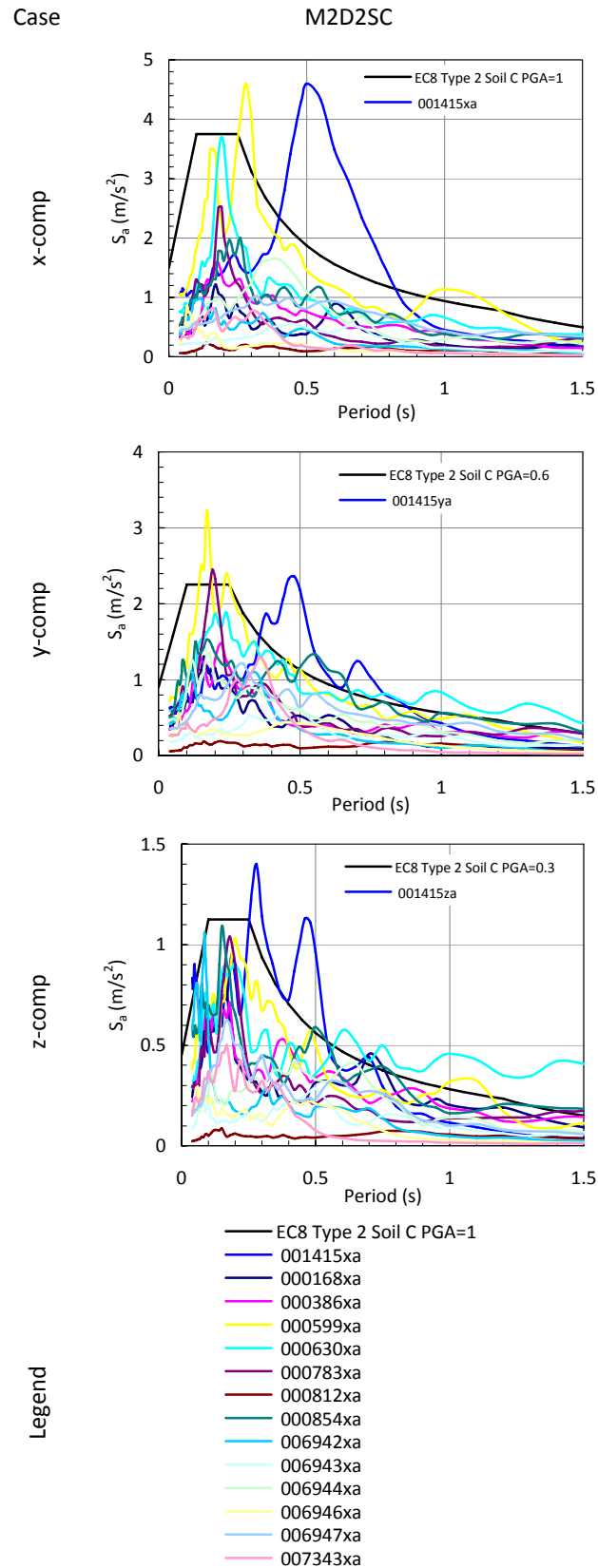


Fig. A2 (b): Acceleration Time History- and Approximated EC8 Response Spectra

Table A2 (c): Earthquake Recording Details for Magnitude Range M2, Distance Range D3

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
25/06/2001	37.238 N	36.206 E	41	4.9	Sellano	Italy	41	847	A
17/06/2000	63.970 N	20.360 W	12	4.9	Poligiros-Prefecture	Greece	42	2027	A
8/26/1983	40.450 N	23.920 E	7	5.2	Gubbio	Italy	42	621	A
17/08/1999	40.702 N	29.987 E	18	5.2	Kavala-Prefecture	Greece	51	2022	A
9/11/1985	41.260 N	23.980 E	28	5.3	Toros	Armenia	51	495	A
4/15/1979	41.980 N	18.980 E	28	5.3	Stepanavan	Armenia	70	494	A
17/06/2000	63.970 N	20.360 W	28	5.3	Spitak-Karadzor	Armenia	77	493	A
1/17/1983	37.960 N	20.260 E	12	4.7	LDEOStationNo.C0375VO	Turkey	131	6433	A
26/09/1997	43.031 N	12.862 E	15	4.9	Yapi-KrediPlazaLevent	Turkey	151	7059	A
11/23/1980	40.780 N	15.330 E	12	4.7	LDEOStationNo.C1061	Turkey	123	6432	B
6/5/1976	46.292 N	13.253 E	14	5	Ag.Basilios-Element	Greece	43	6037	B
11/23/1980	40.780 N	15.330 E	10	5.5	Erzincan-Meteorologi	Turkey	45	538	B
15/03/1992	39.530 N	39.930 E	-	5.2	Hella	Iceland	49	5082	B
12/11/1999	40.768 N	31.148 E	4	4.8	Gebze-Arcelik	Turkey	49	6948	B
4/15/1979	41.980 N	18.980 E	15	5.1	Izmir-Meteoroloji-Istas	Turkey	65	1776	B
3/23/1983	38.220 N	20.410 E	5	4.9	Kahramanmaras-Bayindir	Turkey	72	7087	B
6/8/1983	40.080 N	24.810 E	7	5.3	Kernitsa	Greece	99	6841	B
20/06/1990	36.960 N	49.410 E	12	4.7	Botas-GasTerminal	Turkey	106	6930	B
1/10/1995	38.120 N	30.110 E	12	4.7	LDEOStationNo.C1060BU	Turkey	108	6431	B
16/09/2001	37.260 N	21.950 E	15	4.9	Gebze-Arcelik	Turkey	116	7049	B
15/07/1996	45.939 N	6.089 E	10	4.6	Angr-do-Heroismo, GZCAH	Portuga	131	7281	B
6/10/1997	43.019 N	12.838 E	15	4.9	Cekmece-Kucuk	Turkey	170	7053	B
12/11/1999	40.768 N	31.148 E	15	4.9	Istanbul-K.M.Pasa	Turkey	155	7058	C
31/08/1999	40.756 N	29.935 E	12	4.7	Bursa-TofaFabrikasi	Turkey	41	6931	C
31/08/1999	40.756 N	29.935 E	15	4.9	Duzce-MeteorolojiMudurl	Turkey	41	4477	C
1/8/2000	38.758 N	28.973 W	10	5.1	Lefkada-OTE-Building	Greece	42	337	C
26/09/1997	43.023 N	12.892 E	9	5.4	Citta-di-Castello-Regn	Italy	43	360	C
19/08/1999	40.629 N	29.137 E	28	5.3	Bakuriani	Georgia	44	489	C
23/08/2000	40.680 N	30.720 E	12	4.7	Fatih-Tomb	Turkey	46	6933	C
10/3/1981	39.200 N	20.800 E	5	5	Lefkada-OTE-Building	Greece	48	566	C
4/29/1984	43.204 N	12.585 E	10	4.6	Met	Turkey	49	7047	C
16/12/1990	41.370 N	43.720 E	4	4.8	AkyaziGebesKoylmamLojma	Turkey	51	6945	C
19/08/1999	40.629 N	29.137 E	10	4.6	Ballica	Turkey	55	7044	C
17/08/1999	40.702 N	29.987 E	15	4.9	Fatih-Tomb	Turkey	154	7055	C
23/08/2000	40.680 N	30.720 E	15	4.9	Ambarli-Termik	Turkey	174	7050	D
6/10/1997	43.019 N	12.838 E	7	5.2	Rieti	Italy	65	631	D

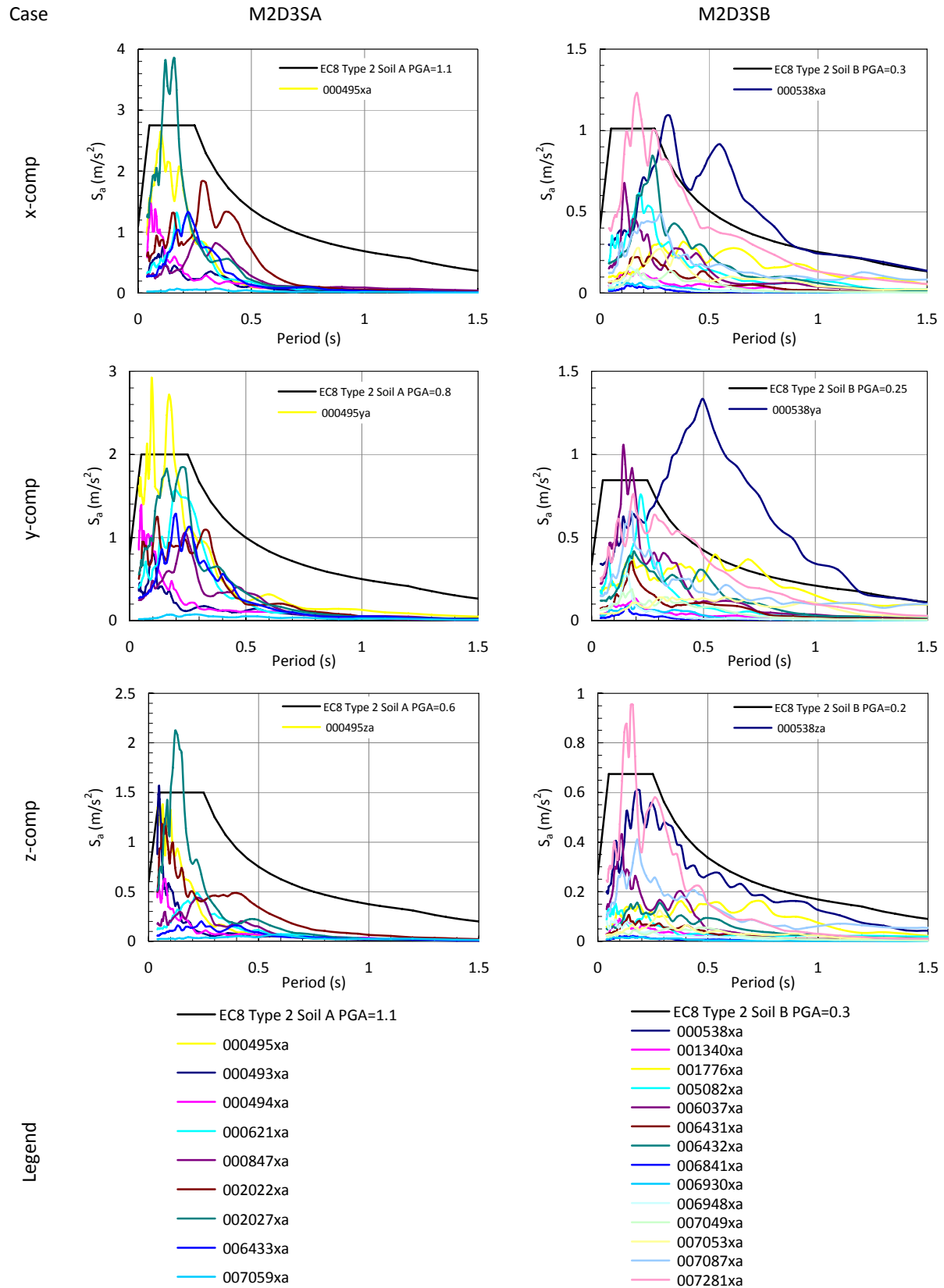


Fig. A2 (c): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

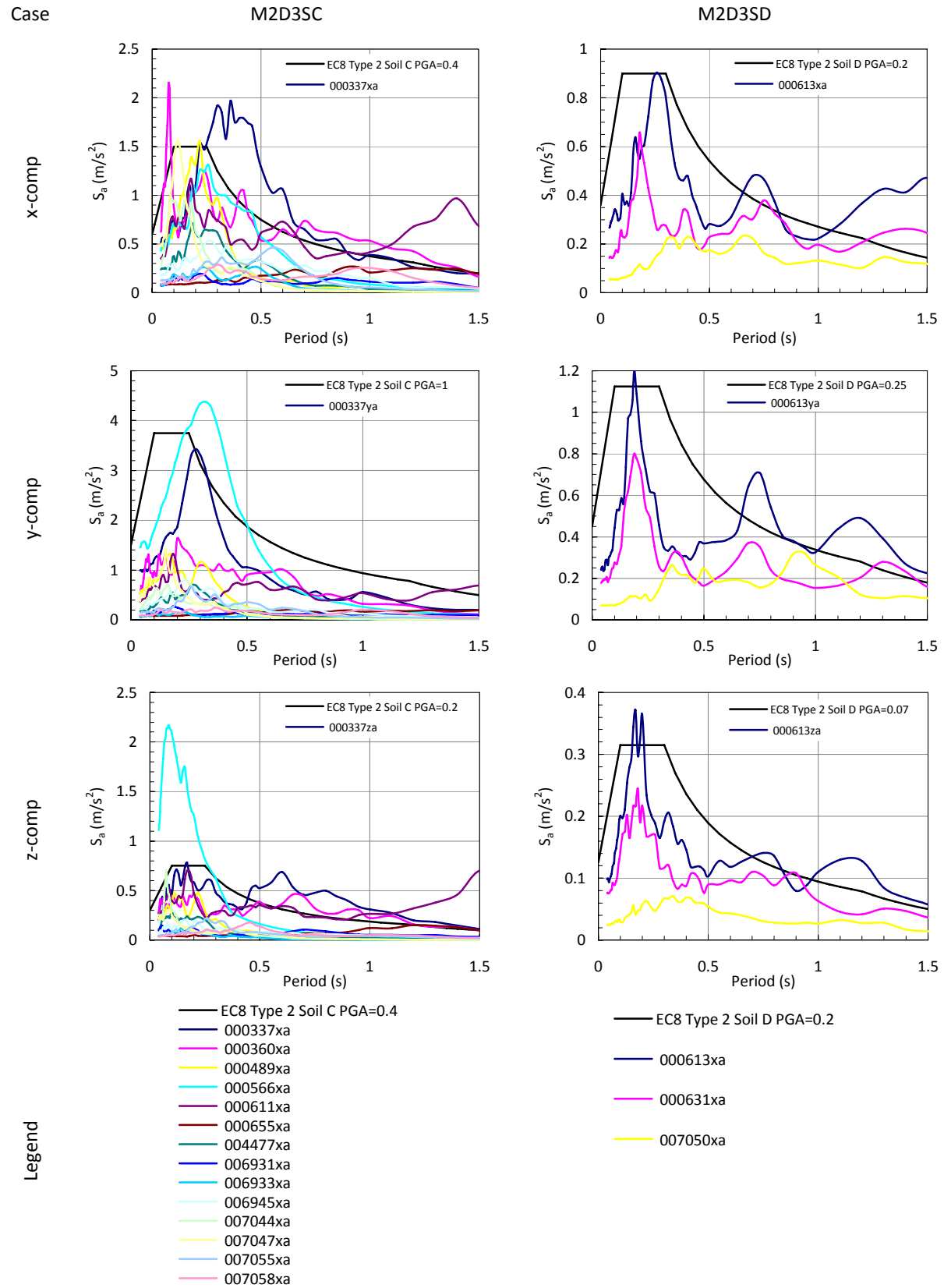


Fig. A2 (c): Acceleration Time History- and Approximated EC8 Response Spectra

Table A3 (a): Earthquake Recording Details for Magnitude Range M3, Distance Range D1

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
7/10/1997	43.025 N	12.865 E	4	5.8	Cascia	Italy	5	242	A
9/19/1979	42.760 N	13.020 E	11	5.8	Atina	Italy	5	365	A
13/11/1998	63.954 N	21.346 W	8	6	Tarcento	Italy	12	149	A
5/4/1998	43.185 N	12.719 E	6	5.9	Nocera	Italy	11	594	A
6/10/1997	43.019 N	12.838 E	8	6	Forgaria-Cornio	Italy	14	146	B
7/9/1999	38.080 N	23.580 E	12	5.52	San-Rocco	Italy	15	120	B
11/8/1980	39.300 N	22.830 E	12	5.55	Forgaria-Cornio	Italy	15	123	B
1/16/1981	40.850 N	15.280 E	6	5.9	Colfiorito	Italy	5	592	B
29/09/1999	40.738 N	29.331 E	7	5.55	Colfiorito	Italy	3	591	B
9/6/1976	46.260 N	13.010 E	1	5.8	Kalamata-Prefecture	Greece	10	413	B
15/05/1995	40.080 N	21.650 E	1	5.8	Kalamata-OTE-Building	Greece	11	414	B
12/11/1999	40.768 N	31.148 E	5	6.1	Forgaria-Cornio	Italy	14	134	B
7/11/1999	40.697 N	30.717 E	5	6.1	San-Rocco	Italy	14	131	B
21/06/2000	63.970 N	20.710 W	8	6	San-Rocco	Italy	14	147	B
9/15/1976	46.291 N	13.153 E	17	5.9	Athens-Sepolia(Metro	Greece	14	1714	B
6/17/1976	46.180 N	12.800 E	17	5.9	Athens	Greece	16	1313	B
4/15/1979	41.980 N	18.980 E	17	5.9	Athens	Greece	17	1314	B
19/05/1995	40.090 N	21.600 E	17	5.9	Athens-Syntagma	Greece	18	1713	B
4/11/1993	38.340 N	21.910 E	17	5.9	Athens-Sygrou-Fix	Greece	19	1710	B
9/11/1985	41.260 N	23.980 E	17	5.9	Athens	Greece	20	1312	B
15/05/1995	40.080 N	21.650 E	17	5.9	Athens-Papagos	Greece	20	1711	B
28/06/1997	38.337 N	26.838 W	17	5.9	Athens-Sepolia(Garage)	Greece	14	1715	B
4/3/2000	38.344 N	26.684 W	15	5.8	Naso	Italy	18	171	B
17/08/1999	40.702 N	29.987 E	15	5.8	Patti-Cabina-Prima	Italy	18	170	C
17/08/1999	40.702 N	29.987 E	17	5.9	Athens-NeoPsihiko	Greece	19	1708	C
15/06/1991	42.465 N	44.030 E	7	5.8	Lefkada-OTE-Building	Greece	15	42	C
17/08/1999	40.702 N	29.987 E	12	5.55	Buia	Italy	9	122	C
23/08/2000	40.680 N	30.720 E	5	6.1	Buia	Italy	9	133	C
7/11/1999	40.697 N	30.717 E	8	6	Buia	Italy	11	151	C
29/08/1999	40.798 N	31.124 E	11	5.8	Cassino-Sant'E-Elia	Italy	16	378	C

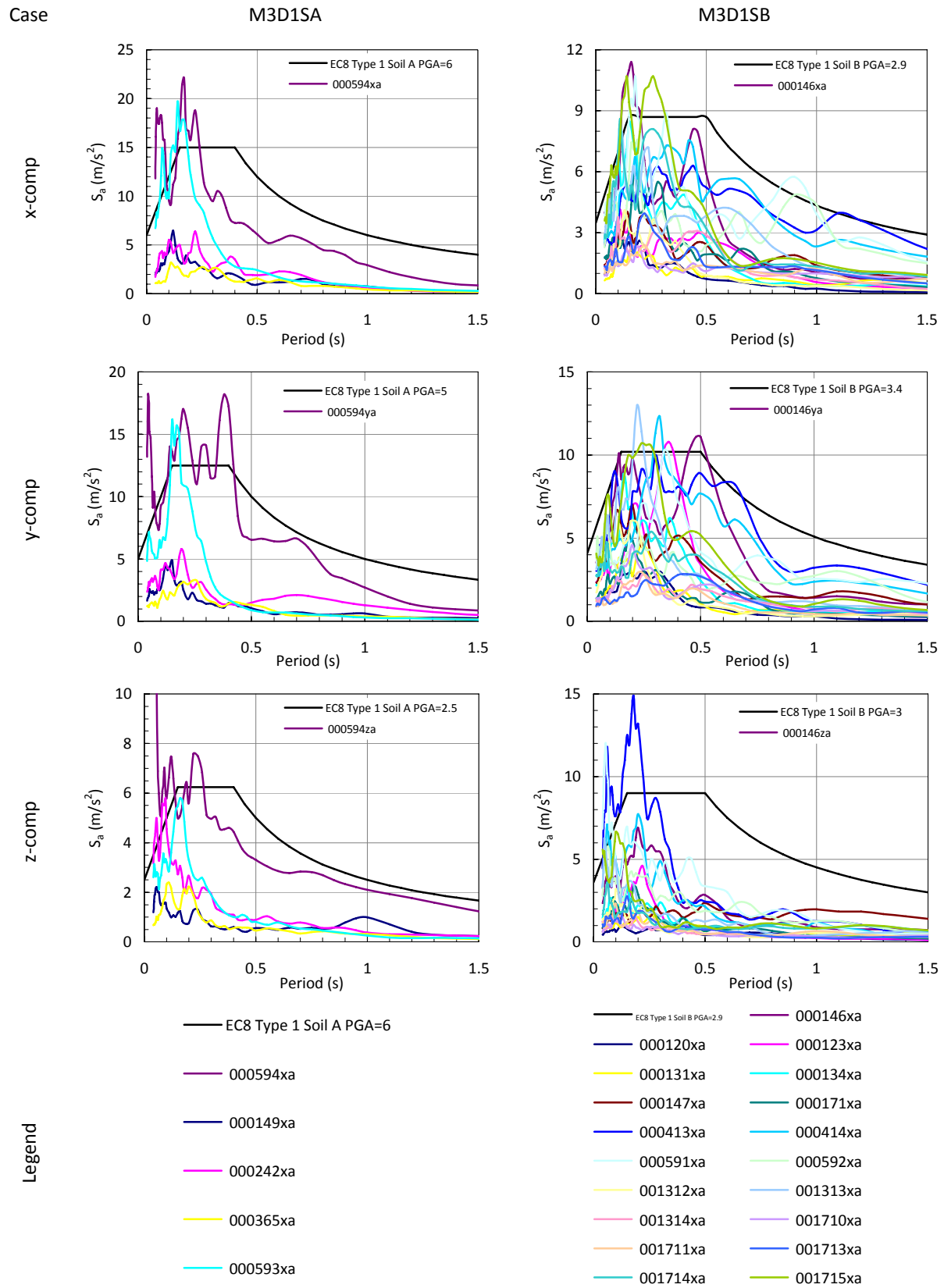


Fig. A3 (a): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

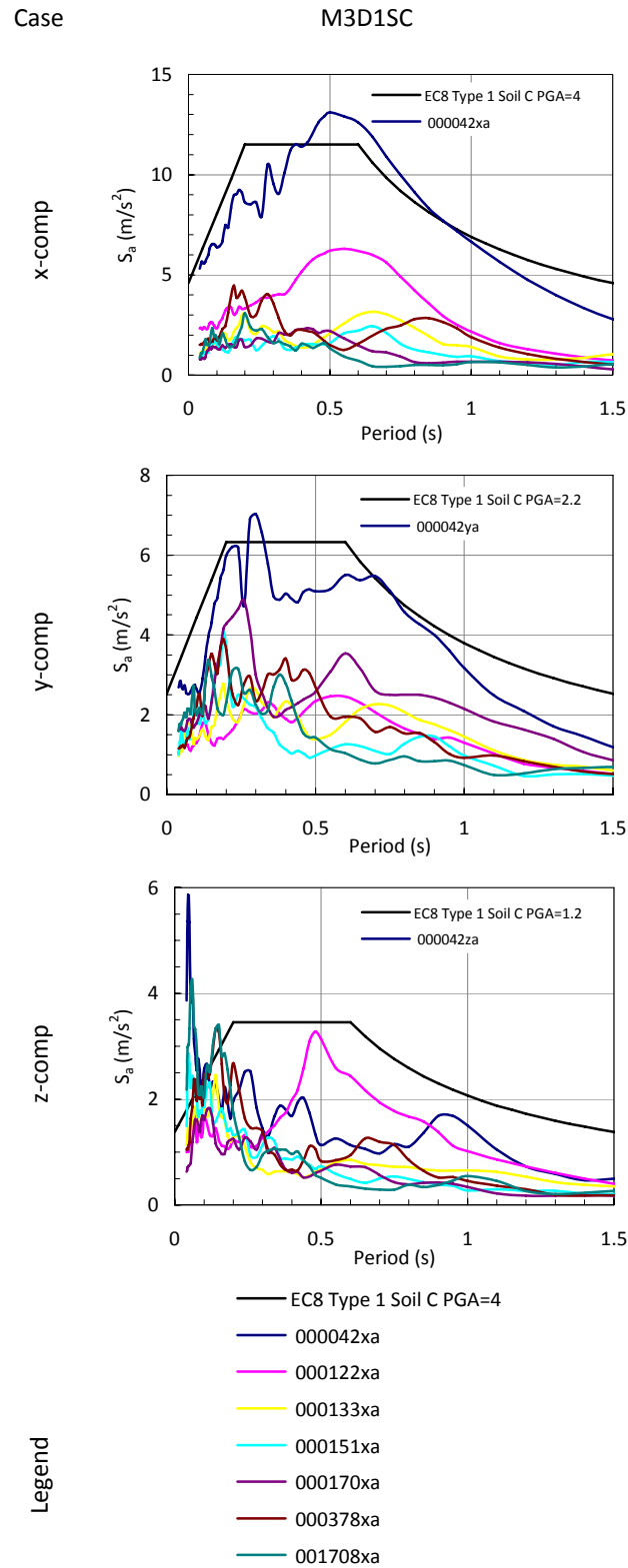


Fig. A3 (a): Acceleration Time History- and Approximated EC8 Response Spectra

Table A3 (b): Earthquake Recording Details for Magnitude Range M3, Distance Range D2

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
21/06/2000	63.970 N	20.710 W	6	5.9	Assisi-Stallone	Italy	21	665	A
9/11/1997	42.854 N	13.000 E	11	5.8	Ponte-Corvo	Italy	22	368	A
5/18/1988	38.350 N	20.470 E	7	5.55	Borgo-Cerreto-Torre	Italy	23	763	A
12/11/1999	40.768 N	31.148 E	7	5.55	Assisi-Stallone	Italy	24	661	A
4/6/1998	64.036 N	21.293 W	5	6.1	Robic	Slov	28	128	A
21/06/2000	63.970 N	20.710 W	4	5.9	Golbasi-DevletHastanesi	Turkey	29	410	A
17/06/2000	63.970 N	20.360 W	15	5.8	Milazzo	Italy	34	173	A
17/06/2000	63.970 N	20.360 W	7	5.55	Cascia	Italy	35	603	A
17/06/2000	63.970 N	20.360 W	6	5.9	Cascia	Italy	37	604	A
21/06/2000	63.970 N	20.710 W	7	5.55	Monte	Italy	24	597	A
26/09/1997	43.023 N	12.892 E	6	5.9	Borgo-Cerreto-Torre	Italy	25	764	A
26/09/1997	43.031 N	12.862 E	6	5.9	Monte	Italy	27	598	A
26/09/1997	43.031 N	12.862 E	8	6	Robic	Slov	32	140	A
12/10/1997	42.910 N	12.946 E	7	5.55	Bevagna	Italy	25	595	B
3/4/1998	43.200 N	12.790 E	6	5.9	Matelica	Italy	27	602	B
4/15/1979	41.980 N	18.980 E	8	6	Kobarid-Osn.Skola	Slov	37	138	B
26/09/1997	43.023 N	12.892 E		6.5	Kaldarholt	Iceland	40	5080	B
5/18/1988	38.350 N	20.470 E	5	6.1	Breginj-Fabrika-IGLI	Slov	21	126	B
11/9/1976	46.286 N	13.160 E	4	5.8	Spoletto	Italy	23	247	B
6/10/1997	43.019 N	12.838 E	8	6	Breginj-Fabrika-IGLI	Slov	25	139	B
9/15/1976	46.318 N	13.119 E	8	5.6	Brienza	Italy	28	947	B
19/05/1995	40.090 N	21.600 E	12	5.55	Kobarid-Osn.Skola	Slov	32	117	B
11/9/1976	46.277 N	13.175 E	5	6.1	Kobarid-Osn.Skola	Slov	33	127	B
9/15/1976	46.291 N	13.153 E	11	5.8	San-Agapito	Italy	36	366	B
10/30/1983	40.352 N	42.180 E	1	6.1	Edessa-Prefecture	Greece	36	1875	B
3/4/1998	43.200 N	12.790 E	1	6.1	Kilkis-Hospital	Greece	37	1887	B
7/5/1984	41.666 N	13.820 E	10	6.5	Zakynthos-OTE-Building	Greece	38	5850	B
21/12/1990	40.950 N	22.430 E	4	5.8	Bevagna	Italy	39	244	B
21/12/1990	40.950 N	22.430 E	6	6.2	Iri	Georgia	40	530	B
25/02/1994	38.730 N	20.580 E	6	5.9	Bevagna	Italy	23	596	B
29/11/1994	38.750 N	20.530 E	7	5.55	Matelica	Italy	27	601	B
11/9/1976	46.286 N	13.160 E	6	5.9	Castelnuovo-Assisi	Italy	22	600	C
25/02/1994	38.730 N	20.580 E	7	5.55	Castelnuovo-Assisi	Italy	25	599	C
6/10/1997	43.019 N	12.838 E	8	6.4	Korinthos-OTE-Building	Greece	25	335	C
3/4/1998	43.200 N	12.790 E	6	6.4	Thessaloniki-City-Hotel	Greece	29	175	C
6/20/1978	40.729 N	23.254 E	6	5.9	Gubbio-Piana	Italy	38	612	C

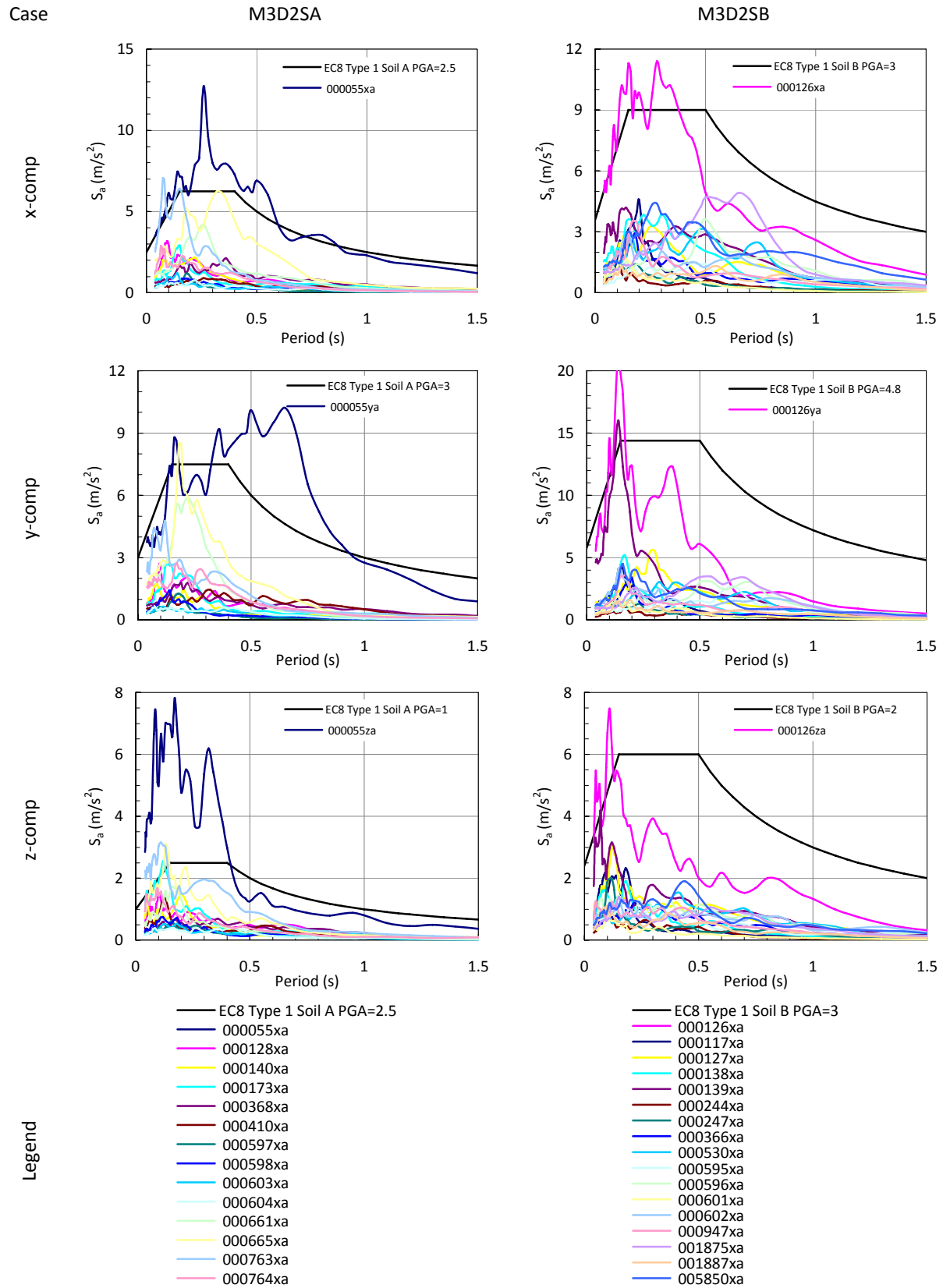


Fig. A3 (b): Acceleration Time History- and Approximated EC8 Response Spectra

(Cont'd)

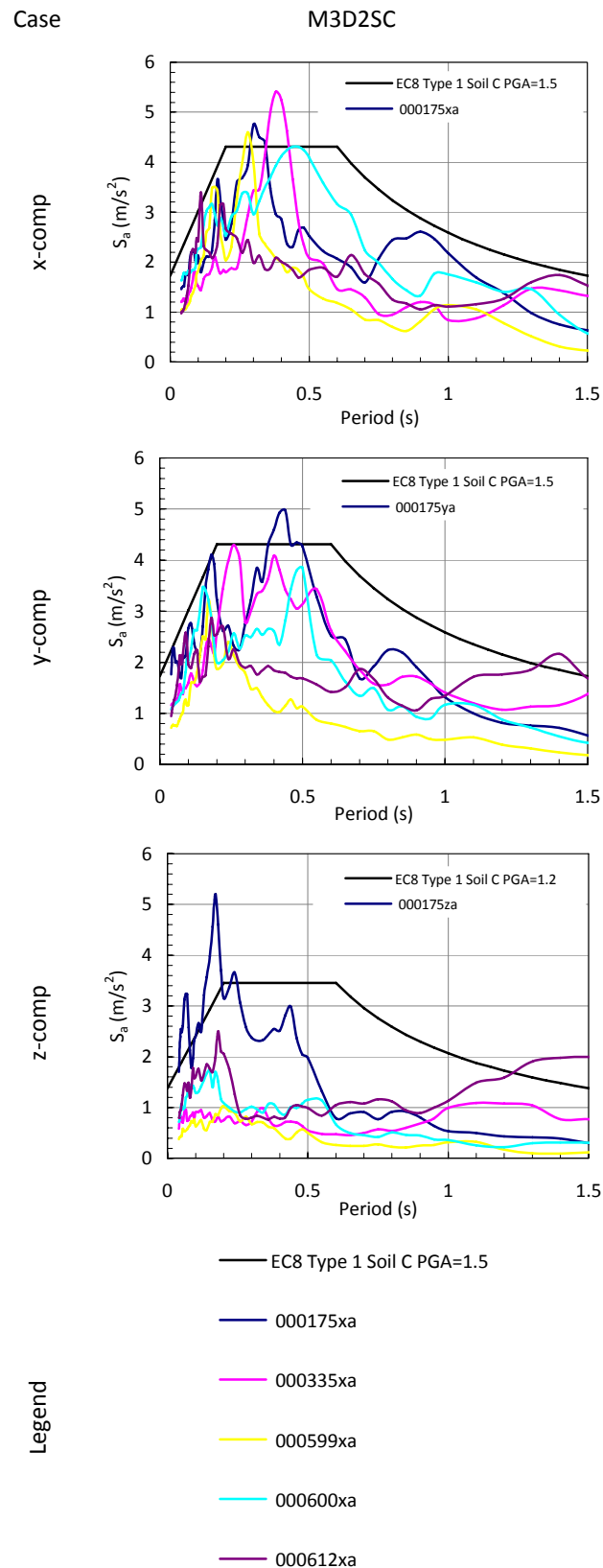


Fig. A3 (b): Acceleration Time History- and Approximated EC8 Response Spectra

Table A3 (c): Earthquake Recording Details for Magnitude Range M3, Distance Range D3

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
21/06/2000	63.970 N	20.710 W	6	5.9	Pietralunga	Italy	55	615	A
21/06/2000	63.970 N	20.710 W	15	5.8	Messina-1	Italy	58	172	A
4/15/1978	38.270 N	14.860 E	11	5.8	Bussi	Italy	58	364	A
21/06/2000	63.970 N	20.710 W	8	5.7	Cerknica	Slov	88	6237	A
12/4/1998	46.309 N	13.632 E	6	5.9	Pennabilli	Italy	100	618	A
26/09/1997	43.031 N	12.862 E	8	5.7	Sleme	Slov	104	6239	A
12/4/1998	46.309 N	13.632 E	7	6.5	Feltre	Italy	108	52	A
6/5/1976	46.292 N	13.253 E	10	6.5	Koroni-Town-Hall	Greece	136	5820	A
18/11/1997	37.482 N	20.692 E	7	6.5	Asiago	Italy	146	46	A
21/06/2000	63.970 N	20.710 W	7	6.5	Tregnago	Italy	185	56	A
9/11/1997	42.854 N	13.000 E	10	6.5	Kyparrisia-Agricultu	Greece	90	5826	A
12/11/1999	40.768 N	31.148 E	11	5.8	Roccamonfina	Italy	44	369	A
12/11/1999	40.768 N	31.148 E	6	5.9	Peglio	Italy	79	617	A
18/11/1997	37.482 N	20.692 E	10	6.3	Patra-SanAlexios-Church	Greece	42	6144	B
15/06/1995	38.362 N	22.200 E	10	6.3	Patra-SanDimitriosChurc	Greece	43	6142	B
31/08/1999	40.756 N	29.935 E	6	6.2	Zemo-Bari	Georgia	60	533	B
17/08/1999	40.702 N	29.987 E	3	6.1	Zakynthos-OTE-Building	Greece	65	2030	B
17/08/1999	40.702 N	29.987 E	10	6	Denizli-Bayindirlik	Turkey	94	1723	B
19/08/1999	40.629 N	29.137 E	10	6.5	Kalamata-OTE-Building	Greece	134	5816	B
12/11/1999	40.768 N	31.148 E	6	6.2	Oni	Georgia	50	532	B
26/09/1997	43.023 N	12.892 E	6	6.2	Oni-Base-Camp	Georgia	50	531	C
31/08/1999	40.756 N	29.935 E	7	5.55	Gubbio-Piana	Italy	41	611	C
14/02/2000	40.980 N	31.700 E	6	6.2	Ambrolauri	Georgia	72	529	C
20/06/1990	36.960 N	49.410 E	7	5.55	Borgo-Ottomila2	Italy	126	655	C
17/08/1999	40.702 N	29.987 E	6	5.9	Borgo-Ottomila2	Italy	128	656	C
7/7/2000	40.800 N	29.260 E	11	5.8	Garigliano-Centrale-	Italy	45	375	D
26/09/1997	43.031 N	12.862 E	6	5.9	Rieti	Italy	67	614	D
7/5/1984	41.666 N	13.820 E	11	5.8	Garigliano-Centrale-	Italy	45	374	D
26/09/1997	43.023 N	12.892 E	7	5.55	Rieti	Italy	66	613	D

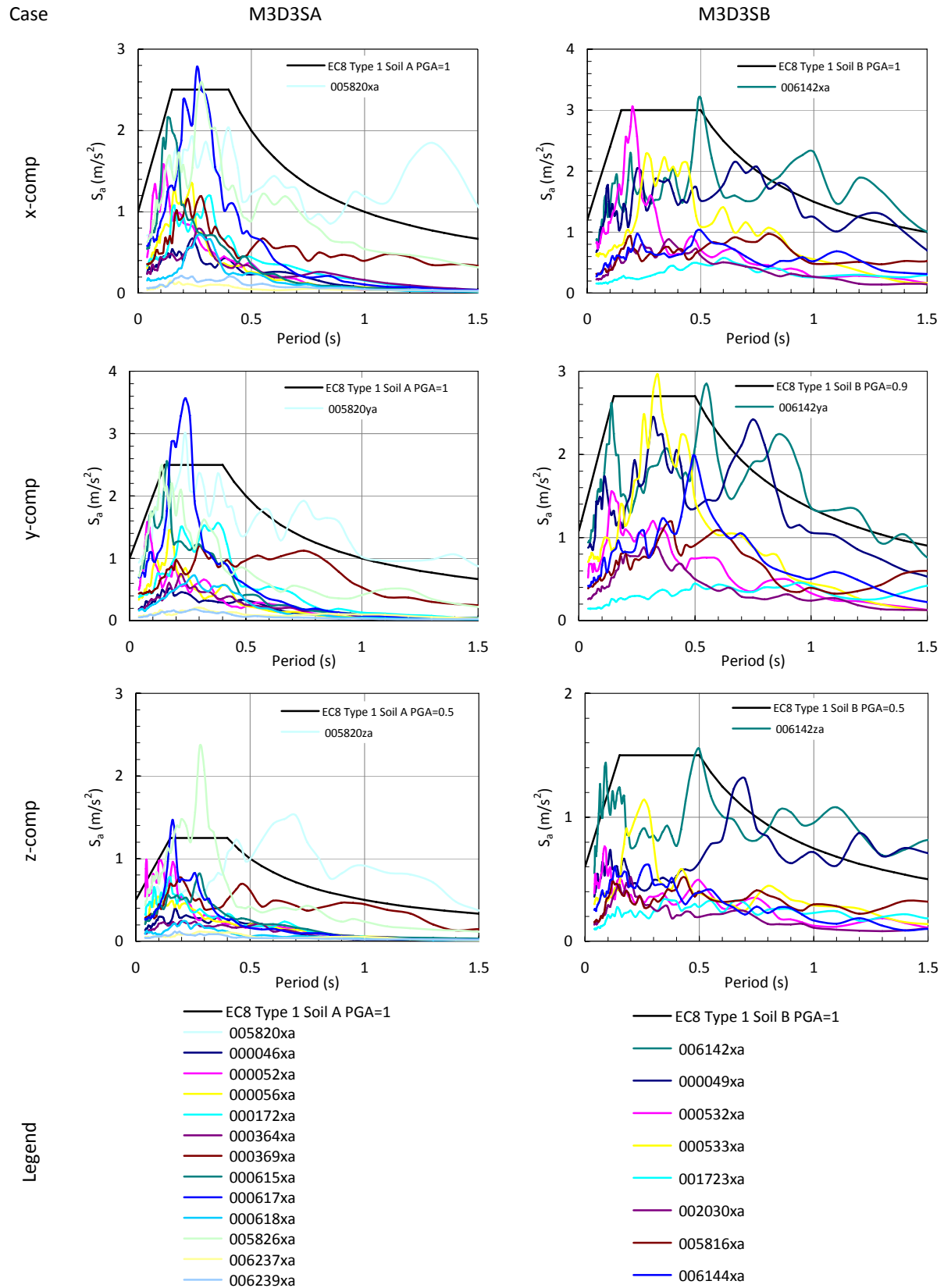


Fig. A3 (c): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

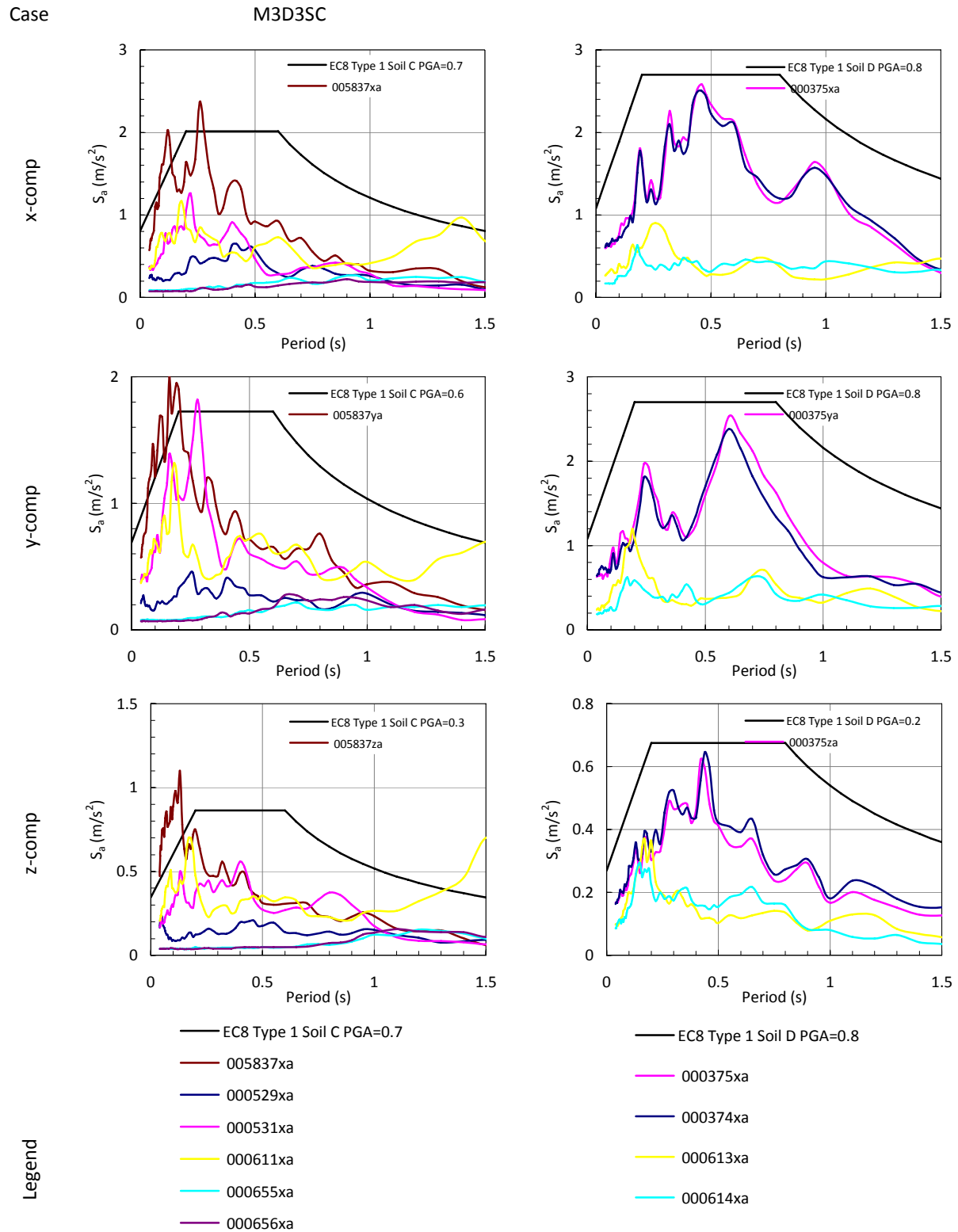


Fig. A3 (c): Acceleration Time History- and Approximated EC8 Response Spectra

Table A4 (a): Earthquake Recording Details for Magnitude Range M4, Distance Range D1

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
7/5/1984	41.666 N	13.820 E	15	6.6	Flagbjarnarholt	Iceland	5	4674	A
6/17/2000	63.970 N	20.360 W	-	6.6	Thjorsarbru	Iceland	5	6349	A
4/6/1998	64.036 N	21.293 W	-	6.6	Thjorsartun	Iceland	6	6332	A
13/11/1998	63.954 N	21.346 W	17	7.8	Izmit-Meteorolojistasy	Turkey	9	1231	A
6/10/1997	43.019 N	12.838 E	4	7.4	Dayhook	Iran	12	182	A
11/5/1976	46.258 N	12.985 E	-	6.6	Selfoss-Hospital	Iceland	14	6326	A
4/6/1998	64.036 N	21.293 W	-	6.6	Selfoss-City-Hall	Iceland	15	6335	A
10/6/1987	37.240 N	21.480 E	14	6.55	Kozani-Prefecture	Greece	17	6115	A
21/03/1998	42.958 N	12.879 E	-	6.6	Ljosafoss-Hydroelectr	Iceland	20	6342	A
16/10/1997	43.034 N	12.891 E	15	6.6	Minni-Nupur	Iceland	13	4675	A
7/11/1999	40.697 N	30.717 E	15	6.6	Thjorsarbru	Iceland	15	6277	A
16/12/1990	41.370 N	43.720 E	-	6.6	Irafoss-Hydroelectric	Iceland	20	6341	A
4/3/1977	45.830 N	26.720 E	-	6.6	Solheimar	Iceland	11	6334	B
7/11/1999	40.697 N	30.717 E	15	6.6	Hella	Iceland	15	4673	B
9/15/1976	46.318 N	13.119 E	15	6.6	Solheimar	Iceland	17	4676	B
9/16/1977	46.280 N	12.980 E	15	6.6	Kaldarholt	Iceland	7	6263	B
11/5/1976	46.258 N	12.985 E	14	7.3	LDEOStationNo.C1058BV	Turkey	11	6494	B
4/11/1993	38.340 N	21.910 E	-	6.6	Kaldarholt	Iceland	12	6328	B
9/13/1986	37.100 N	22.190 E	10	6.9	Erzincan-Meteorologi	Turkey	13	535	B
11/9/1976	46.277 N	13.175 E	12	7	Bar-Skupstina-Opstine	Serb	16	199	B
9-Nov-76	46.277 N	13.175 E	16	6.9	Calitri	Italy	16	291	B
9/15/1976	46.318 N	13.119 E	10	6.7	Xilokastro-OTE-Building	Greece	19	334	C
7/5/1984	41.666 N	13.820 E	10	6.7	Korinthos-OTE-Building	Greece	20	333	C
2/24/1981	38.099 N	22.842 E	17	7.8	Yarimca-Petkim	Turkey	20	1257	C
5/17/1976	40.280 N	63.380 E	30	7.1	Karakyr Point	Uzbkstn	11	74	D

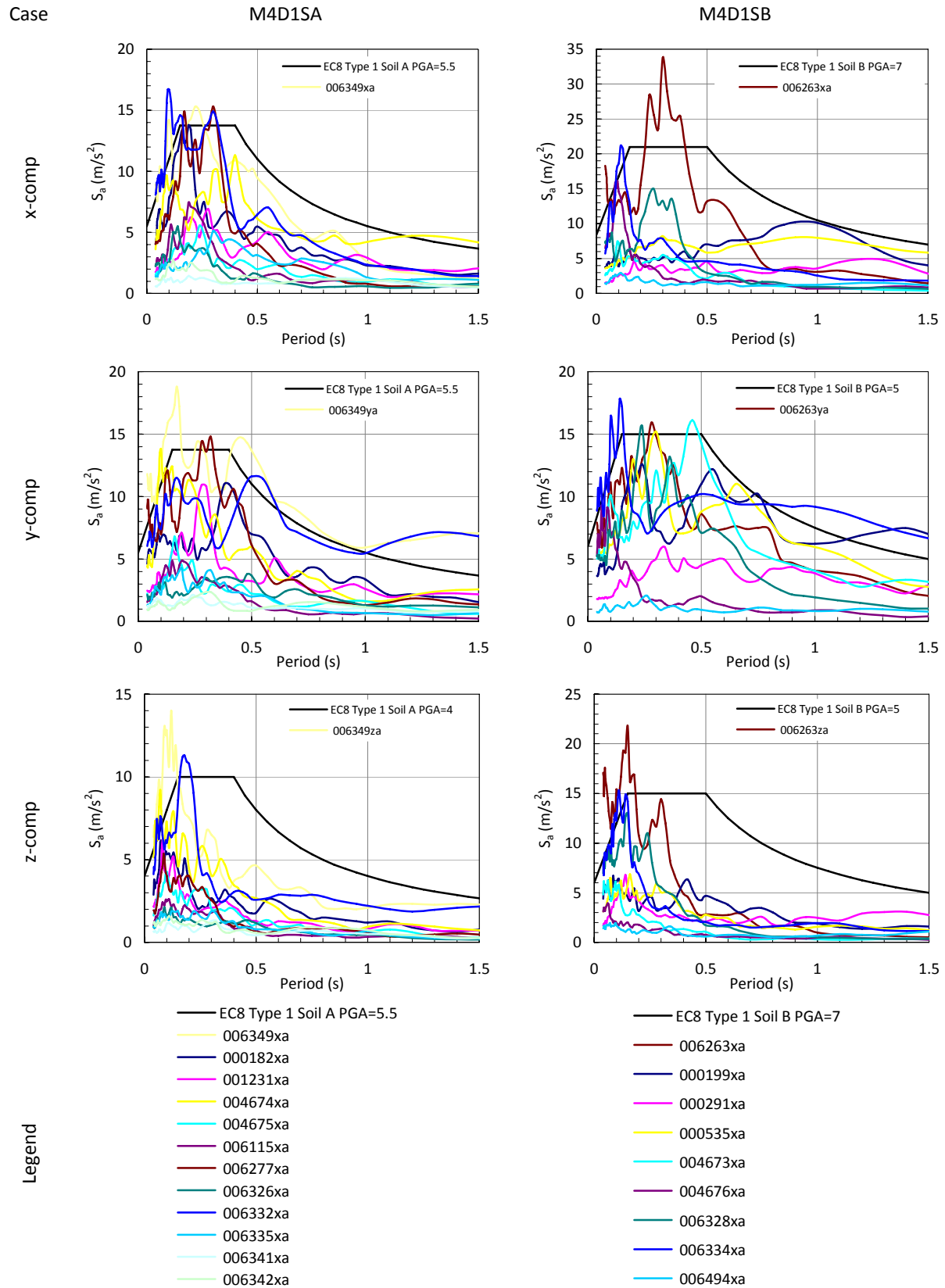


Fig. A4 (a): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

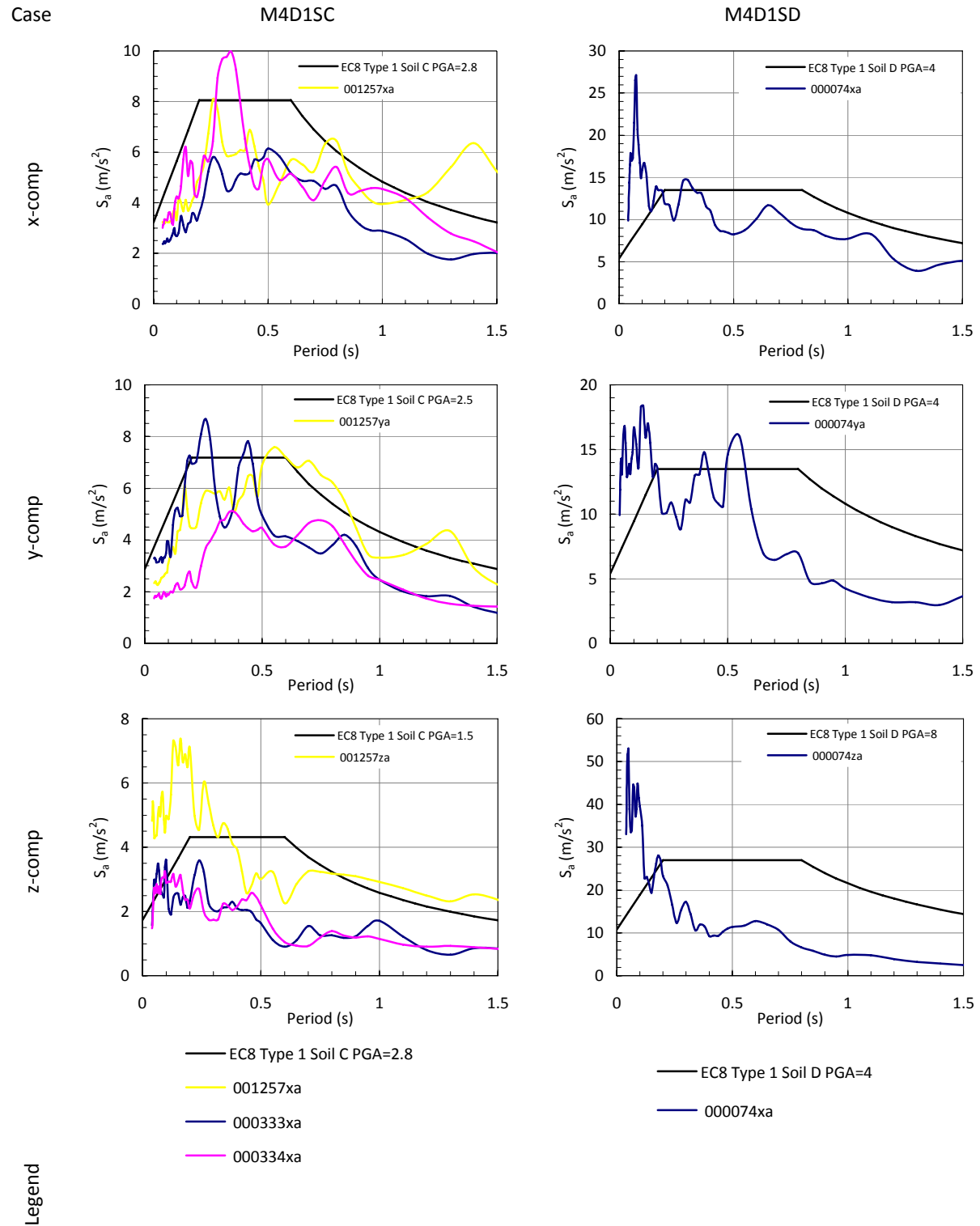


Fig. A4 (a): Acceleration Time History- and Approximated EC8 Response Spectra

Table A4 (b): Earthquake Recording Details for Magnitude Range M4, Distance Range D2

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
4/10/1997	42.933 N	12.936 E	7	6.501	Tolmezzo-Diga-Ambiesta	Italy	23	55	A
6/5/1976	46.292 N	13.253 E	16	6.9	Bagnoli-Irpino	Italy	23	287	A
4/6/1998	64.036 N	21.293 W	14	7.3	LDEOStationNo.C0375VO	Turkey	23	6500	A
21/10/1997	38.971 N	22.073 E	-	6.6	Hveragerdi-Church	Iceland	24	6327	A
21/06/2000	63.970 N	20.710 W	-	6.6	Hveragerdi-Retirement	Iceland	24	6336	A
21/06/2000	63.970 N	20.710 W	16	6.9	Auletta	Italy	25	292	A
11/23/1980	40.780 N	15.330 E	16	6.9	Bisaccia	Italy	26	294	A
4/5/1995	40.590 N	23.600 E	-	6.6	Minni-Nupur	Iceland	28	6333	A
9/7/1984	40.660 N	21.890 E	15	6.6	Selfoss-Hospital	Iceland	31	6262	A
17/06/2000	63.970 N	20.360 W	15	6.6	Selfoss-City-Hall	Iceland	32	4678	A
13/11/1998	63.954 N	21.346 W	15	6.6	Irafoss-Hydroelectric	Iceland	34	6269	A
26/09/1997	43.023 N	12.892 E	15	6.6	Ljosafoss-Hydroelectr	Iceland	35	6270	A
21/06/2000	63.970 N	20.710 W	12	7	Ulcinj-Hotel-Albatros	Serb	21	198	A
4/15/1979	41.980 N	18.980 E	-	6.6	Flagbjarnarholt	Iceland	22	6331	A
9/15/1976	46.318 N	13.119 E	16	6.9	Sturno	Italy	32	290	A
17/06/2000	63.970 N	20.360 W	-	6.6	Hella	Iceland	21	6330	B
17/06/2000	63.970 N	20.360 W	12	7	Petrovac-Hotel-Oliva	Serb	25	196	B
21/06/2000	63.970 N	20.710 W	16	6.9	Rionero-in-Vulture	Italy	33	293	B
9/15/1976	46.291 N	13.153 E	12	7	Ulcinj-Hotel-Olimpic	Serb	24	197	B
11/5/1984	41.732 N	13.921 E	16	6.7	Horasan-Meteoroloji-Mu	Turkey	33	354	B
7/9/1999	38.080 N	23.580 E	14	7.3	LDEOStationNo.D0531WF	Turkey	26	6501	B
4/15/1978	38.270 N	14.860 E	14	7.3	LDEOStationNo.C1061	Turkey	31	6497	B
29/11/1994	38.750 N	20.530 E	14	7.3	LDEOStationNo.C1062FI	Turkey	28	6498	C
7/9/1999	38.080 N	23.580 E	14	7.3	Bolu-Bayindirlik	Turkey	39	1560	C
3/10/1997	43.033 N	12.838 E	17	7.8	Iznik-KarayollariSefli	Turkey	39	1230	C

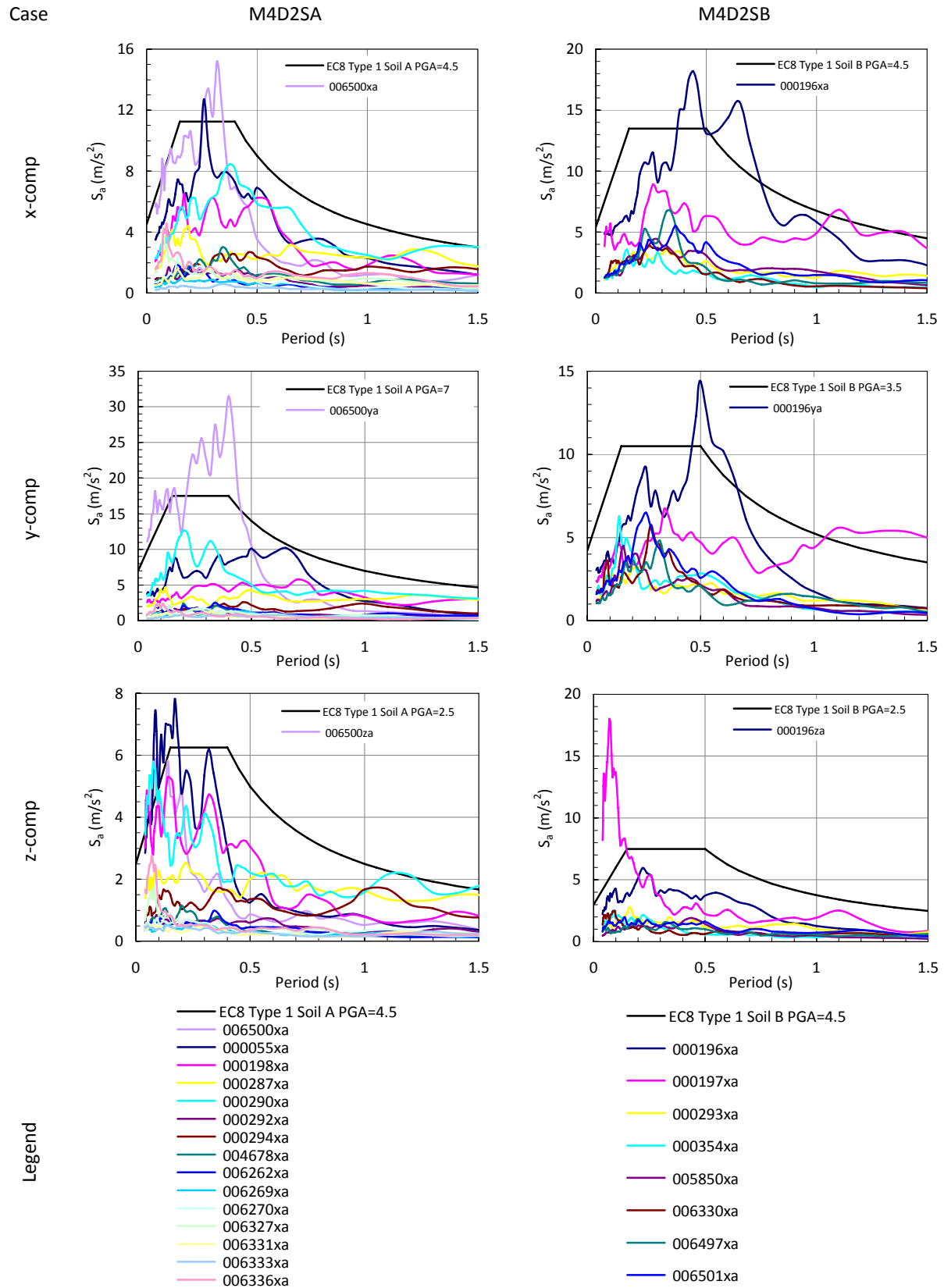


Fig. A4 (b): Acceleration Time History- and Approximated EC8 Response Spectra

(Cont'd)

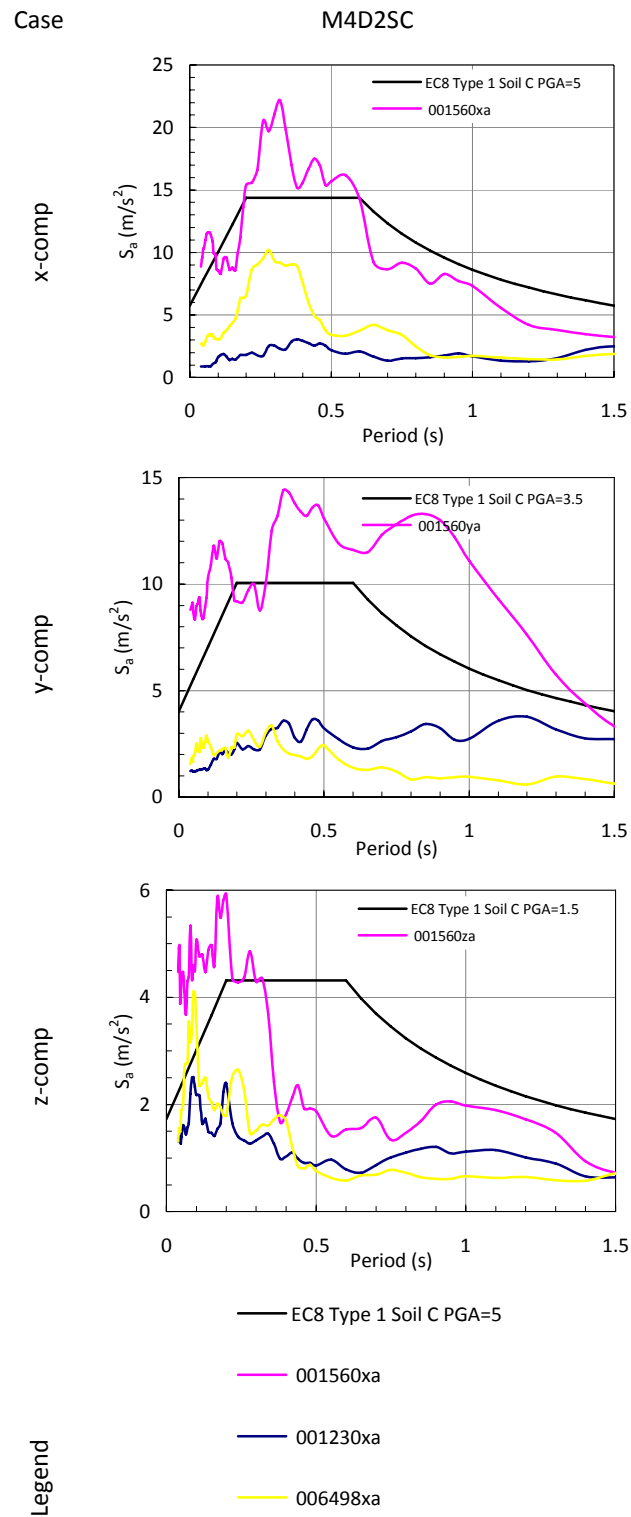


Fig. A4 (b): Acceleration Time History- and Approximated EC8 Response Spectra

Table A4 (c): Earthquake Recording Details for Magnitude Range M4, Distance Range D3

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
26/03/1998	43.238 N	12.931 E	15	6.6	Hveragerdi-Church	Iceland	41	6278	A
17/06/2000	63.970 N	20.360 W	15	6.6	Hveragerdi-RetirementH	Iceland	41	4679	A
6/10/1997	43.019 N	12.838 E	15	6.6	Sultartanga-Hydroelectr	Iceland	42	6272	A
17/06/2000	63.970 N	20.360 W	17	7.8	Gebze-Tubitak	Turkey	47	1228	A
16/12/1990	41.370 N	43.720 E	-	6.6	Reykjavik-Heidmork	Iceland	53	6348	A
26/09/1997	43.031 N	12.862 E	-	6.6	Reykjavik-Foldaskoli	Iceland	56	6347	A
7/5/1984	41.666 N	13.820 E	-	6.6	Sultartanga-Hydroelect	Iceland	58	6344	A
21/06/2000	63.970 N	20.710 W	15	6.6	Hrauneyjafoss-Hydroele	Iceland	61	6266	A
17/06/2000	63.970 N	20.360 W	-	6.6	Reykjavik-HusVerslunari	Iceland	61	6346	A
21/06/2000	63.970 N	20.710 W	12	7	Hercegnovi-Novi-O.S.D.	Serb	65	200	A
16/12/1990	41.370 N	43.720 E	15	6.6	Reykjavik-Heidmork	Iceland	70	6276	A
17/06/2000	63.970 N	20.360 W	15	6.6	Reykjavik-Foldaskoli	Iceland	72	6275	A
16/12/1990	41.370 N	43.720 E	15	6.6	Reykjavik-HusVerslunar	Iceland	78	6274	A
17/06/2000	63.970 N	20.360 W	16	6.9	Torre-del-Greco	Italy	80	296	A
11/23/1980	40.780 N	15.330 E	-	6.6	Sigolduvirkjun-Hydroel	Iceland	81	6343	A
4/6/1998	64.036 N	21.293 W	16	6.9	Arienzo	Italy	78	286	A
17/08/1999	40.702 N	29.987 E	4	7.4	Ferdoos	Iran	111	183	B
26/09/1997	43.031 N	12.862 E	16	6.9	Tricarico	Italy	72	297	B
16/06/1990	39.130 N	20.380 E	7	6.55	Codroipo	Italy	42	49	B
9/19/1979	42.760 N	13.020 E	14	7.3	LDEOStationNo.C0362CH	Turkey	42	6499	B
15/06/1991	42.465 N	44.030 E	16	6.9	Brienza	Italy	43	288	B
15/06/1995	38.362 N	22.200 E	14	7.3	LDEOStationNo.C1060BU	Turkey	45	6496	B
13/11/1998	63.954 N	21.346 W	16	6.9	Mercato-San-Severino	Italy	48	289	B
11/23/1980	40.780 N	15.330 E	17	7.8	Gebze-Arcelik	Turkey	55	1248	B
4/6/1998	64.036 N	21.293 W	12	7	Titograd-Geoloski-Zavod	Serb	56	202	B
13/11/1998	63.954 N	21.346 W	4	7.4	Tabas	Iran	57	187	B
9/16/1978	33.249 N	57.384 E	14	7.3	Sakarya-Bayindirlik	Turkey	65	1566	B
15/06/1991	42.465 N	44.030 E	4	7.4	Boshroyeh	Iran	68	181	B
2/4/1996	37.830 N	26.990 E	17	7.8	Goynuk-Devlet-Hastanesi	Turkey	73	1229	B
12/11/1999	40.768 N	31.148 E	17	7.8	Istanbul-Bayindirlik	Turkey	86	1218	B
9/16/1978	33.249 N	57.384 E	22	7.1	Ierissos-Police-Station	Greece	87	2019	B
25/06/2001	37.238 N	36.206 E	17	7.8	Bursa-Sivil-Savunma-M	Turkey	93	1216	B
17/08/1999	40.702 N	29.987 E	19	7.5	Qazvin	Iran	94	476	B
26/09/1997	43.023 N	12.892 E	16	6.9	Benevento	Italy	58	295	B
12/11/1999	40.768 N	31.148 E	17	7.8	Istanbul-Mecidiyekoy	Turkey	93	4340	B
11/23/1980	40.780 N	15.330 E	17	7.8	Istanbul-Atakoy	Turkey	101	4337	C
29/09/1999	40.738 N	29.331 E	14	7.3	Baglaralti(Yalova)	Turkey	159	6919	C
17/08/1999	40.702 N	29.987 E	14	7.3	Istanbul-K.M.Pasa	Turkey	189	3726	C
12/11/1999	40.768 N	31.148 E	14	7.3	Istanbul-Atakoy	Turkey	195	4338	C
18/11/1997	37.482 N	20.692 E	17	7.8	Yapi-Kredi-Plaza-Levent	Turkey	92	1256	A
17/08/1999	40.702 N	29.987 E	17	7.8	Istanbul-Maslak	Turkey	93	4341	A
4/15/1979	41.980 N	18.980 E	14	7.3	Golcuk	Turkey	113	4339	C
17/08/1999	40.702 N	29.987 E	19	7.5	Tonekabun	Iran	131	480	C
12/11/1999	40.768 N	31.148 E	12	7	Veliki-Ston-F-Ka-Soli	Croatia	144	1436	C
4/15/1979	41.980 N	18.980 E	14	7.3	Kasif	Turkey	157	6924	C
12/11/1999	40.768 N	31.148 E	14	7.3	Bahcevan(Yalova)	Turkey	158	6920	C
12/11/1999	40.768 N	31.148 E	14	7.3	Ruzgar(Yalova)	Turkey	159	6926	C
12/11/1999	40.768 N	31.148 E	14	7.3	Tar	Turkey	161	6927	C

Cont'd

Table A4 (c): Earthquake Recording Details for Magnitude Range M4, Distance Range D3

Date	Latitude	Longitude	Depth (km)	Ms	Station Name	Country	de (km)	Waveform ID	Soil
12/11/1999	40.768 N	31.148 E	14	7.3	Kutahya-SivilSavunma	Turkey	179	1564	C
12/11/1999	40.768 N	31.148 E	14	7.3	Bursa-TofaFabrikasi	Turkey	185	3719	C
12/11/1999	40.768 N	31.148 E	14	7.3	Fatih-Tomb	Turkey	187	3723	C
12/11/1999	40.768 N	31.148 E	14	7.3	Istanbul-Zeytinburnu	Turkey	190	4344	C
12/11/1999	40.768 N	31.148 E	12	7	Gacko-Zemlj.-Zadruga	BosnHer	137	203	C
7/5/1984	41.666 N	13.820 E	17	7.8	Kutahya-Sivil	Turkey	142	1219	C
4/15/1979	41.980 N	18.980 E	14	7.3	Hastane	Turkey	160	6922	C
6/10/1997	43.019 N	12.838 E	10	6.501	Pyrgos-Agriculture	Greece	69	5837	C
5/4/1998	43.185 N	12.719 E	17	7.8	Fatih-Tomb	Turkey	94	1254	C
26/09/1997	43.031 N	12.862 E	17	7.8	Istanbul-K.M.Pasa	Turkey	95	6918	C
12/11/1999	40.768 N	31.148 E	16	6.9	Bovino	Italy	52	299	C
6/10/1997	43.019 N	12.838 E	17	7.8	Bursa-Tofa-Fabrikasi	Turkey	92	1251	C
31/08/1999	40.756 N	29.935 E	19	7.5	Rudsar	Iran	81	479	C
14/02/2000	40.980 N	31.700 E	17	7.8	Istanbul-Zeytinburnu	Turkey	96	4343	C
29/08/1999	40.798 N	31.124 E	19	7.5	Abhar	Iran	98	475	C
15/06/1991	42.465 N	44.030 E	17	7.8	Duzce-Meteoroloji-	Turkey	100	1226	C

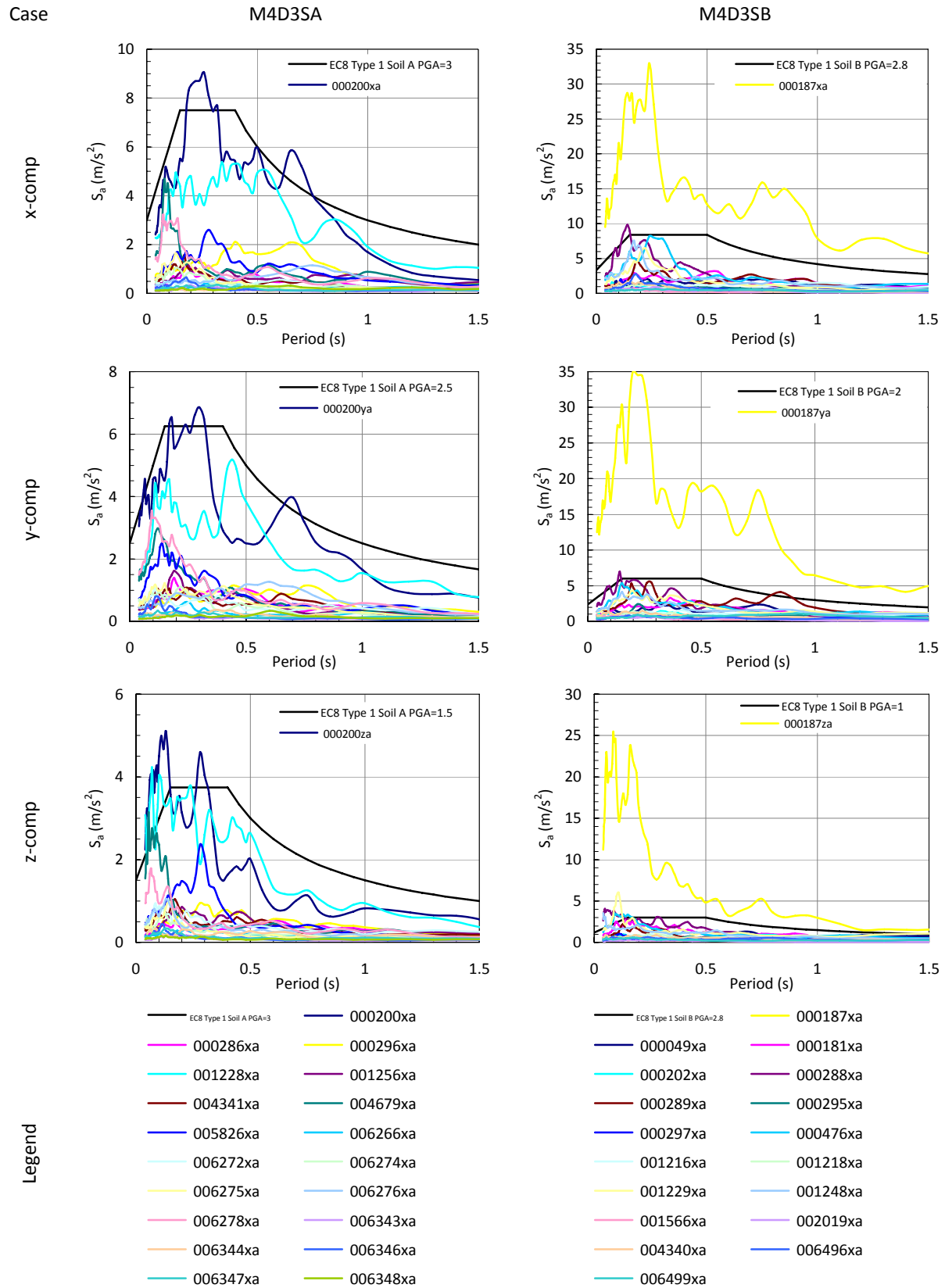


Fig. A4 (c): Acceleration Time History- and Approximated EC8 Response Spectra (Cont'd)

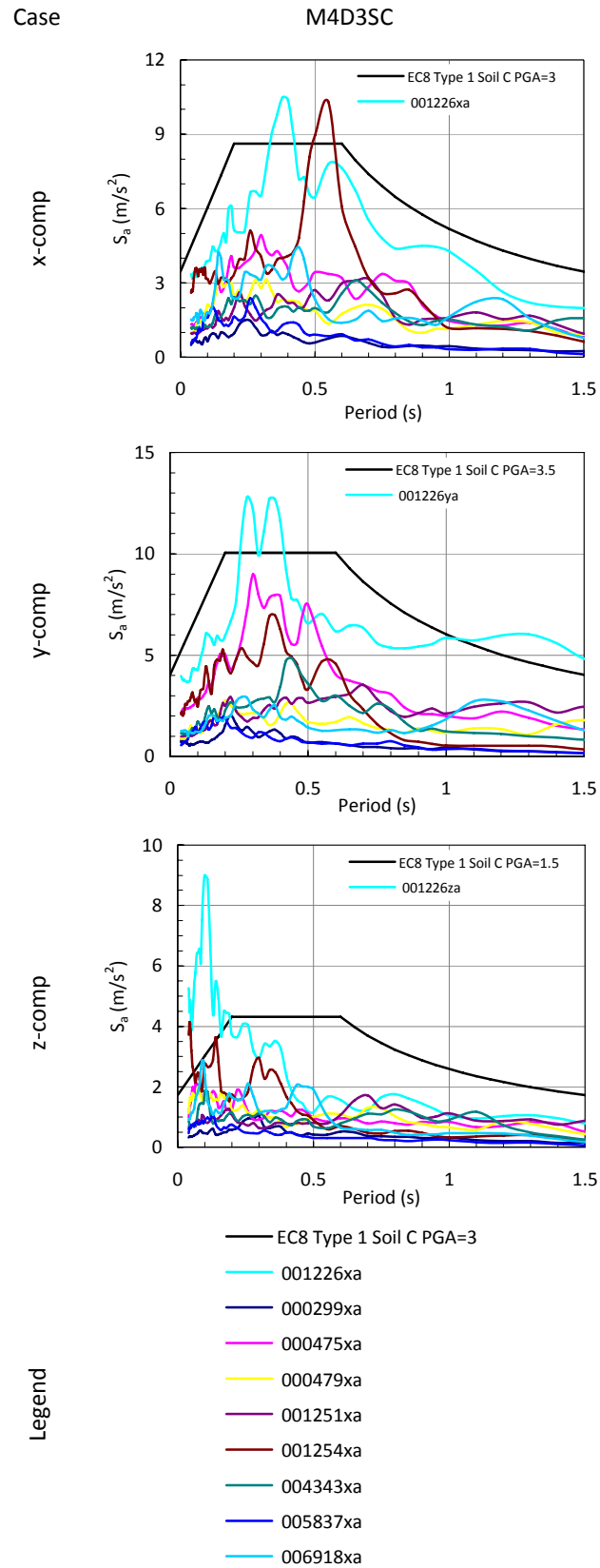


Fig. A4 (c): Acceleration Time History- and Approximated EC8 Response Spectra

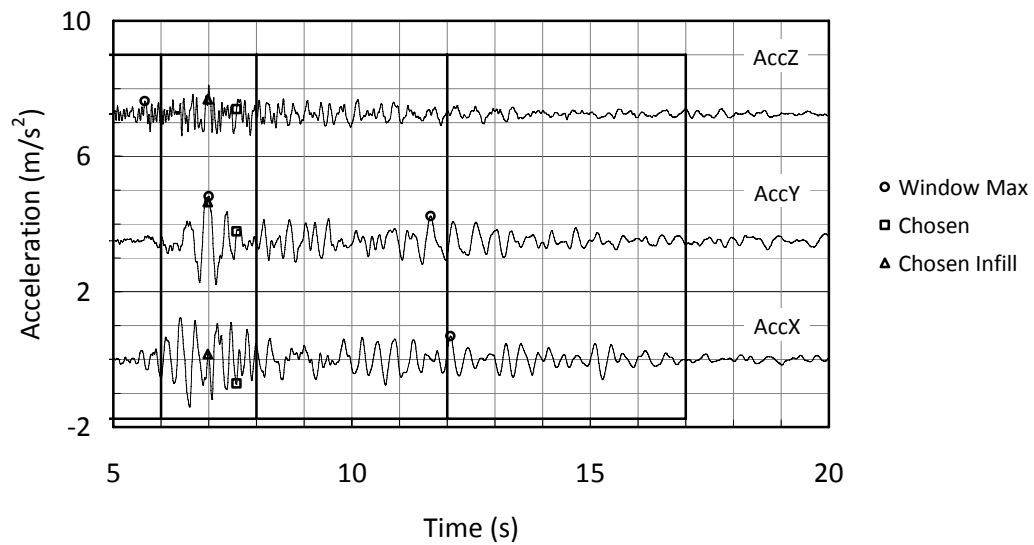
ANNEXURE B

EXAMPLES FOR VERIFICATION OF CRITERION

This part is an annexure of section [4.3.4](#) of the main text. It contains the examples presented for the verification of the criterion used to develop the load patterns.

Example 1 (given in the main text)

Example 2



(a) Acceleration Time History M2D2SB with Pre-Event, Event and Post-Event Windows

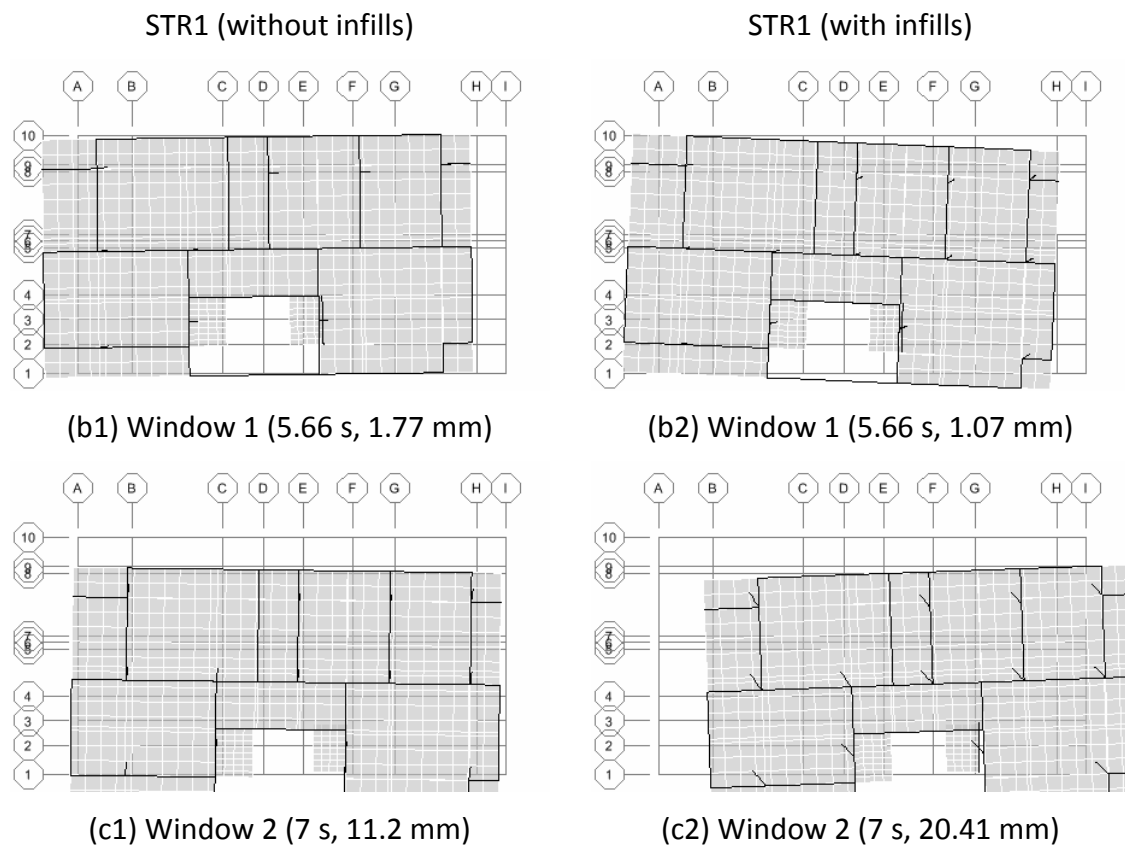
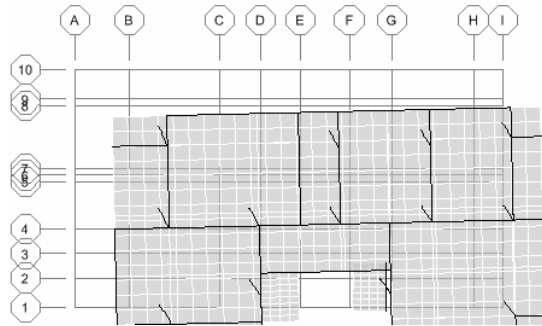


Fig. B1: Time History Response of STR2 for Different Time Windows

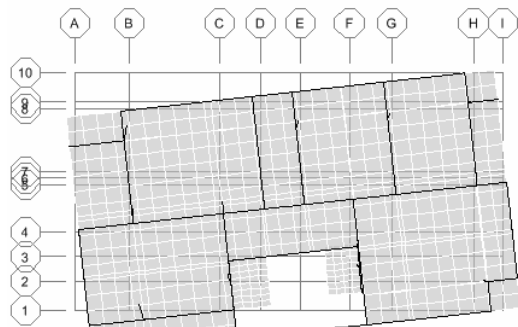
Cont'd



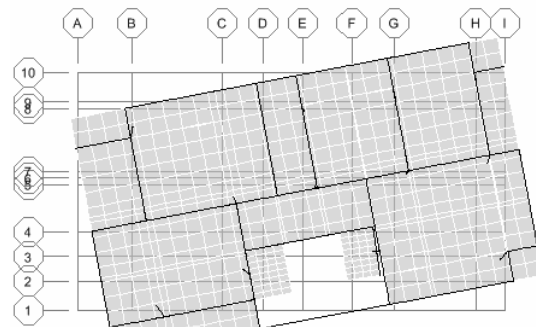
(d1) Window 3 (11.65 s, 9.82 mm)



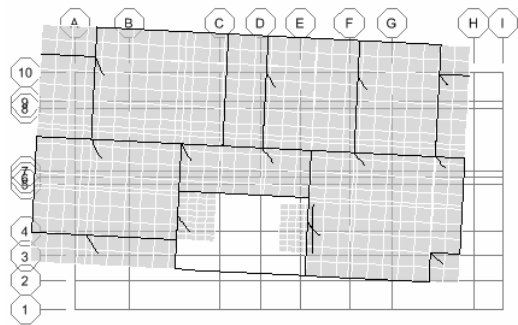
(d2) Window 3 (11.65 s, 5.33 mm)



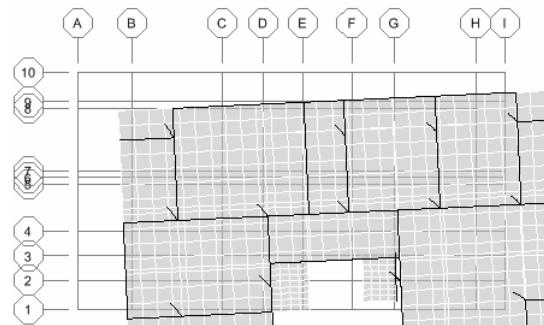
(e1) Window 4 (12.07 s, 3.4 mm)



(e2) Window 4 (12.07 s, 1.64 mm)

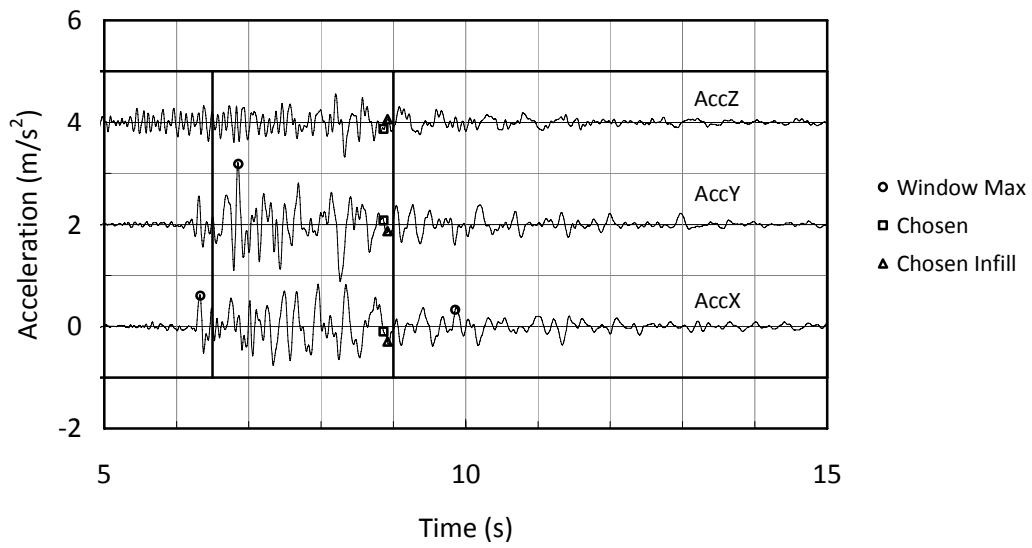


(f1) Chosen (7.585 s, 20.22 mm)



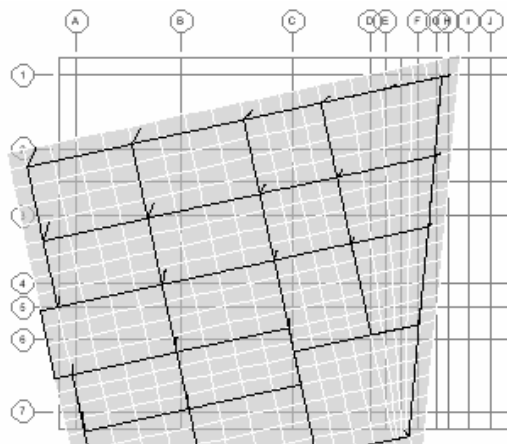
(f2) Chosen (6.98 s, 20.4 mm)

Fig. B1: Time History Response of STR2 for Different Time Windows

Example 3

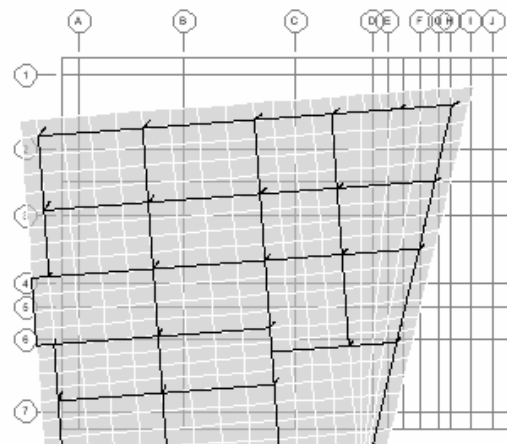
(a) Acceleration Time History M1D1SB with Pre-Event, Event and Post-Event Windows

STR2 (without infills)

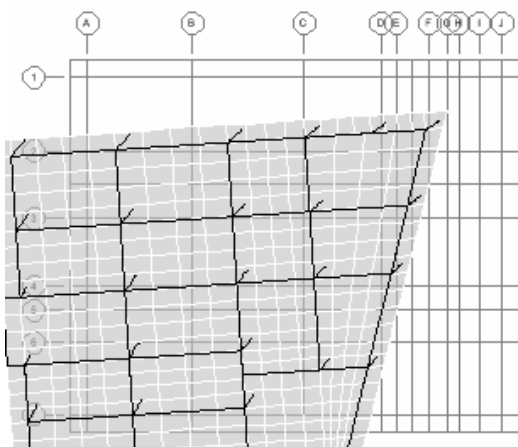


(b1) Window 1 (6.33 s, 0.55 mm)

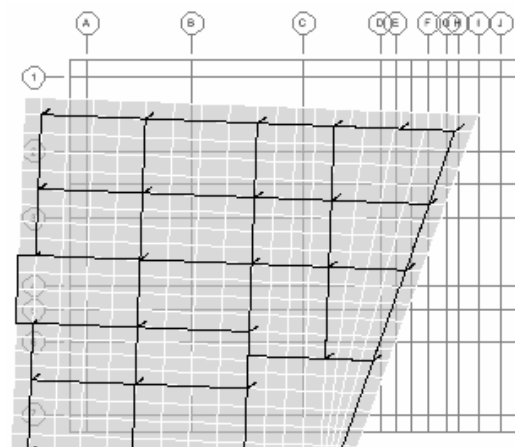
STR2 (with infills)



(b2) Window 1 (6.33 s, 0.47 mm)



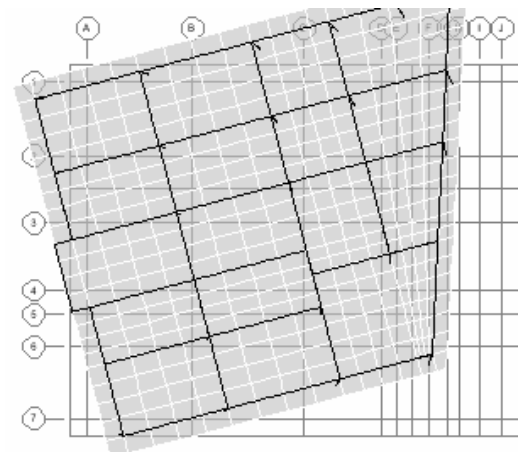
(c1) Window 2 (6.855 s, 4.56 mm)



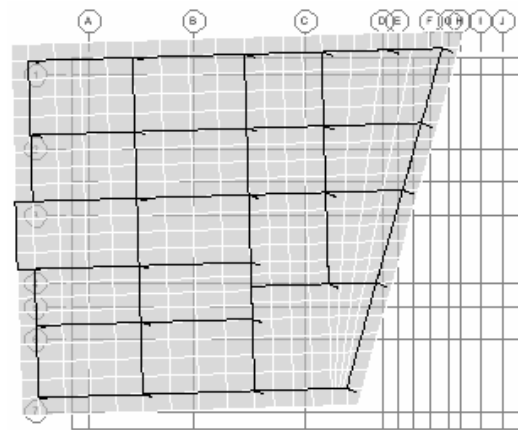
(c2) Window 2 (6.855 s, 1.4 mm)

Fig. B2: Time History Response of STR2 for Different Time Windows

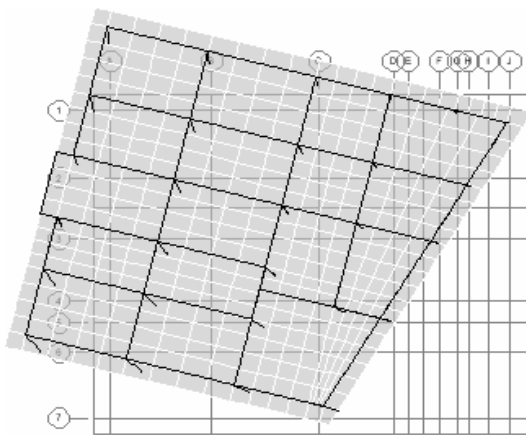
Cont'd



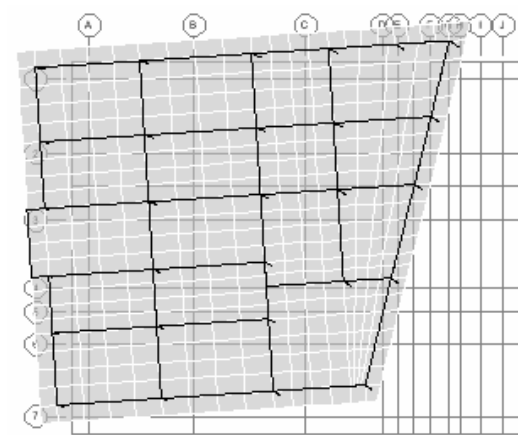
(d1) Window 3 (9.855 s, 3.98 mm)



(d2) Window 3 (9.855 s, 1.66 mm)



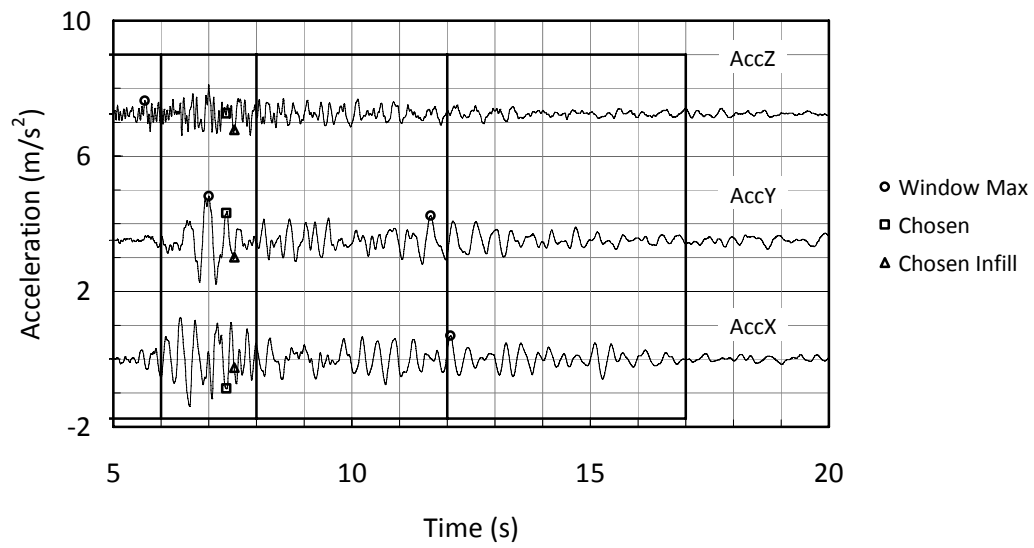
(e1) Chosen (8.345 s, 8.865 mm)



(e2) Chosen (8.745 s, 8.92 mm)

Fig. B2: Time History Response of STR2 for Different Time Windows

Example 4



(a) Acceleration Time History M2D2SB with Pre-Event, Event and Post-Event Windows

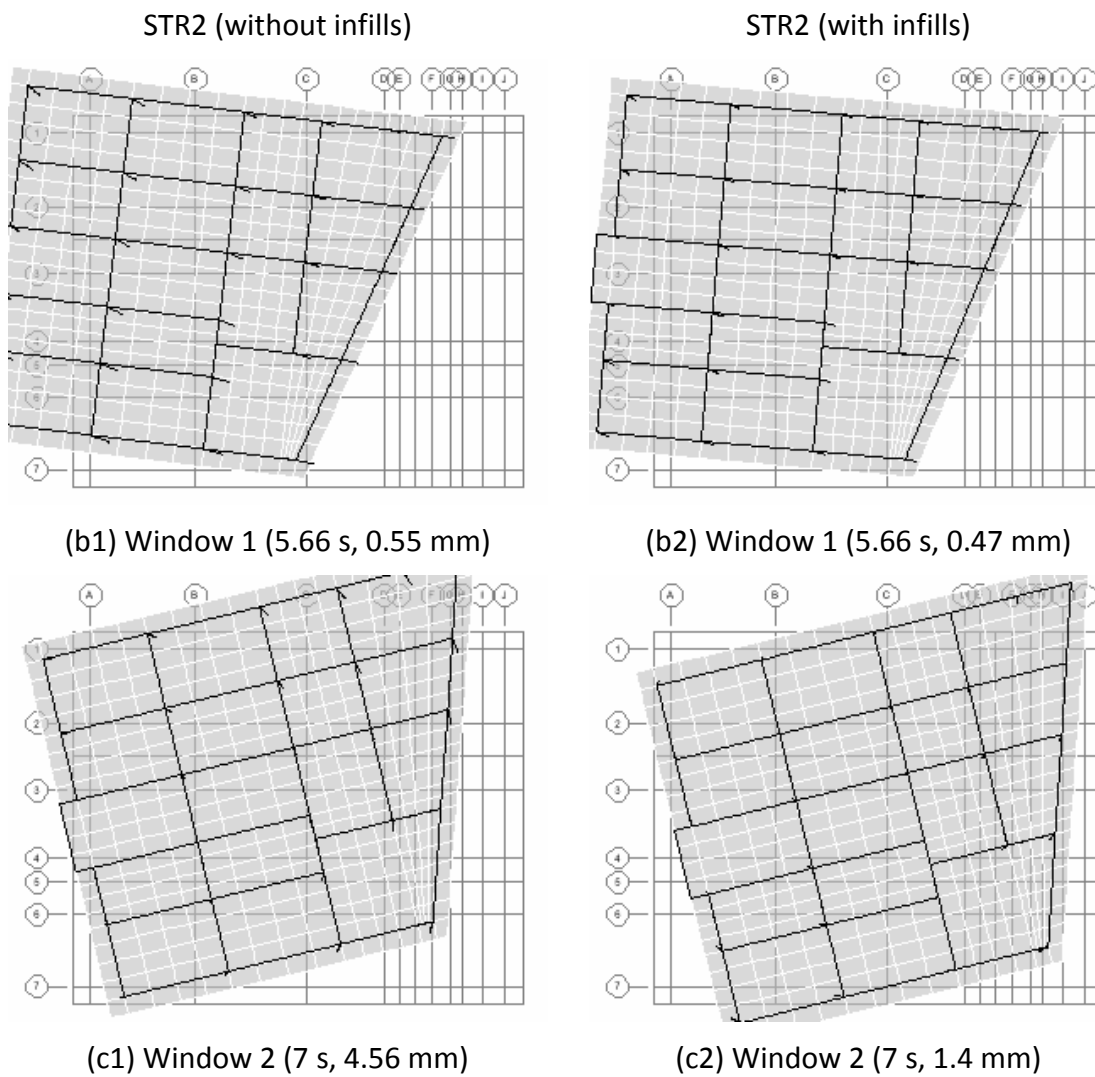
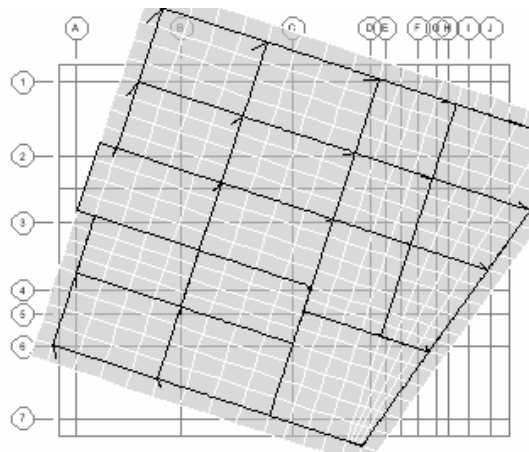
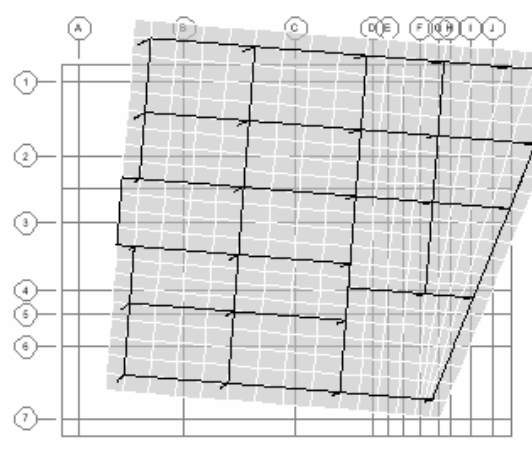


Fig. B3: Time History Response of STR2 for Different Time Windows

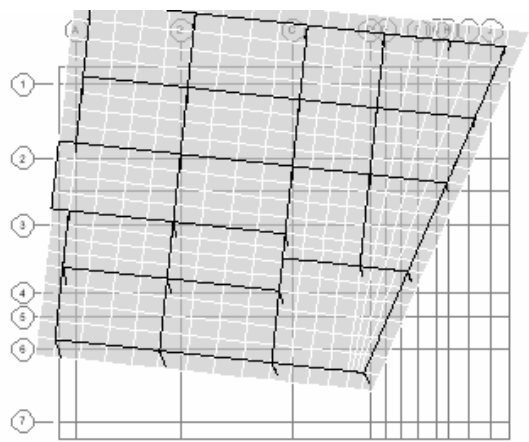
Cont'd



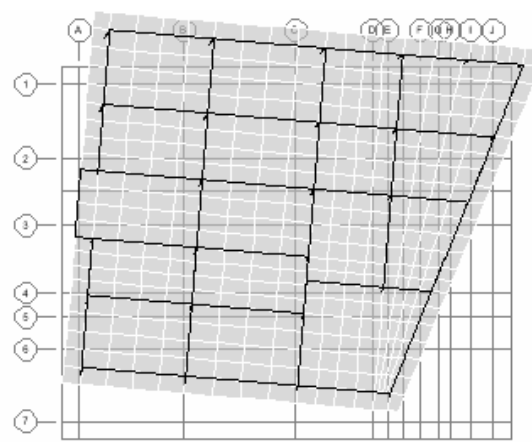
(d1) Window 3 (11.65 s, 3.98 mm)



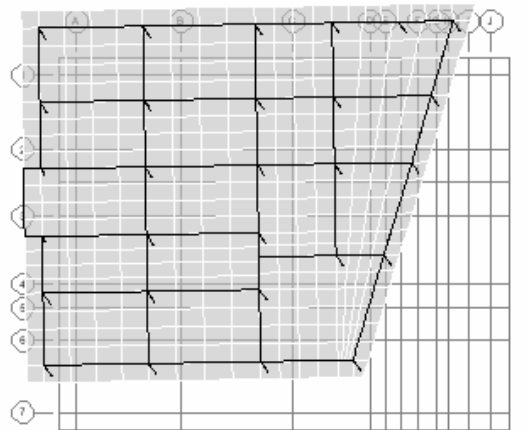
(d2) Window 3 (11.65 s, 1.66 mm)



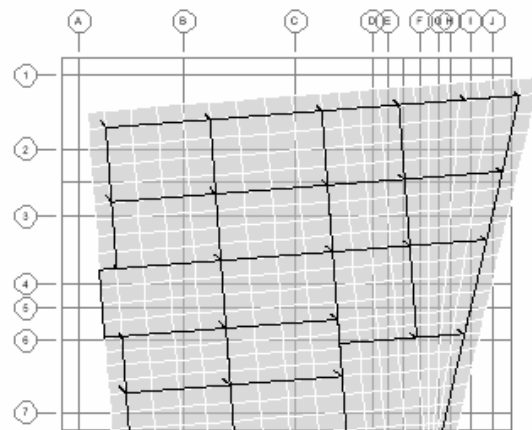
(e1) Window 4 (12.07 s, 3.98 mm)



(e2) Window 4 (12.07 s, 1.66 mm)



(f1) Chosen (8.345 s, 7.37 mm)



(f2) Chosen (8.745 s, 7.535 mm)

Fig. B3: Time History Response of STR2 for Different Time Windows

ANNEXURE C

DEVELOPED LOAD PATTERNS

This part is an annexure of section [4.3.5](#) of the main text. It contains all the load patterns developed for a combination of four different magnitude ranges, three epicentral distance ranges and four different soil classes.

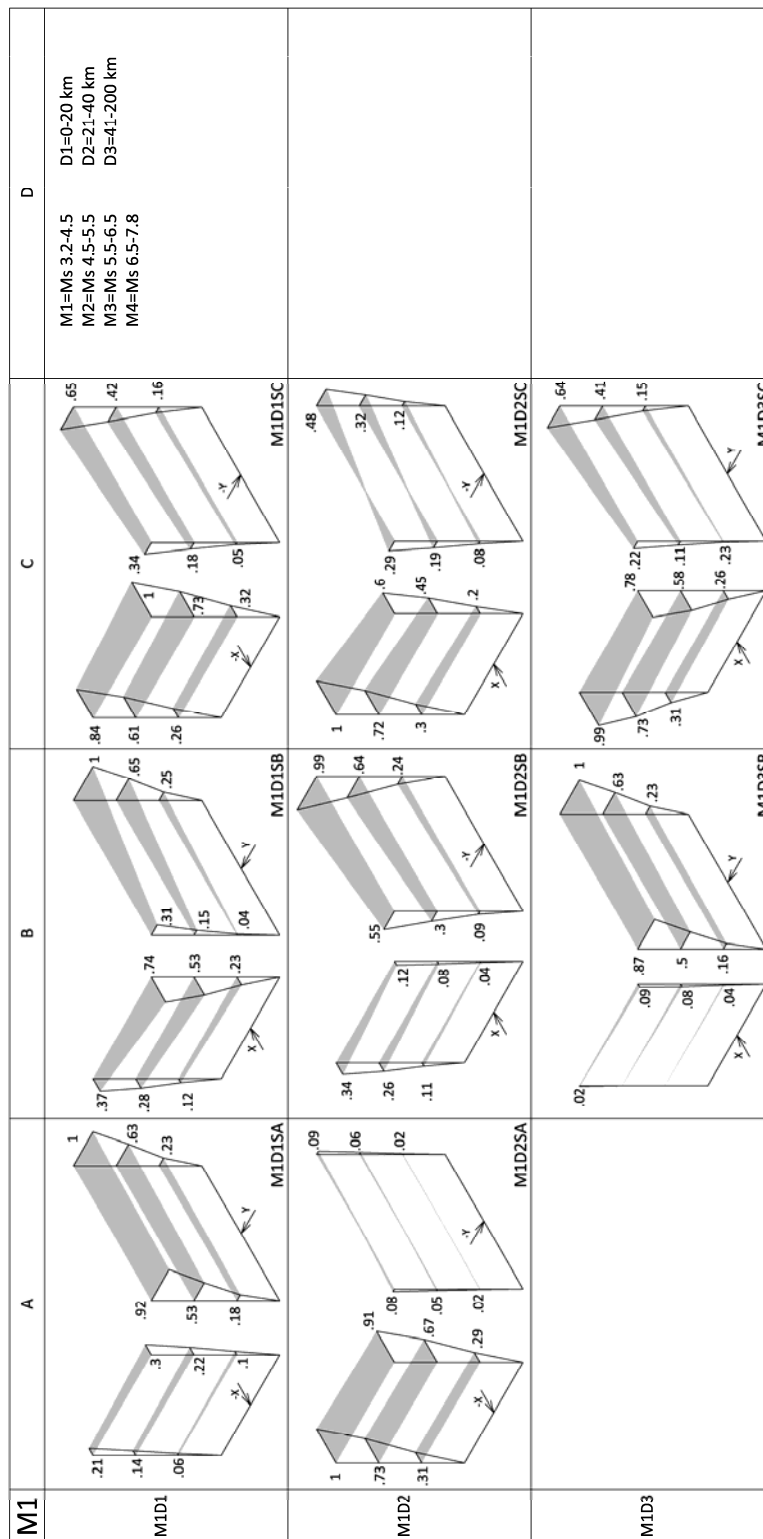


Fig. C1 (a): Load Patterns for STR1 (Magnitude Range M1)

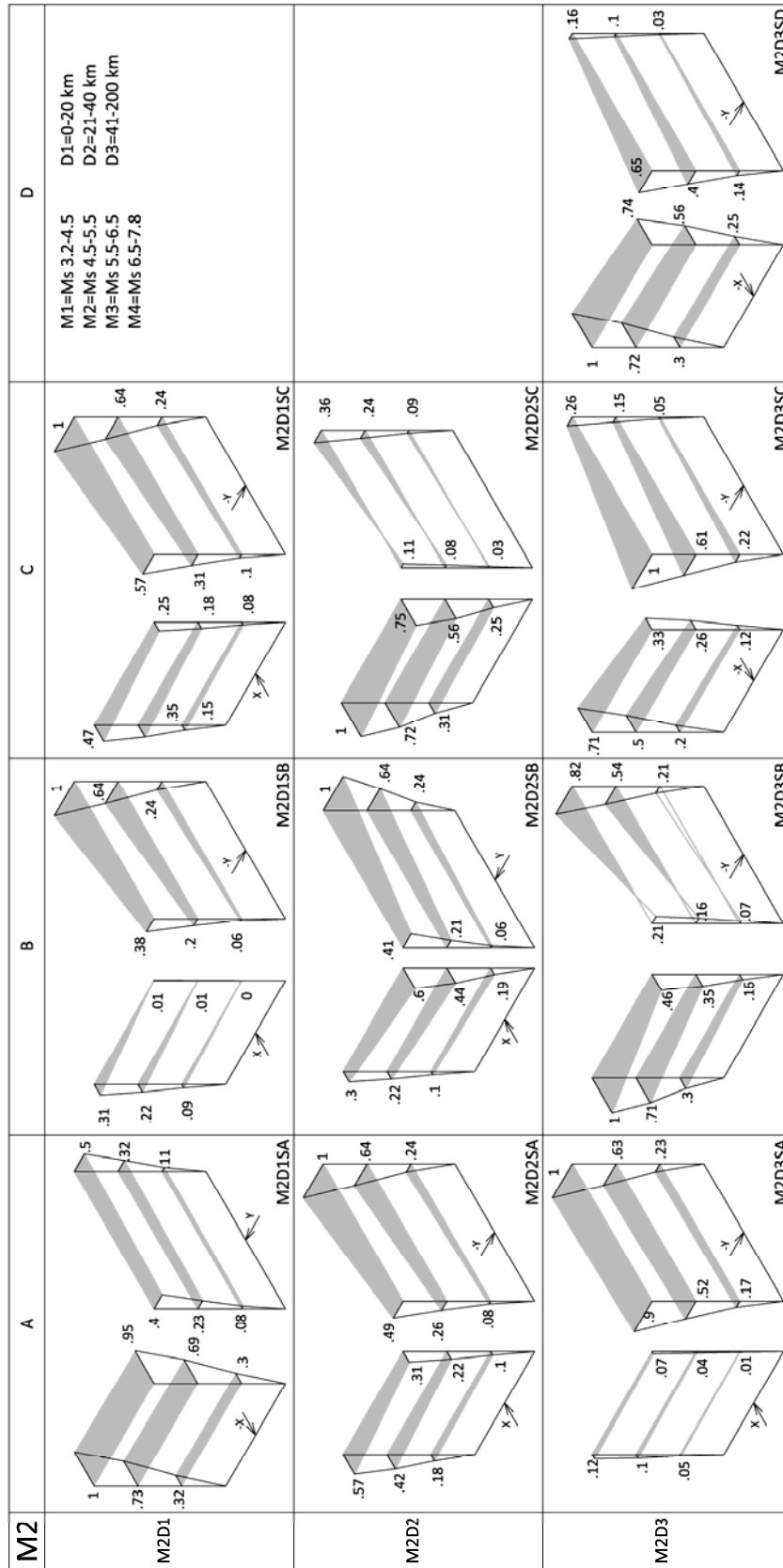


Fig. C1 (b): Load Patterns for STR1 (Magnitude Range M2)

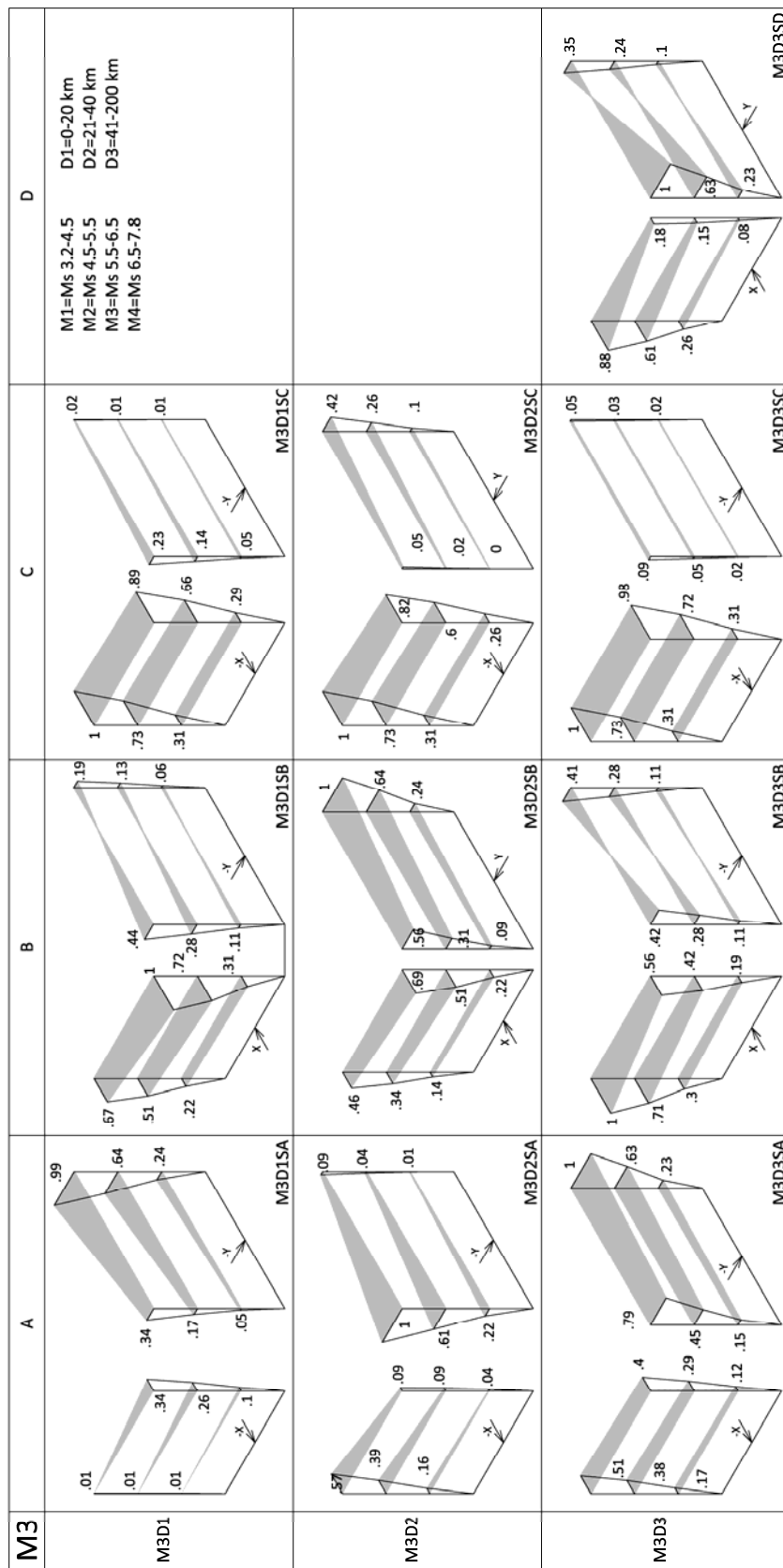


Fig. C1 (c): Load Patterns for STR1 (Magnitude Range M3)

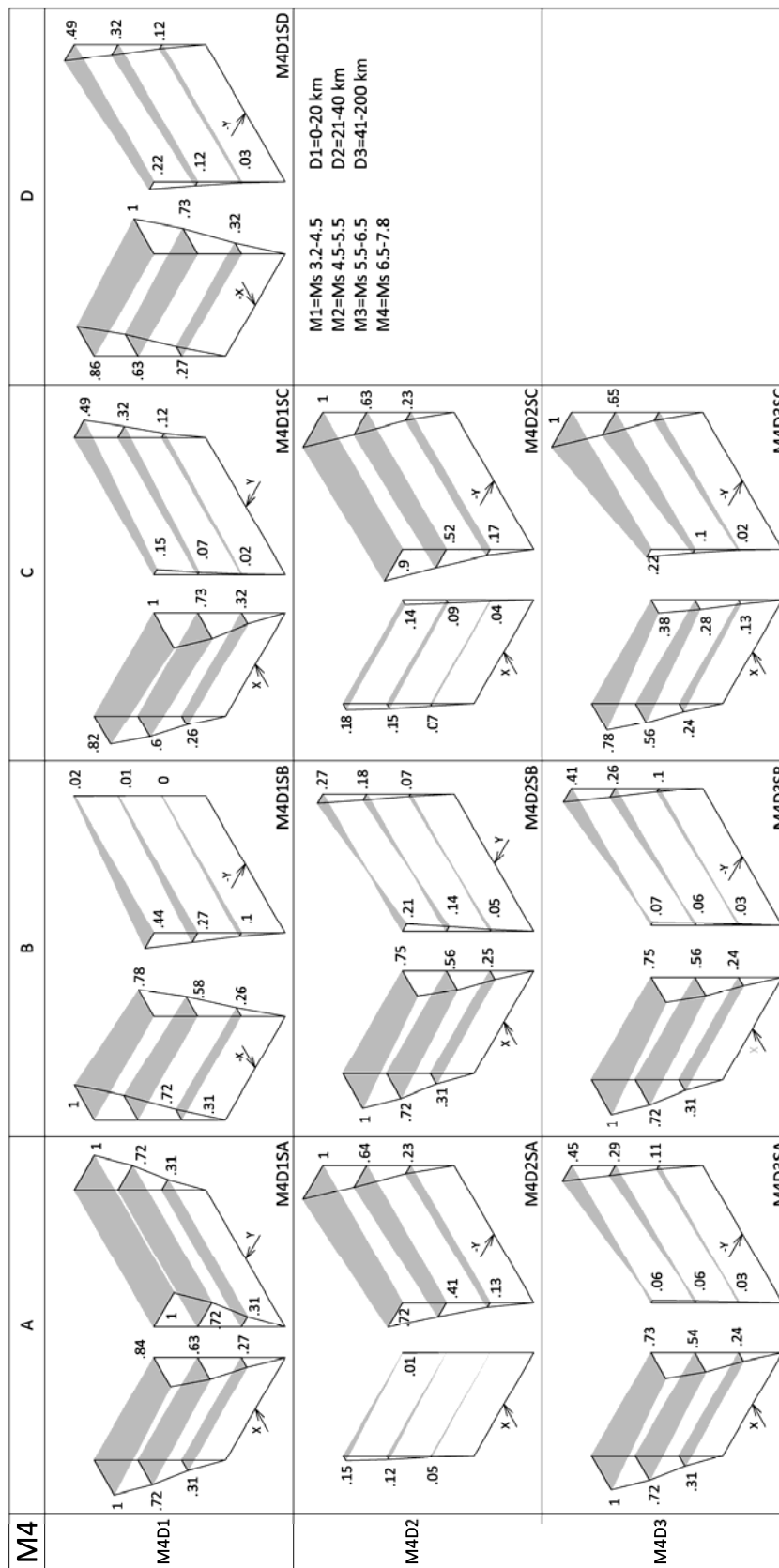


Fig. C1 (d): Load Patterns for STR1 (Magnitude Range M4)

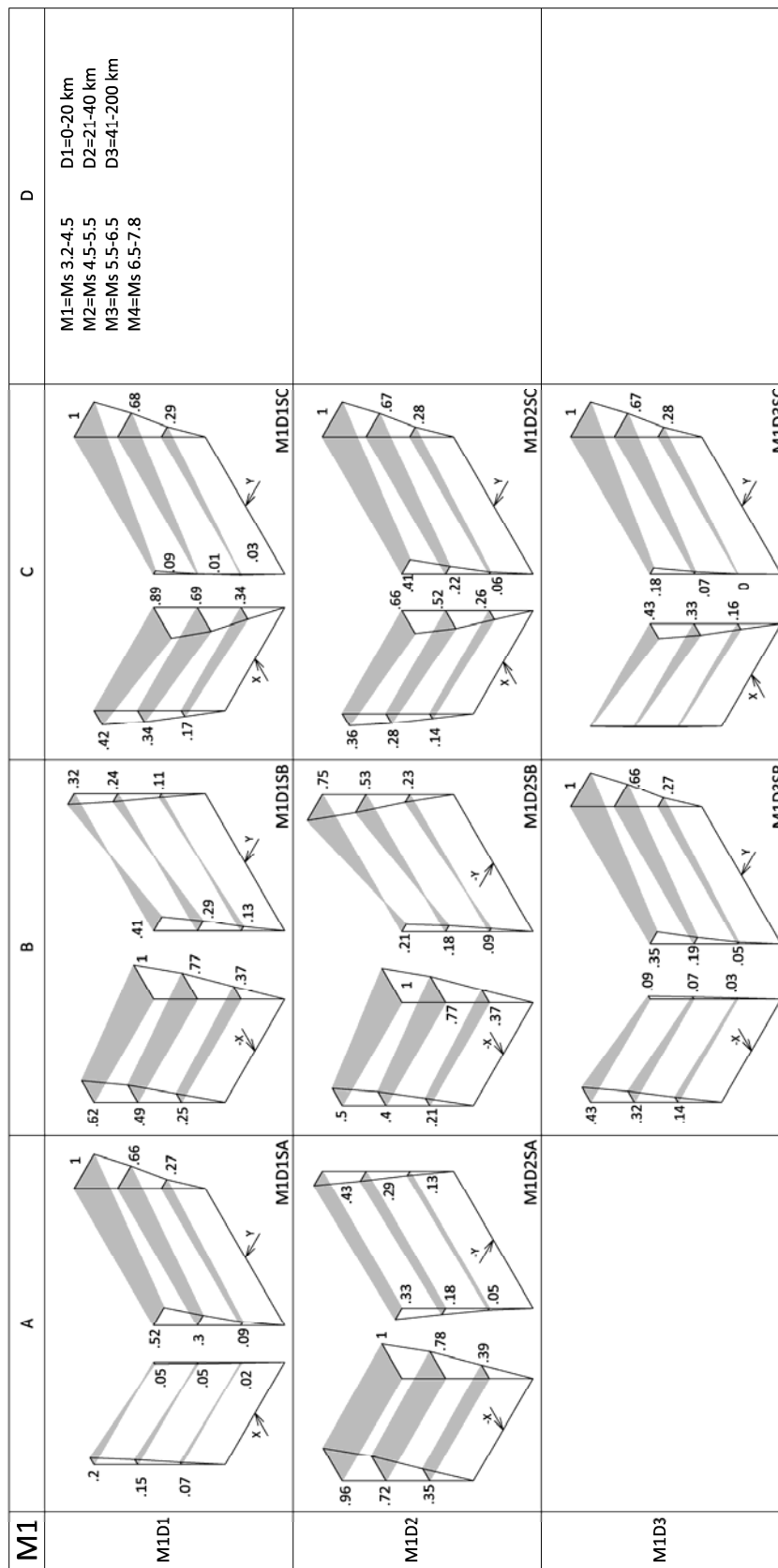


Fig. C2 (a): Load Patterns for STR1-Infill (Magnitude Range M1)

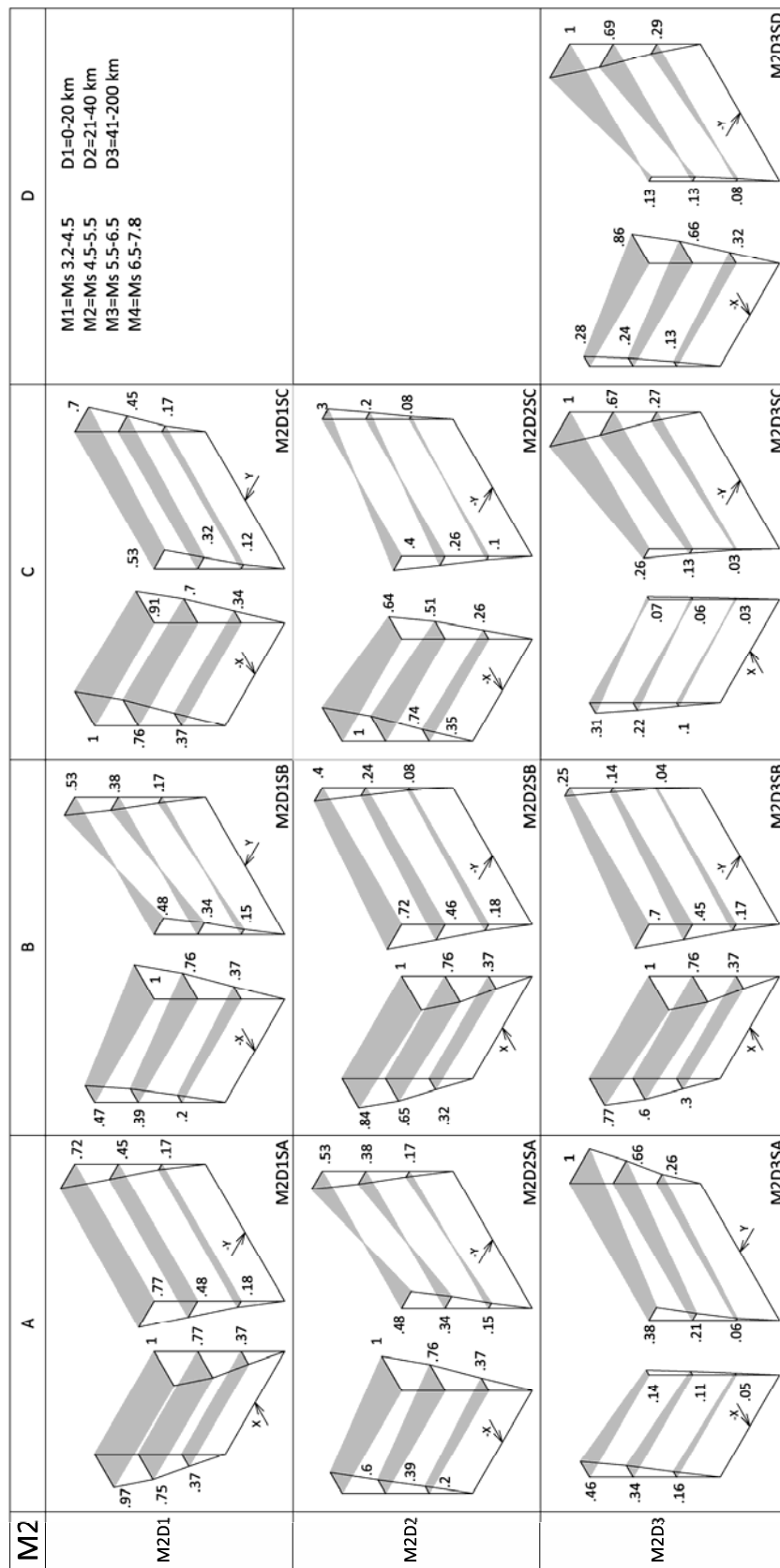


Fig. C2 (b): Load Patterns for STR1-Infill (Magnitude Range M2)

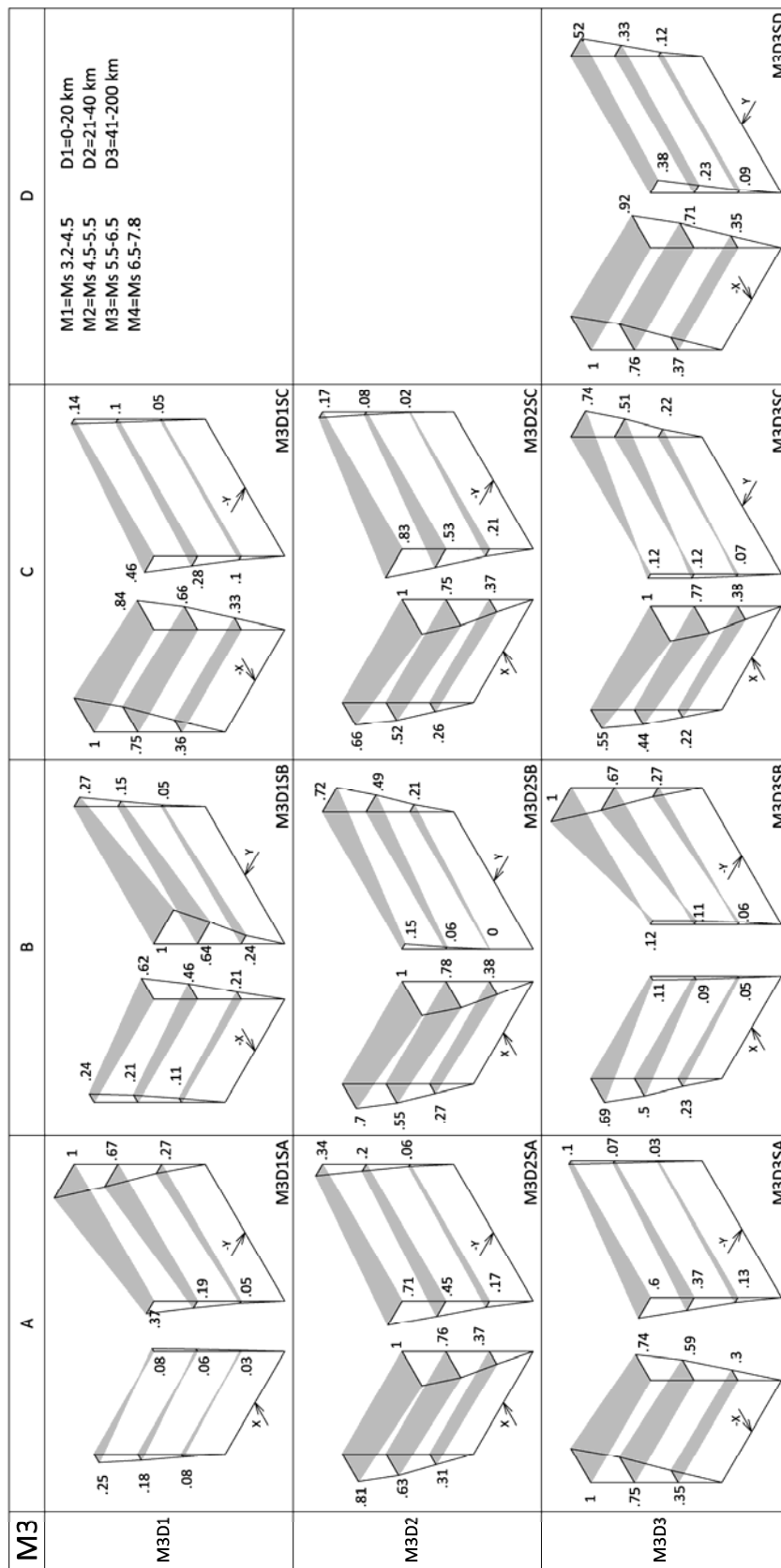


Fig. C2 (c): Load Patterns for STR1-Infill (Magnitude Range M3)

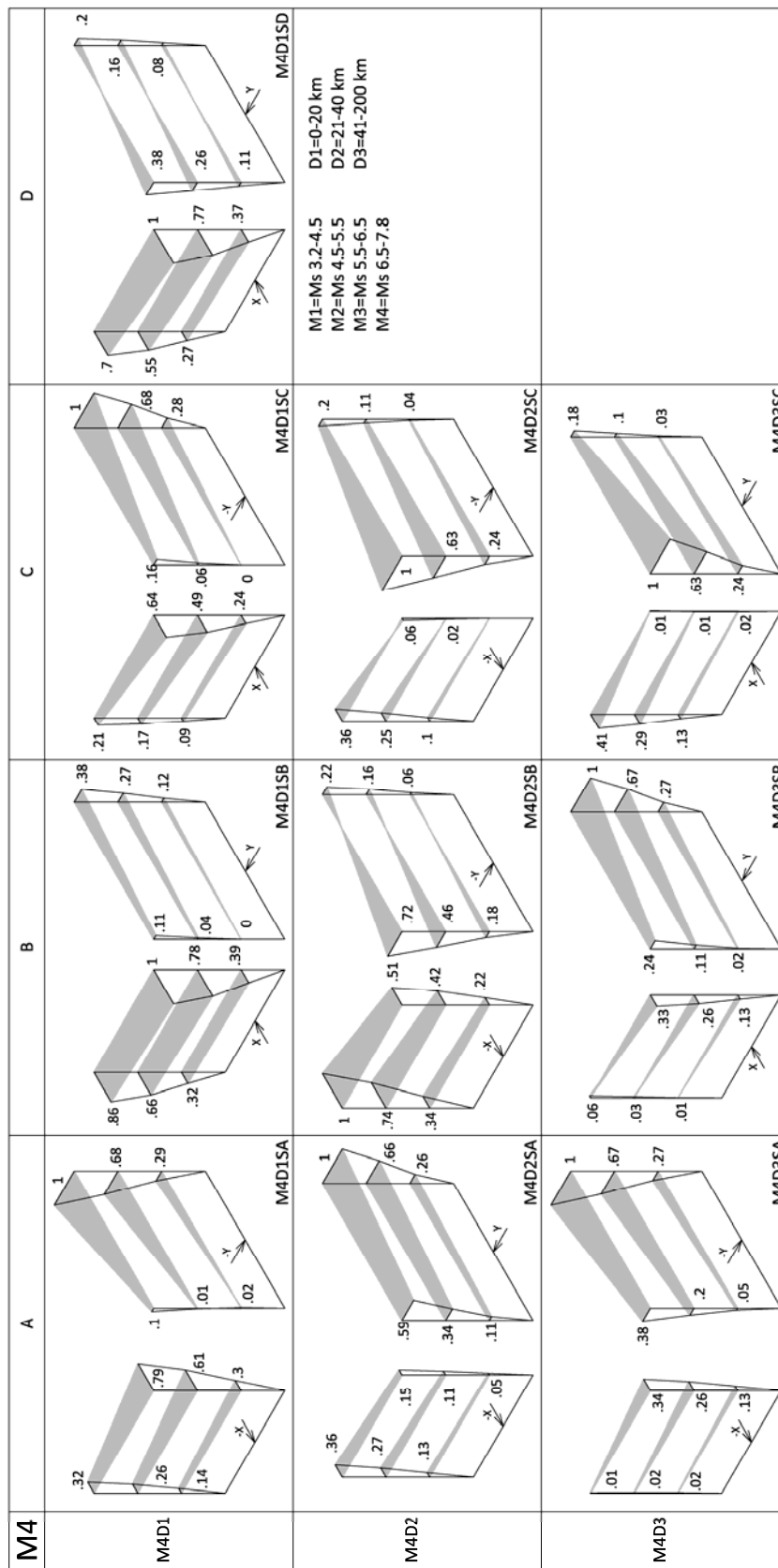


Fig. C2 (d): Load Patterns for STR1-Infill (Magnitude Range M4)

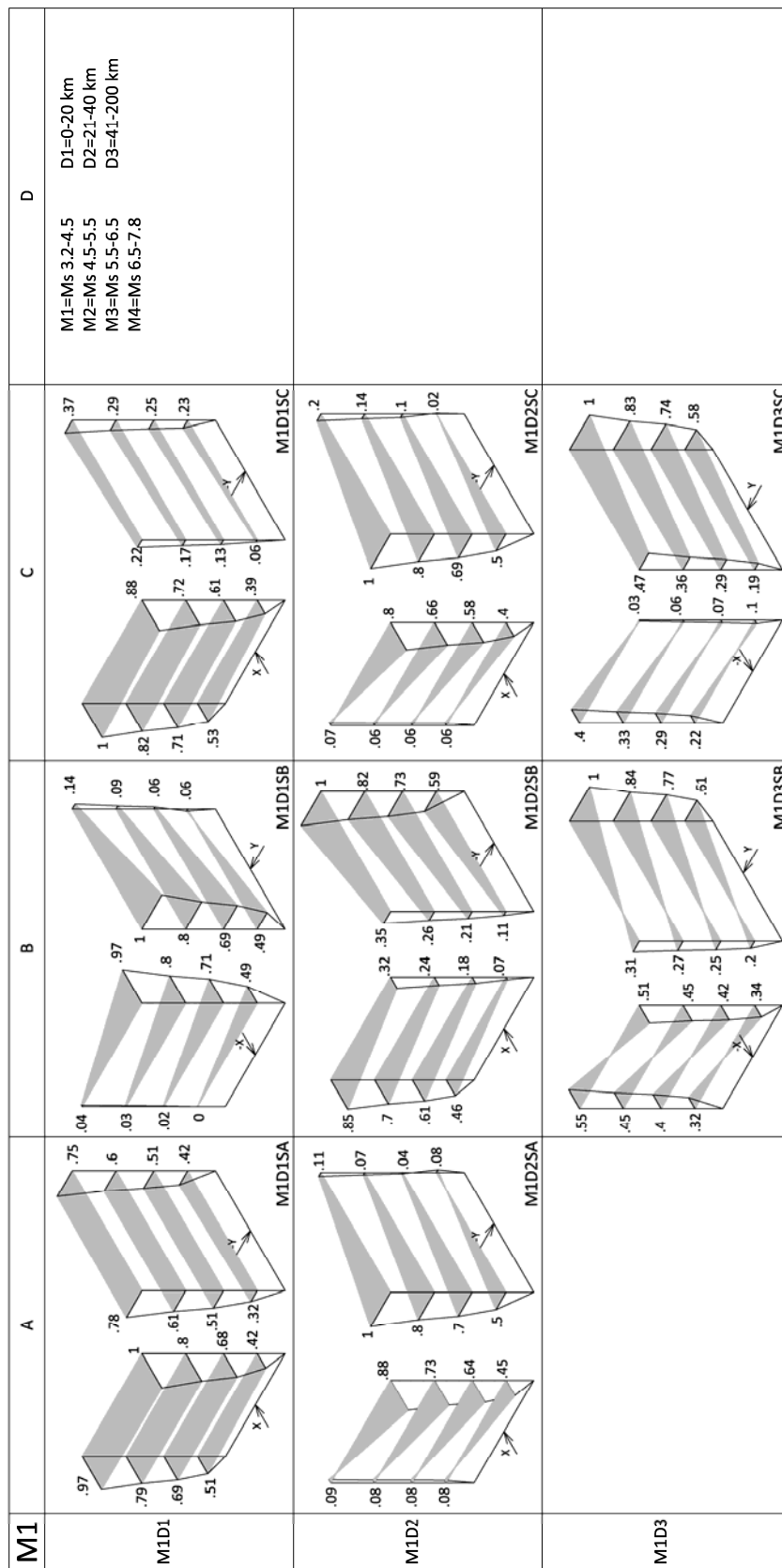


Fig. C3 (a): Load Patterns for STR2 (Magnitude Range M1)

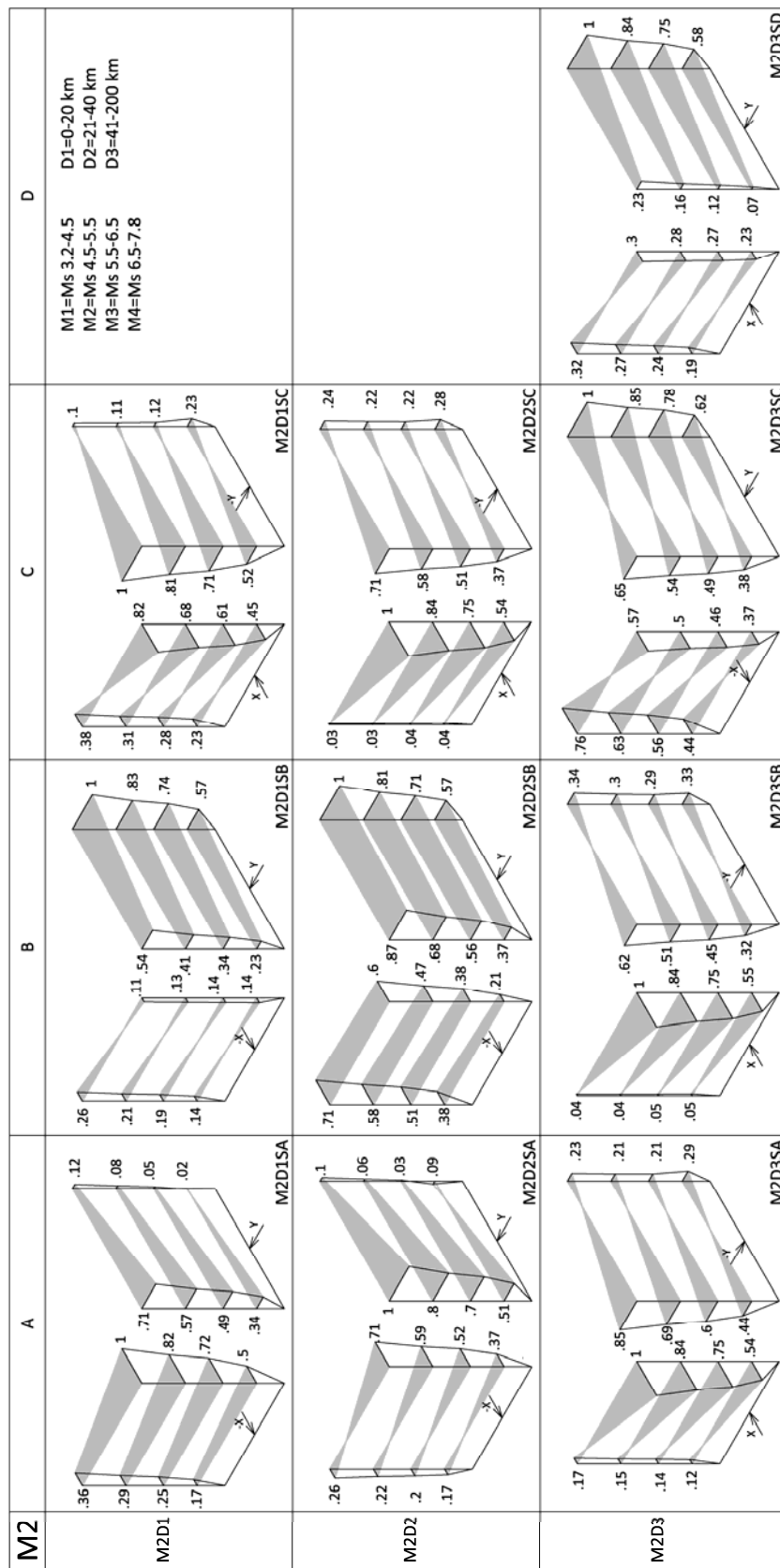


Fig. C3 (b) Load Patterns for STR2 (Magnitude Range M2)

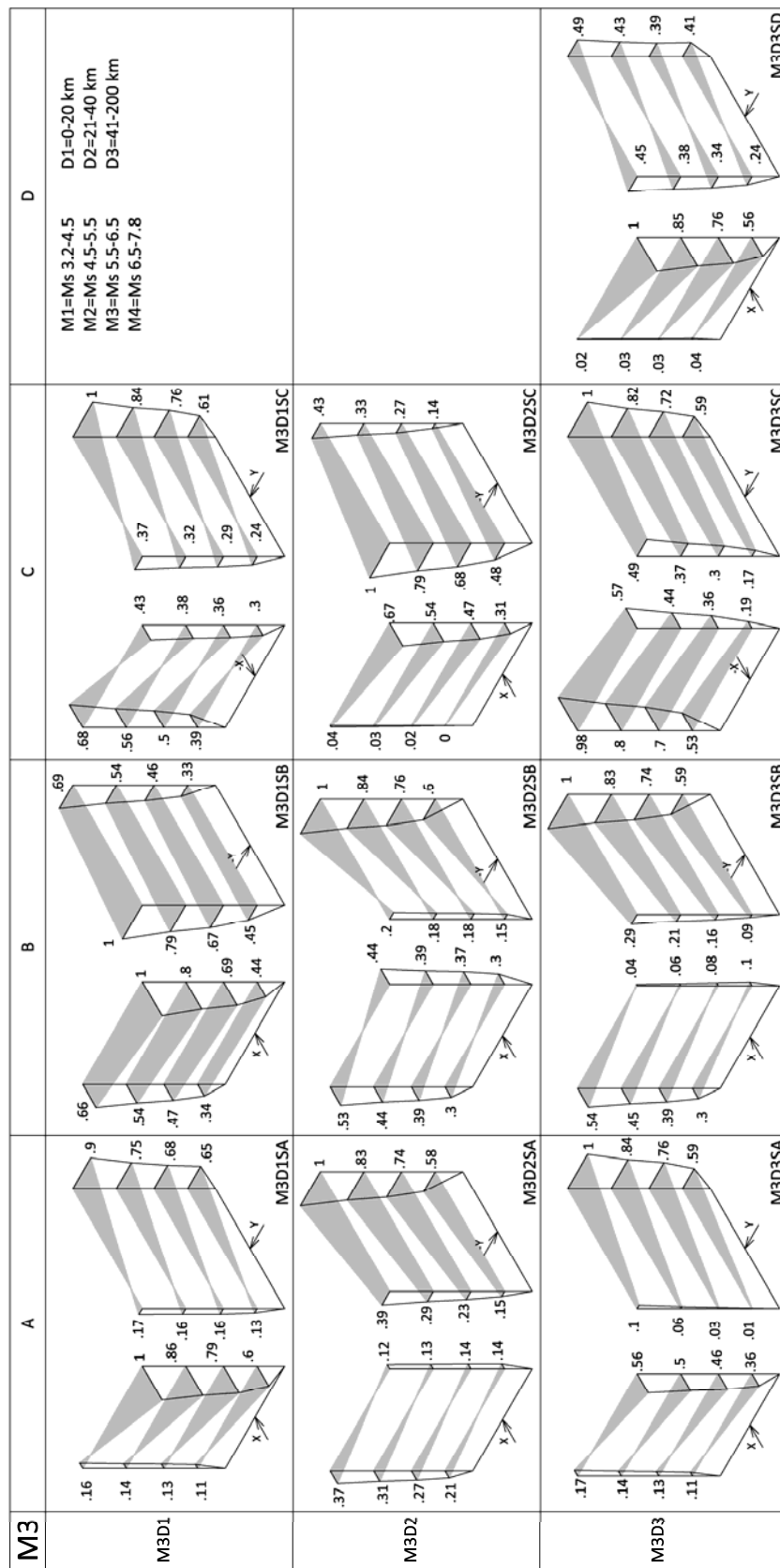


Fig. C3 (c): Load Patterns for STR2 (Magnitude Range M3)

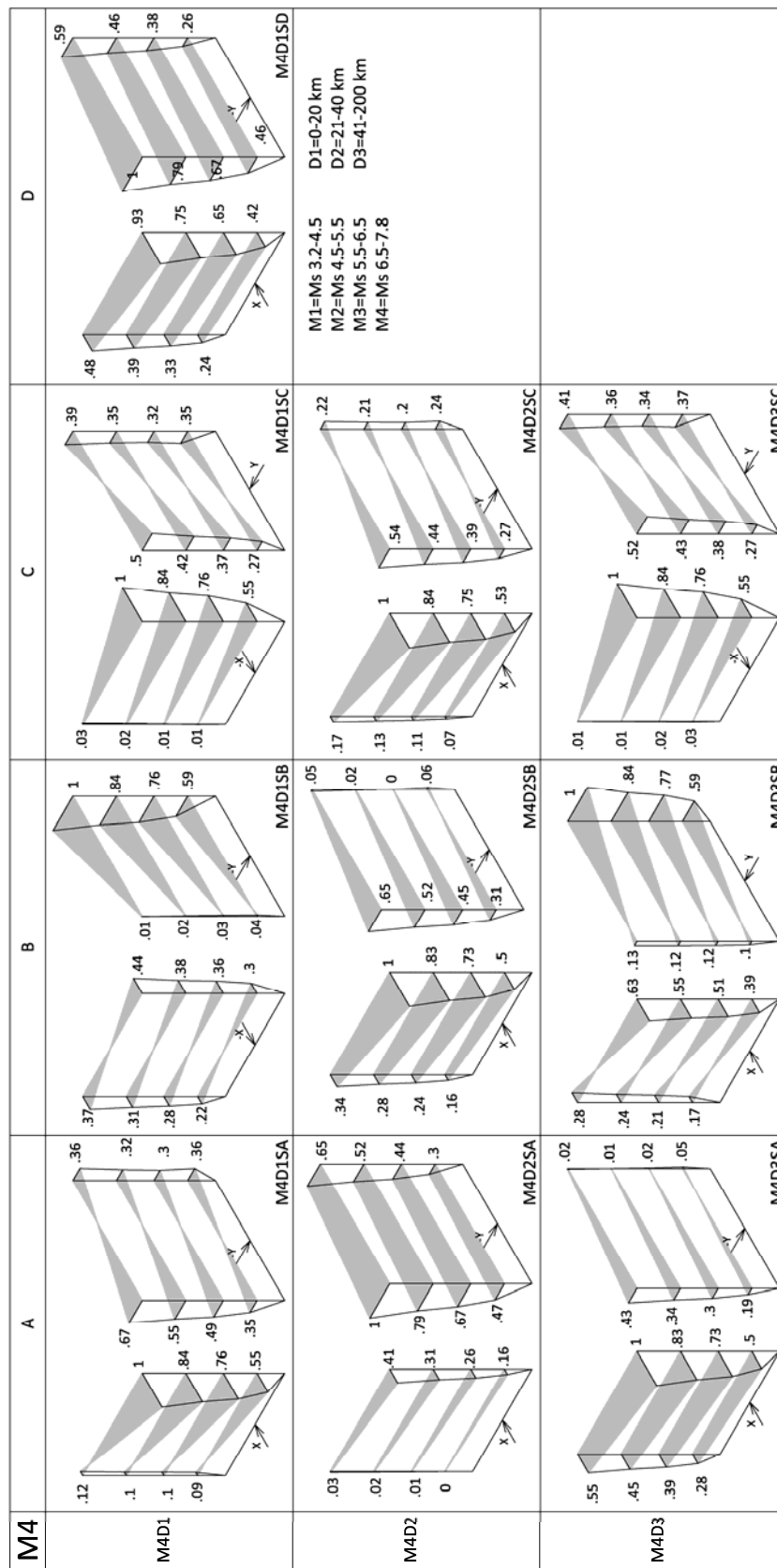


Fig. C3 (d): Load Patterns for STR2 (Magnitude Range M4)

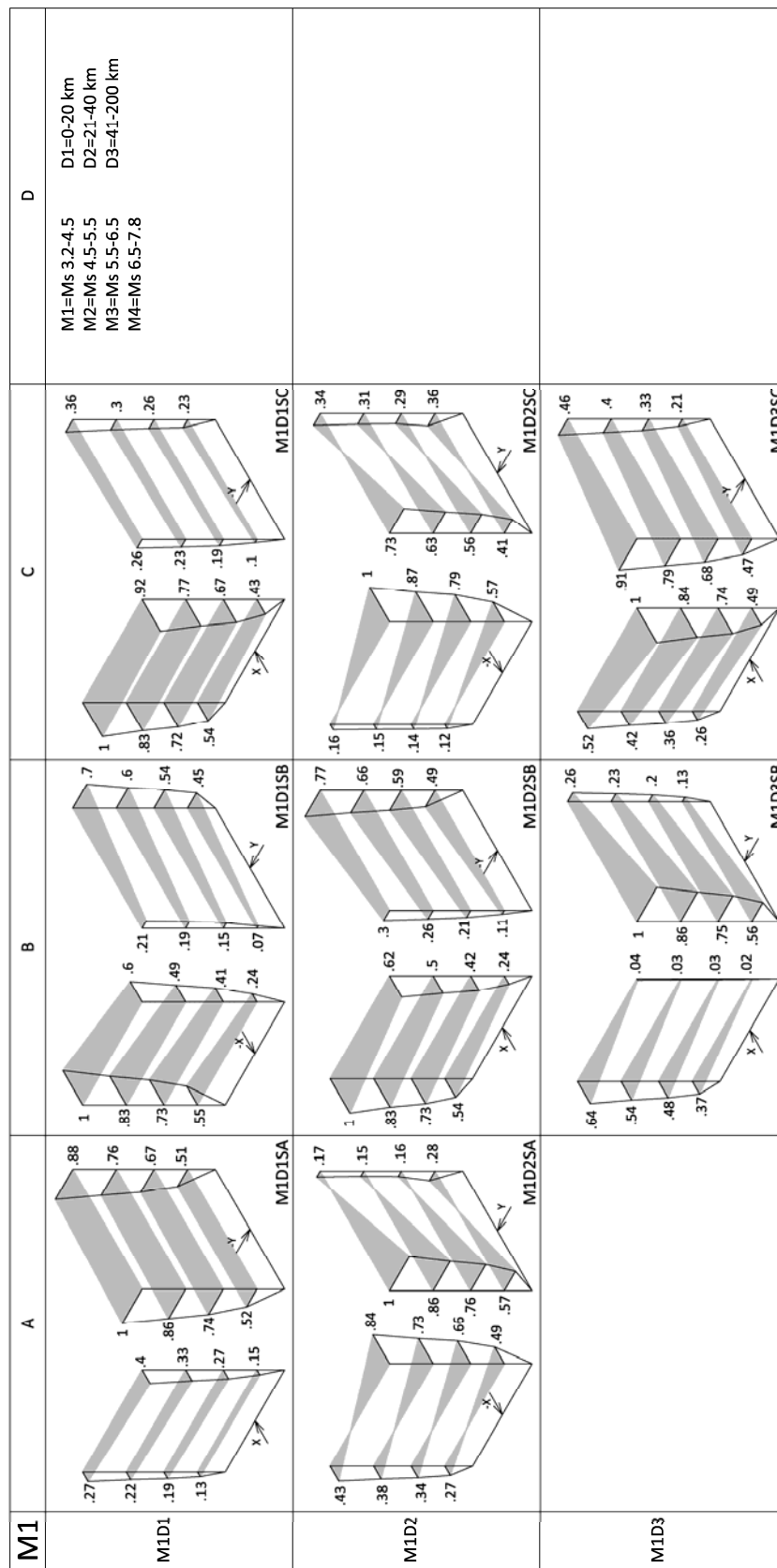


Fig. C4 (a): Load Patterns for STR2-Infill (Magnitude Range M1)

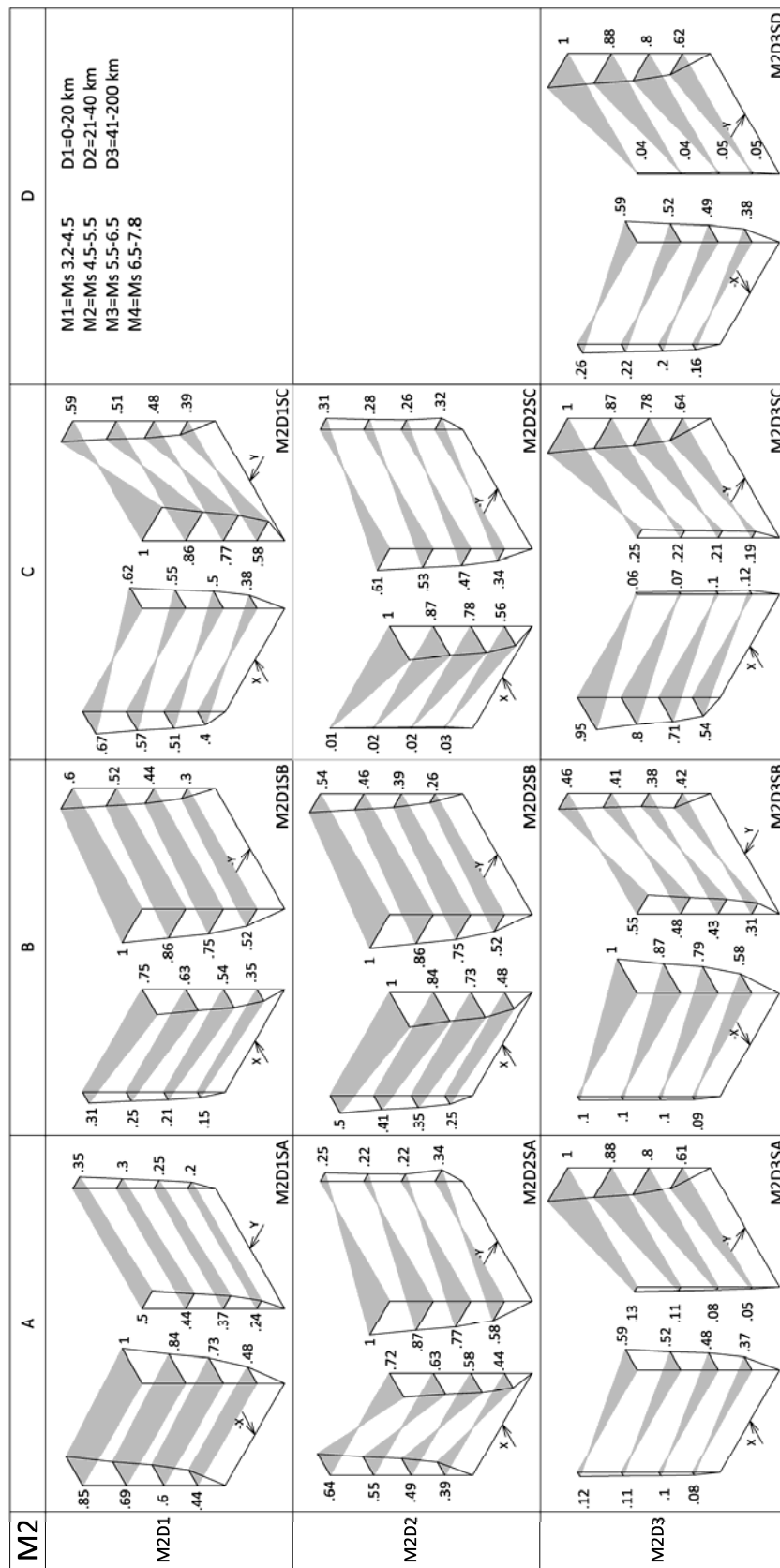


Fig. C4 (b): Load Patterns for STR2-Infill (Magnitude Range M2)

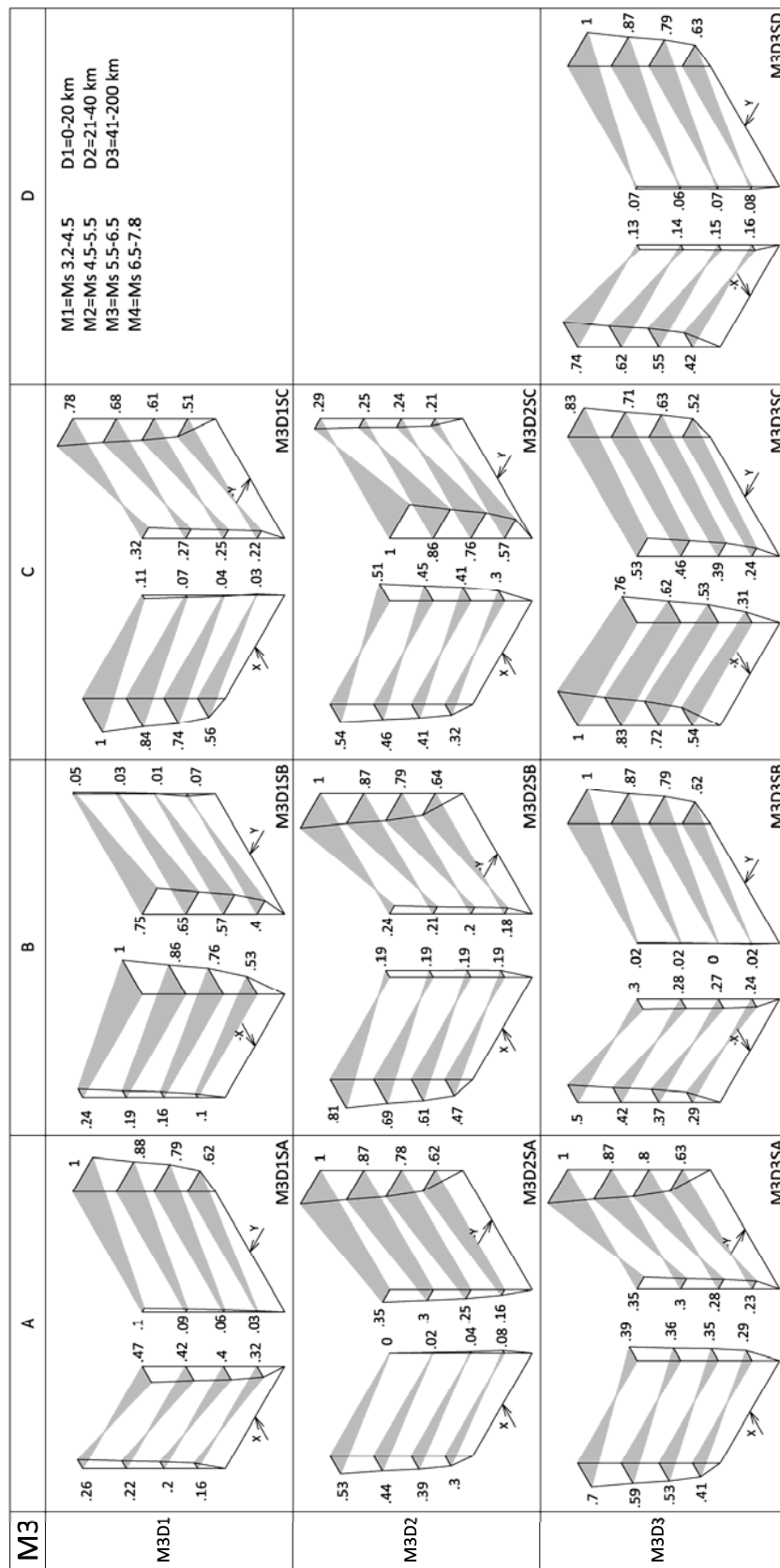


Fig. C4 (c): Load Patterns for STR2-Infill (Magnitude Range M3)

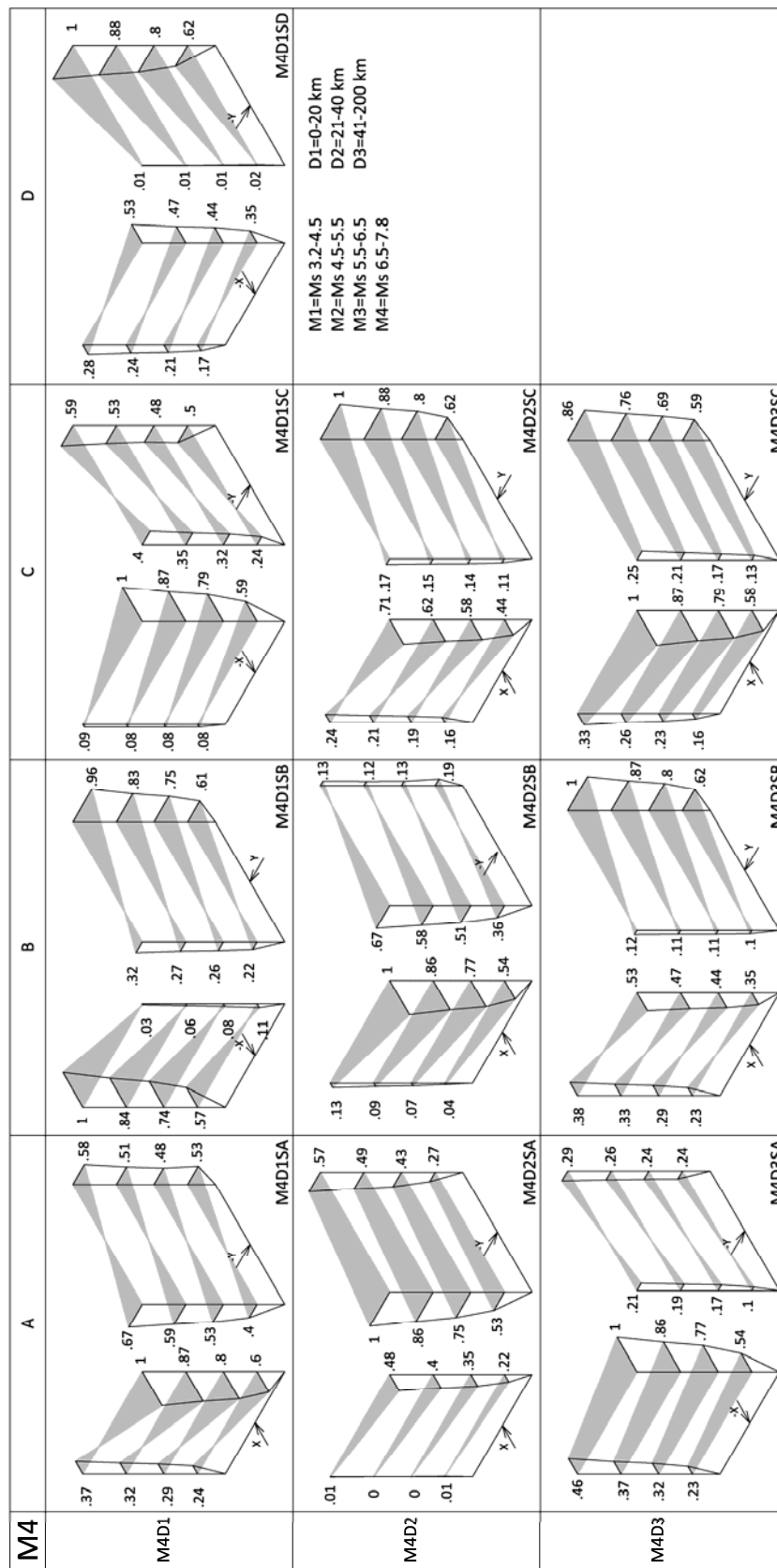


Fig. C4 (d): Load Patterns for STR2-Infill (Magnitude Range M4)

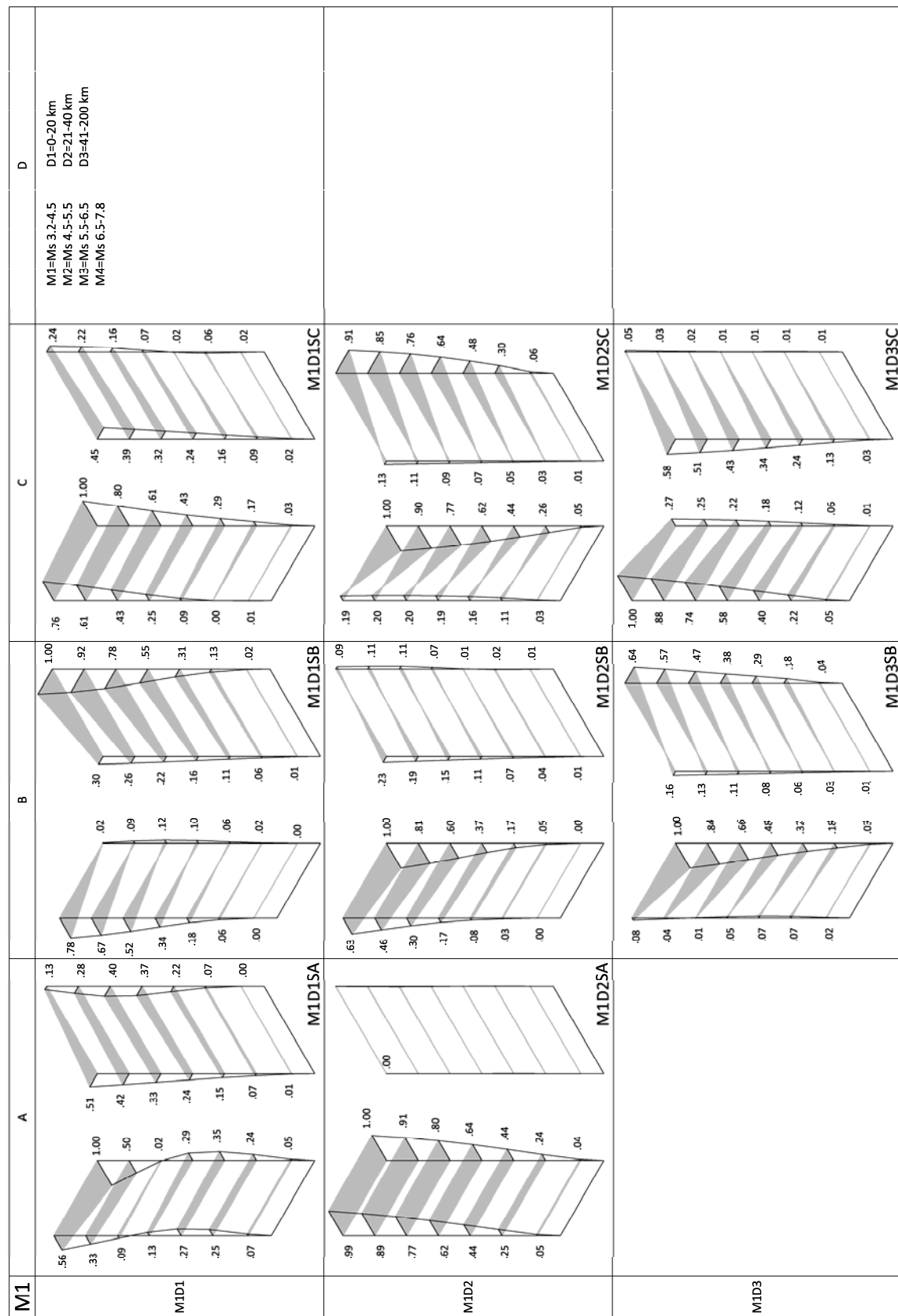


Fig. C5 (a): Load Patterns for STR3 (Magnitude Range M1)

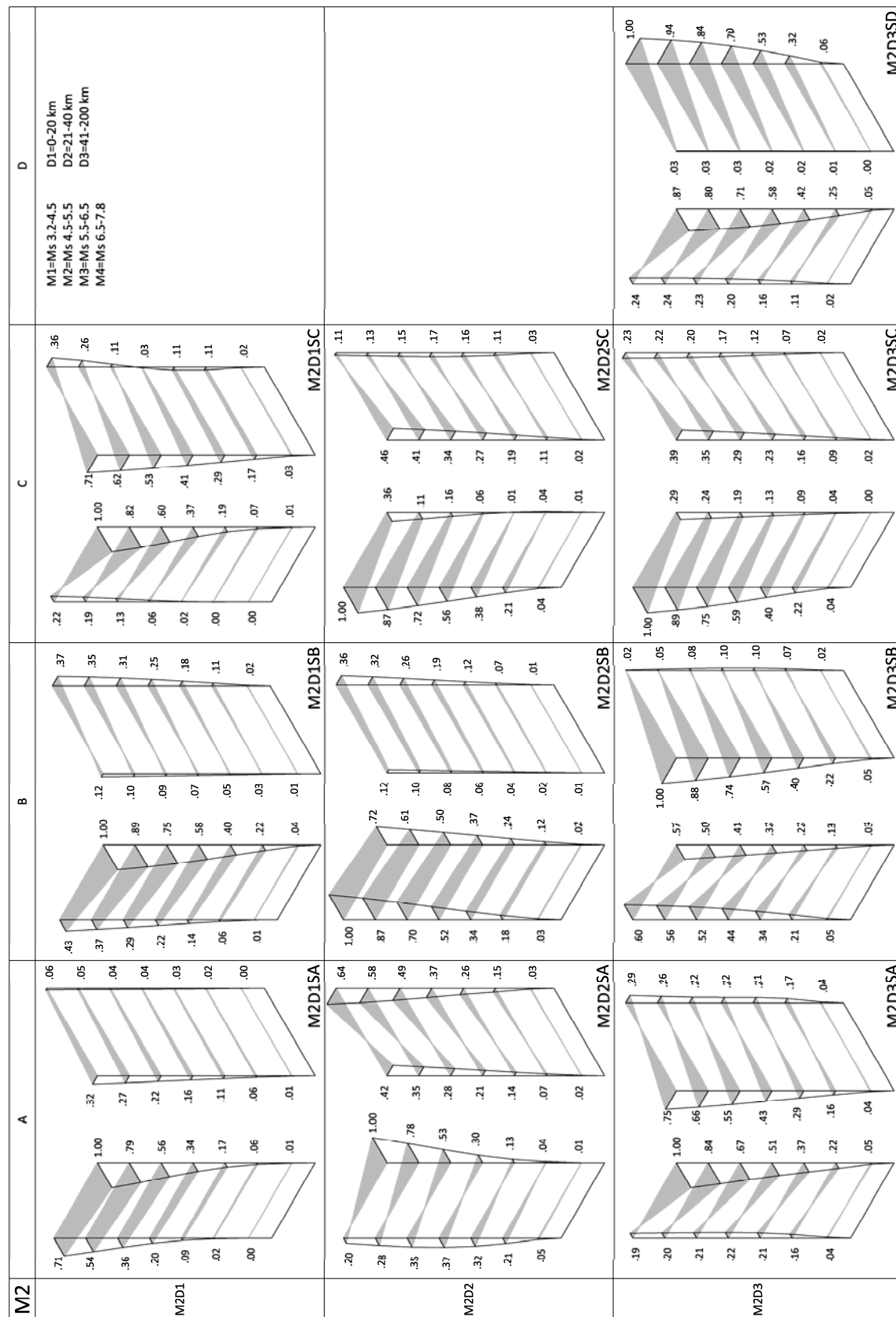


Fig. C5 (b): Load Patterns for STR3 (Magnitude Range M2)

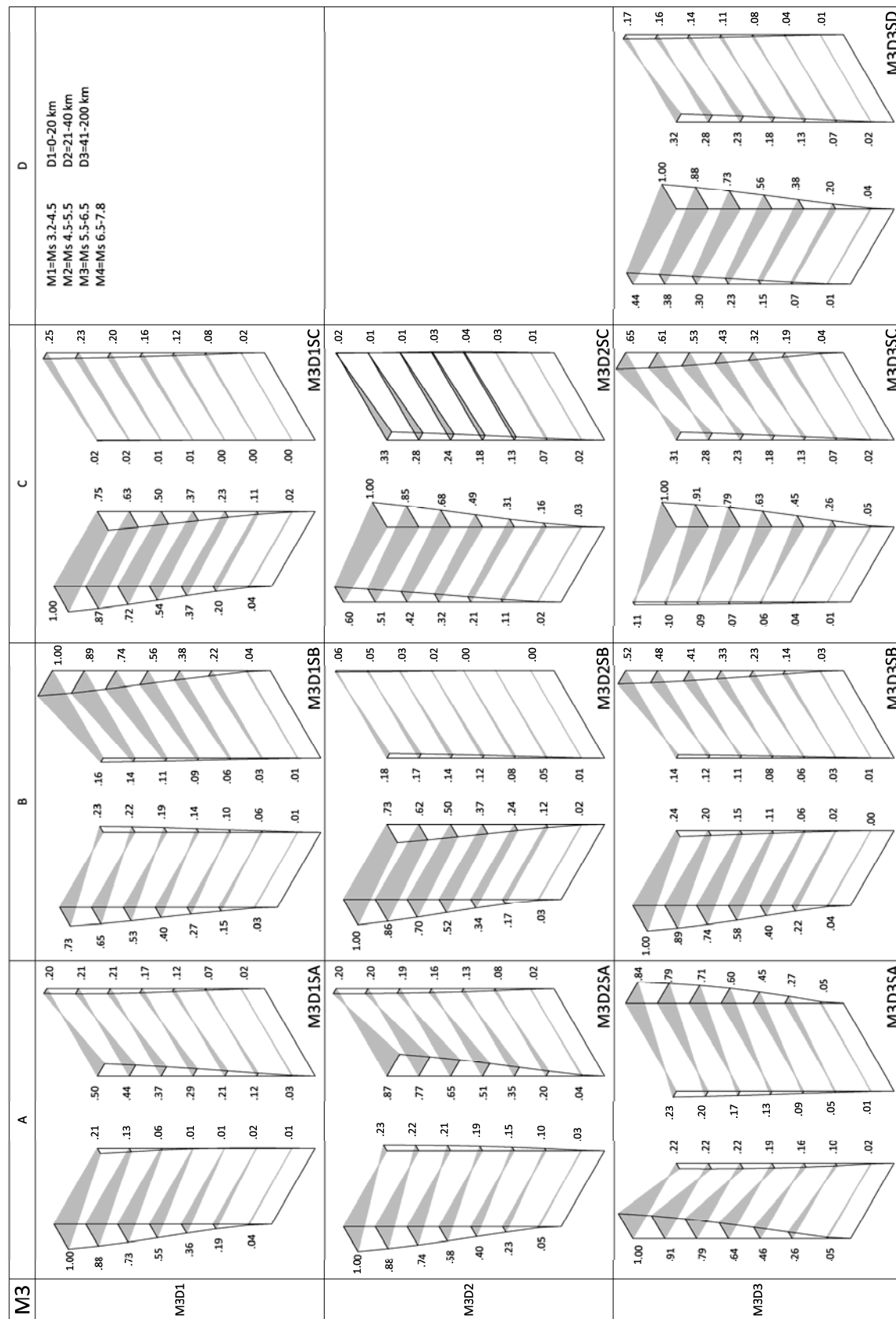


Fig. C5 (c): Load Patterns for STR3 (Magnitude Range M3)

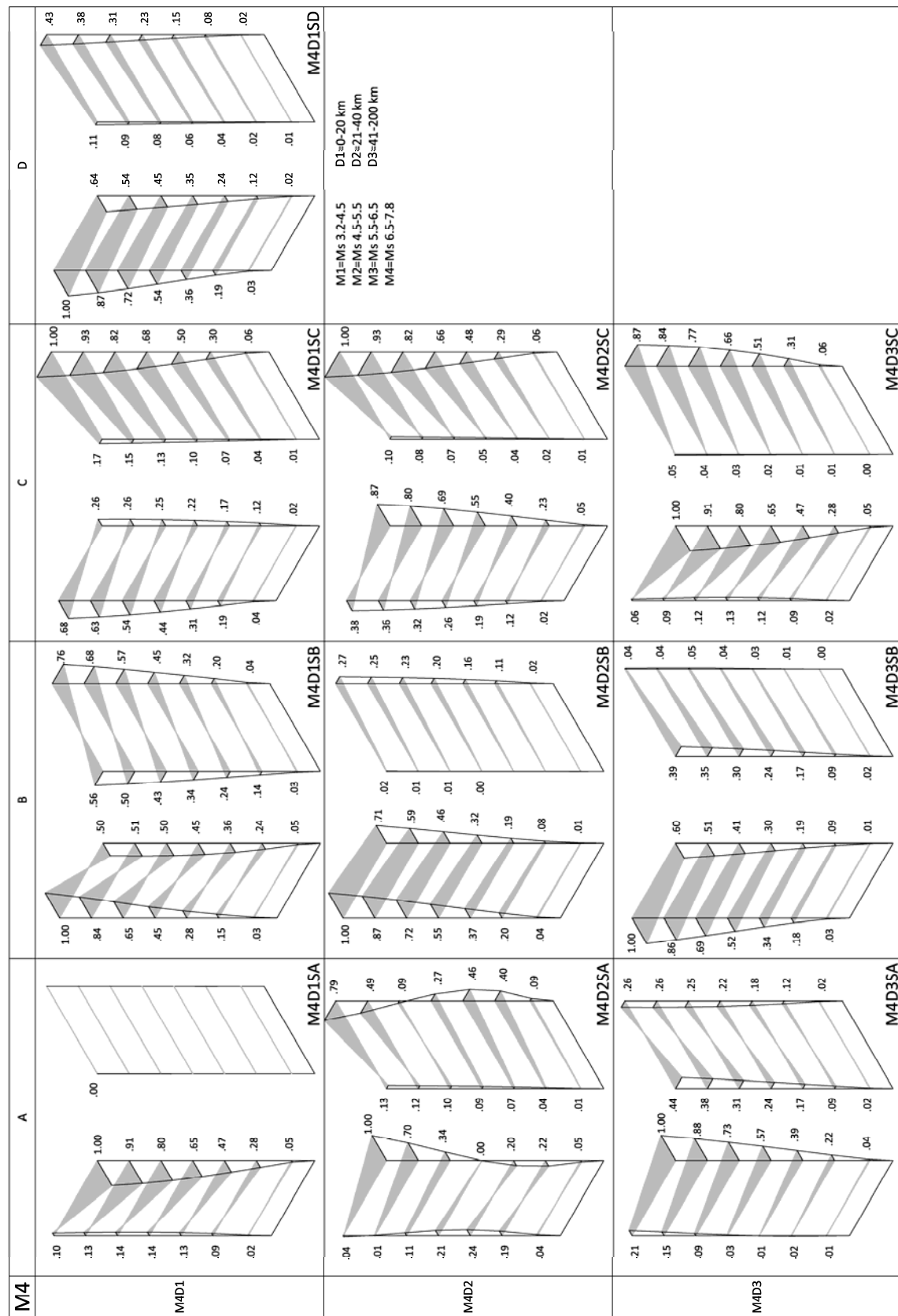


Fig. C5 (d): Load Patterns for STR3 (Magnitude Range M4)

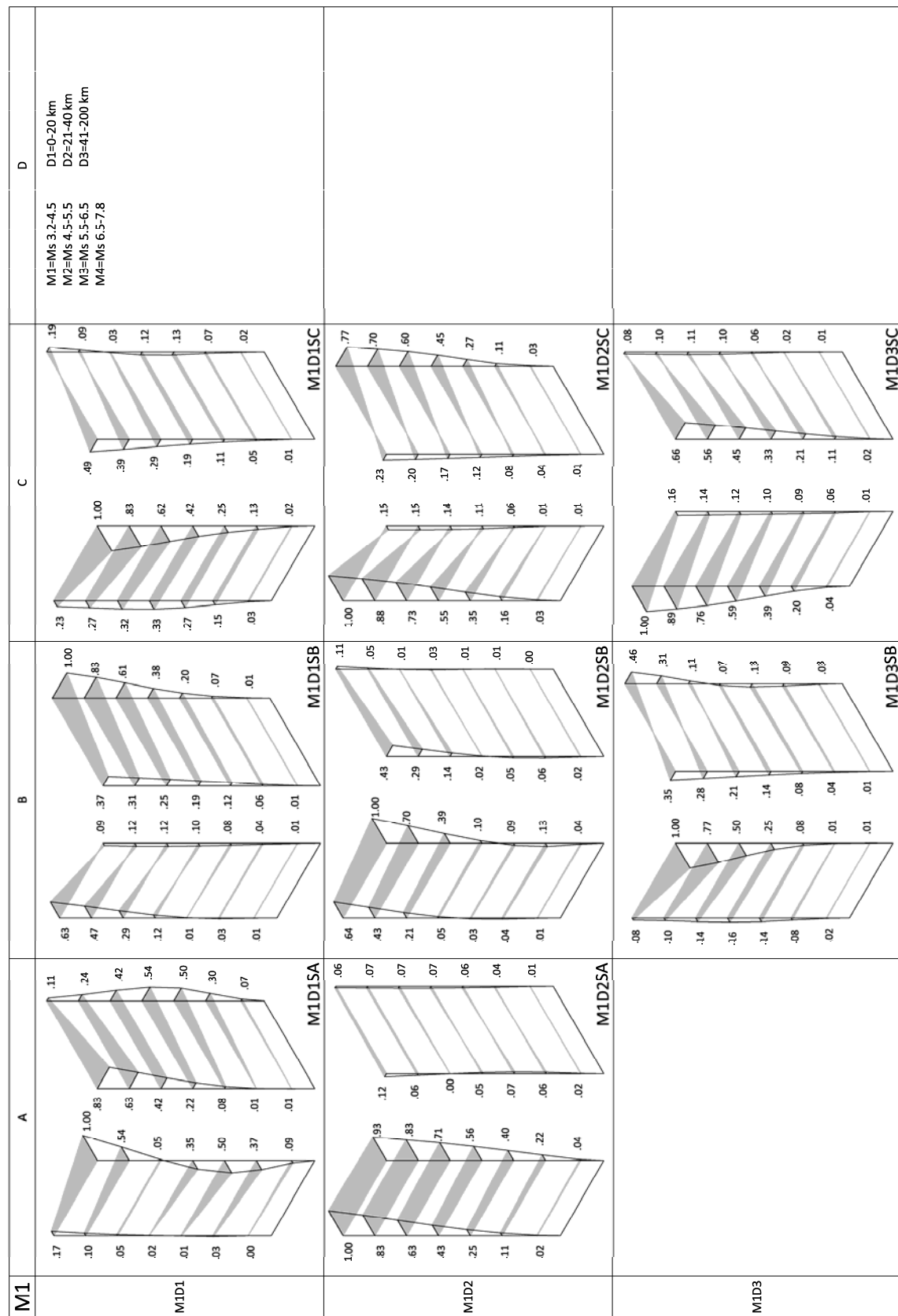


Fig. C6 (a): Load Patterns for STR3-Infill (Magnitude Range M1)

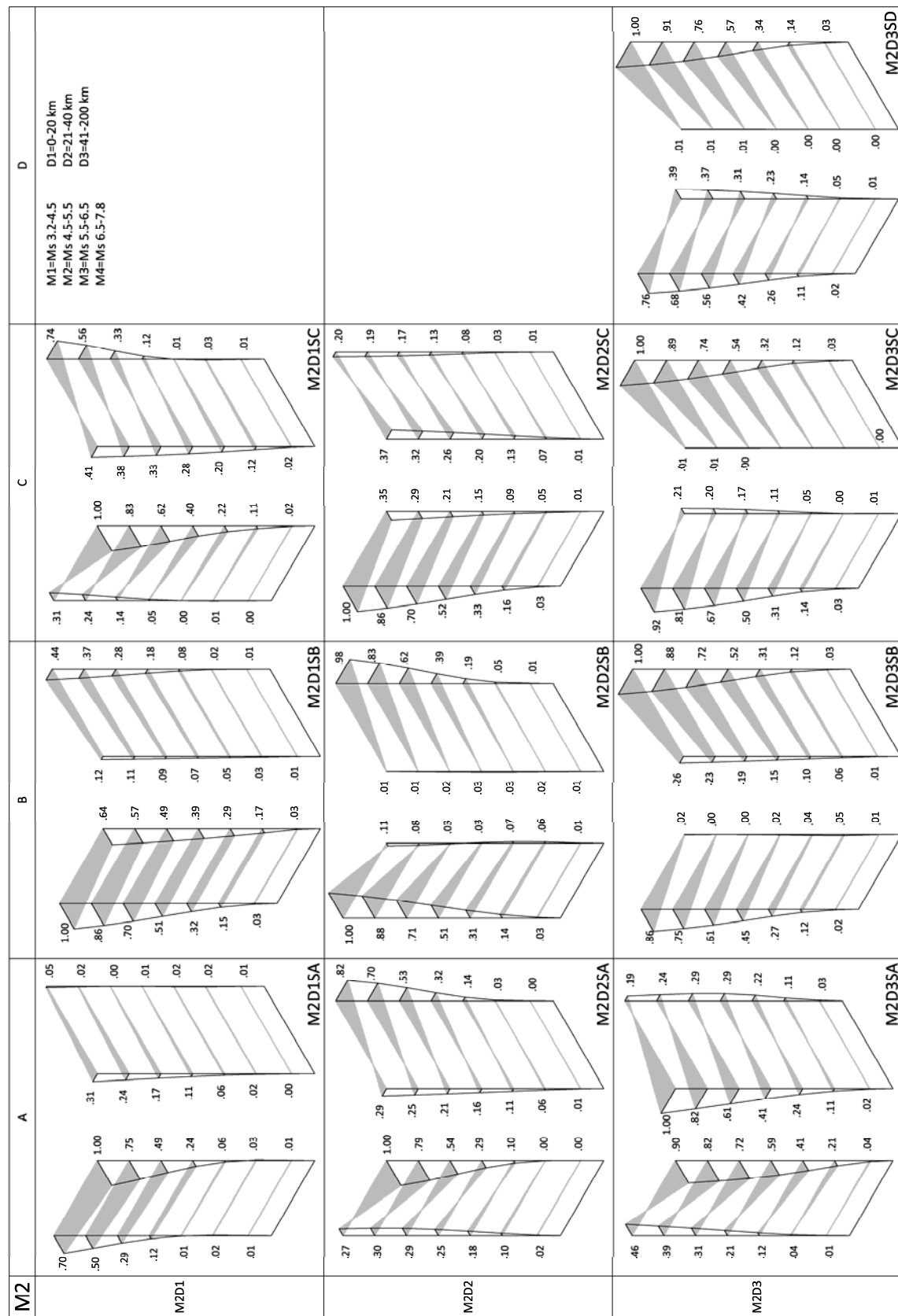


Fig. C6 (b): Load Patterns for STR3-Infill (Magnitude Range M2)

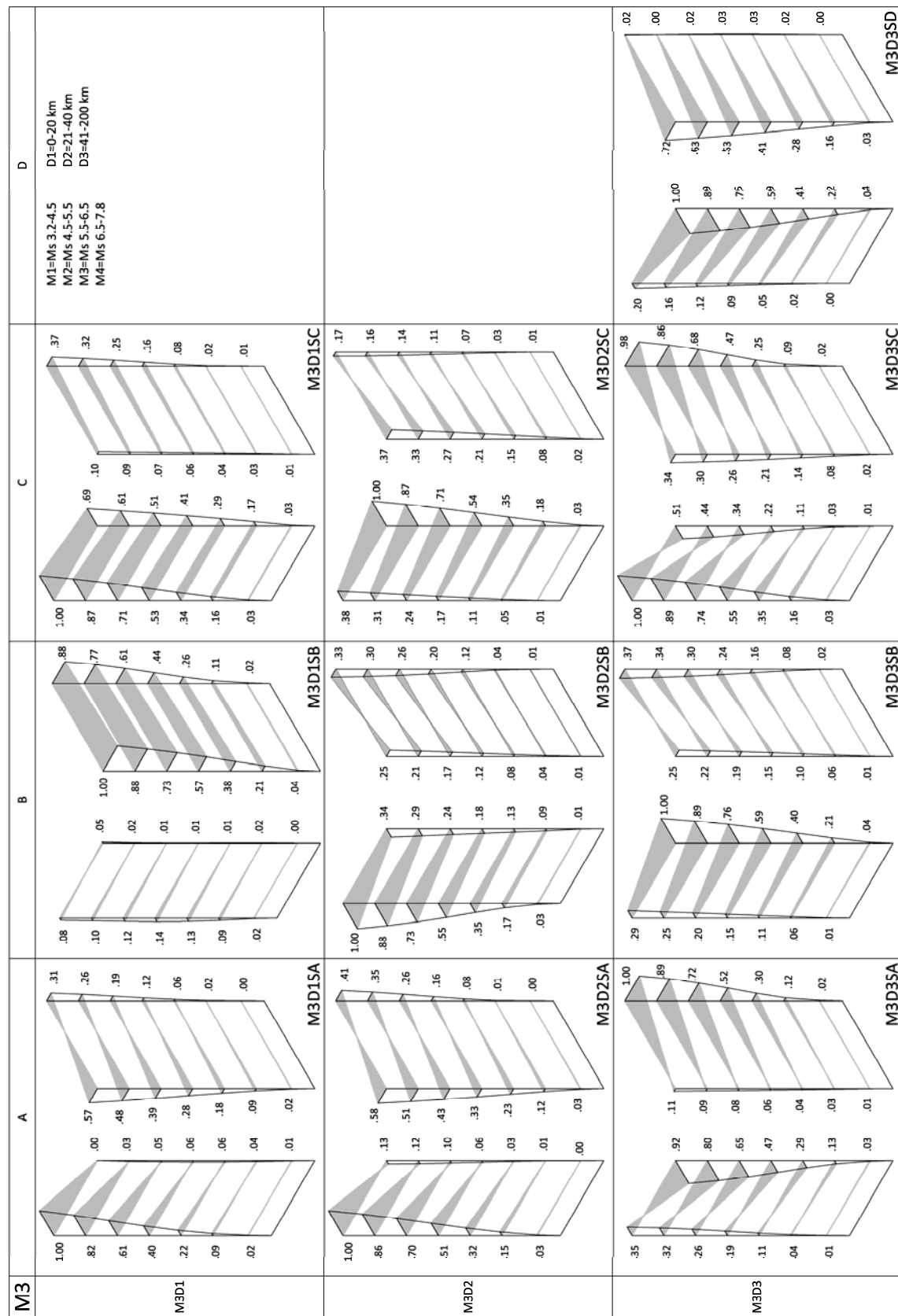


Fig. C6 (c): Load Patterns for STR3-Infill (Magnitude Range M3)

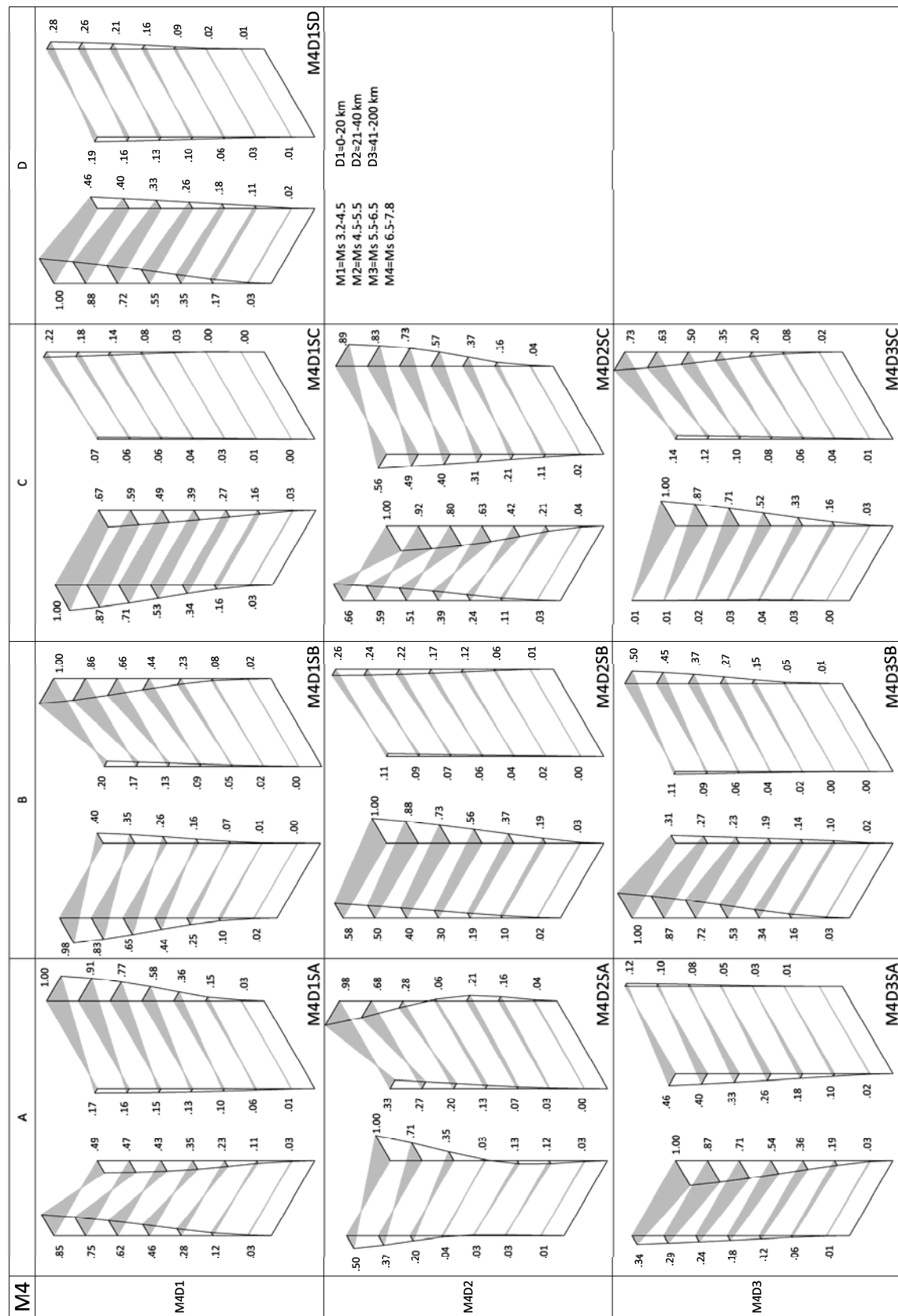


Fig. C6 (d): Load Patterns for STR3-Infill (Magnitude Range M4)

ANNEXURE D

DAMAGE DISTRIBUTION FOR DIFFERENT LOAD PATTTTERNS

This part is an annexure of section [5.2](#) of the main text. Damage distribution for all the structures is given. Fig. D1, Fig. D2 and Fig. D3 show damage distribution for STR1, STR2 and STR3 respectively.

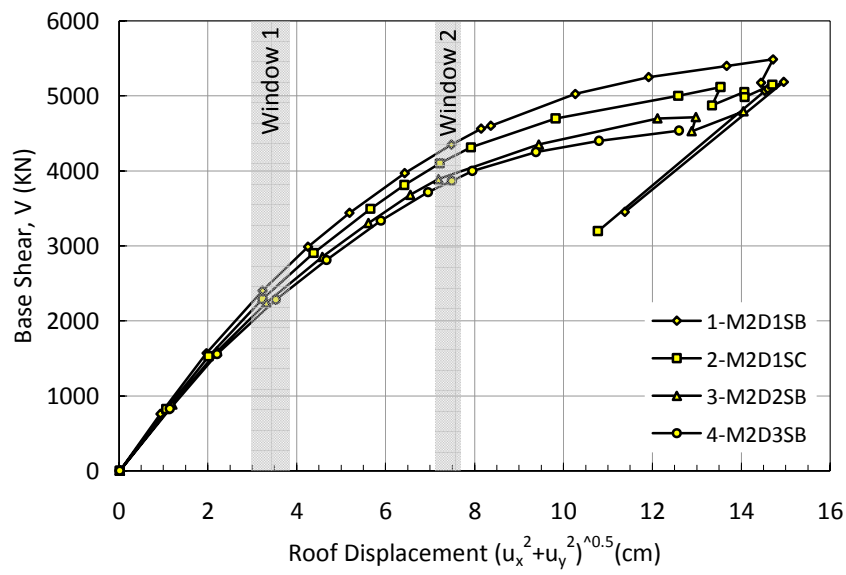


Fig. D1 (a): Capacity Curve of STR1-Infill

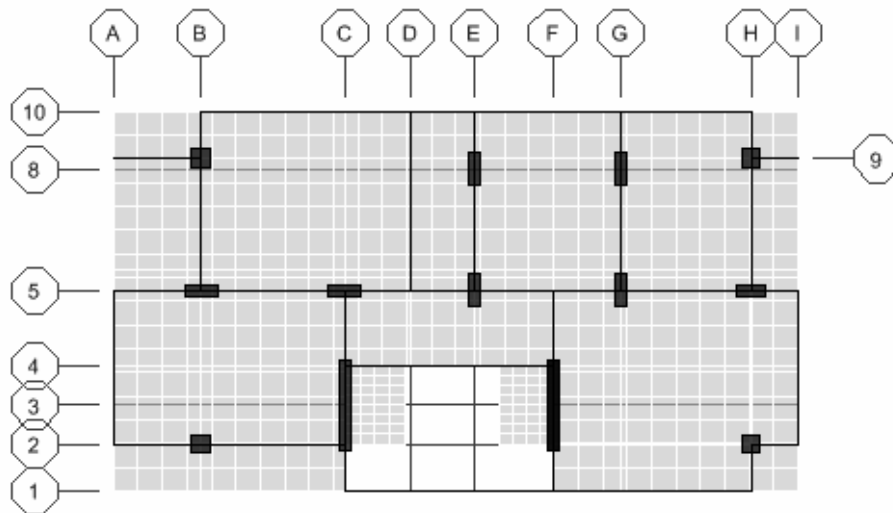


Fig. D1 (b): Reference Plan and Elevation Lines of STR1

Fig. D1 (c): Damage Distribution for STR1-Infill (Roof Displacement 3.2-3.5 cm, window 1)

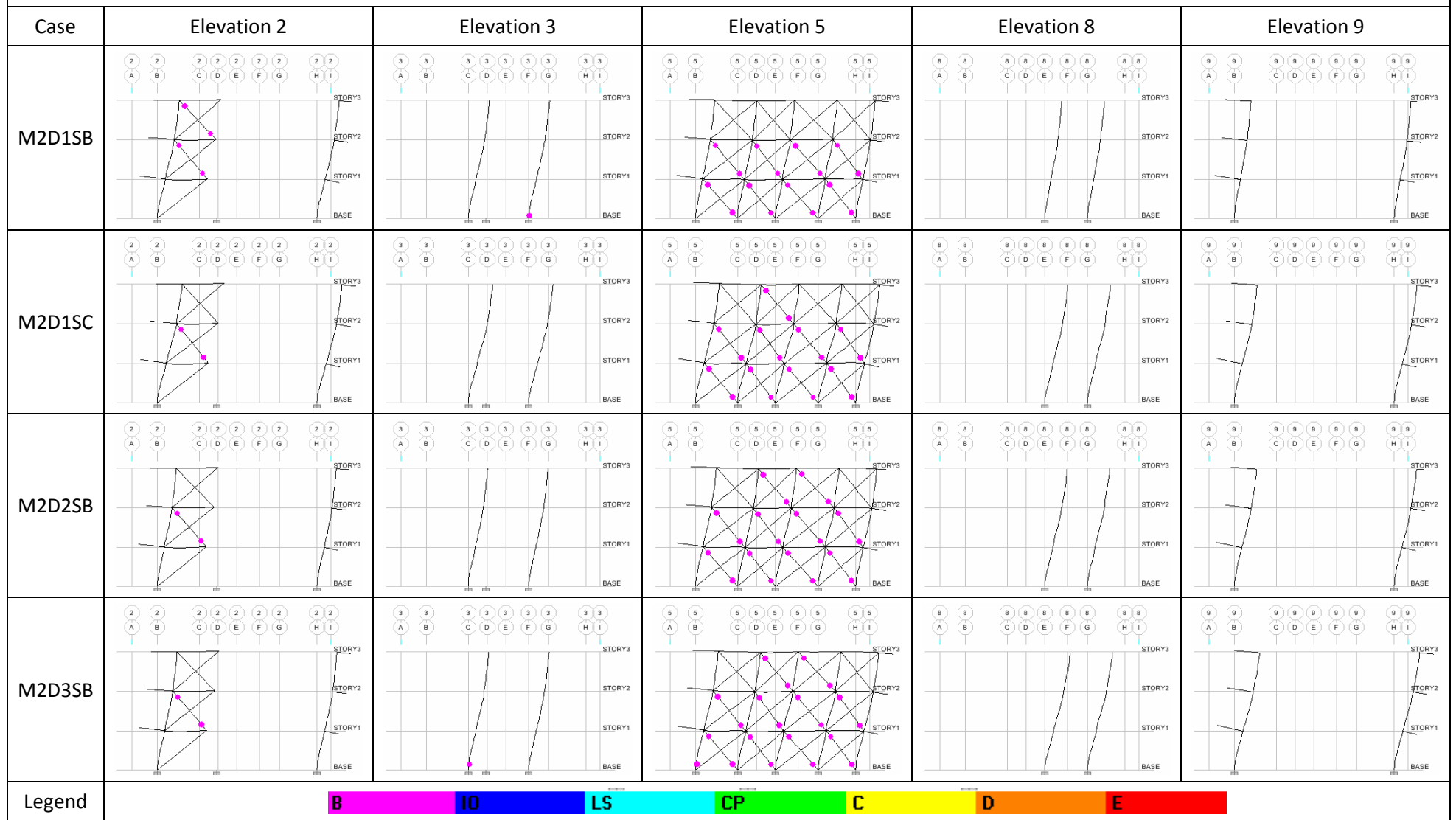


Fig. D1 (d): Damage Distribution for STR1-Infill (Roof Displacement 7.2-7.5 cm, window 2)

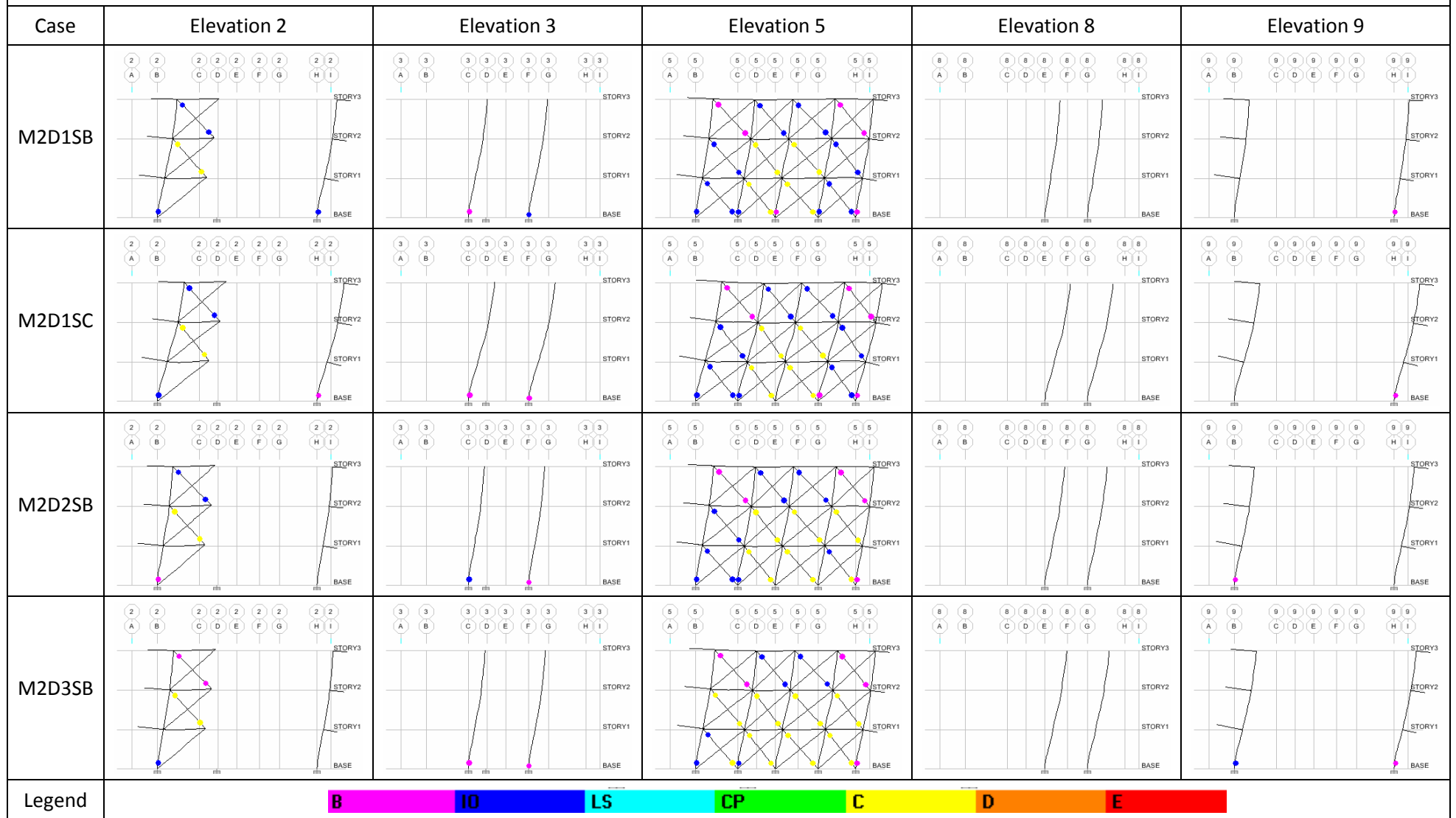
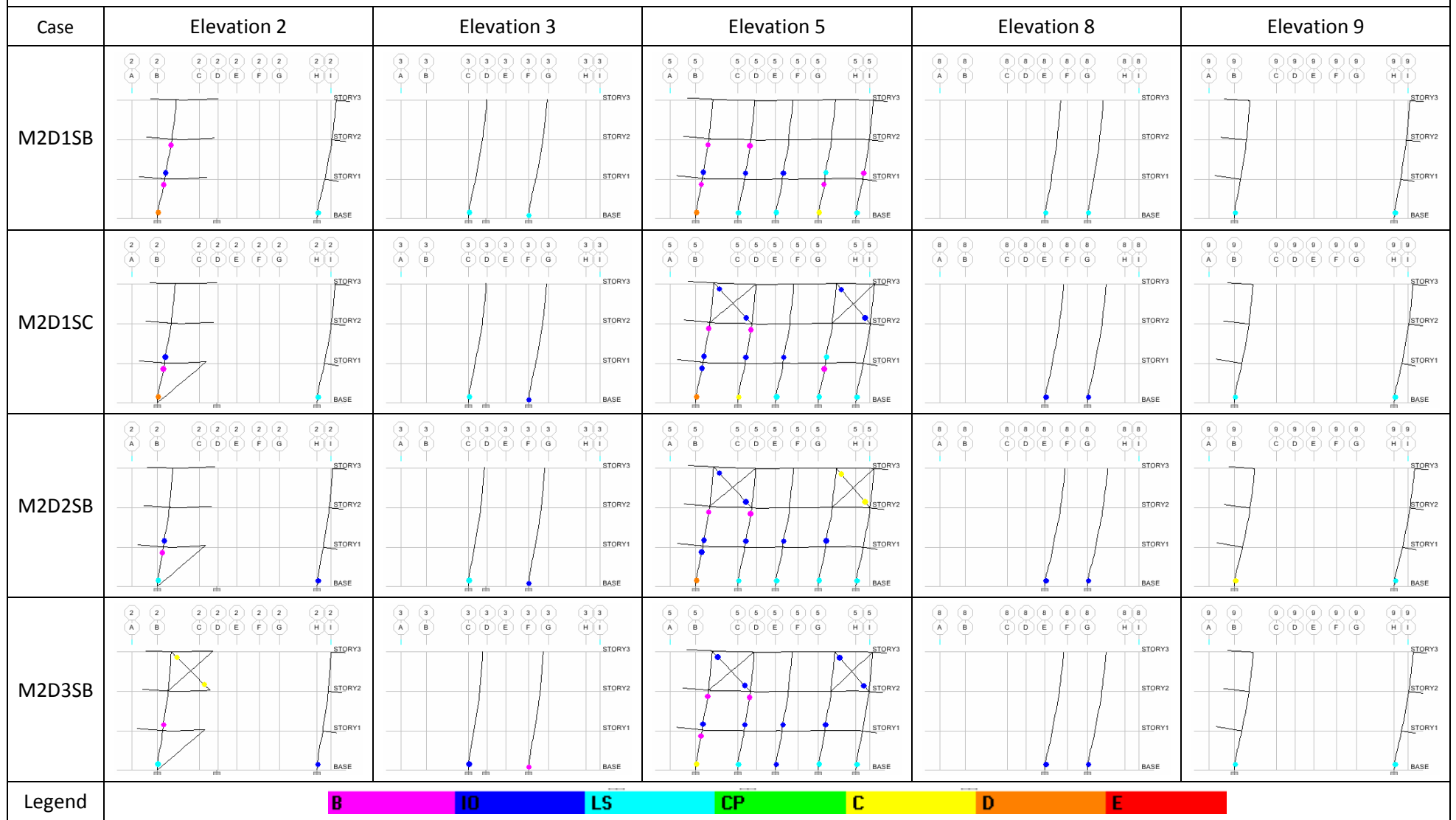


Fig. D1 (e): Damage Distribution for STR1-Infill (at ultimate state)



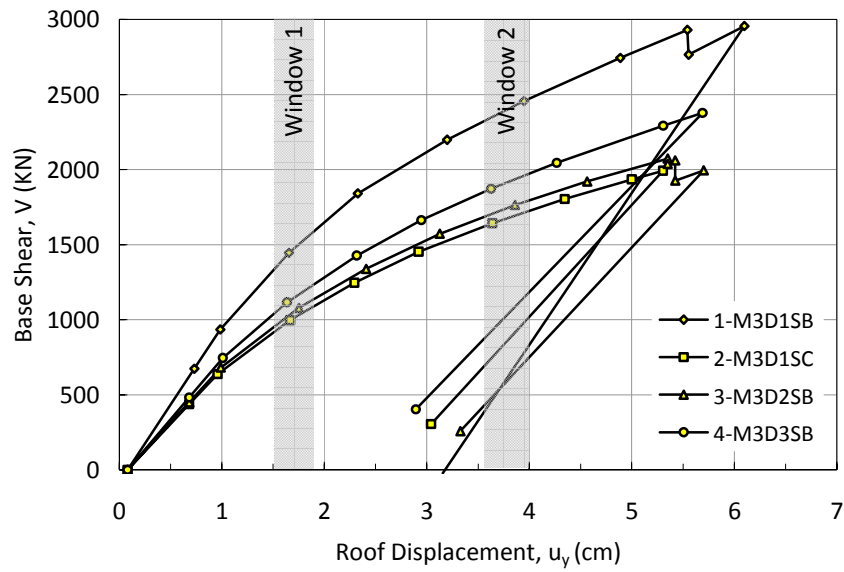


Fig. D2 (a): Capacity Curve of STR2

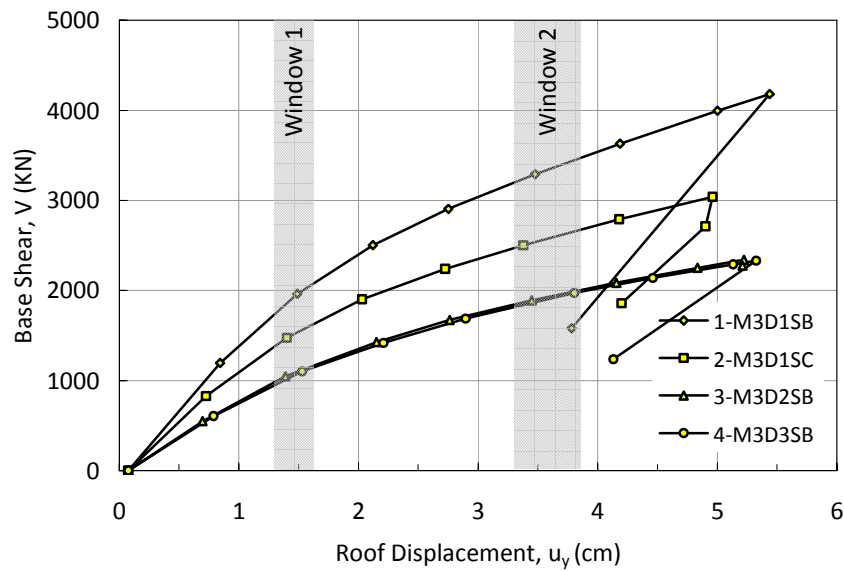


Fig. D2 (b): Capacity Curve of STR2-Infill

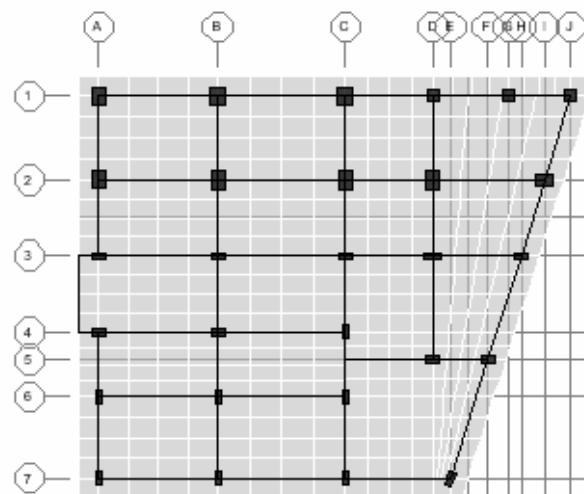


Fig. D2 (c): Plan and Reference Elevation Lines of STR2

Fig. D2 (d): Damage Distribution for STR2 (Roof Displacement cm, window 1)

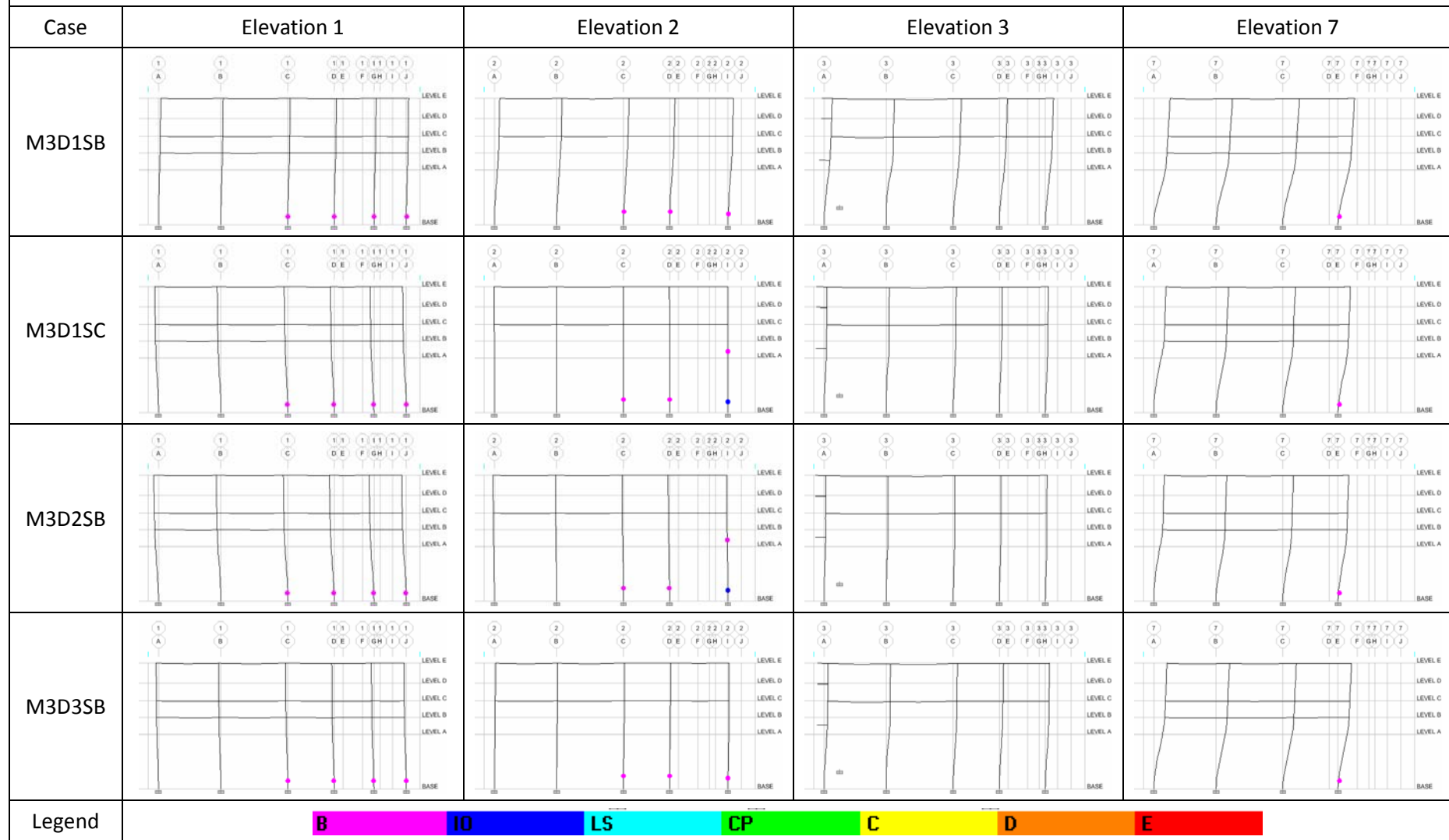


Fig. D2 (e): Damage Distribution for STR2 (Roof Displacement cm, window 2)

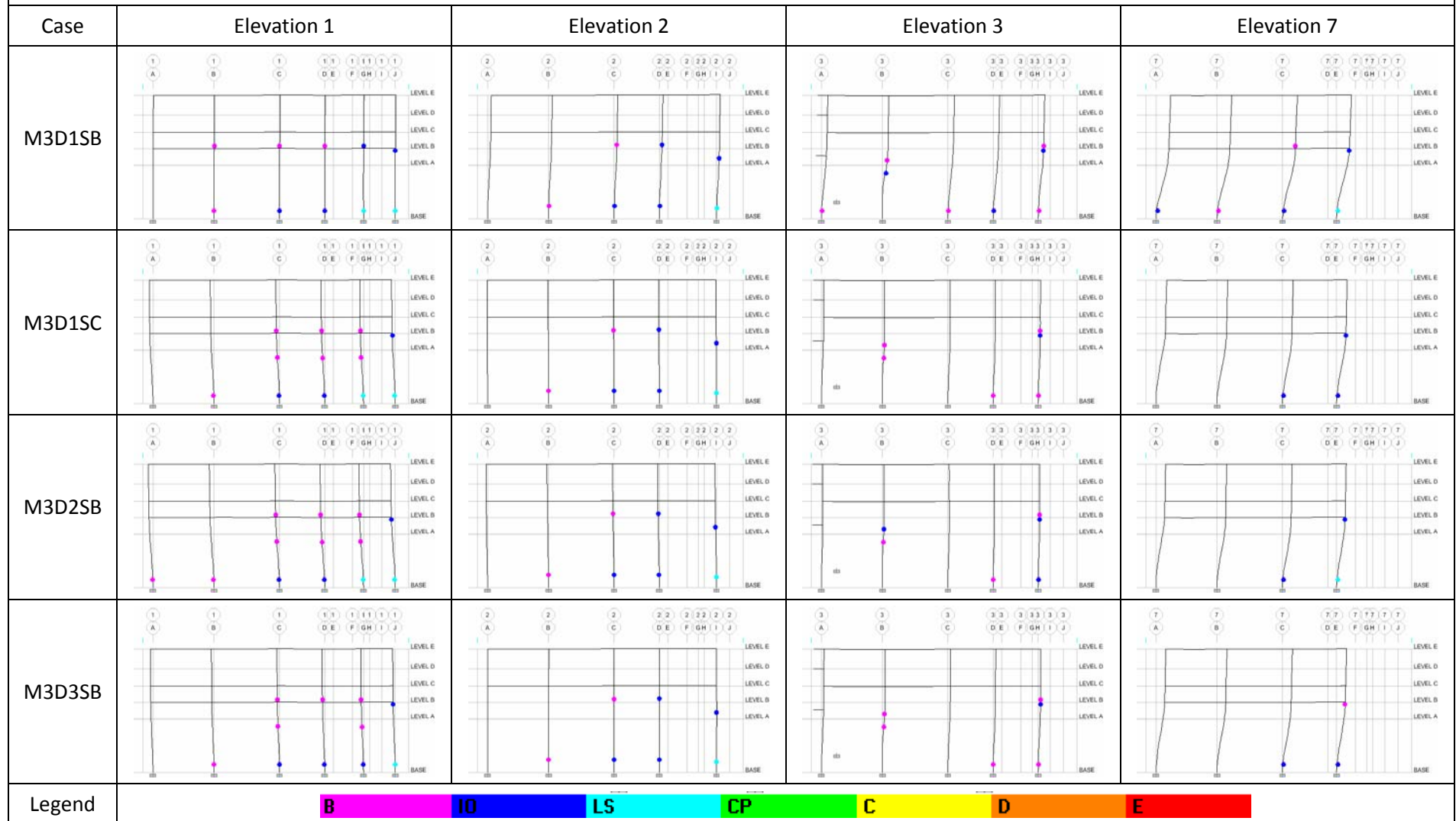


Fig. D2 (f): Damage Distribution for STR2 (at ultimate state)

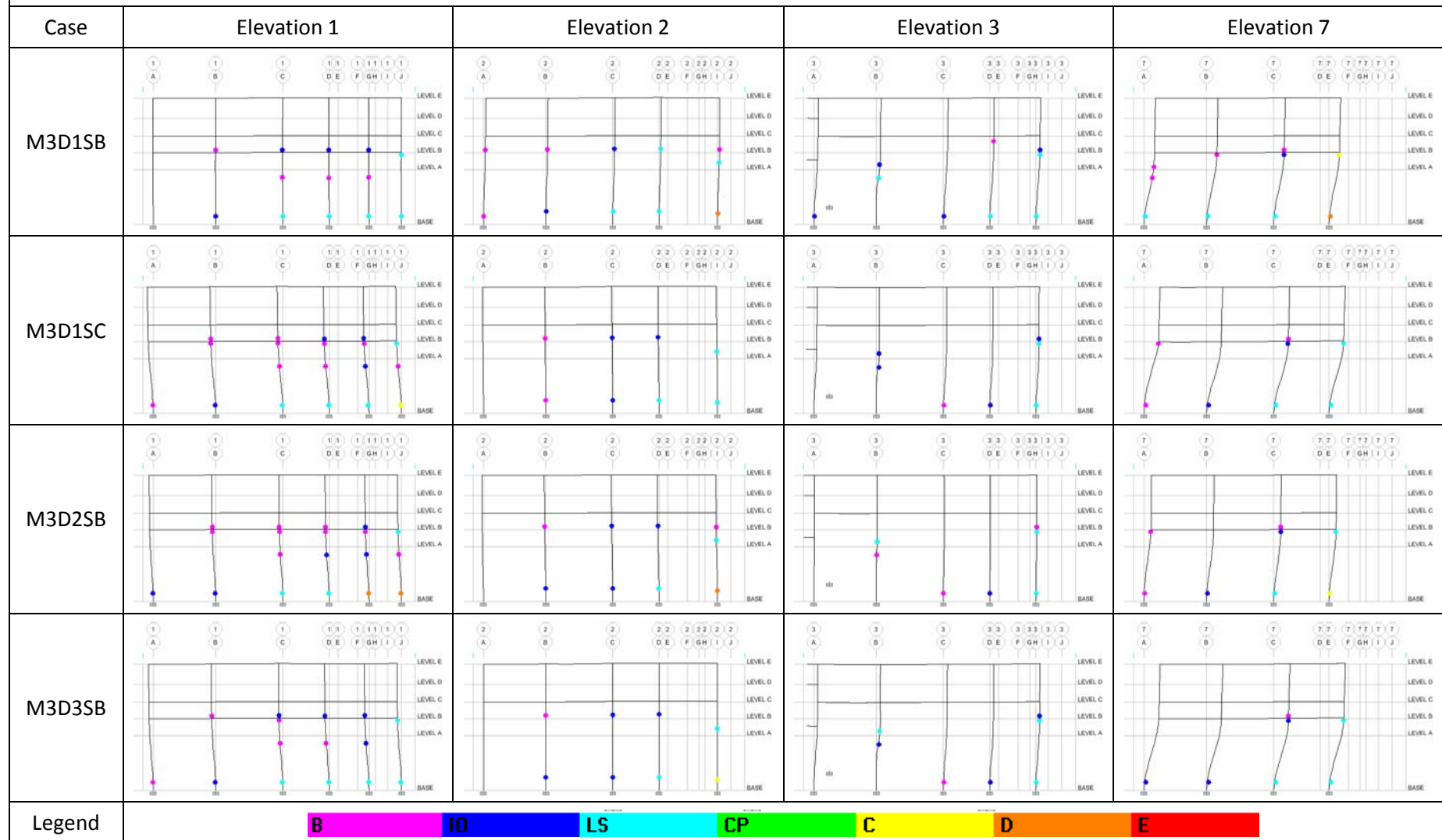


Fig. D3 (g): Damage Distribution for STR2-Infill (Roof Displacement cm, window 1)

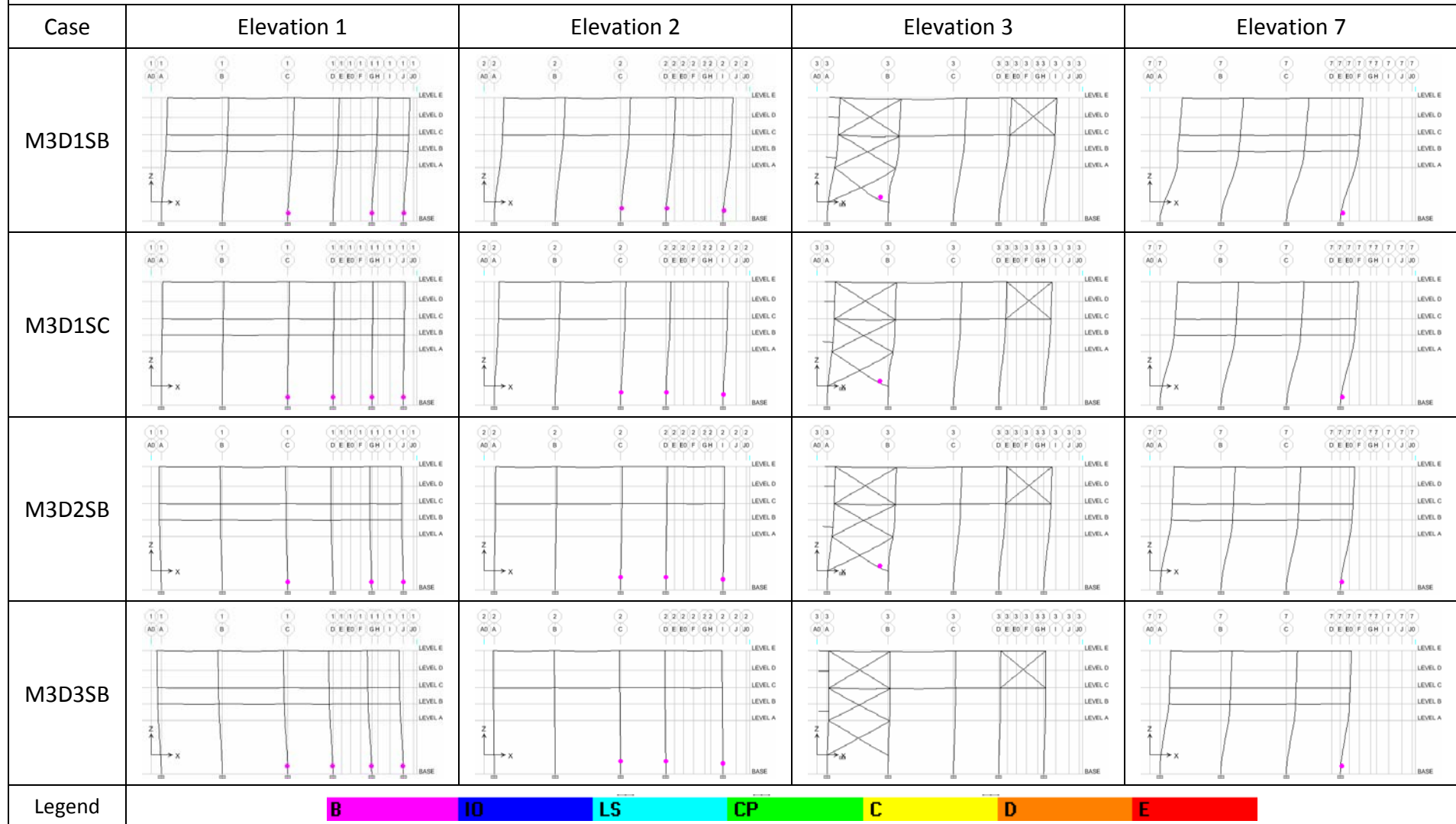


Fig. D3 (h): Damage Distribution for STR2-Infill (Roof Displacement cm, window 2)

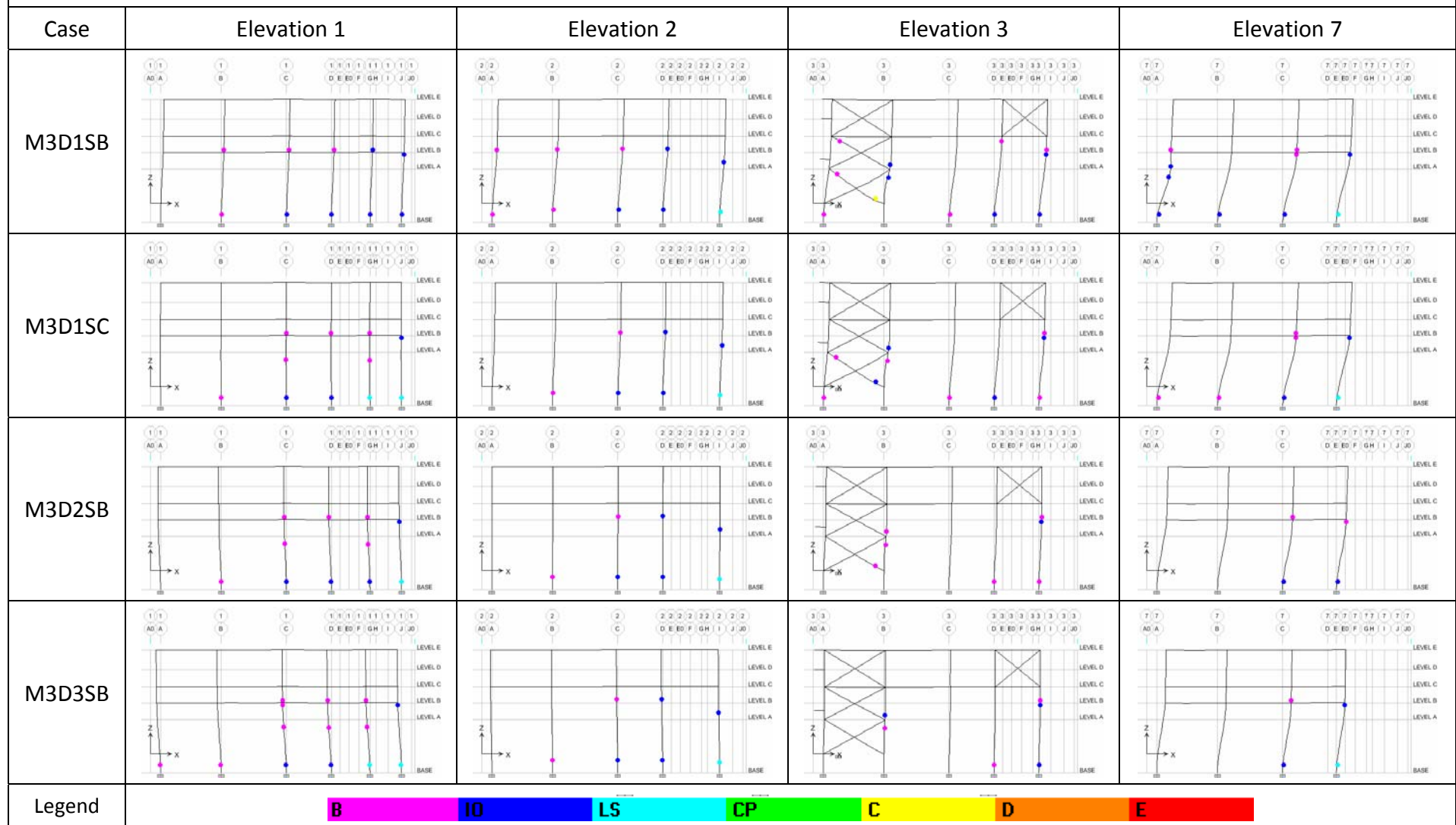
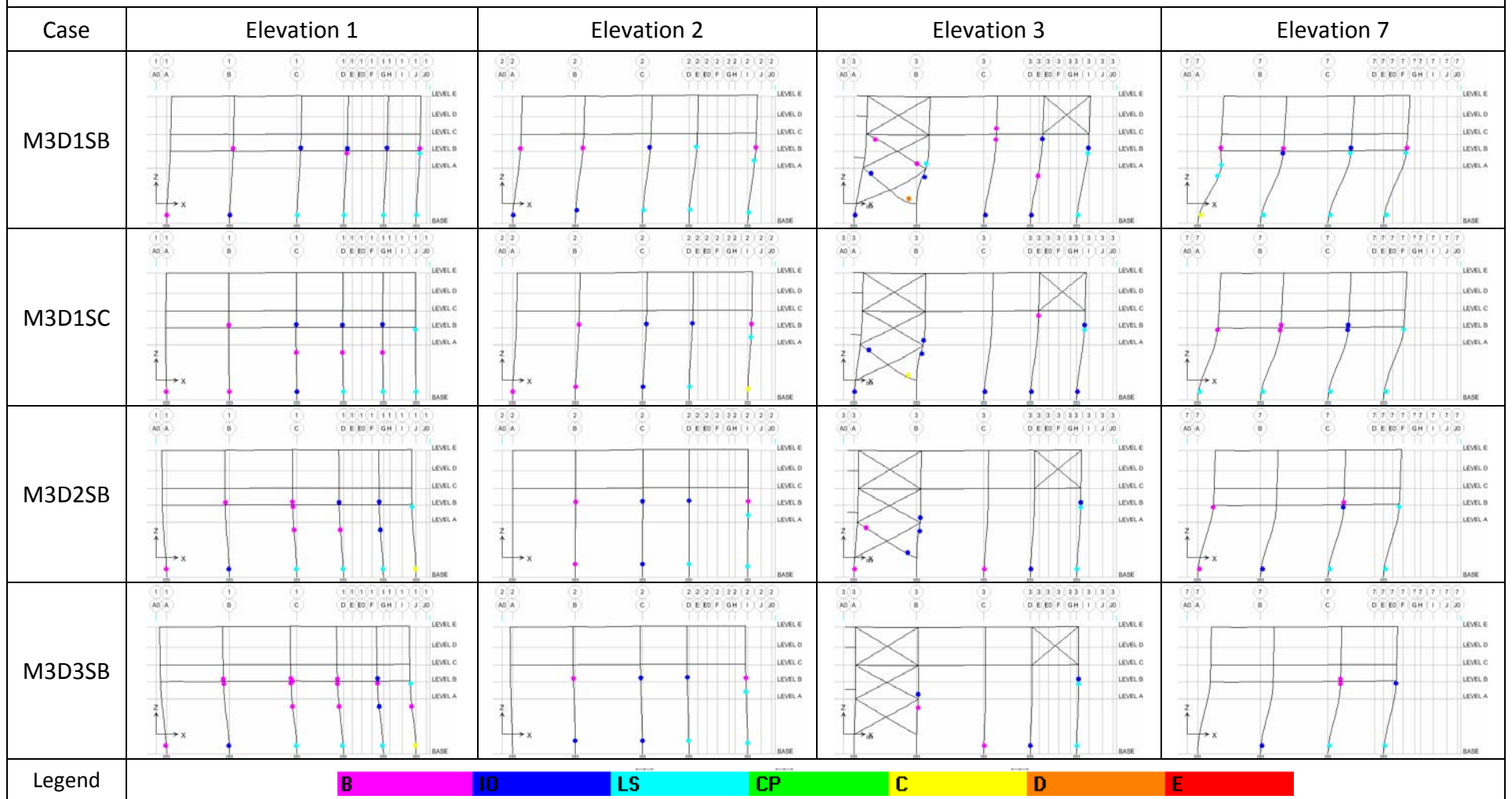


Fig. D3 (i): Damage Distribution for STR2-Infill (at ultimate state)



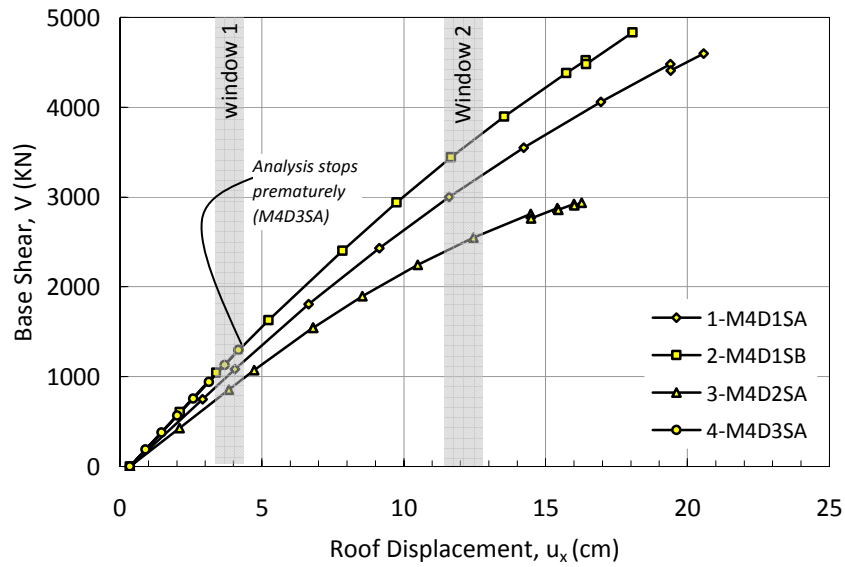


Fig. D3 (a): Capacity Curve of STR3

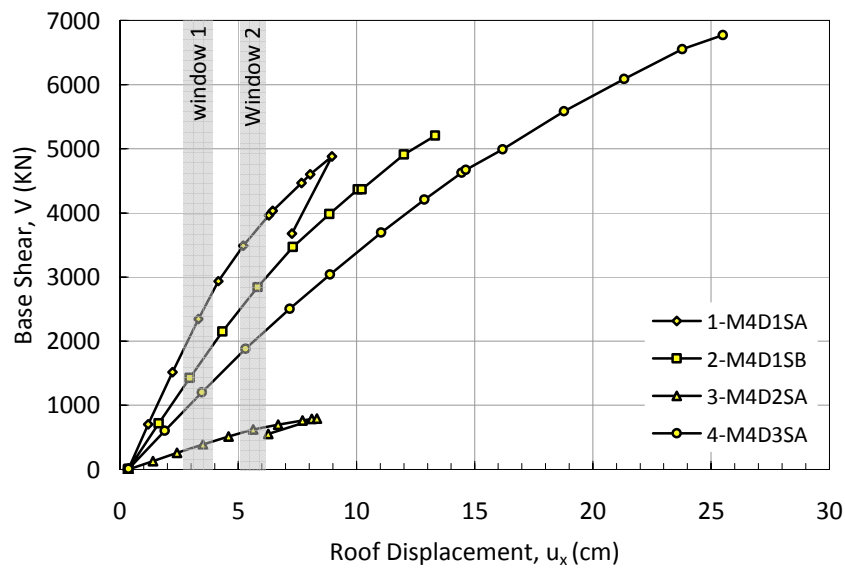


Fig. D3 (b): Capacity Curve of STR3-Infill

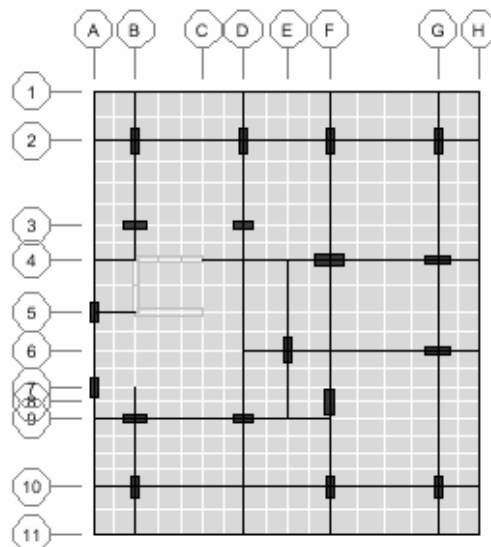


Fig. D3 (c): Plan and Reference Elevation Lines of STR3

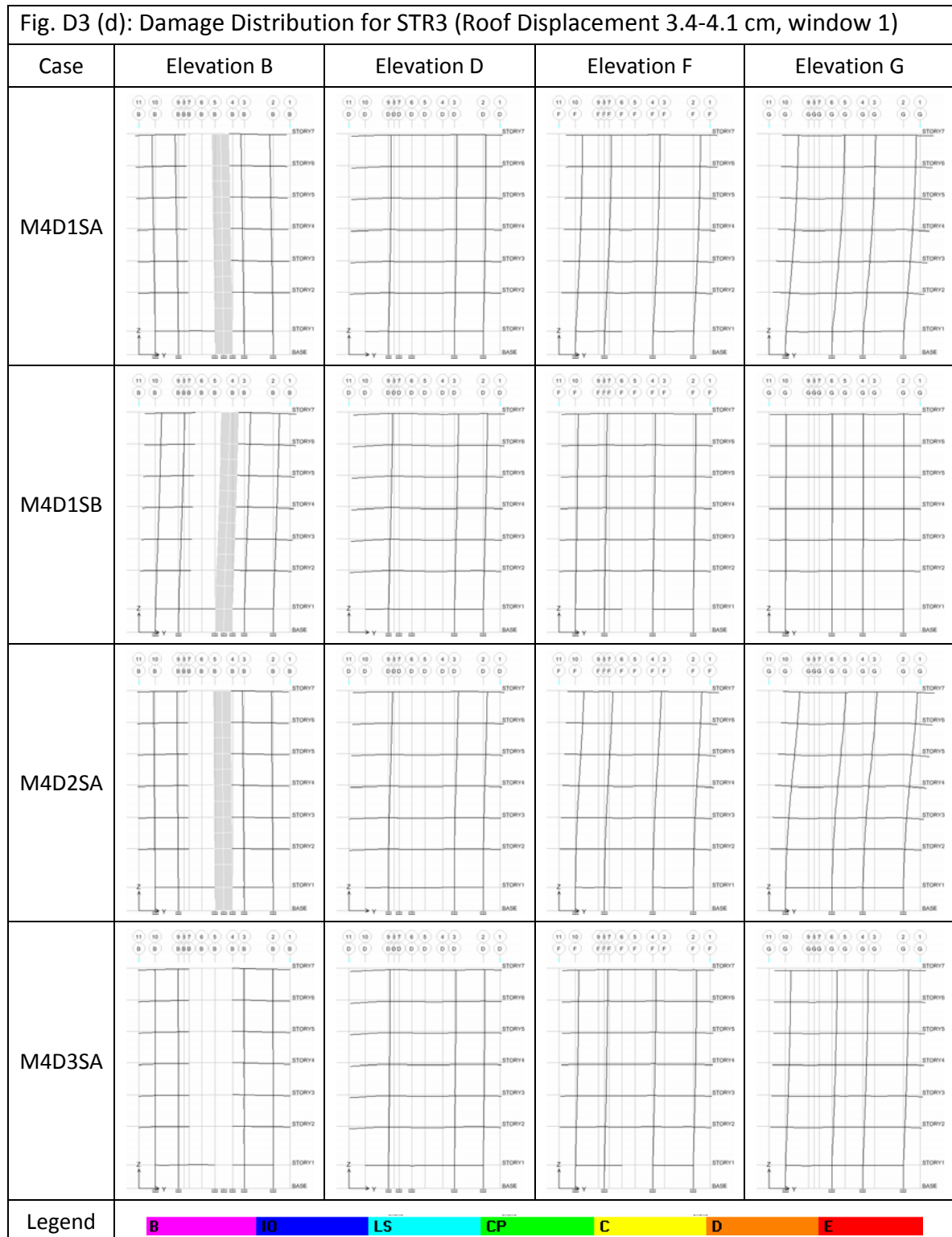


Fig. D3 (e): Damage Distribution for STR3 (Roof Displacement 11.6-12.5 cm, window 2)

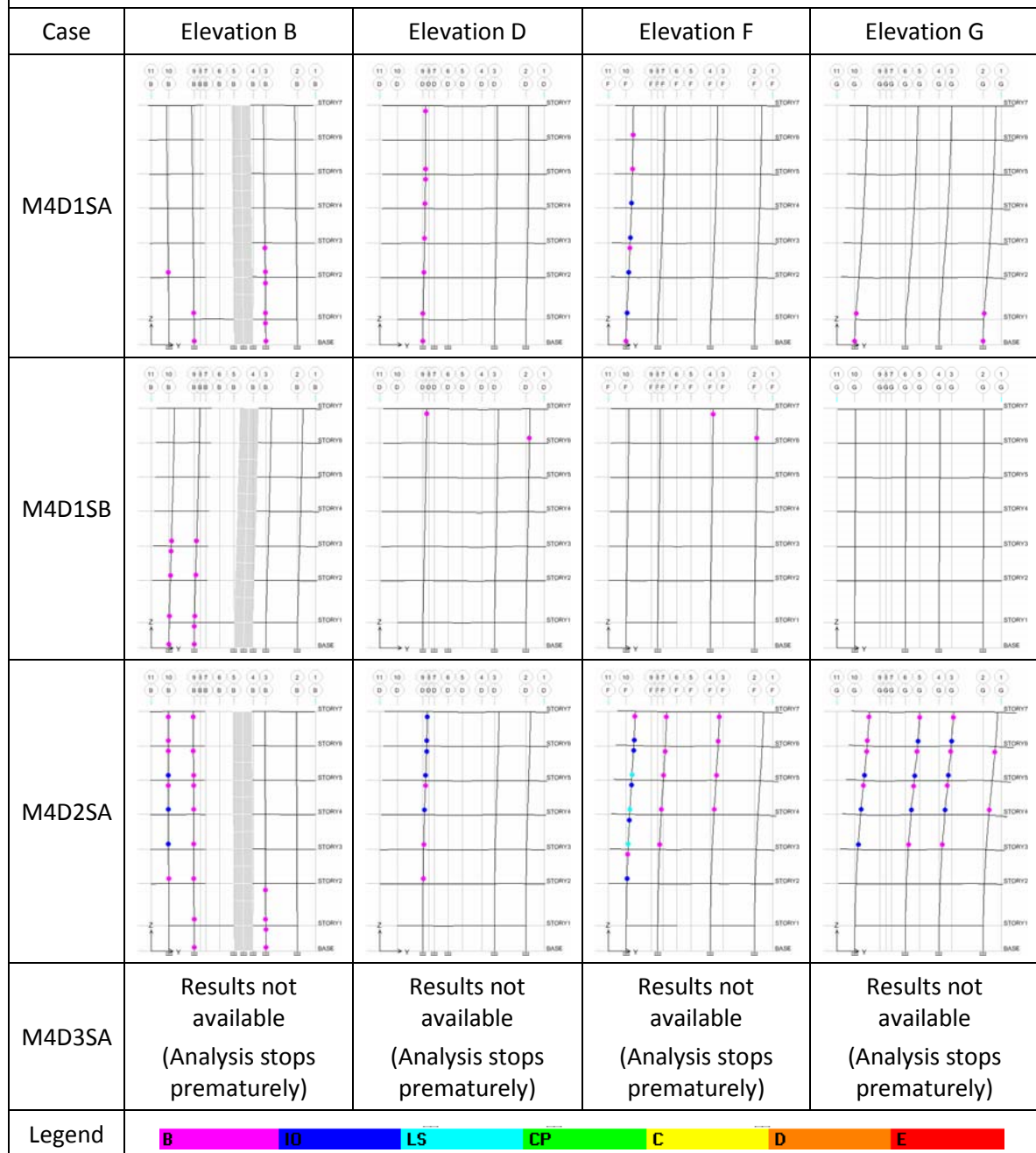
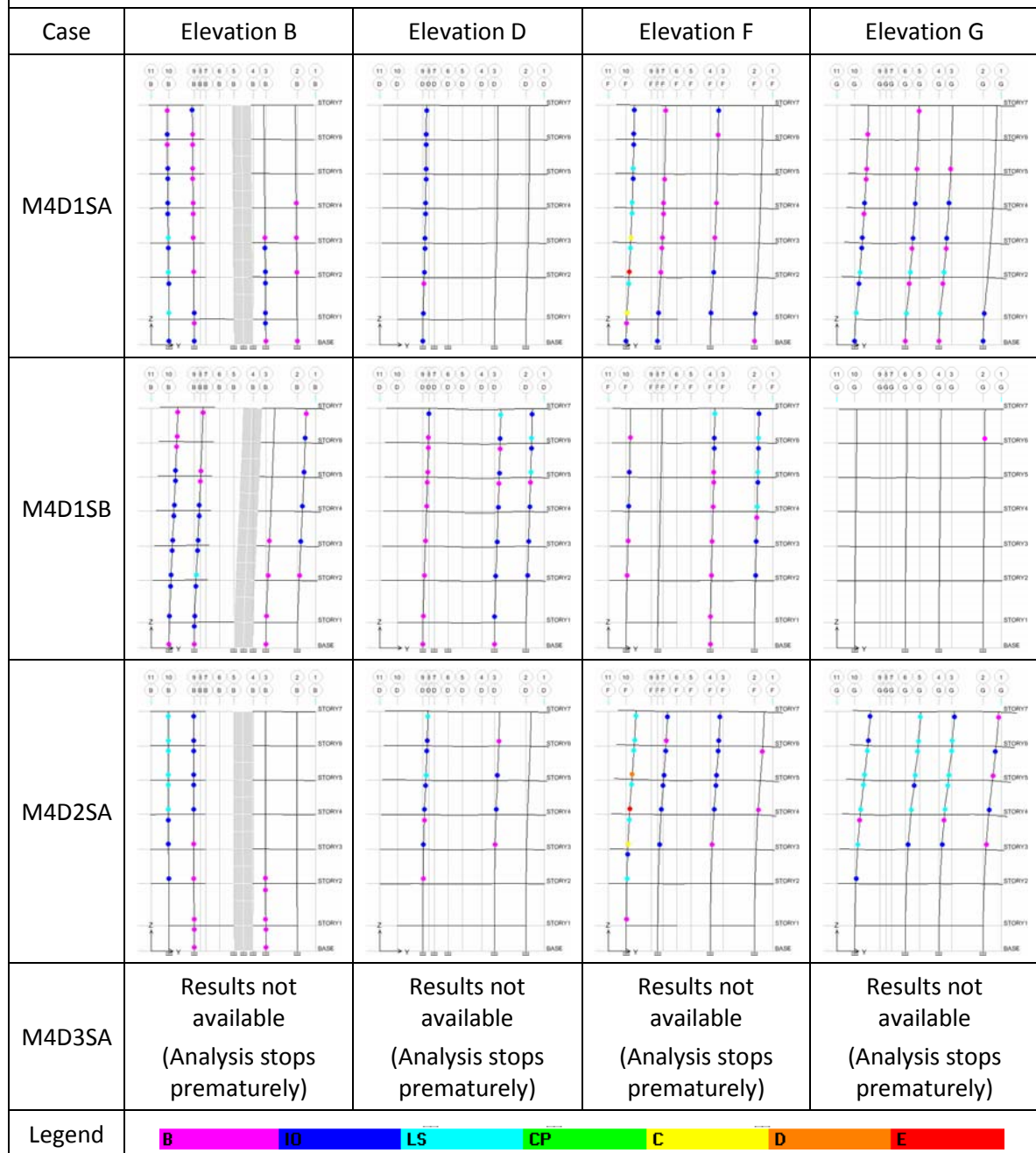


Fig. D3 (f): Damage Distribution for STR3 (at ultimate state)



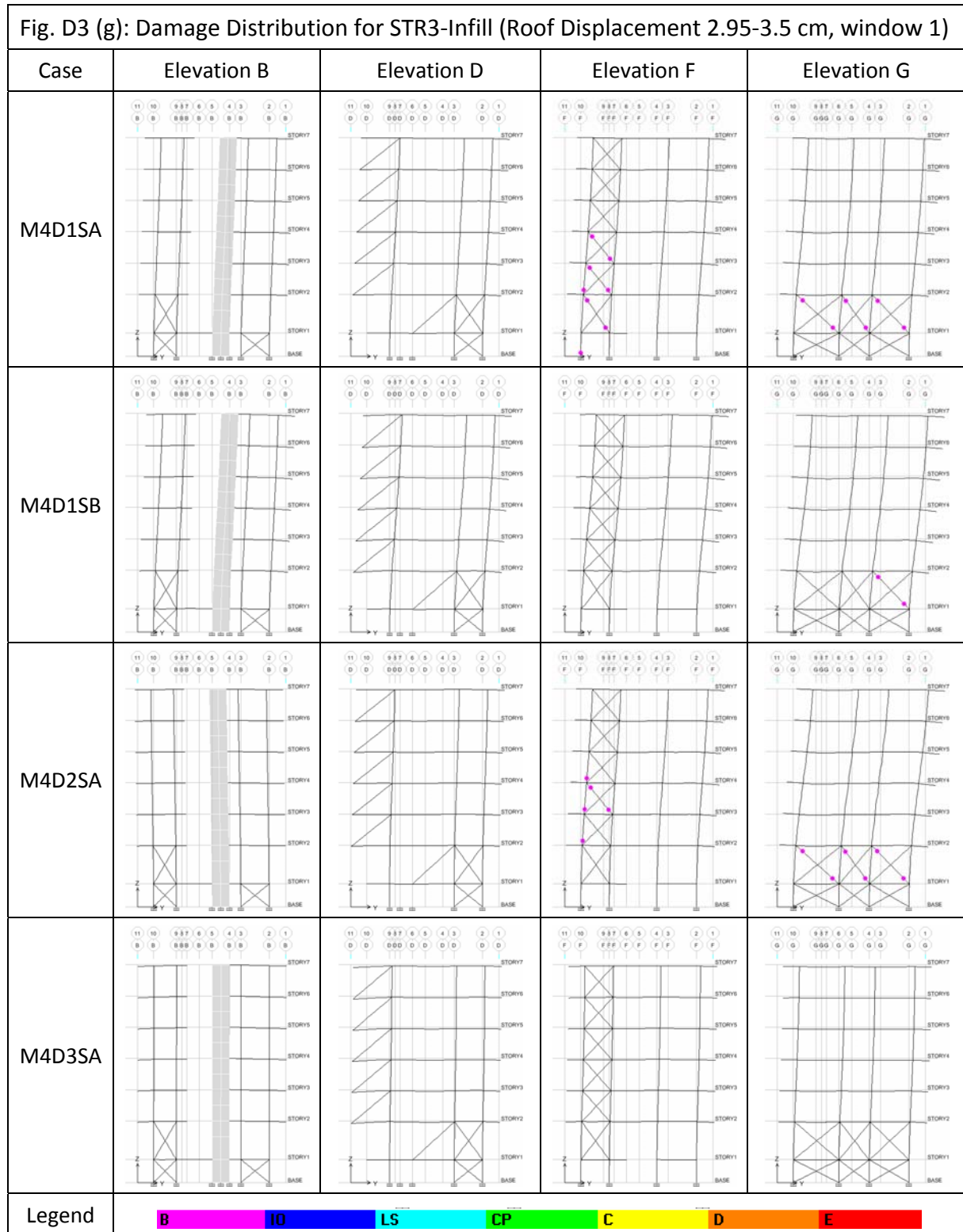
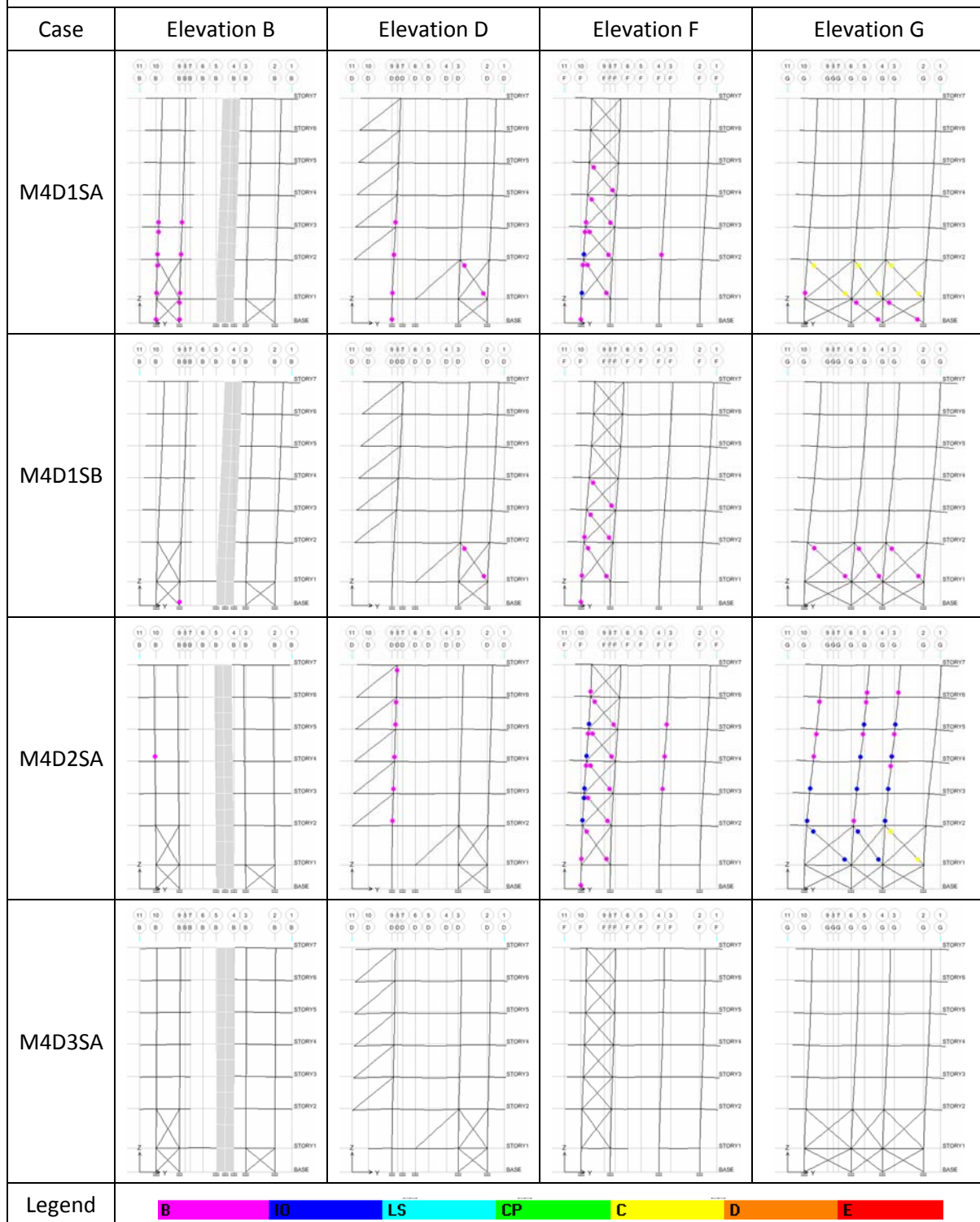
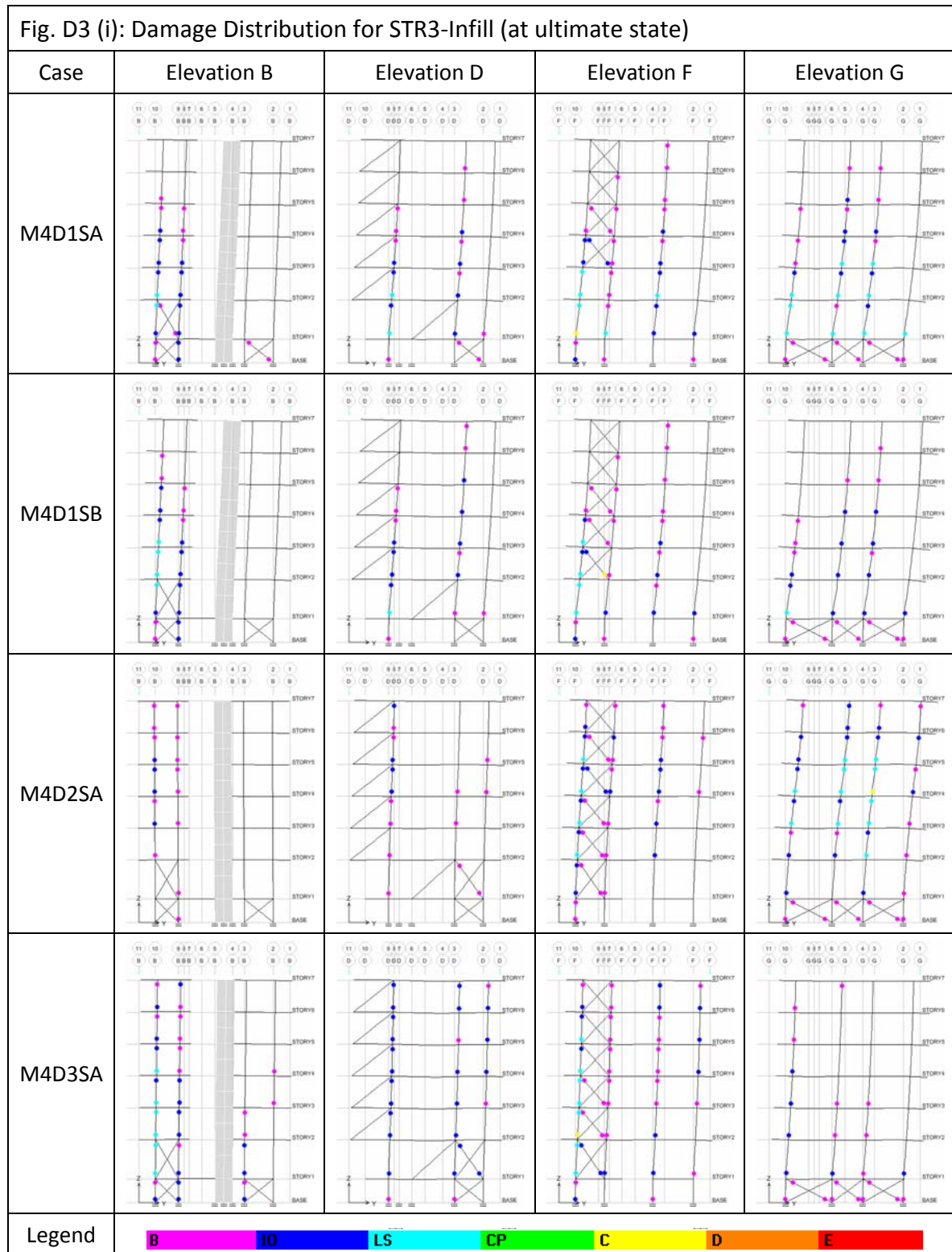


Fig. D3 (h): Damage Distribution for STR3-Infill (Roof Displacement 5.2-5.8 cm, window 2)

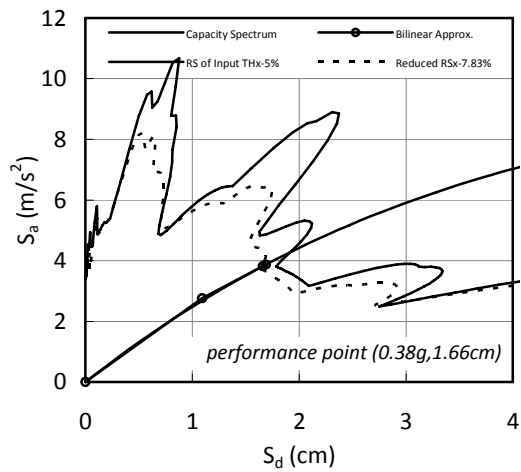




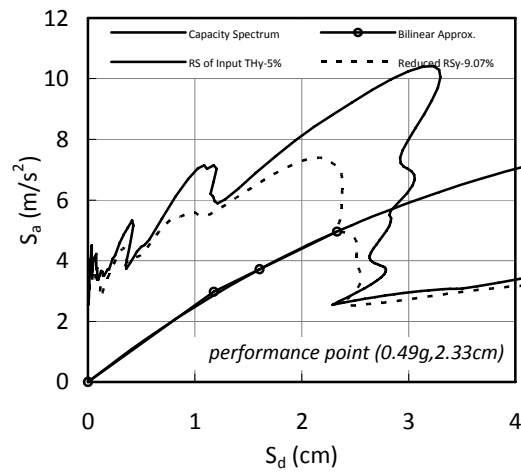
ANNEXURE E

COMPARISON OF DAMAGE DISTRIBUTION BETWEEN NONLINEAR TIME HISTORY ANALYSIS AND LOAD PATTERN PUSHOVER ANALYSIS

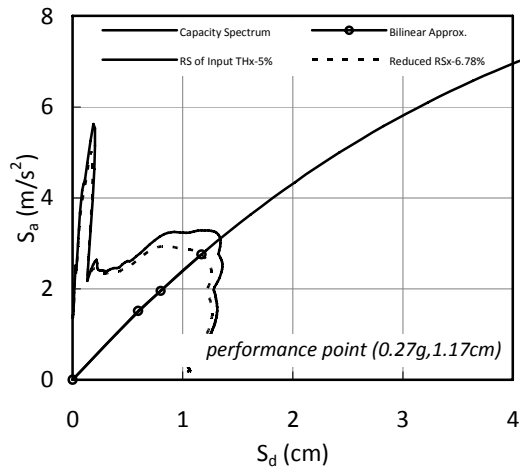
This part is an annexure of section [5.3.1](#) of the main text. Comparison of damage distribution for structure STR1-infill is given.



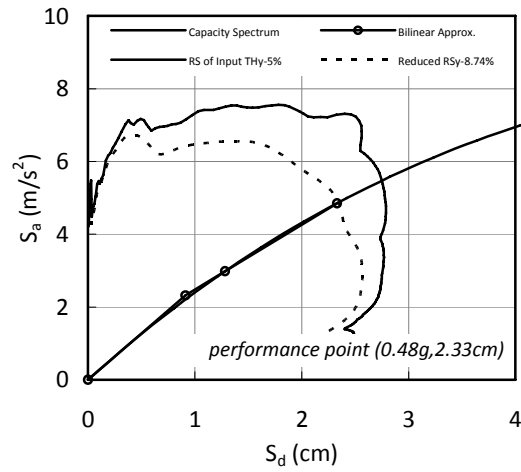
(a) M2D1SB (x-comp)



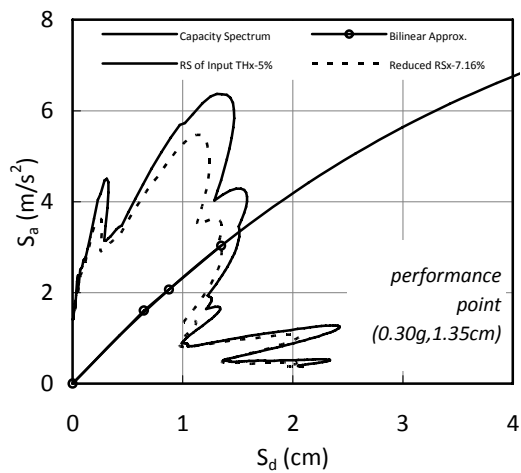
(b) M2D1SB (y-comp)



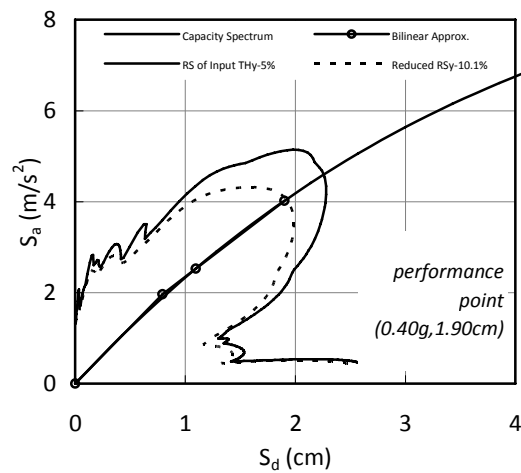
(c) M2D1SC (x-comp)



(d) M2D1SC (y-comp)



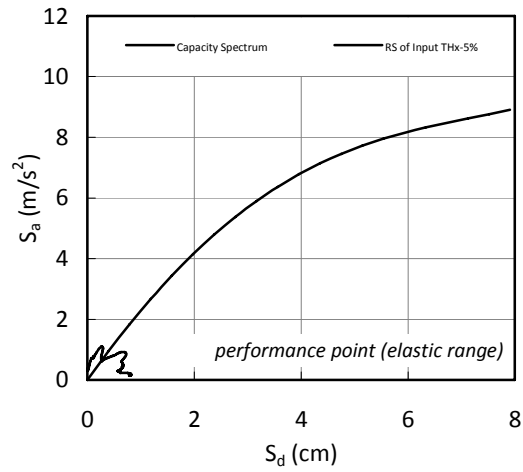
(e) M2D2SB (x-comp)



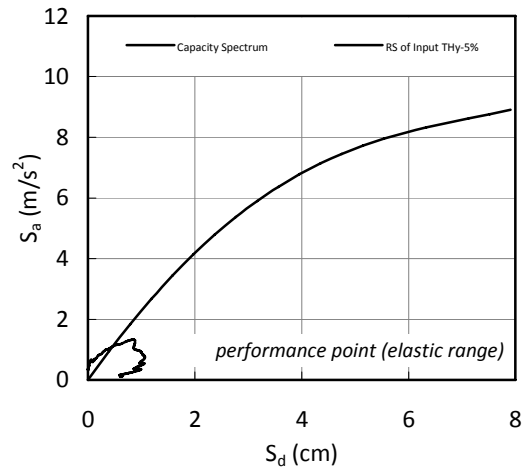
(f) M2D2SB (y-comp)

Fig. E1: Performance Calculations for STR1-Infill

Cont'd



(g) M2D3SB (x-comp)



(h) M2D3SB (y-comp)

Fig. E1: Performance Calculations for STR1-Infill

Fig. E2 (a): Comparison of Damage Distribution for NLTH and LP PO Analyses (STR1-infill)

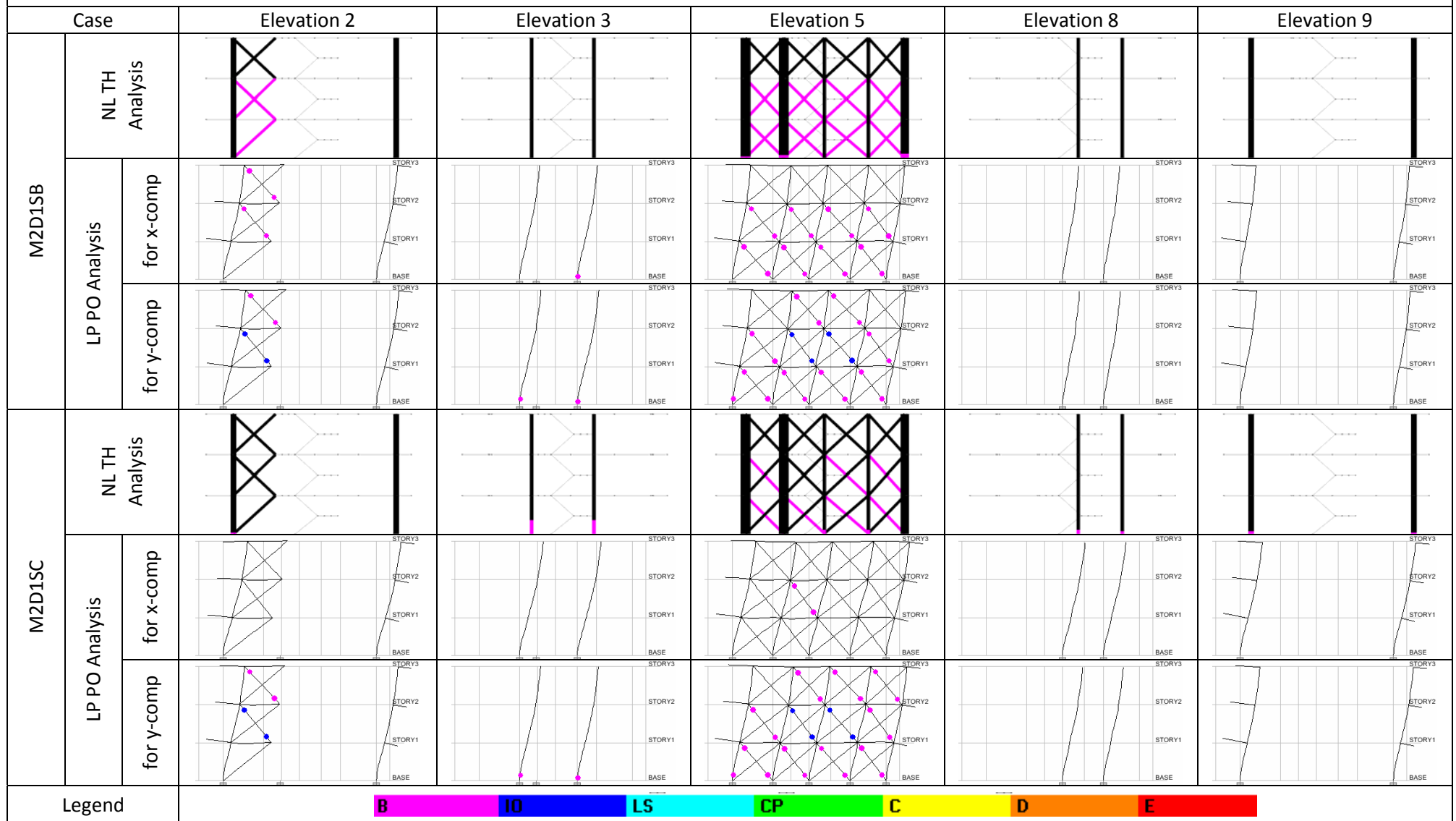
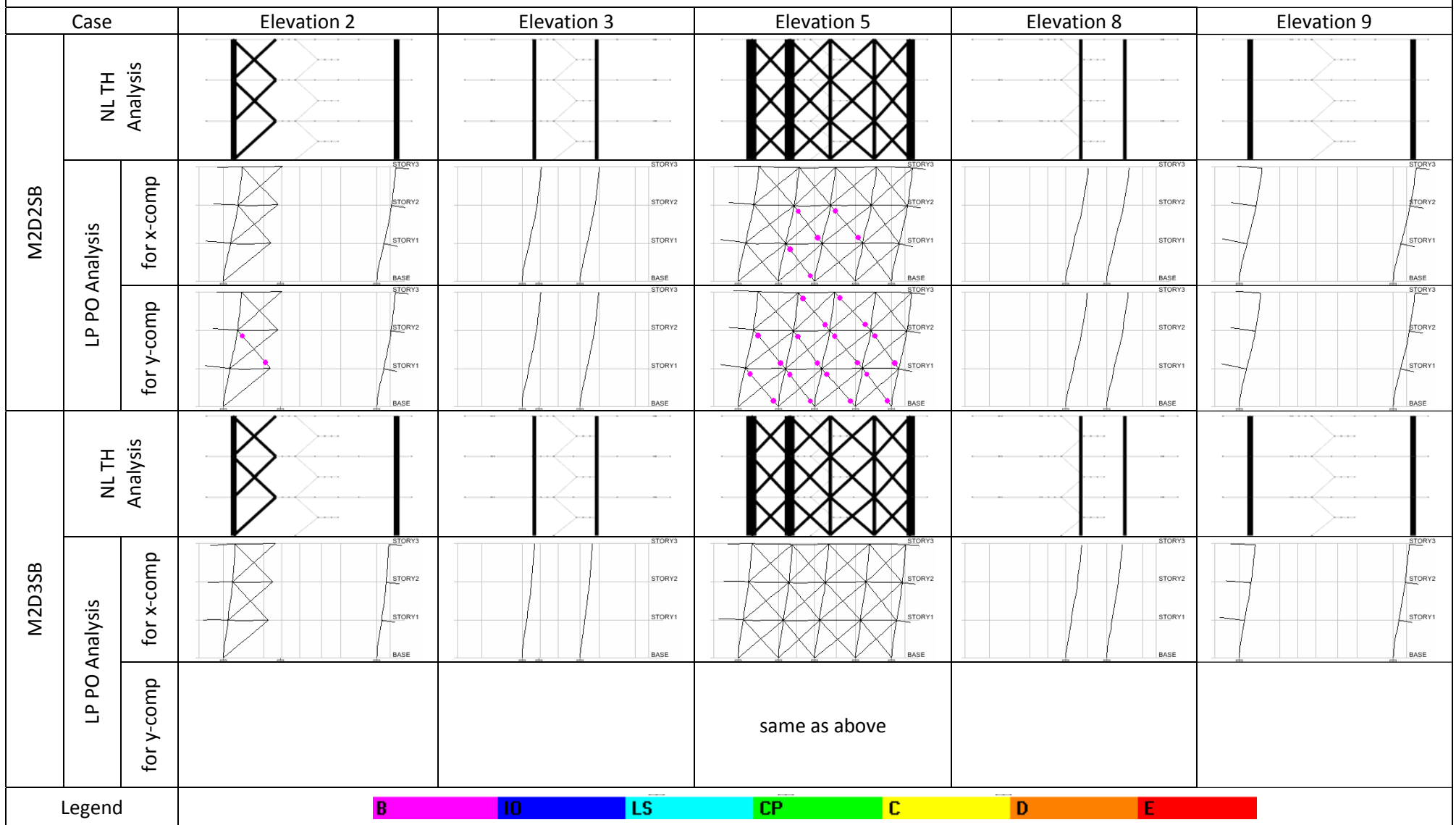


Fig. E2 (b): Comparison of Damage Distribution for NLTH and LP PO Analyses (STR1-infill)

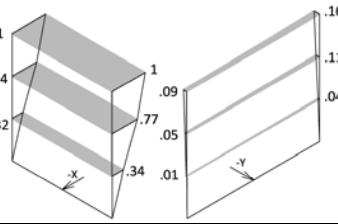
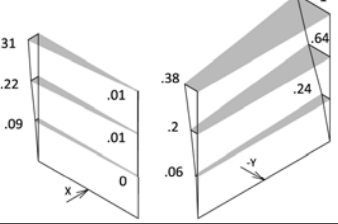
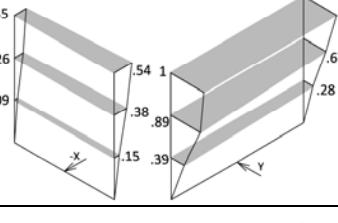
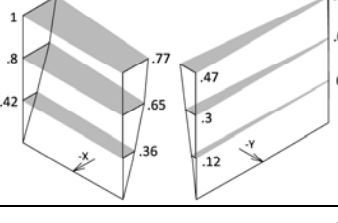
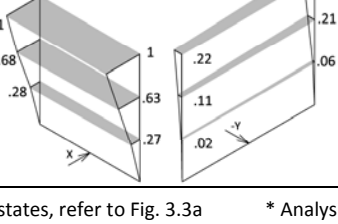
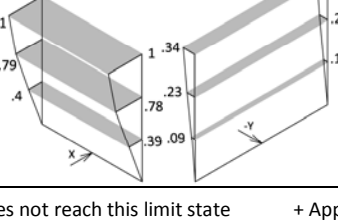


ANNEXURE F

**RESULTING LOAD PATTERNS FOR
NONLINEAR SCALED TIME HISTORY ANALYSIS**

This part is an annexure of section [5.4](#) of the main text. Nonlinear analysis results for scaled time histories for building STR1 are given.

Fig. F1 (a): Load Patterns for Scaled Time Histories for STR1 (Analysis Case: M2D1SB)

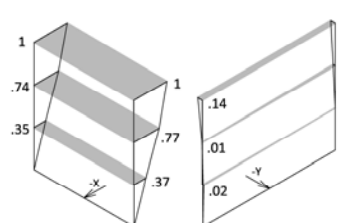
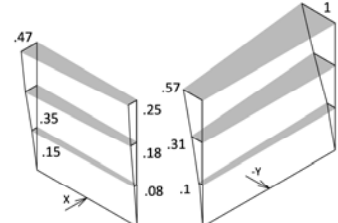
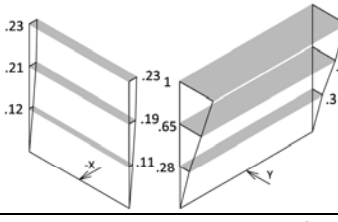
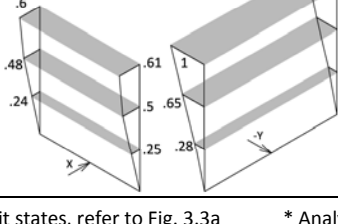
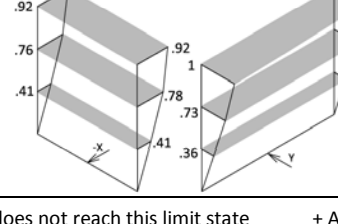
Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X		*	*	
	3X			*	+
	5X			*	+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

Fig. F1 (b): Load Patterns for Scaled Time Histories for STR1 (Analysis Case: M2D1SC)

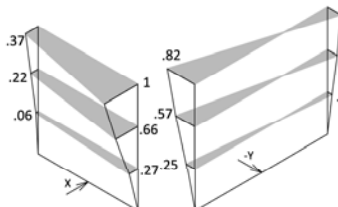
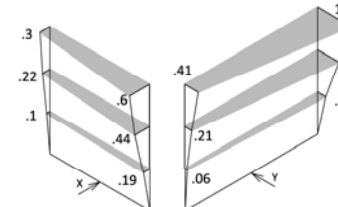
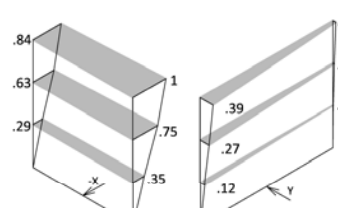
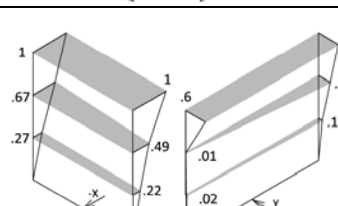
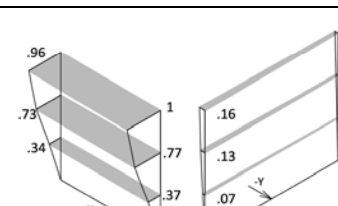
Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X		*	*	
	3X		*	*	+
	5X			*	+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

Fig. F1 (c): Load Patterns for Scaled Time Histories for STR1 (Analysis Case: M2D2SB)

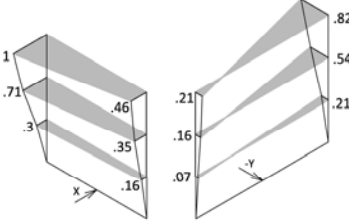
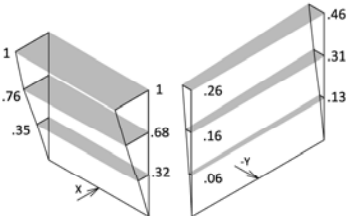
Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X		*	*	
	3X		*	*	+
	5X			*	+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

Fig. F1 (d): Load Patterns for Scaled Time Histories for STR1 (Analysis Case: M2D3SB)

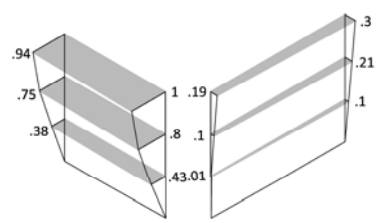
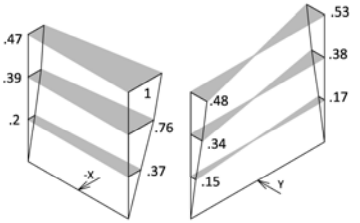
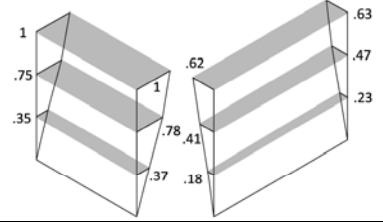
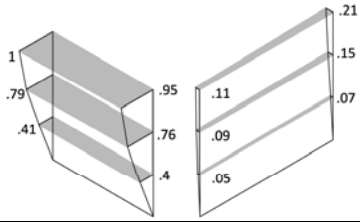
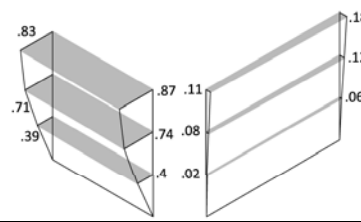
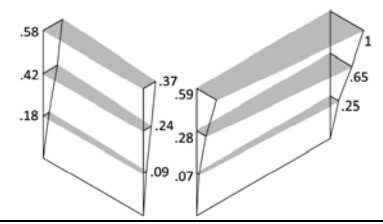
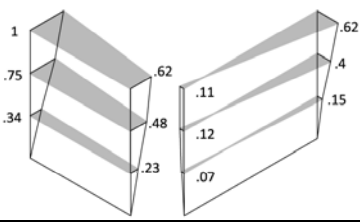
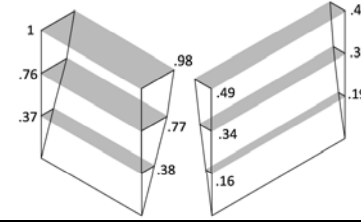
Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X	*	*	*	
	3X	*	*	*	+
	5X		*	*	+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

Fig. F2 (a): Load Patterns for Scaled Time Histories for STR1-infill (Analysis Case: M2D1SB)

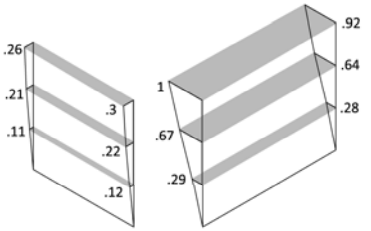
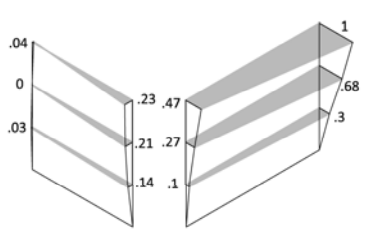
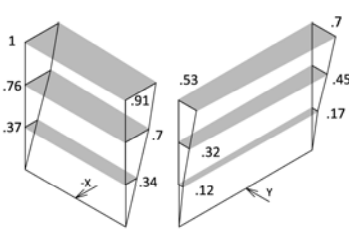
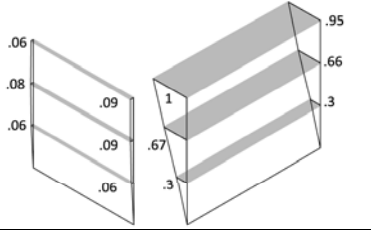
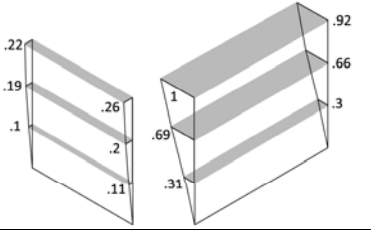
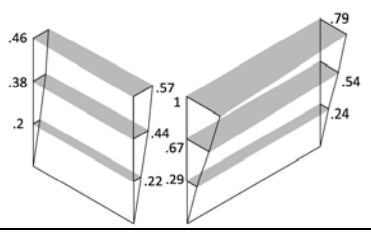
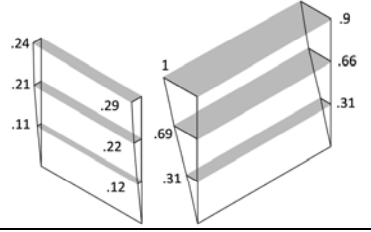
Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X		*	*	
	3X				+
	5X				+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

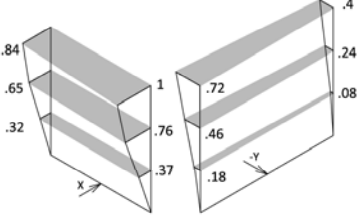
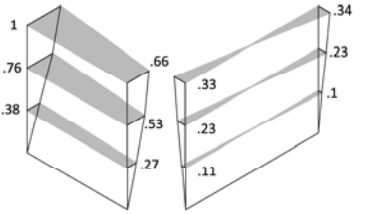
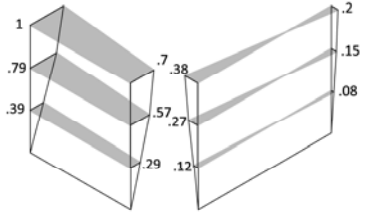
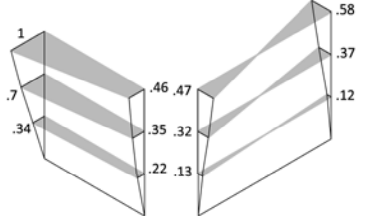
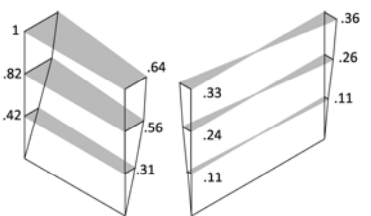
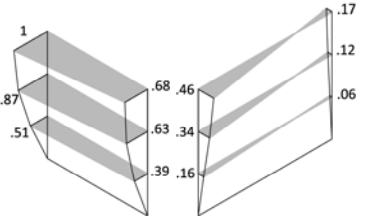
Fig. F2 (b): Load Patterns for Scaled Time Histories for STR1-infill (Analysis Case: M2D1SC)

Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X			*	
	3X			*	+
	5X			*	+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

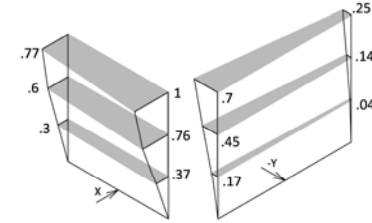
Fig. F2 (c): Load Patterns for Scaled Time Histories for STR1-infill (Analysis Case: M2D2SB)					
Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X	*	*	*	
	3X			*	+
	5X				+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

Fig. F2 (d) Load Patterns for Scaled Time Histories for STR1-infill (Analysis Case: M2D3SB)

Limit State $\diamond$		B	IO	LS	Applied Load Pattern
Scale Factor	1X	*	*	*	
	3X	*	*	*	+
	5X	*	*	*	+

 $\diamond$ For definition of limit states, refer to Fig. 3.3a

* Analysis does not reach this limit state

+ Applied load pattern from Linear TH Analysis is same

Statement

Herewith I declare that the present thesis has been completed by me and no other sources have been used than those mentioned.

Zahid Hanif
Weimar, March 12, 2007