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Seismic Performance of Steel Moment-Resisting Frames with Spines Examined by Shake-Table Tests

by

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ABSTRACT

Seismic resisting multi-story building structures are expected to distribute the seismic force demands evenly throughout the structure height, achieving the global mechanism and thus utilizing the full potential of the structure. However, soft story is still quite common in real constructions as observed in past large earthquakes. These story collapse may be the results of brace failure in braced frame systems or column end hinging in moment resisting frames. Such story mechanism is difficult to avoid even if the structure is code compliant due to the stochastic nature of earthquakes.

The spine, which is a vertically continuous pivoting substructure with large lateral stiffness and strength, is introduced as a possible solution to the drift concentration in conventional multi-story buildings. When connected to a base structure, the spines will redistribute the demands in the system after yielding happens, coordinating the deformation of different stories and force a global deformation distribution via its large stiffness. Such systems have been successfully adopted in both retrofitting and new constructions, and showed its effectiveness.

Despite the effectiveness of spine systems, it has been found in various research works that there may be non-negligible higher mode responses in such systems compared to conventional seismic force resisting systems. Such higher mode contribution will lead to large force demands in the spine members and also floor acceleration demand in the base structure, causing troubles in member design and cost control. Although some researchers have suggested design methods on such systems considering higher mode contribution, considering the cost-effective point of view, it is still necessary to try to reduce the higher mode demands in such systems.

A possible solution to the higher mode issues is to use deformable connections between the spines and the base structure, limited to frames in this article. Such connections can allow some relative motion between the spines and the base structure and hopefully to reduce the higher mode demands. Besides, yielding of such connections is also expected to limit the force transferred from the base structure to the spines. Similar connections have been proposed in a RC flat-slab rocking-wall system. Although numerical simulation illustrates the effectiveness of the system, shake table test did not show very satisfactory results on this purpose. Thus, application of this type of connections in a frame-spine system requires more validation to build confidence, especially when considering that the structural system also changed.

A series of full-scale shake table tests on the Frame-Spine system with deformable connections, specifically termed as Force-Limiting-Connections (FLCs), were conducted in E-Defense. The purpose of the tests was to validate the effectiveness of FLCs in real world and build confidence of such applications. During the test, two structure systems with and without the FLCs were tested. Two ground

motions were selected to activate the structure differently. A comprehensive data analysis was conducted after the test to understand the behavior of the tested structure systems.

In addition to the conventional data analysis on the shake table tests, modal property analysis, including modal parameter identification and time-frequency analysis, were also conducted to further understand the modal behavior of the Frame-Spine systems in real world. To avoid possible inaccuracies brought by numerical modeling, real world test data taken from several more shake table tests on other types of structural systems were used in addition to the Frame-Spine test, including the RC flat-slab rocking-wall test. The supplementary tests were selected to provide comparison on different possible influencing factors, including but not limited to the existence of spines, scale of the specimen, structure height, property of the base structure.

Results from the data analysis of the Frame-Spine shake table test and modal property analysis showed the effectiveness of FLCs in reducing the higher mode acceleration and shear force responses in the system. It was also confirmed that in higher modes were relatively stronger in the system with spines than pure moment resisting frames. Influence of other factors were also investigated.

In the work reported here, basic confidence on the Frame-Spine-FLC systems was built on the test and analysis results. The concept of such a structural system was proved to be worth further investigating. Further work requires more comprehensive analysis on the specific design of the connection properties, as well as other influencing factors as above-mentioned. A series of detailed parametric study may be necessary to get more systematic understanding of such systems. Code compliant design approaches on such systems also require more effort.

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With the final words written, a six-year journey finally comes to an end. The past six years may be the toughest in my 35 years of life, but also the most fruitful. It has been my dream to get a doctor's degree since I was a child, although at that time I was not very clear about what this word means. Although I may not continue with academia, I am glad that I finally contributed something new to the body of knowledge, no matter how small this contribution would be.

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1. INTRODUCTION

1.1 BACKGROUND

For multi-story seismic resistant building structures, although design provisions ensure their capacity, there is still possibility that story mechanisms forms. Considering the rather complicated influencing factors around a building structure, provisions may not be sufficient to ensure what is expected. Story mechanisms may come from many reasons, including but not limited to unexpected exterior loads, deterioration of structure members, and so on.

For moment resisting frames, a common practice to prevent the formation of story mechanism is to ensure the “strong-column-weak-beam” criterion. Ideally, such criterion can ensure a more favorable beam hinge yielding mechanism rather than the column hinge mechanism which may lead to story failure. However, existing research indicates that properly designed structures may still form the story failure mechanism if encountering excitation exceeding the provision assumption [1,2]. For braced frames, in addition to column hinging, the story mechanisms may also stem from the strength loss of the buckled braces and consequent story strength and stiffness deterioration. Even utilization of the Buckling Restraint Braces (BRBs), which avoid the buckling of conventional braces, the low post-yielding stiffness still make it possible that braced frames concentrate demands in one or few stories [3,4].

These deformation concentrations will increase the local damage of structural and non-structural components, increase the P- Δ effects, increase the residual deformation after earthquakes, and finally make it difficult or even impossible to repair these structures. In worse conditions, the deformation concentration will cause building collapse during the earthquake, as illustrated in Figure 1.1.



Figure 1.1 Examples of building story failure during earthquake

(Source: <https://kobe117shinsai.jp/>)

Regardless of the specific structure systems, a key issue to deal with the story mechanisms is to redistribute the lateral demands to other stories when soft stories fail. To achieve the redistribution, a possible solution is to provide an alternative load path, especially a direct vertical load path to the nearby stories. Such load paths can be provided implicitly by making use of the flexural stiffness and strength of the columns and beams, such as utilizing the continuity of columns [5,6], or providing a back-up

moment resisting frame in dual systems [7]. Alternatively, the extra load paths can also be provided explicitly with some vertical connections across stories, such as configuring the brace frames using zipper [8] or tied braces [9]. Using the spine-like substructure is also within the explicit alternative load paths.

Spines are a type of vertically continuous substructure connecting several or all floor levels in a multi-story building structure. They are usually designed to stay essentially elastic under given excitation levels. With the relatively large lateral stiffness, the spines can force a uniform deformation over the height of the structure, and thus utilizing the full potential of the structure. Spines can come in various configurations, such as pin-supported walls [10], spine core frames [11], rocking cores or walls [12], strong back frames [13], and so on. The common part of these various configurations is that they provide some kind of base pivoting mechanism, either by a real mechanical pin, or by rocking behavior. With such pivoting mechanism, the spines are not intended to provide additional lateral resistance, but just to redistribute the deformation demands over the structure height. Figure 1.2 illustrates some applications of the spine concept in building structures.

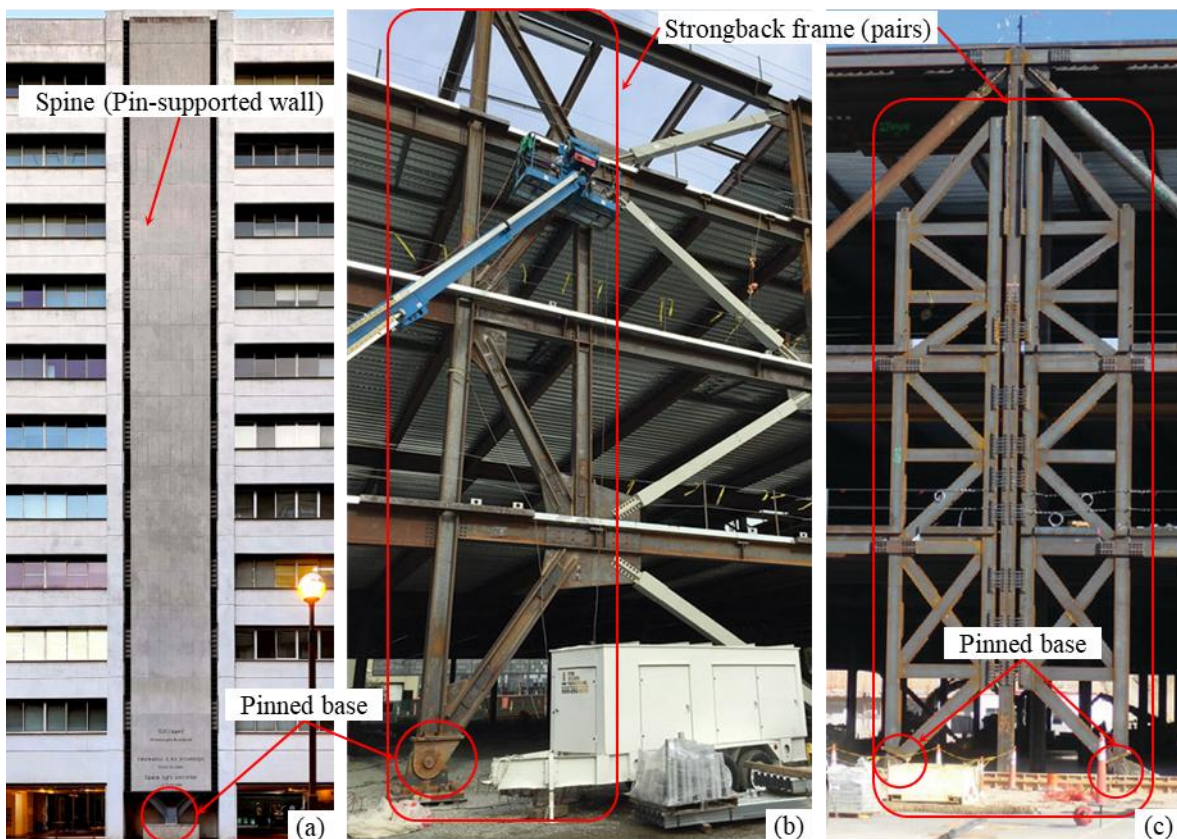


Figure 1.2 Structure systems with spines

(a) Building G3, Tokyo Institute of Technology, Tokyo, Japan; (b) Heinz Avenue Building, Berkeley, CA, US; (c) Tesla Gigafactory, Reno, NV, US.

(Source: (a) courtesy of the author; (b) and (c) reference [14])

Despite its practical applications in both retrofitting and new construction [10,15], previous research

found that the insertion of spines may introduce large story shear force and floor acceleration demands due to near-elastic higher mode responses [16,17]. Such extra demands may not be automatically fulfilled by current design provisions. The large demands may be considered during design by taking the higher mode demand into account, but such large demands may still raise issues, including but not limited to extra demands in the base structure to be retrofitted, or in the proportion of spine members.

Efforts have been made to address the higher mode issues in structure systems with spines. Some of the attempts focused on providing more “flexibility” of the spine, such as using segmental configuration in the spine [18,19]. Another possible way is to focus on the connections between the frame and the spines. A type of deformable connections, termed as Force-Limiting-Connections (FLCs), has been proposed for the application in connecting floor system and lateral force resisting system [20]. Its application to the spine systems may help address the higher mode issues also. By using deformable connections, and thus allowing some relative motion between the base structure and the spines, it is possible to reduce the higher mode demands in the system, making it easier and more economic to design the structure members.

Although it seems promising to address the higher mode issues using deformable connections, real world validation is still needed to build confidence on such systems. In addition, although the higher mode issues were identified in previous research, the work was mainly conducted using analytical or numerical analysis. There is a need to investigate the higher mode issues directly using real world data. Moreover, the interaction of the base structure, the spines and the connections require further investigation to build fundamental understanding for designing such systems. As such, this study focuses on the seismic performance of structure systems with spines and deformable connections, aiming to build the foundation of the design approach of such systems.

1.2 RESEARCH OBJECTIVES

The overall goal of this study is to investigate the seismic performance of the aforementioned structure systems. To achieve this, the following objectives are considered:

- 1) Validate the concept of the structure system with spines and deformable connections in real world.
- 2) Investigate the vibration properties of such systems based on real world test data.

The study mainly focuses on the investigation of the behavior and performance of the new structure system. Practical design approaches are left for future work.

1.3 DISSERTATION ORGANIZATION

This dissertation is organized into 7 chapters. After the current introduction chapter, Chapter 2 provides a literature review of the relevant research on the structure system under investigation. Chapter 3 and 4 introduce the design and results of a series of full-scale shake table test on a Frame-Spine-FLC system,

respectively. Chapter 5 and 6 introduce the methods and results of vibration property analysis on such systems using real world test data, respectively. Finally, Chapter 7 summarizes the work and gives suggestions for future work. Appendices provides additional information for the work in addition to the content of the body.

1.4 TERMINOLOGY

The term “spine”, as aforementioned, can refer to a wide range of structural systems. For the current study reported hereafter, a narrower definition, which requires the existence of a real pinned base connection, instead of rocking mechanism or flexural yielding plastic hinge, is used. Thus, the rocking systems or conventional wall systems are excluded.

Considering that the major type of base structure investigated is Moment Resisting Frames (MRFs), the base structure is termed more specifically as “MRF”. The deformable connections, to be more specific, are termed as “Force-Limiting Connections” and in short as “FLC”. With the spines still termed as “spine”, the proposed system is thus termed as “MRF-Spine-FLC” system.

Other terminologies will be defined when needed.

2. LITERATURE REVIEW

2.1 INTRODUCTION

This chapter is intended to discuss the relevant research on structural systems with spines. Due to the relatively wide extent of such topics, only those closely relevant to this study are summarized here. Considering the research interest of this project, attention is mainly placed on steel structures.

2.2 SOFT-STORY MECHANISM AND MITIGATION

2.2.1 General

As aforementioned in Chapter 1, story mechanism is unwanted but not rare to see in real buildings, as observed in past large earthquakes. The very basic concept accepted by most of the individuals of the seismic engineering field and adopted by many of the design provisions may be the “strong-column-weak-beam” concept, which was discussed by Paulay and Priestley [21]. However, this concept is not easy to ensure in real constructions due to the composite behavior, material overstrength and so on. Thus, even code compliant designs may still result in soft-story mechanisms and cause collapse of the structure [22]. Due to this concern, researchers also have focused on other methods to avoid soft-story mechanism. The basic idea is to provide some extra load paths to transfer and redistribute the demands to several stories, increase the integrity of adjacent stories, and finally achieve a global mechanism. Among the various attempts to deal with soft-story mechanisms, considering the research interest of the work reported here, this section focuses on the efforts in mitigating soft-story mechanism using structural systems with spines.

The spine is a vertically continuous element that utilizes the stiffness and strength of the element to redistribute the demands in different stories, enforcing a uniform distribution of structure deformation and achieving the wanted global mechanism. The spines are not intended to provide lateral resistance but only as a coordinator to redistribute the deformation over the height. Thus, regardless of the detailed configuration, spines introduced here all include a moment release mechanism at the base connection, such as a pin or rocking mechanism.

Figure 2.1 illustrates a conceptual configuration of a structural system with spines (MRF-Spine system), showing the story mechanism and global mechanism. The spine in the figure is only conceptual, which can be a wall, an elastic frame or truss, or even a strong column.

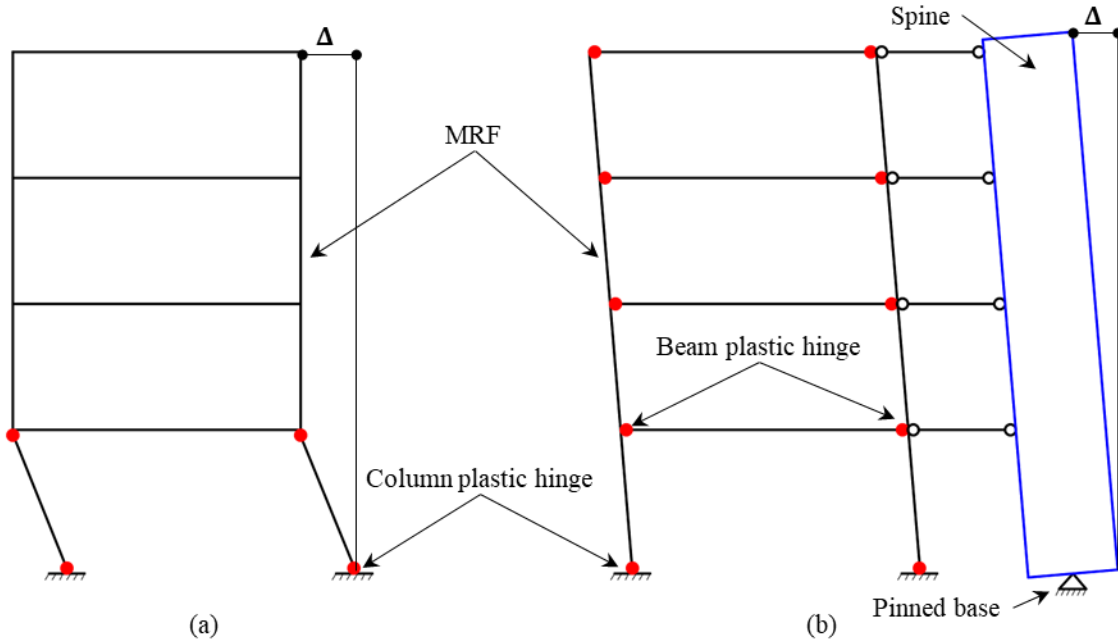


Figure 2.1 Conceptual configuration of an MRF-Spine system
(a) Story mechanism of the MRF; (b) Global mechanism of the MRF-Spine system

2.2.2 Pin-base columns

Some researchers paid attention to the effect of vertical continuous stiffness in controlling drift concentration. This section introduced part of those directly connected to the spine concept, i.e., those with pinned column or walls.

Akiyama and Takahashi [23] investigated the damage distribution in structural systems with the so-called “spreader column”, which is a column connected to the frame members and the ground by pinned connections. The relationship between drift concentration index and the stiffness ratio between the spreader column and the frame K_{eq}/K_R was quantified. Figure 2.2 (a) illustrates the conceptual model of a frame connected to a spreader column.

MacRae et al [5] investigated a similar concept in concentrically braced frames with continuous columns. The continuous column was also pinned to the ground and connected to the frame via truss connections. It was examined by numerical simulations that increase of the stiffness of the continuous columns resulted in a reduction in story drift concentration. Ji et al. [6] conducted similar work in a more analytical way. Figure 2.2 (b) illustrates the conceptual model of the continuous column system.

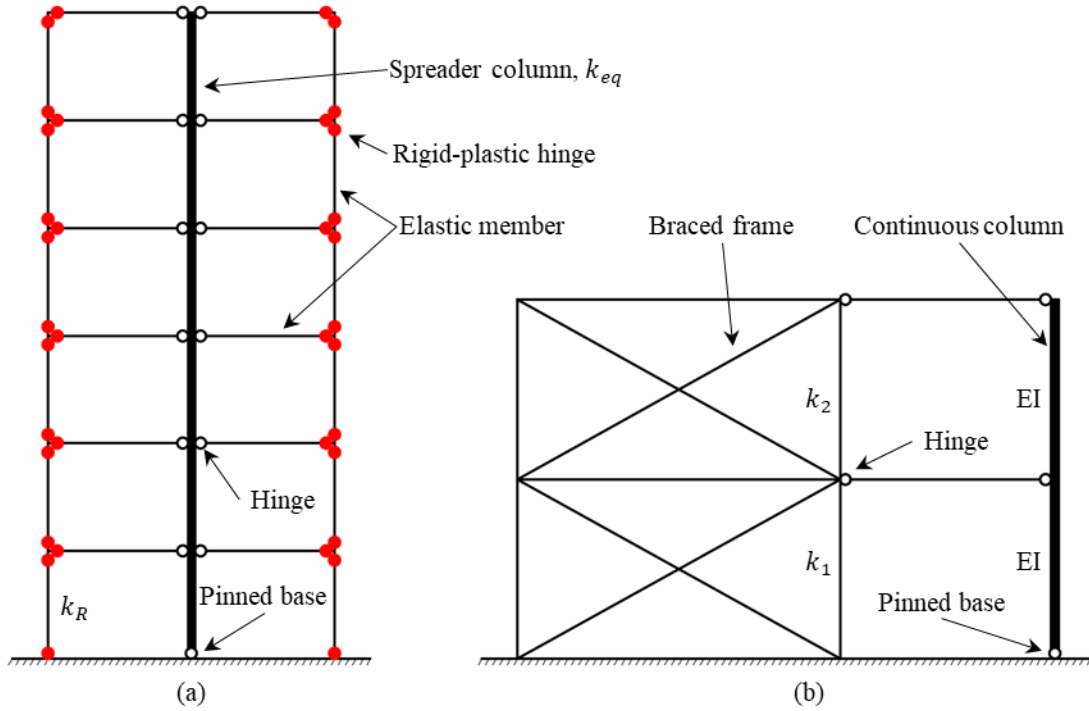


Figure 2.2 Conceptual models with pinned columns or walls:

(a) Spreader column by Akiyama and Takahashi; (b) Continuous column by MacRae et al.

(Reproduced according to the original papers)

2.2.3 Pin-base wall systems

With a deep cross section, shear walls can be an effective means to control the structure deformation over different stories. The pin-base walls can form a stiff vertical element without enlarging the structure stiffness and consequent larger seismic excitation. As proposed by Paulay and Priestly [21], reduced constraint at wall bases would not significantly impair the seismic performance of the wall-frame systems.

Alavi and Krawinkler [24] investigated the effectiveness of strengthening frame using walls and compared the differences between fix-base and pin-base walls. Results indicated that the pin-base walls were effective in reducing drift concentration as well as the maximum story drift demands. In addition, moment and shear demands in the system with pin-base walls were much lower than with fix-base walls. Figure 2.3 illustrates the two conceptual models in this study.

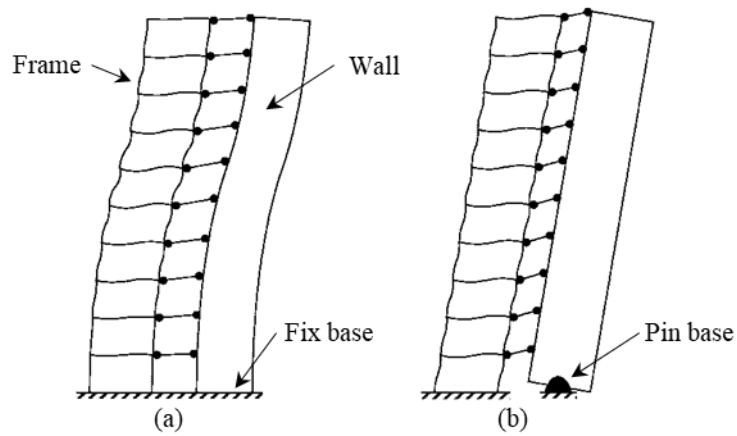


Figure 2.3 Elastic deflection shapes of the frame-wall systems
 (a) Fix base wall; (b) Pin base wall
 (Modified from [24])

Qu et al [10] proposed a pin-supported wall for the retrofitting of a 11-story SRC frame building of Tokyo Institute of Technology. The retrofitting utilized 6 post-tensioned RC shear walls placed symmetrically along the longitudinal direction of the building. The wall was supported by a pin which resisted shear force and compression vertical force but allow finite rotation. Shear type steel dampers were placed between the walls and the RC frame. The retrofit experienced the 2011 Tohoku earthquake and functioned as expected [25]. Figure 2.4 illustrates the retrofit plan using the pin-supported walls.

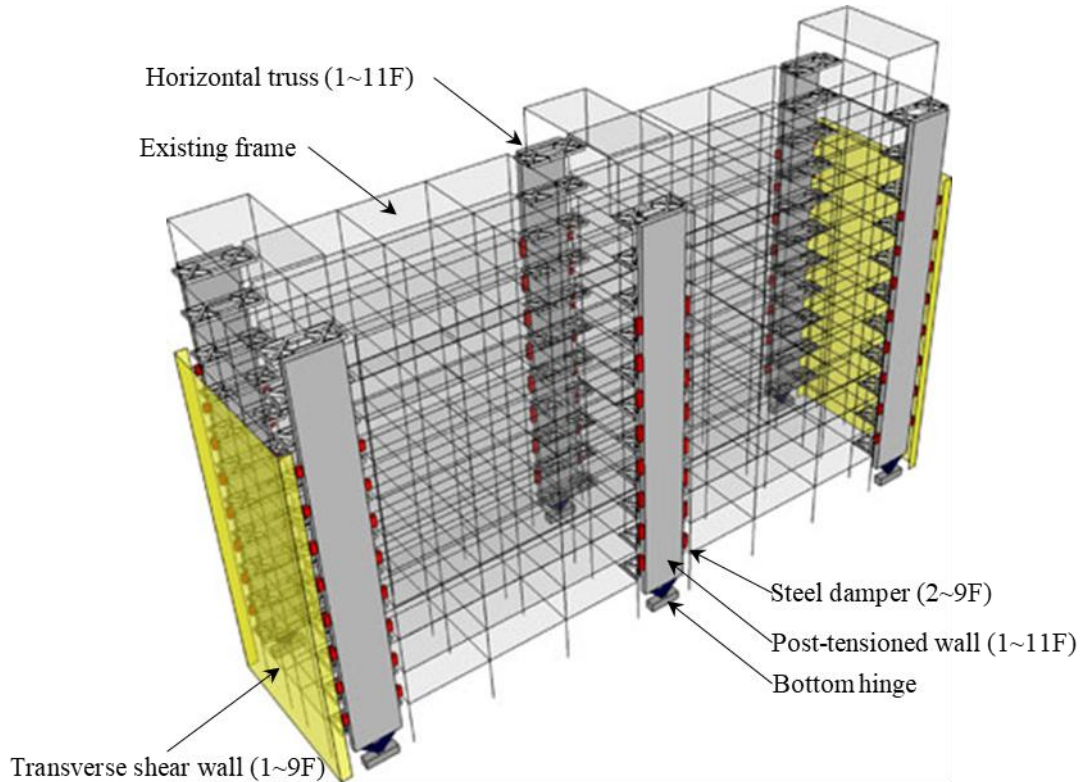


Figure 2.4 Retrofit plan using pin-supported walls
 (Modified from [10])

Similar systems were also proposed by other researchers. Janhunen et al proposed similar retrofitting scheme using a pin-based RC wall in a 14-story building in the US to enforce more uniform drift distribution [26]. Djojo et al introduced the use of steel panel shear walls with additional damping devices at the column bases [27]. Casprini et al proposed a retrofitting design on existing RC frames using pin-supported walls [28].

In all the examples listed above, the major purpose was to use the stiffness and strength of the pin-based walls to coordinate the deformation of the base structure. Most of the cases were used for retrofitting, but there is no difficulty in applying similar designs to new constructions.

2.2.4 Pin-base frame systems

The pin-base frame system is similar to the pin-base wall system, just by changing the walls to an elastic frame. The elastic frame in such systems can be equipped with energy dissipation devices. Such systems may be more applicable in steel buildings since shear walls are usually not first choices in such systems.

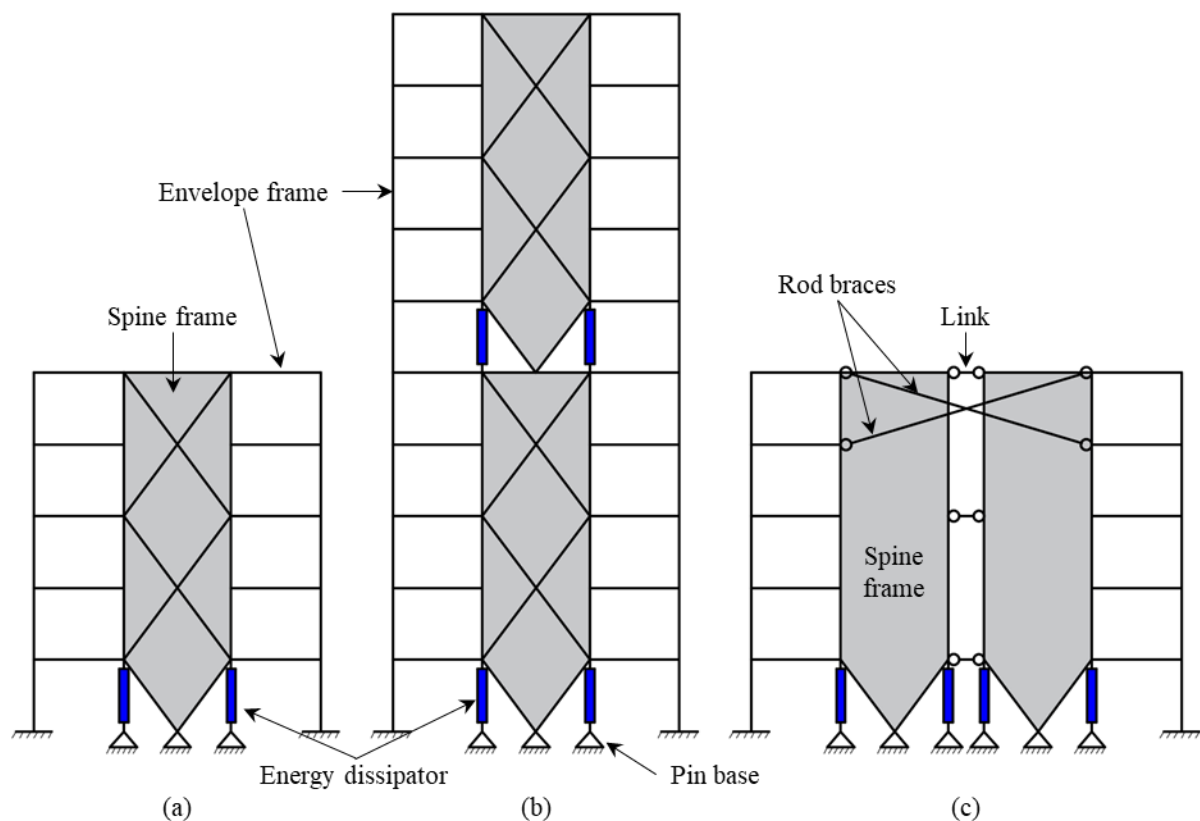


Figure 2.5 Models of spine frames

(a) Spine frames [11]; (b) Segmented spine frames [29]; (c) Parallel spine frames [30]

(Reproduced according to referred papers)

Takeuchi et al. [11] proposed a pin-base spine frame system using buckling restrained columns as energy dissipators. Envelope moment frames were placed to provide partial self-centering ability. The system behaved similarly to systems using post-tensioned strands for self-centering and was used in a building

in Japan. Chen et al. [29] further proposed a stiffness-based design method for the spine frame using a single equivalent column to represent the spine frame. An inelastic-to-elastic stiffness ratio of the frame and a moment frame-to-spine frame stiffness ratio were used to characterize the property of the equivalent column. A simplified design procedure using an equivalent SDOF model was also proposed. This system was further developed by Chen et al. into high rise buildings using a segmented configuration of the spine frame [19], and super elastic system using parallel spine frames [30]. Figure 2.5 illustrates the conceptual models of these spine frames.

2.2.5 Strongback-braced systems

The strongback-braced system is essentially the same as the pin-base frame system, only that the application by far mainly focused on braced frames in US. The strongback systems have been applied to several construction projects in US [15,31].

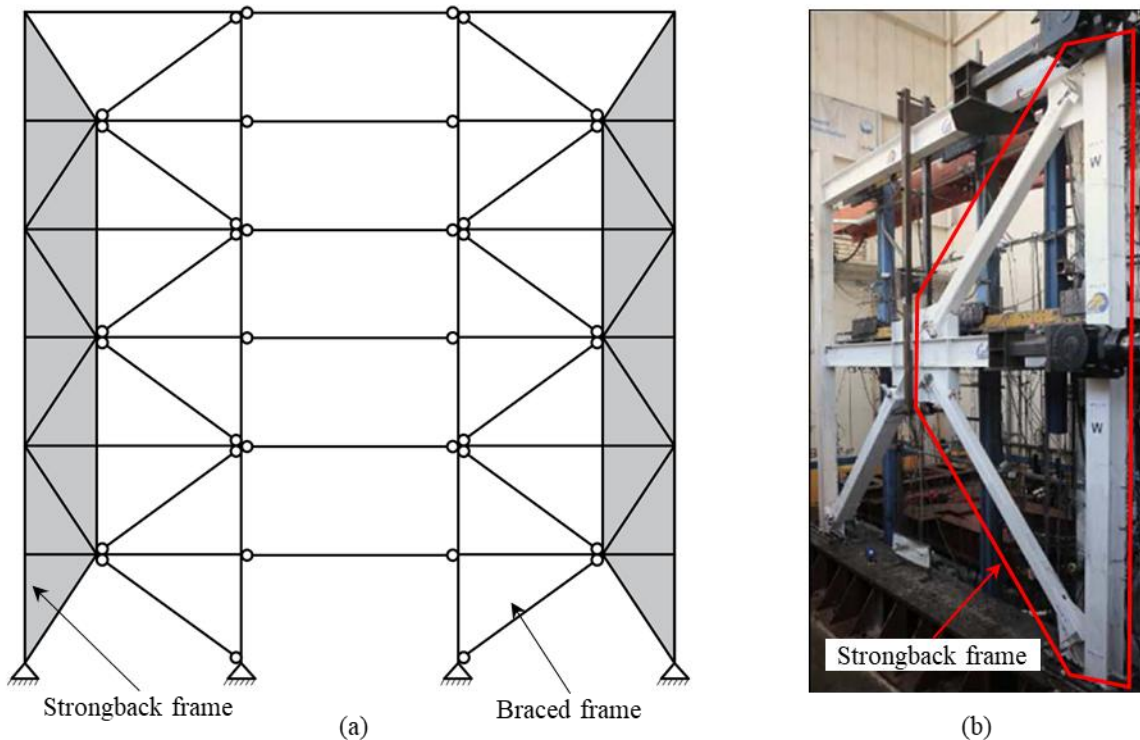


Figure 2.6 Example of strongback systems

(a) Offset configuration (reproduced according to [13]); (b) Static test (modified from [32])

Lai and Mahin [13] investigated chevron and X-braced frames with BRB-frames and strongback frames. The study proposed an offset configuration of the two frames where the connections between the strongback frame and the inelastic frame were moved away from the centerline. The strongback frame members were designed to remain essentially elastic that some members may buckle under severe ground motions. The investigation observed a near uniform distribution of deformation over the structure height by using a strongback. Figure 2.6 (a) illustrates the offset configuration in the system.

Simpson et al. [32] conducted near-full scale static tests on a strongback retrofit braced frame. The retrofit was applied to two previously tested vintage concentrically braced frames representing the 1970s and 1980s common practice. Results indicated that the strongback effectively controlled the soft-story mechanism and resulted in a uniform distribution of structure deformation. Figure 2.6 (b) illustrates a photo of the test taken from the referred paper.

2.2.6 Studies on the design of spine systems

Design of the spine systems require proper proportioning of the mechanical properties of the spines and the base structure. Palermo et al. [33] investigated the coupled response of moment resisting frames connected to a strongback through analytical analysis and numerical simulation. The work obtained a set of close-form analytical results of the interaction between the frame and the strongback, aiming to assist engineers for design of such systems. Lin et al. [34] developed a simplified MDOF numerical model to investigate the stiffness configuration of strongbacks. A distribution of spine stiffness over the height was proposed through comprehensive parametric study using the simplified model, suggesting a distribution with large stiffness at the 1st story and decreasing stiffness going up to the top. Pan et al. [35] proposed a distributed parameter model to evaluate the stiffness and strength demand of pin-supported walls. Bosco et al. [36] proposed a displacement-based design procedure for pin-supported rocking buckling restrained brace frames. Qu et al. [37] proposed an investigation on the stiffness and strength demand for pin-supported walls in RC frames. In general, previous studies mainly utilized numerical or analytical methods instead of dealing with real-world data.

2.3 HIGHER MODE ISSUES IN SPINE SYSETMS

2.3.1 General

Although systems with spines have shown promising abilities in controlling the deformation of a structural system, some studies also show that higher mode response is non-negligible. This section summarizes some of the studies on the higher mode issues in structural systems with spines.

2.3.2 Studies on higher mode responses in spine systems

Higher mode responses in spine systems cannot be divided from previous studies on wall systems. From previous studies, the 1st mode governs the base overturning moment, while the higher modes contribute more to shear force and moment in upper region [38–40]. For the condition in spine systems, the 1st base-overturning mode no longer exist since there is already a pin at the base, thus, for the spine's vibration, the influence of higher mode in shear force is generally large.

Regarding the higher mode responses in spine systems, researchers have mainly focused on analytical and numerical investigations. Wiebe and Christopoulos [41] utilized a cantilever beam analogy to quantify higher mode effects and concluded that shear force demand in the structure was less affected

by the pin at the base. Wu et al. [17] investigated the higher mode responses in frame pin-supported wall structures using a distributed parameter model. The investigation indicated that design based on the fundamental mode is not conservative, and the frame is less affected by higher modes than the walls. Chen et al. [19] noted that higher mode effects may dominate the spine force demand, especially in taller buildings, which also suggested that higher mode must be included in the design of such systems. Simpson [16] evaluated the higher mode force response in strongback-braced frames using nonlinear time history analysis. Analysis indicated that the force demands in strongback arose from a yielding first mode “pivoting” and elastic higher mode “bending”. The analysis also illustrated the dependency of higher mode responses on ground motion properties. In the analysis of stiffness and strength demand of pin-supported wall systems, Qu et al. [37] also concluded that higher mode responses may be heavily underestimated if analyzed using static method such as pushover.

Similar observations were also made in the rocking systems which are quite close to the spine systems. In the rocking systems, the base overturning moment is dominated by the first mode, but the story shear is heavily affected by higher mode effects, such as documented in [42,43].

2.3.3 Estimating higher mode contributions in spine systems

Since higher mode responses are important in spine systems, their contributions must be considered in design. A common conclusion from various studies is that conventional static design approaches considering only first mode contribution is insufficient, and higher modes must be considered. Researchers have suggested various design methods to better consider the higher mode contribution.

In the analysis of spine frame system utilized in high-rise building, Chen et al. [19] suggested using modal pushover and spectral method to consider higher mode responses, and combine the contribution of different modes using SRSS (Square Root of the Sum of Squares) method. Qu et al. [37] suggested using dynamic analysis methods such as Incremental Dynamic Analysis (IDA) for evaluation of system response of pin-supported wall systems, which may be very computational costly. Simpson et al. [44] proposed a simplified modal pushover method (MPA) to estimate modal force demands. Results from the simplified MPA procedure show that the proposed method can give reasonable estimation of system response compared to dynamic analysis, promising possible applications in design.

2.3.4 Reducing higher mode responses

Although proper design methods can incorporate higher mode responses in the design of spine systems, there are still difficulties when the computed demand is too big to be realize. Thus, reducing the higher mode responses becomes another choice. The general concept in reducing higher mode responses is to provide more “flexibility” to the system, or from another point of view, provide yielding mechanisms for higher modes.

One way to provide more yielding mechanisms is to divide the spine or rocking frame into segments, allowing pivoting or uplifting at multiple locations, such as introduced in [18,19,45]. The other way is to work with the connections between the spines and the base structure. A type of force limiting connections were proposed for application in RC flat slab-rocking wall systems [20,46–48]. The connections were designed to limit the inertia force transferred from the gravity system to the rocking wall, thus reducing the higher mode induced large floor acceleration responses. A shake table test was conducted to validate the design, but results were not as good as in numerical simulation [49]. A force-based design approach was also proposed for this system [50].

The Force Limiting Connection is the method to reduce higher mode responses selected for the work reported. More details of it will be introduced in following chapters.

2.4 INVESTIGATION OF VIBRATION PROPERTIES OF BUILDING STRUCTURES

2.4.1 General

As aforementioned, previous work looking into the modal properties, especially higher mode properties, of the spine systems mainly utilized numerical methods. Numerical simulation can give rather “clean” results under well controlled conditions, but may also raise concerns about the uncertainties brought by the modeling [51], and further consistency between the simulation and reality, for example found in the work on Force-Limiting Connections [20,49]. As an alternative, direct investigation on the vibration properties of a building structure using data from real-world, either monitoring or tests, can give results without influence from the modeling procedures. This section summarizes some of the studies on the vibration property analysis methods, including modal analysis, time-frequency analysis and spectral analysis methods. It should be noted that, as a well developed discipline, a huge number of possible methods have been and are being proposed. This section is neither intended nor able to make a very thorough introduction. Only the very typical methods which are also closely related to the work introduced here will be touched.

2.4.2 Modal analysis

Modal analysis here refers to the process of extracting modal parameters, such as natural periods, damping ratios, mode shapes, and so on, from the monitored or tested data of a structure. Different methods may work in frequency or time domains and require either input-output signals or output-only signals of the system. In the application on civil engineering, these methods are mainly adopted by the structural health monitoring community but not so often by the seismic engineering community.

Among the frequency domain methods, when the input is available, or in other words in Experimental Modal Analysis (EMA), the Frequency Response Function (FRF) method may be simplest and most widely used. This method uses the input and output signals of the structural system in frequency domain,

and extracts the modal parameters through curve fitting on the FRF curves [52]. When the input signals are not available, or in other words in “Operational Modal Analysis (OMA)”, some output-only methods can be used. For example, the Frequency Domain Decomposition method proposed by Brincker et al [53]. This method uses the output signals of the structural under flat excitations (white noise or ambient) to construct the FRF of the structural system and uses Singular Value Decomposition (SVD) on the FRF matrix to extract the modal parameters. This method was further developed to the Enhanced Frequency Domain Decomposition method (EFDD) to enable the identification of modal damping ratios using the Auto-Correlation Function (ACF) and curve fitting [54]. Some further development was also made to address some practical issues in this method [55,56]. The FRF and FDD series of methods, and almost all the frequency domain methods, have the same problem of frequency resolution which was limited by the number of data points in the signal due to the nature of the Fourier transform. This may cause problems when the system has closely spaced modes.

For the time domain methods, the limitation of frequency resolution no longer exists. One of the most widely used time domain methods is the Auto-Regressive (AR) methods. These methods belong to the family of recursive methods which represent the current output using information from previous outputs. These methods can consider the input (extra input, the “X”) or not and come in various forms with different modeling of the noise component, such as Auto-Regressive with Moving Average (ARMA), Auto-Regressive with Moving Average and eXtra input (ARMAX), and so on [57]. A series of similar but different time domain use the State Space to model a dynamic system, considering both input and output signals (Deterministic State Space method) or output-only signals (Stochastic State Space method)[58]. These methods all use a certain model to depict the dynamic behavior of the structural system, and extract the modal parameters using Engen Value Decomposition (EVD) on the system matrix, which is obtained differently in the AR series and State Space series methods. For these methods, a major difficult is to determine a proper order of the model, which can be realized by Stabilization Diagram [59]. In addition to these methods, there are also some methods using the free decay property of a structural system in time domain, such as the Random Decrement Technique (RDT) [60], the Ibrahim Time Domain (ITD) method [61], the Natural Excitation Technique (NExT) [62], and so on. Limited by the content, these methods will not be further introduced. Readers can refer to the original references for more information.

2.4.3 Time-frequency analysis

The aforementioned modal analysis methods mainly utilize the full-length signals of a structure, which represent the properties without time information. When considering time-invariant systems such as building structures under white noise excitations, this is acceptable. But when considering time-variant systems, such as building structures under ground motion excitations, time variation of the properties within a single excitation is also of interest, and these modal analysis methods no longer fit. In order to

consider the time variation of the structural properties, time-frequency analysis methods are necessary.

The Short-Time Fourier Transform (STFT) method may be the most straightforward [63]. This method is a direct development of the conventional Fourier Transform. In this method, the Fourier Transform is not applied to the whole signal but to segmented signals taken using a moving window. In this way, time dependency of structural properties can be observed. A major drawback of the STFT method is that there must be a trade-off between the time resolution and frequency resolution of the results which is due to the fixed window length in the analysis. The longer the window, the better the frequency resolution but worse the time resolution, and vice versa.

As a method with variable base functions, the wavelet method, either Continuous Wavelet Transform (CWT) or Discrete Wavelet Transform (DWT), can address the fixed resolution in STFT method. This method uses a series of wavelet bases to transfer the time domain signal into time-frequency domain. This method can provide a reasonable trade-off between the time and frequency resolution and have been widely used in the investigation of vibration properties of building structures, for example summarized in [64]. An improvement to the conventional CWT method was introduced by Daubechies et al [65] as the Synchro-Squeezed Wavelet Transform (SSWT). The SSWT method uses a synchro-squeezing method [66] to reallocate the energy on the time-frequency plane in the frequency direction to “sharpen” the results and make it easier to identify the instantaneous frequencies in the system. The SSWT method also enables reconstruction of the time domain modal components from the time-frequency domain.

It should be noted that the time-frequency methods are mainly signal-based, meaning that when applied to system identification of building structures, they are output-only methods. Although there have been efforts to consider input-output data structure using wavelet-based methods, for example in [67–69], they are not yet developed to be used for ground motion excitations.

2.4.4 Spectral analysis

As a relatively conventional concept in structural dynamics, the response spectra, for example the pseudo-acceleration, can also be used to evaluate the mode strength in a structural system. For elastic systems, the pseudo-acceleration of each mode, the Modal Pseudo-Acceleration (MPA) can be determined, and the peak value of the MPAs of each mode can be used to quantify the mode strength in the system response. As an extension of this concept in nonlinear conditions, Roke proposed the Effective Modal Pseudo-Acceleration (EMPA) [70]. This concept has been used in previous work on spine systems to evaluate the modal strength in these systems, for example in [16,71]. Although this method, which uses the whole-length signal, cannot provide time information, it can provide a point of view much closer to seismic engineering. Since this method considers only responses of a structure, it is also an output-only method.

2.5 SUMMARY

This chapter summarizes part of the studies on spine systems. Different types of spine systems are introduced, mainly focusing on those with pin base connections. Higher mode issues in spine systems and the efforts to address them are also introduced. Considering the wide extent of the topic, only those closely related to the current study are included. The purpose of this chapter is to give some basic information on how the current study comes. A comprehensive state-of-the-art summary is out of the scope of this chapter.

Major contents of this chapter are summarized as follows:

- Various structural systems can be used to overcome the soft-story issues in multi-story buildings, of which the spine system is one of the choices.
- Spines can come in various forms including but not limited to walls, elastic frames, strong-back frames, and so on. These systems share a common point of pin-base to avoid increasing the overturning moment resistance.
- Higher mode issues have been observed in spine systems. These large higher mode demands may increase the cost in structure design and cause possible damage to non-structural components.
- Deformable connections may be effective to control the higher mode responses in spine systems, but their effectiveness still requires real-world validation.
- Various methods can be used to investigate the vibration properties of building structures under dynamic loadings. These methods work in different domains and use different input-output data structure. Application of these methods on real-world data may be effective for the investigation of higher mode responses in spine systems.

3. FULL-SCALE SHAKE TABLE TESTS: TEST PLAN

3.1 GENERAL

A series of full-scale shake table tests were conducted to provide real-world validation of the MRF-Spine-FLC concept and build the foundation for further development of such systems in practical projects. Test design and data reduction algorithms are introduced in this chapter.

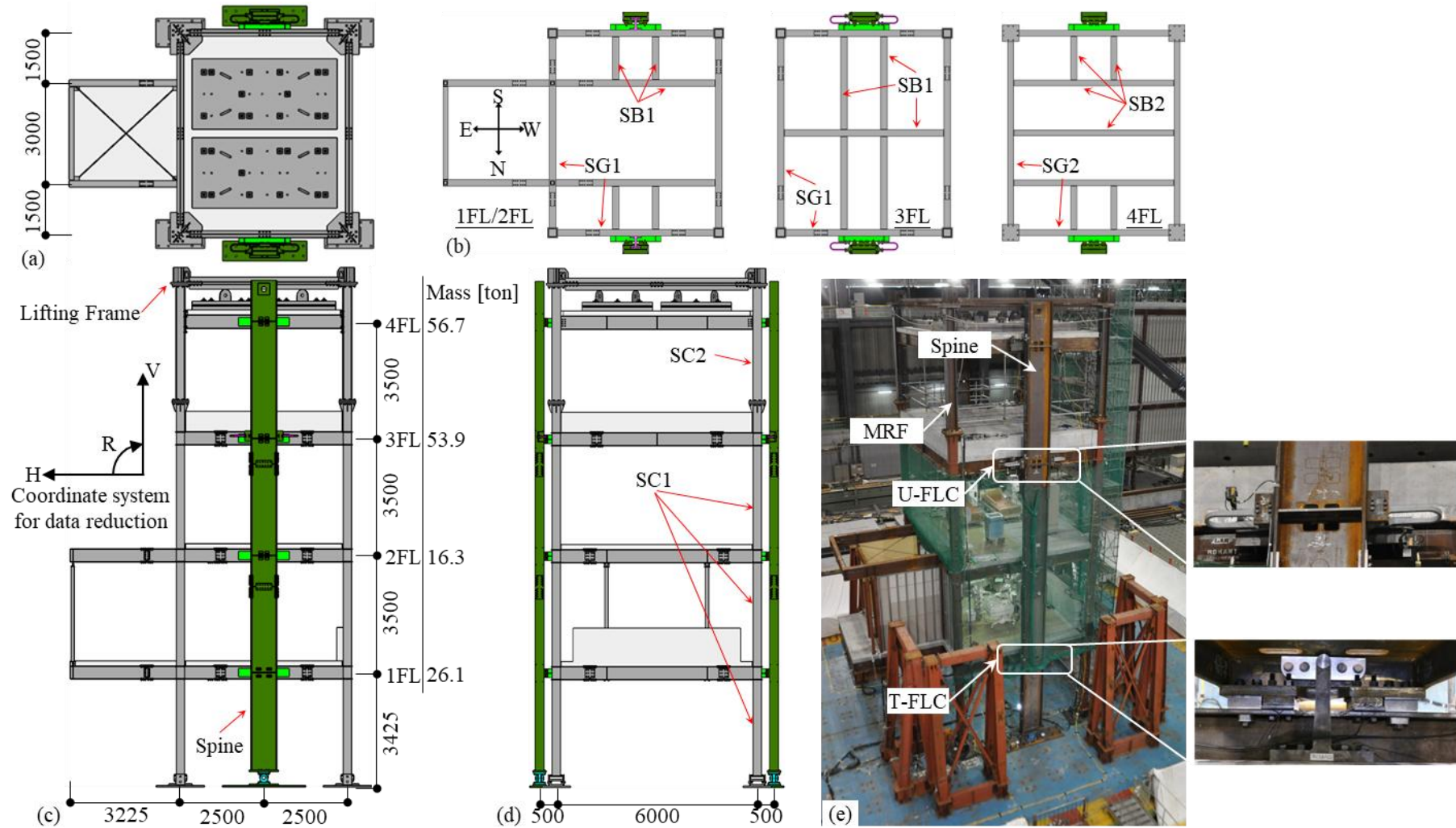
3.2 TEST SPECIMEN

3.2.1 Layout of the whole specimen

Figure 3.1 illustrates the layout of the test specimen. The specimen was a full-scale steel structure consisting of three major parts: a 4-story 1-by-1 bay Moment Resisting Frame (MRF) serving as the base structure, two spines, and the connections between them. The columns of the MRF were pinned at the base to ensure a weak first story. The spines were deep girders pinned at the base and connected to the MRF at each floor level. The connections were set at the mid-span of the MRF girders at each floor, totaling eight locations. The lower three stories of the MRF were fabricated in Japan while the remaining parts were fabricated in US. It should be noted that the lifting frame was not part of the structure, but it was left on top due to practical issues. Bolts at the lift frame were loosened so that it only contributed to the structural weight but did not affect the mechanical properties. Table 3.1 summarizes the major cross sections, materials, and weight of the specimen. Figure 3.2 illustrates the major cross sections and plots of the FLCs with nominal dimensions.

Table 3.1 Cross sections, materials, and weight of the specimen

Story/Floor	Member	Section	Material	Weight [ton]
4	Column	W10×100	ASTM A992	56.8
	Girder/beam	W16×40	ASTM A992	
	Spine	W30×148	ASTM A992	
3	Column	HSS-250×250×9	BCR295	53.9
	Girder/beam	H-400×200×8×13	SN490B/SS400	
	Spine	W30×148	ASTM A992	
2	Column	HSS-250×250×9	BCR295	16.3
	Girder/beam	H-400×200×8×13	SN490B/SS400	
	Spine	W30×148	ASTM A992	
1	Column	HSS-250×250×9	BCR295	26.1
	Girder/beam	H-400×200×8×13	SN490B/SS400	
	Spine	W30×148	ASTM A992	



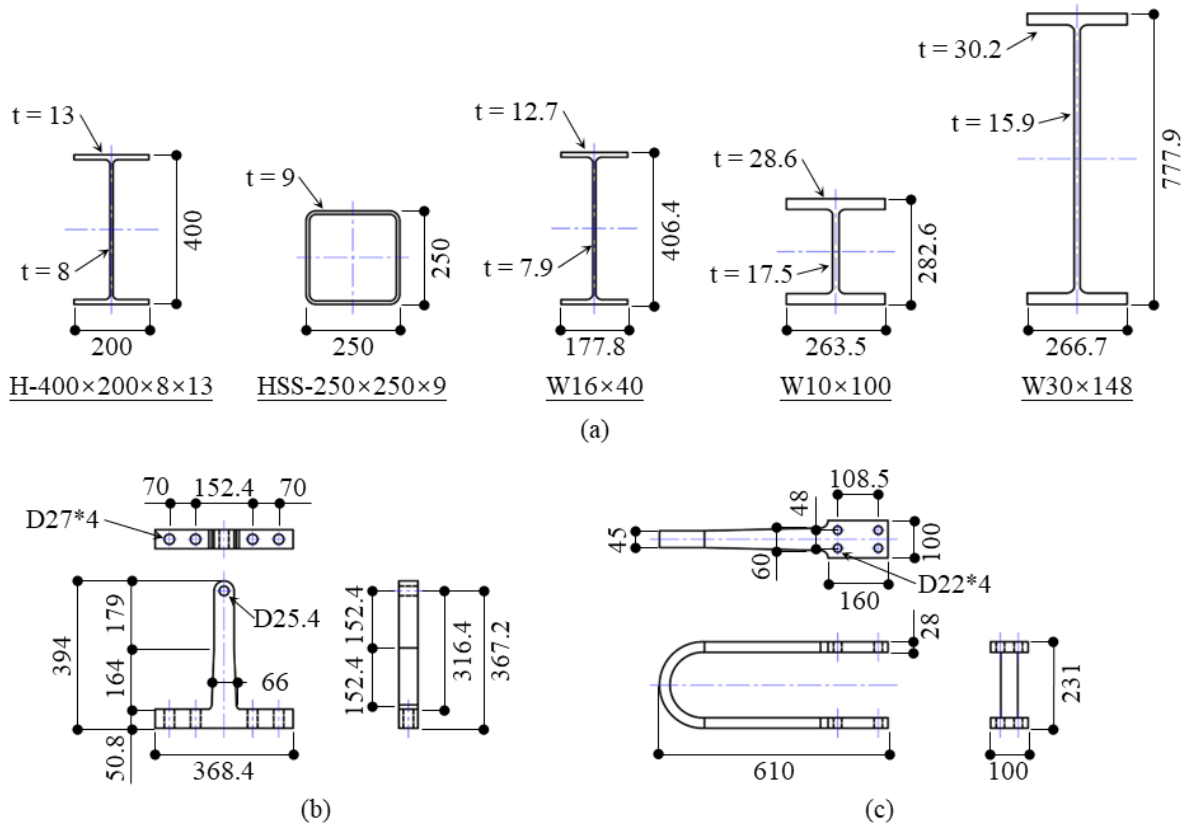


Figure 3.2 Major cross sections and FLCs [Unit: mm]

(a) Major cross sections; (b) TFLC; (c) UFLC

3.2.2 Configuration of connections between the MRF and spines

In the tests, two connection configurations were applied without and with the FLCs activated, forming the MRF-Spine and MRF-Spine-FLC systems, respectively. Connections were specially designed to achieve fast conversion on site. At each connection, a stiffening beam was bolted to the MRF girder at mid-span. A thick connection plate was welded to the spine at the corresponding location of the stiffening beam. A pair of “bearing-and-tie” devices were set at each connection to provide constraint on the out-of-plane rotation of the spines, which slid on the surface of the stiffening beam. A pair of TFLCs were set at 1FL, while a pair of UFLCs (UD-40 damper from Nippon Steel Engineering) set at 3FL. Figure 3.3 illustrates the configuration of the connections for both MRF-Spine and MRF-Spine-FLC systems.

The key for system change, or more specifically, activation of the FLCs, was the bolts connecting the stiffening beam and the connection plate. In the MRF-Spine system shown in Figure 3.3(a), the bolts in the red boxes were fastened in the MRF-Spine system, allowing (theoretically) no relative motion at the connections, forming the MRF-Spine system, in which the force was transferred through these bolted connections. In the MRF-Spine-FLC system shown in Figure 3.3(b), these bolts at the lower floors were removed, allowing relative motion at these connections. Force was then transferred through the FLCs at

the 1st and 3rd floors, and not force transferred at the 2nd floor. The top floor bolts were kept ensuring that the spines and the MRF worked together.

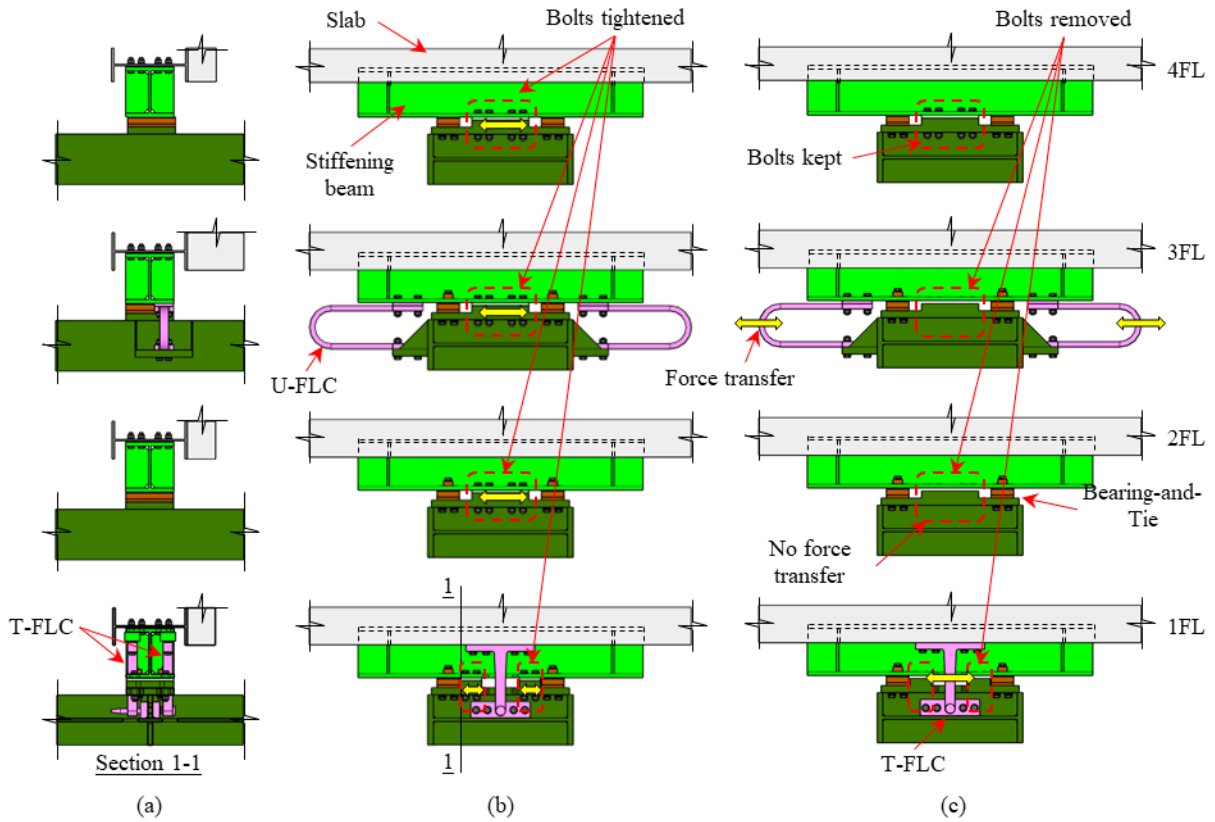


Figure 3.3 Connection configuration

(a) Elevation of the connections (Section at 1FL); (b) Plan of the MRF-Spine system; (c) Plan of the MRF-Spine-FLC system

3.3 MATERIAL AND MEMBER PROPERTY

3.3.1 Material property

Table 3.2 summarizes the material properties of the test specimen. Due to the lack of coupon results, nominal strength of the ASTM A992 steel was used.

Table 3.2 Material properties of the specimen

Material	Yield strength [MPa]	Tensile strength [MPa]	Yield to Tensile Ratio	Compression strength [MPa]	Note
BCR295	361	431	0.78	-	Coupon
SN490B	419	537	0.86	-	Coupon
ASTM A992	345	448	0.77	-	Nominal
Concrete	-	-	-	35	4FL
(cylindrical)	-	-	-	47	1FL/3FL
	-	-	-	37	2FL

3.3.2 Member capacity

Using the above material strength and nominal dimension of the steel members, the member capacity can be calculated as follows:

$$M_{p,bc} = Z_{p,bc} * f_{y,bc} \quad (3.1)$$

$$V_{p,p} = \tau_y \cdot t_p \cdot d_{c,p} \quad (3.2)$$

Where,

$M_{p,bc}$ is the plastic moment of columns and beams.

$Z_{p,bc}$ is the plastic section modulus of the columns and beams.

$f_{y,bc}$ is the yielding strength of the corresponding material.

$V_{p,p}$ is the plastic shear force of the panel zone.

τ_y is the shear strength of the panel material.

t_p is the thickness of the panel zone.

$d_{c,p}$ is the width of the panel zone.

Table 3.3 summarizes the calculated member capacity and capacity ratio. Capacity ratios were calculated in comparison to the beam plastic moment. The plastic moment of the columns was summed up. The panel plastic moment was calculated using the panel plastic shear times panel height. Capacity ratio indicates that the panel zones are the weakest at all locations, while the column capacity is close to the beam at lower floors. Considering the possible overstrength brought by the slab, a strong-beam-weak-column mechanism is expected at the lower three stories, as will be seen in the following chapter.

Table 3.3 Member properties of the specimen

Story/Floor	Member	$M_{p,bc}$ [kN·m]	$V_{p,p}$ [kN]	Ratio
1~3	Beam	549	-	1.0
	Column	271	-	1.1
	Panel zone	-	906	0.7
4	Beam	414	-	1.0
	Column	734	-	1.9
	Panel zone	-	873	0.9

3.3.3 Hysteresis of FLCs

Properties of the FLCs used in the test were obtained by static cyclic loading. The UFLCs were tested by Nippon Steel Engineering [72]. The TFLCs were tested by Lehigh University prior to the shake table tests and by Hokkaido University after the shake table tests. Figure 3.4 illustrates the hysteresis curves of the FLCs. The elastic stiffness and yielding force of the devices were evaluated using the backbone curve of the hysteresis and summarized in Table 3.4. Details of the static test on the FLCs conducted at Hokkaido University can be found in Appendix B.

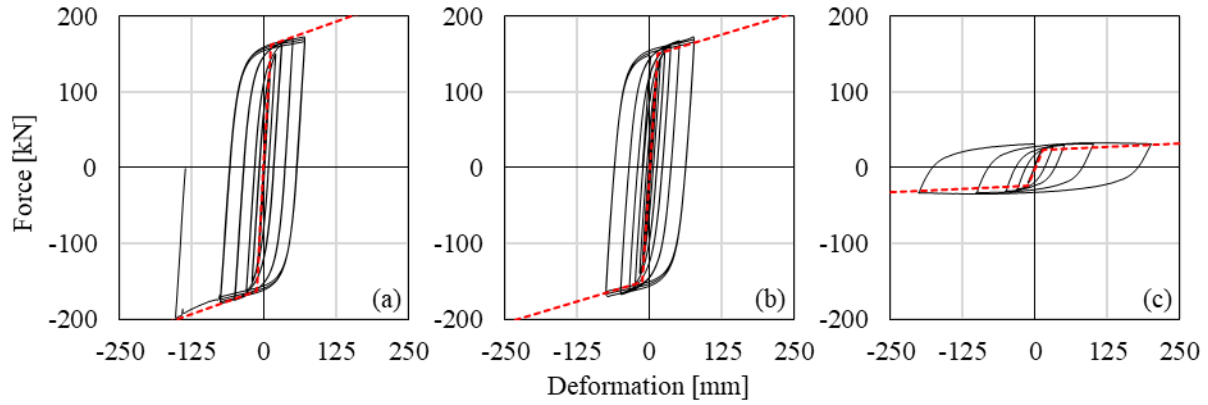


Figure 3.4 Hysteresis curves of the FLCs

(a) TFLC, Hokkaido University; (b) TFLC, Lehigh University; (c) UFLC, Nippon Steel Engineering

Table 3.4 Mechanical properties of the FLCs evaluated using a bilinear model

Device	Data source	F_y [kN]	K_e [kN/mm]	Hardening ratio
TFLC	Hokkaido University	162.9	13.3	0.02
	Lehigh University	150.2	11.6	0.03
UFLC	Nippon Steel Engineering	24.1	1.8	0.02

3.4 LOADING PLAN

3.4.1 Excitations

Selection of the input ground motion followed the following basic principles: 1) Spectral pseudo-acceleration should be large near the higher mode periods; 2) Peak story drift ratio for the MRF with the spine should be moderate; 3) Yielding should not happen to the spine; and 4) Scale factor of the ground motions should be less than 1.5. Considering these principles, the fault normal component of the LA Sepulveda VA Hospital station record in the 1994 Northridge earthquake was selected. However, preliminary numerical simulations showed that the Northridge VA motion was not strong enough to cause significant damage in the specimen. Considering this, and to make the test result comparable to similar results in Japan, the NS component of the JMA Kobe record was also selected.

Figure 3.5 illustrates the time history and 2% damping pseudo-acceleration and displacement spectra of the two motions, with the predicted natural periods illustrated in the spectra. It is seen that the Northridge VA motion was stronger in acceleration near the 2nd natural period, while the JMA Kobe motion was stronger in displacement near the 1st natural period. With such properties, the Northridge VA motion was expected to excite more higher mode acceleration responses in the system, while the JMA Kobe motion introduced more damage in structural members.

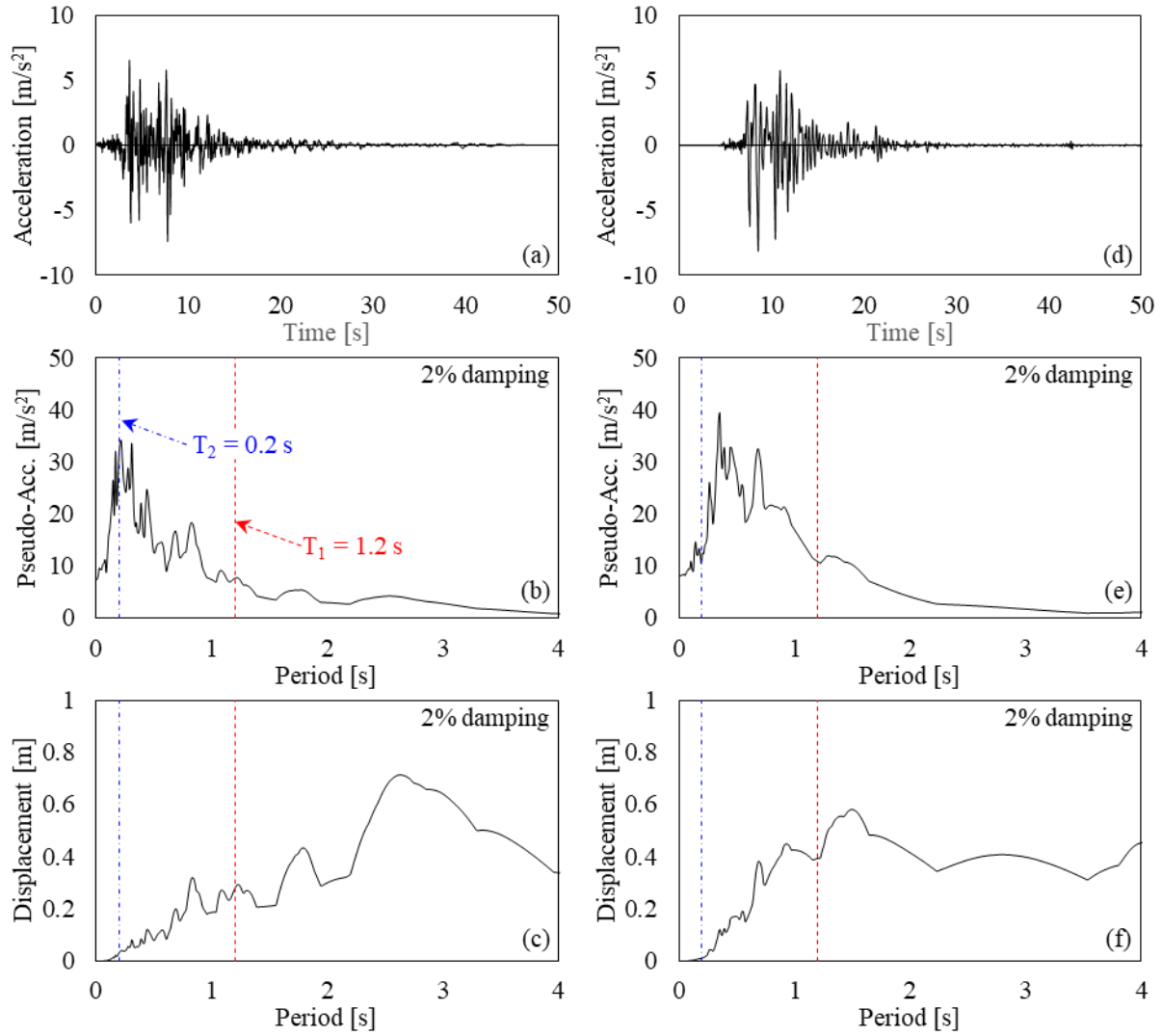


Figure 3.5 Selected ground motions and spectra

Northridge VA: (a) time history; (b) Pseudo-acceleration spectrum; (c) Displacement spectrum;
 JMA Kobe: (d) time history; (e) Pseudo-acceleration spectrum; (f) Displacement spectrum.

3.4.2 Load cases

Two scale levels were applied to the ground motions, being 40% and 100% for the Northridge VA motion and 50% and 100% for the JMA Kobe motion. Together with the two structural systems, totally 6 major excitations were conducted in the shake table tests, as summarized in Table 3.5. White noise excitations were conducted before and after each major excitation to evaluate the dynamic properties of the specimen. The white noise excitation was a synthetic signal with RMS (Root Mean Square) of 0.55m/s^2 and bandwidth of 0 to 35Hz. The original white noise signal was triaxial, but in the test only the Y direction was used.

Table 3.5 Summary of excitations

Load case	Ground motion	Scale level	PGA [m/s^2]	System
Case 01	White noise	100%	0.5	MRF-Spine
Case 02	Northridge VA	40%	3.0	
Case 03	White noise	100%	0.5	
Case 04	Northridge VA	100%	7.4	
Case 05	White noise	100%	0.5	
Case 06	White noise	100%	0.5	MRF-Spine-FLC
Case 07	Northridge VA	40%	3.0	
Case 08	White noise	100%	0.5	
Case 09	Northridge VA	100%	7.4	
Case 10	White noise	100%	0.5	
Case 11	JMA Kobe	50%	4.1	
Case 12	White noise	100%	0.5	
Case 13	JMA Kobe	100%	8.2	
Case 14	White noise	100%	0.5	

3.5 INSTRUMENTATION

3.5.1 General

Various sensors were used for the test instrumentation. Uniaxial and biaxial strain gauges were set at columns and spines to obtain the internal force distribution. Displacement transducers were set to measure the local deformation at interested locations. Triaxial accelerometers were placed at each floor to measure the floor acceleration. An independent optical measurement system was used to get the story drift. Figure 3.6 illustrates the overall layout of the sensors (not all cameras illustrated). Table 3.6 summarizes their general information. Details of sensor information are introduced in the following sections. The full list of channel tags can be found in Appendix B.

Table 3.6 Summary of sensor information

Type	Brand and Model	# of sensors	# of channels	Note
Strain gauges	TML FLA-5-11	88	88	Uniaxial
	TML FCA-5-11	8	16	Biaxial
Displacement transducer	TML SDP-50CT	4	4	Strain gauge type
	TML SDP-100CT	12	12	
	TML CDP-25	16	16	Laser type
	KEYENCE LK-500	28	28	
Accelerometer	Tokyo Keiki TA-25E-10-1	8	24	Triaxial servo type
Load cell	TOMOE TORD-PS-750KN	2	4	Biaxial pin type
E-Defense total			192	
Optical system	-	8	16	Independent system
Cameras	-	23	23	Independent system

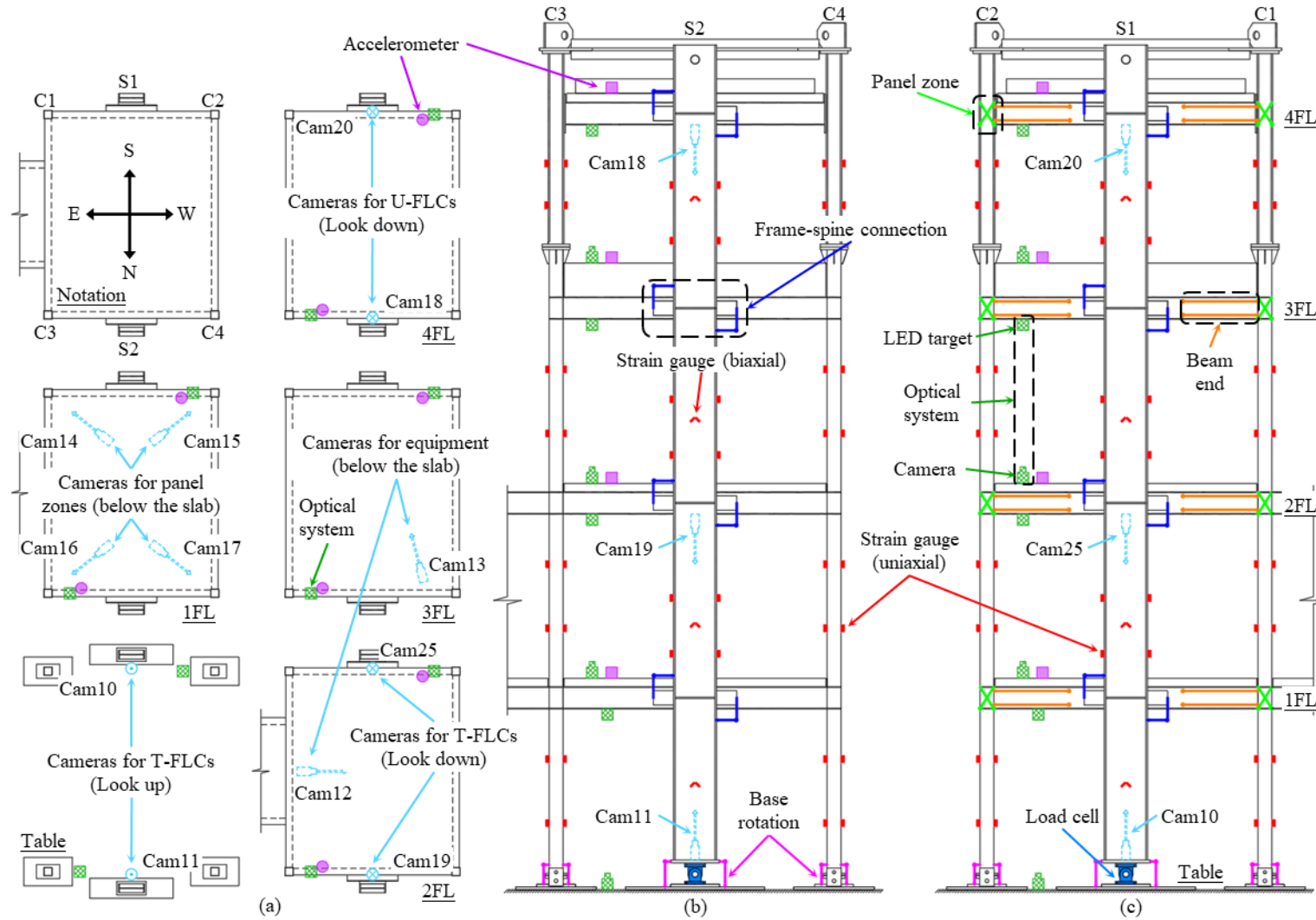


Figure 3.6 Layout of sensors on the specimen
 (a) Plan of typical floor; (b) North frame; (c) South frame.

3.5.2 Arrangement of strain gauges

All four columns and both spines were instrumented. For the column, at two sections within each story, one uniaxial strain gauge was placed on each of the east and west surface of the column with the intention of measuring axial force and bending moment. For the spine, at the second through fourth stories, a total of four uniaxial strain gauges were placed at two sections at two flanges and one biaxial strain gauge was placed on the web with the intention of measuring, via calibration with the load cell reading, shear force. At the first story, one pair of biaxial strain gauges were placed with the intention to compare the shear force measured from strain gauges against the measurement from the load cell.

Figure 3.7 illustrates the general layout of strain gauges on the members. Specific values of D1 to D3 are summarized in Table 3.7. For the spines, the distances were measured from centerline of the upper and lower stiffeners. For the columns, the distances were measured from the bottom and top surfaces of the upper and lower beams, respectively. For both the spines and columns, lower reference at the 1st story was taken as the center of column base pins and spine base load cells (NOT the top of end plates). The layout is the same for all columns and spines, respectively.

For all the strain gauges, regardless of uniaxial or biaxial, reading is positive when the gauge is in tension and negative in compression.

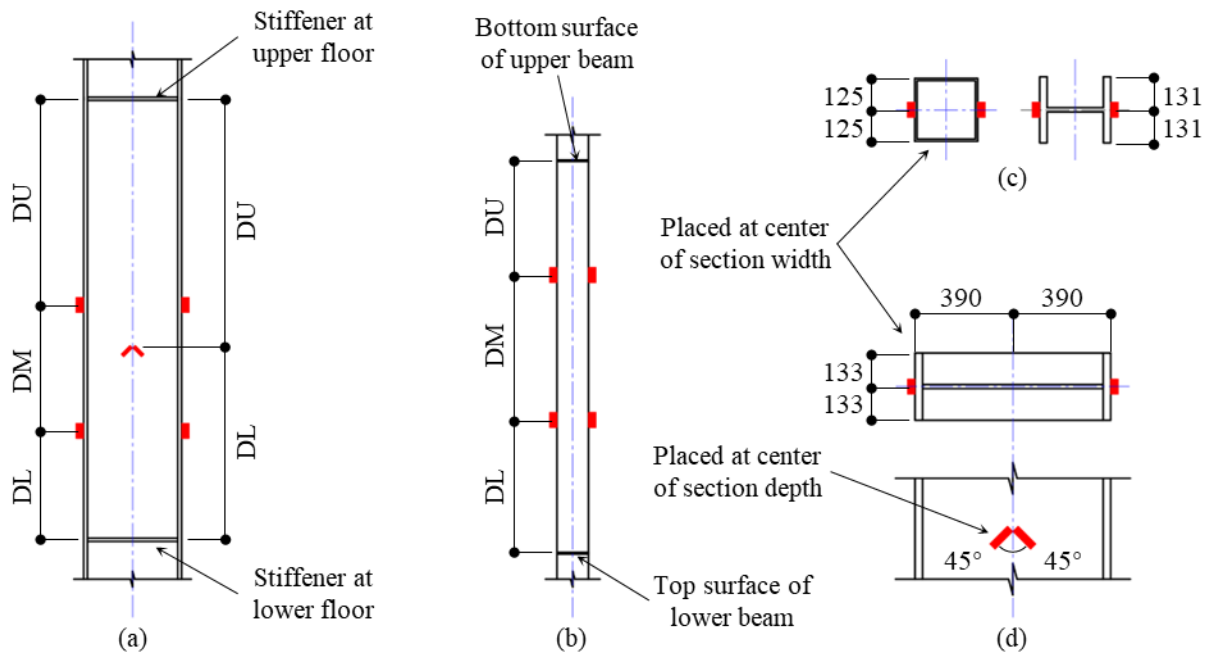


Figure 3.7 Layout of strain gauges [Unit: mm]

- (a) Vertical location on the spines; (b) Vertical location on the columns;
(c) Horizontal location on the columns; (d) Horizontal location on the spines.

Table 3.7 Reference distances of strain gauges [Unit: mm]

Story	Column Uniaxial			Spine Uniaxial			Spine Biaxial	
	DL	DM	DU	DL	DM	DU	DL	DU
4	1560	850	687	1200	1000	1300	2000	1500
3	600	1750	750	850	1000	1650	1530	1970
2	600	1750	750	780	1000	1720	1340	2160
1	800	1750	515	-	-	-	1630	1515

3.5.3 Arrangement of displacement transducers

Displacement transducers were used to measure local deformation of the specimen, including (a) MRF-Spine relative displacement, (b) beam-end rotation, (c) panel-zone shear deformation, and (d) column/spine base rotation. MRF-Spine relative displacement was measured at all occurrences: two frames at four floors, totaling 8 locations. Beam-end rotation and panel-zone shear deformation were measured at the South Frame at four floors and two ends, totaling 8 locations. Column and spine base rotation were measured at all 6 occurrences. Two transducers were used for each measurement location. LK-500 transducers were used for (a) and (d), while SDP-50/100CT and CPD-25 transducers for (b) and (c), respectively. Figure 3.8 illustrates the general layout of the transducers. Specific values of the transducer locations are summarized in Table 3.8. All transducer pairs were placed symmetrically about the member centerline, unless specifically mentioned.

For all the transducers, the reading is positive when the target point moves away from the transducer, and negative when the target point moves towards the transducer.

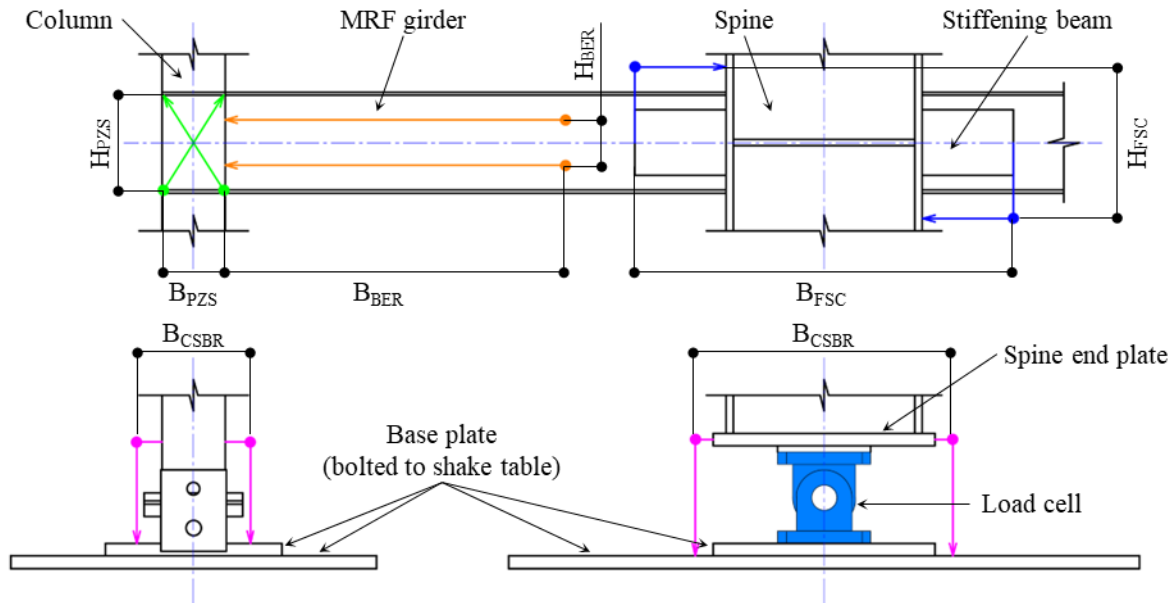


Figure 3.8 Layout of displacement transducers

Table 3.8 Reference distances of displacement transducers [Unit: mm]

Floor	Measurement	Parameters	C1	C2	C3	C4	S1	S2
4	Panel zone shear	B _{PZS}	256	256	-	-	-	-
		H _{PZS}	397	398	-	-	-	-
	Beam end rotation	B _{BER}	1280	1280	-	-	-	-
		H _{BER}	180	180	-	-	-	-
	MRF-Spine connection deformation	B _{FSC}	-	-	-	-	1500	1500
		H _{FSC}	-	-	-	-	592	605
3	Panel zone shear	B _{PZS}	236	238	-	-	-	-
		H _{PZS}	377	379	-	-	-	-
	Beam end rotation	B _{BER}	1350	1350	-	-	-	-
		H _{BER}	180	180	-	-	-	-
	MRF-Spine connection deformation	B _{FSC}	-	-	-	-	1500	1500
		H _{FSC}	-	-	-	-	607	605
2	Panel zone shear	B _{PZS}	232	243	-	-	-	-
		H _{PZS}	382	389	-	-	-	-
	Beam end rotation	B _{BER}	1350	1350	-	-	-	-
		H _{BER}	180	180	-	-	-	-
	MRF-Spine connection deformation	B _{FSC}	-	-	-	-	1500	1500
		H _{FSC}	-	-	-	-	608	602
1	Panel zone shear	B _{PZS}	238	242	-	-	-	-
		H _{PZS}	387	376	-	-	-	-
	Beam end rotation	B _{BER}	1350	1350	-	-	-	-
		H _{BER}	180	180	-	-	-	-
	MRF-Spine connection deformation	B _{FSC}	-	-	-	-	1500	1500
		H _{FSC}	-	-	-	-	604	610
Table	Base rotation	B _{CSBR}	454	454	452	452	1022	1019

3.5.4 Arrangement of accelerometers

Two triaxial accelerometers were set at each floor, glued to the concrete slab surface. At each floor, the accelerometers were located at the south-west (C2) and north-east (C3) corners. Due to limitation on channel numbers, no accelerometers were set at the shake table surface, where the feedback signals of the shake table control system were used. Figure 3.9 illustrates the location of accelerometers on the slab. Since the layout was the same at all floors, the values are written directly.

For all the accelerometers, positive directions were set as north for x direction, west for y direction and up for z direction.

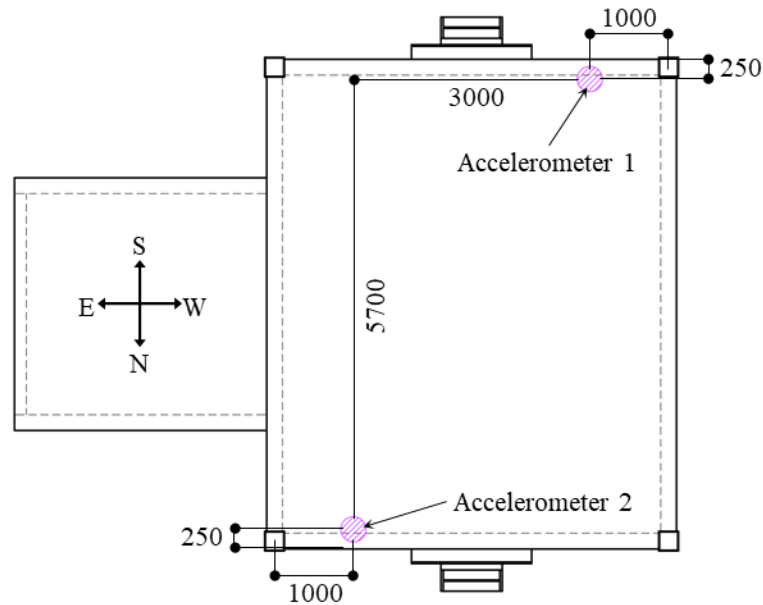


Figure 3.9 Layout of accelerometers at each floor [Unit: mm]

3.5.5 Arrangement of load cells

A pin-type load cell was set at the foot of either spine, serving as the pin and measuring the horizontal and vertical forces at the same time. Locations of load cells can be found in Figure 3.6 and Figure 3.8.

For both load cells, the positive directions were set as west for horizontal and up for vertical forces.

3.5.6 Arrangement of optical measurement system

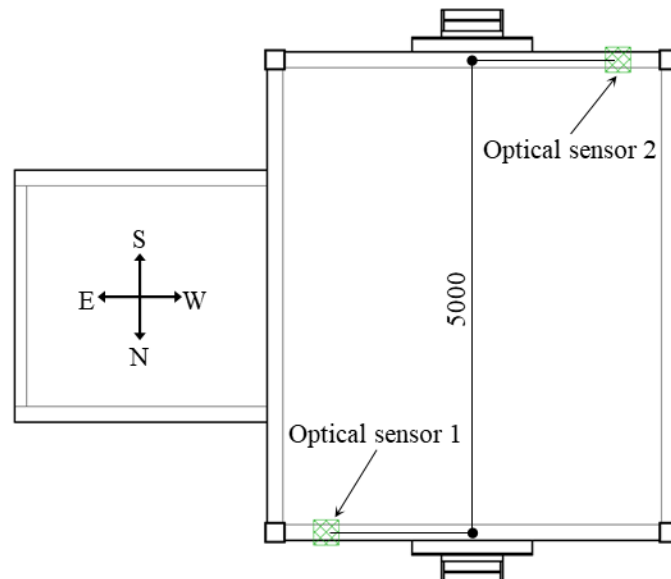


Figure 3.10 Layout of optical sensors at each floor [Unit: mm]

The optical measurement system was not connected to the Data Acquisition System (DAQ) of E-Defense but a stand-alone system. This system was used to measure the story drift in two horizontal directions of all stories. Each location contains a camera set at the slab top surface and an LED target at

the upper floor girder bottom surface. Two frames were all measured, totaling 8 pairs of devices. The devices were placed near the accelerometers on the slab, at the south-west (C2) and north-east (C3) corners, respectively. On each floor, the distance in the NS direction of the two sensors is certain to be the distance of axis (5000 mm), while distance of the other direction is unknown. Thus, rotation will be calculated using this direction of measurement (Y direction) only. Figure 3.10 illustrates layout of optical sensors at each floor. Note that location of sensors set on the shake table was slightly different then in the drawing, but their distance of 5000 mm was ensured.

For the X and Y directions, reading is positive when the upper floor moves to South and West relative to the lower floor, respectively.

3.5.7 Arrangement of cameras

During the test, 23 cameras were used to capture the global and local behavior of the specimen. All the cameras were connected to the E-Defense video capturing system and thus were synchronized. Cameras set on the specimen (totaling 12 locations) are shown in Figure 3.6. Camera set outside the specimen, with 9 fixed cameras of the test hall and 2 portable cameras set at the south and north sides of the specimen are illustrated in Figure 3.11. Information of all cameras is summarized in Table 3.9. Note that camera tag numbers are not continuous due to the setting of the system.

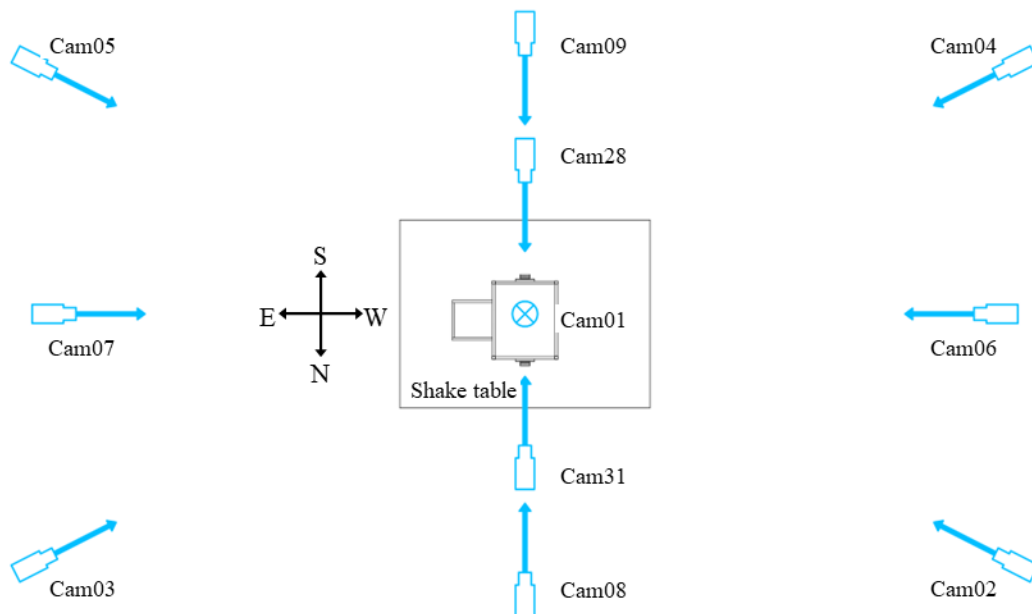


Figure 3.11 Cameras set outside the specimen

Table 3.9 Information of cameras

Camera tag	Camera type	Camera location	Target scene
Cam 01	Test hall fixed	Ceiling	Specimen global movement
Cam 02		North-West	
Cam 03		North-East	
Cam 04		South-West	
Cam 05		South-East	
Cam 06		West	
Cam 07		East	
Cam 08		North	
Cam 09		South	
Cam 10	On specimen	Spine-1 at shake table level	T-FLC from below
Cam 11		Spine-2 at shake table level	T-FLC from below
Cam 12		East side at Story 2	NICU equipment
Cam 13		North side at Story 3	Dialysis room equipment
Cam 14		South-East under 1FL slab	Panel zone of C1
Cam 15		South-West under 1FL slab	Panel zone of C2
Cam 16		North-East under 1FL slab	Panel zone of C3
Cam 17		North-West under 1FL slab	Panel zone of C4
Cam 18		Spine-2 at 4FL level	U-FLC from above
Cam 19		Spine-2 at 2FL level	T-FLC from above
Cam 20		Spine-1 at 4FL level	U-FLC from above
Cam 25		Spine-1 at 2FL level	T-FLC from above
Cam 28	On floor	South of specimen on floor	South elevation of specimen
Cam 31		North of specimen on floor	North elevation of specimen

3.6 DATA REDUCTION

3.6.1 General

Data obtained from the E-Defense DAQ system require further reduction to get useful structure responses. Data from the optical measurement system were already treated by the Waseda University team and can be readily used after synchronized with the E-Defense DAQ system. Table 3.10 summarizes the basic information of data reduction conducted. All the data were preprocessed prior to calculation.

Data reduction presented here uses a global coordinate system in which the horizontal axis points EAST, vertical axis points UP, and rotation points from horizontal to vertical. The three axes are denoted as H, V and R, respectively, as illustrated in Figure 3.1. All related measurements should be adjusted accordingly.

The data reduction scheme assumes no gravity load. Responses under gravity load is calculated separately and added to the “dynamic” responses to get the “real” responses. Determination of the initial force responses is introduced in the following section.

Table 3.10 Brief information of data reduction

Data type	Target responses	Data source
Global response	Floor acceleration	Accelerometer
	Story drift	Optical system
	Story drift ratio	
	Floor displacement	
Local deformation	Beam end rotation	TML SDP-50/100CT
	Panel zone shear deformation	TML CDP-25
	Column/spine base rotation	KEYENCE LK-500
	MRF-Spine connection horizontal deformation	
	MRF-Spine connection rotation	
Member internal force	Column axial force	Strain gauges
	Column bending moment	
	Column shear force	
	Spine axial force	
	Spine bending moment	
	Spine shear force	Column bending moment
	Beam bending moment	
	Beam shear force	
	Panel zone shear force	
	MRF-Spine connection force	

3.6.2 Data preprocessing

Raw data were first checked channel by channel and processed for further calculations. For all the channels, they were first checked for any failure or inconsistency with instrumentation plan (mixed channel tags, etc.), and then preprocessed as needed. Table 3.11 summarizes some brief information of the preprocessing applied on different data types. A full description of data preprocessing can be found in Appendix C.

Table 3.11 Brief information of data preprocessing

Data type	Preprocessing methods	Data source
Acceleration	Outlier removal	Data Acquisition System (E-Defense)
	Filtering (all channels)	
Displacement	Outlier removal	
	Filtering (necessary channels)	
Strain	Outlier removal	
	Failed gauge compensation	Optical measurement system (Waseda University)
	Yielding gauge compensation	
Story drift	Up sampling (50Hz to 200Hz)	Optical measurement system (Waseda University)
	Data synchronization with E-Defense DAQ system	

3.6.3 Global response

Global response refers to the floor and story responses of the specimen. Floor acceleration, story drift, story drift ratio, and floor displacement were calculated. Positive direction of the accelerometers was WEST, while the optical sensor was EAST.

3.6.3.1 Floor acceleration

Floor acceleration is calculated as the average of data from the two accelerometers in the East-West direction (Y axis of the sensors) as follows:

$$FACC_i = \frac{-ACC_{i,1} - ACC_{i,2}}{2} \quad (3.3)$$

Where,

$FACC_i$ is the floor acceleration at the i-th floor.

$ACC_{i,1}$ and $ACC_{i,2}$ are signals from the two accelerometers at the i-th floor.

3.6.3.2 Story drift, story drift ratio and floor displacement

The story drift, story drift ratio and floor displacement are calculated using story drift measurement of the optical system. The story drift in the East-West direction (Y axis of the sensors) from the two sensors of the same story are averaged as the specimen story drift. Story drift ratio and floor displacement are then calculated from the story drift data. The responses are calculated as follows:

$$SD_i = \frac{SD_{i,1} + SD_{i,2}}{2} \quad (3.4)$$

$$SDR_i = \frac{SD_i}{H_i} \quad (3.5)$$

$$FDSP_i = \sum_1^i SD_i \quad (3.6)$$

Where,

SD_i is the specimen story drift.

$SD_{i,1}$ and $SD_{i,2}$ are the measurement results from the optical system.

SDR_i is the story drift ratio.

H_i is the story height, being 3425 mm for the 1st story and 3500 mm for the 2nd to 4th story.

$FDSP_i$ is the floor displacement.

3.6.4 Local deformation

Local deformation refers to beam end rotation, panel zone shear deformation, MRF-Spine connection lateral deformation and rotation, and column/spine base rotation. All sensors except for the beam end

rotation ones share the same polarity that positive results are given when the distance between the sensor and the target increases. The beam end rotation ones are opposite since they were connected by strings instead of direct contact.

3.6.4.1 Beam end rotation

Beam end deformation was measured at both ends of the MRF girder in the south frame. Figure 3.12 illustrates the layout of the measurement.

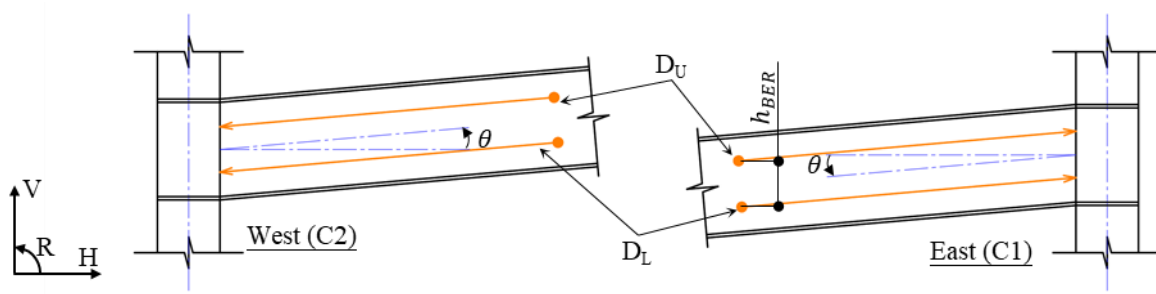


Figure 3.12 Beam end rotation measurement

For both sides, the beam end rotation and horizontal deformation are calculated as follows:

$$\theta = \frac{D_U - D_L}{H_{BER}} * \text{sign}_{EW} \quad (3.7)$$

$$\Delta = -\frac{D_U + D_L}{2} \quad (3.8)$$

Where,

θ is the beam end rotation in radian, $\theta < 0$.

Δ is the beam end horizontal deformation, positive for elongation and negative for contraction.

D_U is the measurement from the upper transducer. $D_U > 0$ at west, $D_U < 0$ at east.

D_L is the measurement from the lower transducer. $D_L < 0$ at west, $D_L > 0$ at east.

H_{BER} is the distance between two transducers.

sign_{EW} is a factor according to the location: west, -1; east, 1.

3.6.4.2 Panel zone shear deformation

Panel zone shear deformation was measured at both panels of the south frame columns (C1 and C2). Figure 3.13 illustrates the layout of the measurement. The plot only shows the HSS section, but the same layout also applies to the H section at the 4th story.

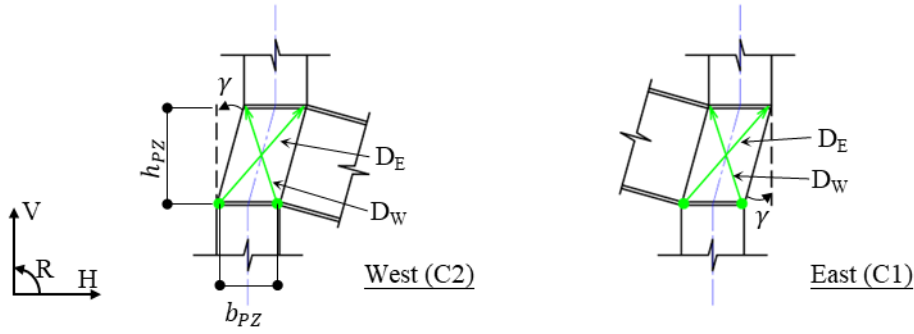


Figure 3.13 Panel shear deformation measurement

For both panels, the shear angle is calculated as follows:

$$\gamma_E = -\tan^{-1} \left(\frac{\sqrt{(L_{PZS} + D_E)^2 - H_{PZS}^2} - B_{PZS}}{H_{PZS}} \right) \quad (3.9)$$

$$\gamma_W = -\tan^{-1} \left(\frac{-\sqrt{(L_{PZS} + D_W)^2 - H_{PZS}^2} + B_{PZS}}{H_{PZS}} \right) \quad (3.10)$$

$$\gamma = \text{average}(\gamma_E, \gamma_W) \quad (3.11)$$

Where,

γ_E is the panel zone shear angle deduced using D_E .

γ_W is the panel zone shear angle deduced using D_W .

γ is the panel zone shear angle in radian, taken as the average of γ_E and γ_W , $\gamma < 0$.

D_E is the measurement from the lower-west to upper-east transducer, $D_E > 0$.

D_W is the measurement from the lower-east to upper-west transducer, $D_W < 0$

$L_{PZS} = \sqrt{B_{PZS}^2 + H_{PZS}^2}$ is the panel diagonal length.

B_{PZS} is the panel width.

H_{PZS} is the panel height.

3.6.4.3 MRF-Spine connection horizontal deformation and rotation

All MRF-Spine connections were instrumented during the test, totaling 8 locations. At each location, the horizontal deformation and rotation are calculated. Figure 3.14 illustrates the layout at the connections. The transducers placed here are tagged as D1 and D2 for the WEST and EAST sensors, respectively. Thus, location of D1 and D2 is different at the south and north frames.

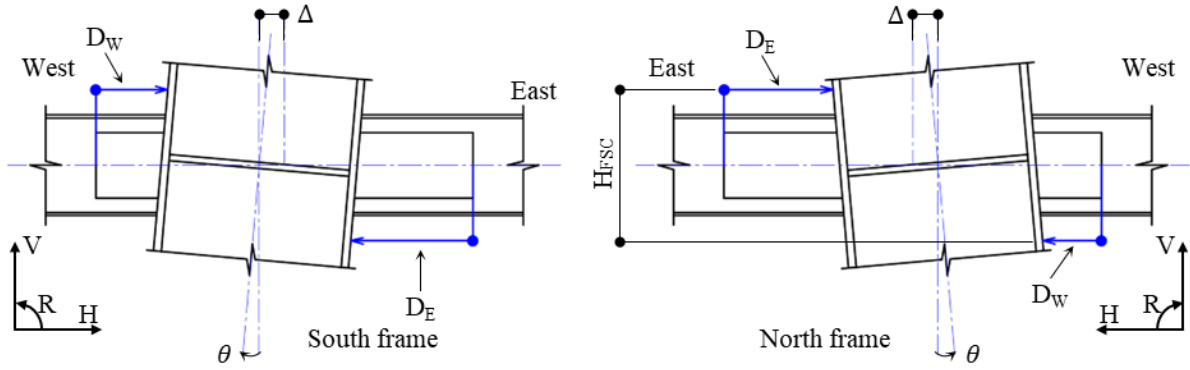


Figure 3.14 MRF-Spine connection measurement

For both frames, the horizontal deformation and rotation at the connections are calculated as follows:

$$\Delta = \frac{D_W - D_E}{2} \quad (3.12)$$

$$\theta = \frac{D_W + D_E}{H_{FSC}} * \text{sign}_{NS} \quad (3.13)$$

Where,

Δ is the horizontal deformation, $\Delta < 0$.

θ is the rotation, $\theta < 0$.

D_W is the measurement from the west transducer.

D_E is the measurement from the east transducer.

H_{FSC} is the distance between two transducers.

sign_{NS} is a factor according to the location: south, -1; north, 1.

3.6.4.4 Column and spine base rotation

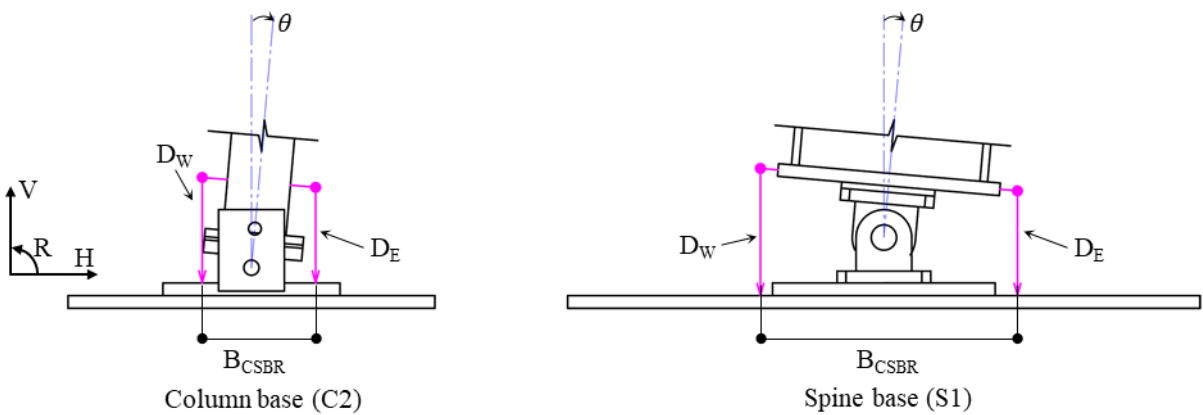


Figure 3.15 Column and spine base measurement

Base rotation was measured at all 4 columns and 2 spines. At each location, the west transducer is tagged as D1 while the east as D2. Equations of all 6 locations are the same. Figure 3.15 illustrates the layout of base rotation measurement, showing a column (C2) and a spine (S1) from the south frame.

At all the locations, base rotation and vertical deformation are calculated as follows:

$$\theta = \frac{D_E - D_W}{B_{CSBR}} \quad (3.14)$$

$$\Delta = \frac{D_E + D_W}{2} \quad (3.15)$$

Where,

θ is the column/spine base rotation angle, $\theta < 0$.

Δ is the vertical deformation at the base, positive for elongation and negative for contraction

D_W is the measurement from the west transducer, $D_W > 0$.

D_E is the measurement from the east transducer, $D_E < 0$.

B_{CSBR} is the distance between two transducers.

3.6.5 Member internal force

The internal force of structural members is calculated in generally two steps. The column and spine internal forces are calculated from strain gauge measurement, while the other members calculated using these results.

3.6.5.1 Column force

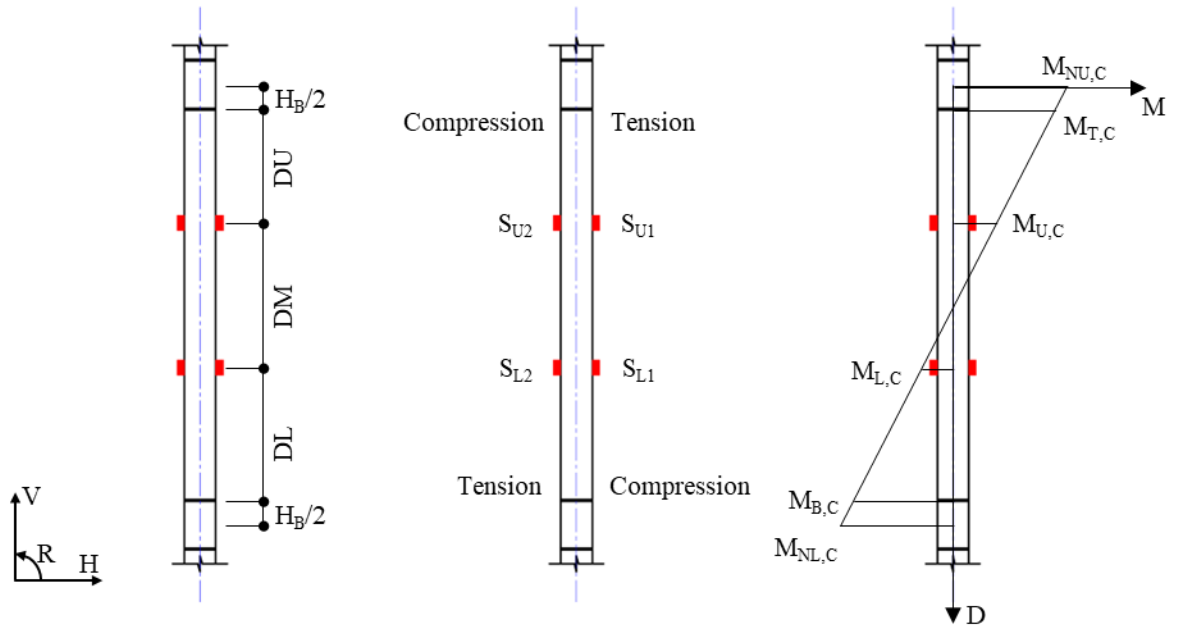


Figure 3.16 Strain and internal force pattern of columns

Column forces are calculated using uniaxial strain gauges. Axial force and moment of the measured sections are calculated first. Shear force is then calculated using the section moment. Figure 3.16 illustrates the scheme of column force calculation, with a HSS column as the example. The M-D coordinate is used for moment extrapolation. It is assumed that axial force is constant within the column

of a story, and the value is taken as the average of the two measured sections. Calculation of the H section of the 4th story follows the same way.

For all the columns, the axial force, bending moment and shear force can be calculated as follows:

$$N_{U,C} = \frac{S_{U1} + S_{U2}}{2} * EA \quad (3.16)$$

$$N_{L,C} = \frac{S_{L1} + S_{L2}}{2} * EA \quad (3.17)$$

$$N_{A,C} = \frac{N_{U,C} + N_{L,C}}{2} \quad (3.18)$$

$$M_{U,C} = \frac{S_{U1} - S_{U2}}{2} * EZ \quad (3.19)$$

$$M_{L,C} = \frac{S_{L2} - S_{L1}}{2} * EZ \quad (3.20)$$

$$V_{T,C} = \frac{M_{U,C} + M_{L,C}}{DM} \quad (3.21)$$

$$V_{B,C} = -\frac{M_{U,C} + M_{L,C}}{DM} \quad (3.22)$$

Where,

S_{U1} , S_{U2} , S_{L1} and S_{L2} are strain of the uniaxial gauges, $S_{U1}, S_{L2} > 0$, $S_{U2}, S_{L1} < 0$.

$N_{U,C}$ and $N_{L,C}$ are the axial force at the upper and lower sections. (NOT global coordinate system!)

$N_{A,C}$ is the average axial force of the column. (NOT global coordinate system!)

$M_{U,C}$ and $M_{L,C}$ are the section moment.

$V_{T,C}$ and $V_{B,C}$ are the shear force at the top and bottom section, taking the column as free body.

E is the Young's modulus of the steel.

A is the section area.

Z is the elastic section modulus.

DM is the section distance, as listed previously in Table 3.7.

In addition to the above column forces, moment at column ends (M_T and M_B) and beam-column interception node (M_{NU} and M_{NL}) can be obtained via linear extrapolation using the M-D coordinate system illustrated in Figure 3.16 and conversion back to global coordinate system as follows:

$$M_{T,C} = M_{U,C} + \frac{M_{U,C} + M_{L,C}}{DM} * DU \quad (3.23)$$

$$M_{NU,C} = M_{U,C} + \frac{M_{U,C} + M_{L,C}}{DM} * (DU + \frac{H_b}{2}) \quad (3.24)$$

$$M_{B,C} = M_{L,C} + \frac{M_{U,C} + M_{L,C}}{DM} * DL \quad (3.25)$$

$$M_{NL,C} = M_{L,C} + \frac{M_{U,C} + M_{L,C}}{DM} * (DL + \frac{H_b}{2}) \quad (3.26)$$

Where,

$M_{T,C}$ is the moment at the top end of the column.

$M_{B,C}$ is the moment at the bottom end of the column.

$M_{NU,C}$ is the moment at the upper beam-column intersection node.

$M_{NL,C}$ is the moment at the lower beam-column intersection node.

DU and DL are the distance of measured sections to the upper and lower column ends.

H_b is the height of the upper and lower MRF girders.

It should be noted that for the 1st story where the bottom of the column is a pin, the “bottom section” and “lower node” are set at the same location, the center of the pin. Thus, for this story, $M_{B,C}$ and $M_{NL,C}$ are the same.

All column force and moment, except for the axial force, are in the global coordinate system. The axial force is defined as “tension positive and compression negative”.

3.6.5.2 Beam force

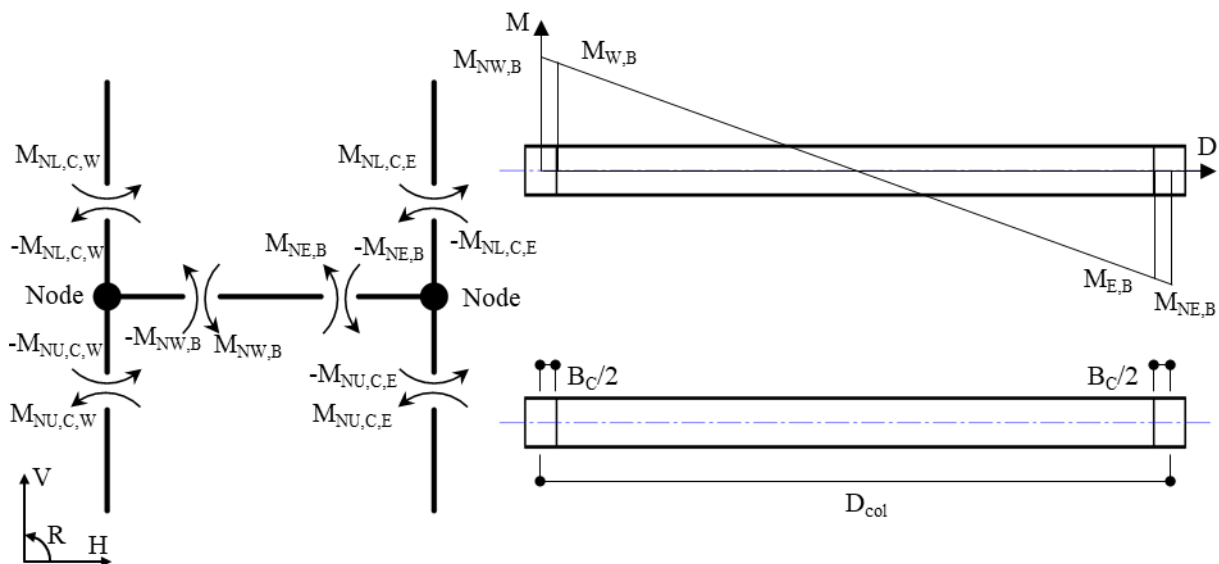


Figure 3.17 Calculation of beam moment

For the beams, axial force is neglected. Beam moment at the beam-column node is obtained using the equilibrium at the node and the extrapolated column moment. Beam moment at beam ends can be obtained from the node moment via linear interpolation, assuming constant shear force within the beam. Figure 3.17 illustrates the scheme of beam moment calculation.

From the moment equilibrium at the beam-column intersection node of both sides, there are

$$M_{NW,B} = -M_{NL,C,W} - M_{NU,C,W} \quad (3.27)$$

$$M_{NE,B} = -M_{NL,C,E} - M_{NU,C,E} \quad (3.28)$$

Where,

$M_{NL,C,W}$ and $M_{NL,C,E}$ are the column node moment of the upper story at the west and east sides.

$M_{NU,C,W}$ and $M_{NU,C,E}$ are the column node moment of the lower story at the west and east sides.

$M_{NW,B}$ and $M_{NE,B}$ are the beam node moment at the west and east sides.

Beam shear force can be calculated as

$$V_{W,B} = \frac{M_{NW,B} + M_{NE,B}}{D_{col}} \quad (3.29)$$

$$V_{E,B} = -\frac{M_{NW,B} + M_{NE,B}}{D_{col}} \quad (3.30)$$

Where,

D_{col} is the distance between column axes, being 5000mm.

$V_{W,B}$ and $V_{E,B}$ are the shear force at the west and east ends of the beam.

Moment at beam ends ($M_{W,B}$ and $M_{E,B}$) can be obtained via linear interpolation in the M-D coordinate system as follows:

$$M_{W,B} = M_{NW,B} - \frac{M_{NW,B} + M_{NE,B}}{D_{col}} * \frac{B_C}{2} \quad (3.31)$$

$$M_{E,B} = M_{NE,B} - \frac{M_{NW,B} + M_{NE,B}}{D_{col}} * \frac{B_C}{2} \quad (3.32)$$

Where,

B_C is the width of columns.

$M_{W,B}$ and $M_{E,B}$ are the beam end moment at the west and east sides.

All the beam force and moment are in the global coordinate system. Axial force is neglected.

3.6.5.3 Panel zone force

The panel zone shear force and center moment are calculated considering the column forces and beam forces acting on the panel [73]. Figure 3.18 illustrates the free body of the panel zones. Note that the figure is different from those provided in the reference, since all the force and moment plotted here are in global coordinate system.

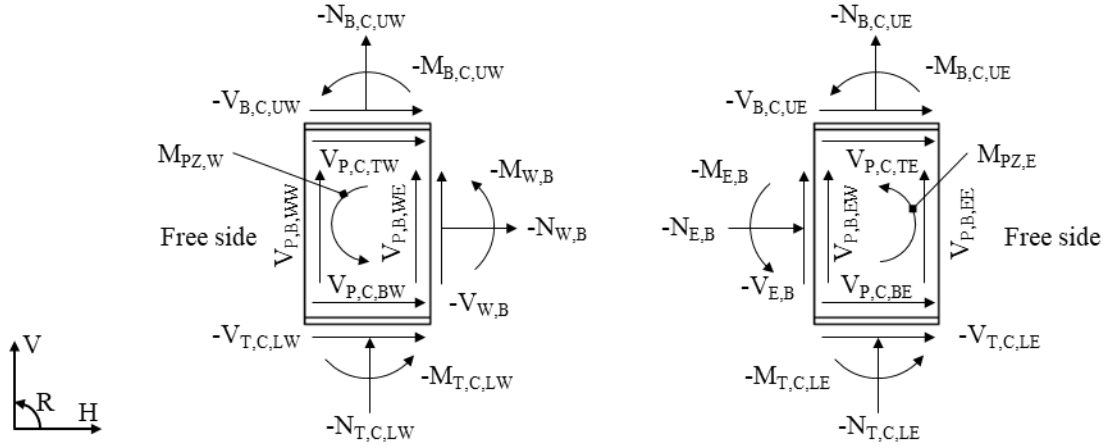


Figure 3.18 Panel force pattern and free body with beams at both sides

For the **west** panel, the panel zone shear force and panel zone center moment can be calculated as:

$$V_{P,C,TW} = \frac{M_{W,B}}{d_b} - \frac{V_{B,C,UW} - V_{T,C,LW}}{2} \quad (3.33)$$

$$V_{P,C,BW} = -V_{P,C,TW} = -\frac{M_{W,B}}{d_b} + \frac{V_{B,C,UW} - V_{T,C,LW}}{2} \quad (3.34)$$

$$M_{PZ,W} = V_{P,C,BW} * d_b \quad (3.35)$$

$$V_{P,B,WW} = -M_{PZ,W}/d_c \quad (3.36)$$

$$V_{P,B,WE} = M_{PZ,W}/d_c \quad (3.37)$$

Where,

$V_{P,C,TW}$ and $V_{P,C,BW}$ are the top and bottom horizontal shear force of the west panel.

$V_{P,B,WW}$ and $V_{P,B,WE}$ are the west and east vertical shear force of the west panel.

$M_{PZ,W}$ is the center moment of the west panel.

$M_{W,B}$ is the beam west end moment.

$V_{B,C,UW}$ is the bottom shear force of the upper column at the west side (C2 and C4).

$V_{T,C,LW}$ is the top shear force of the lower column at the west side (C2 and C4).

d_b is the center-to-center distance of the beam flanges, i.e., height of the panel.

d_c is the center-to-center distance of the column flanges, i.e., width of the panel.

For the **east** panel, the panel zone shear force and panel zone center moment can be calculated as:

$$V_{P,C,TE} = \frac{M_{E,B}}{d_b} - \frac{V_{B,C,UE} - V_{T,C,LE}}{2} \quad (3.38)$$

$$V_{P,C,BE} = -V_{P,C,TE} = -\frac{M_{E,B}}{d_b} + \frac{V_{B,C,UE} - V_{T,C,LE}}{2} \quad (3.39)$$

$$M_{PZ,E} = V_{P,C,BE} * d_b \quad (3.40)$$

$$V_{P,B,EW} = -M_{PZ,E}/d_c \quad (3.41)$$

$$V_{P,B,EE} = M_{PZ,E}/d_c \quad (3.42)$$

Where,

$V_{P,C,TE}$ and $V_{P,C,BE}$ are the top and bottom horizontal shear force of the east panel.

$V_{P,B,EW}$ and $V_{P,B,EE}$ are the west and east vertical shear force of the east panel.

$M_{PZ,E}$ is the center moment of the east panel.

$M_{E,B}$ is the beam east end moment.

$V_{B,C,UE}$ is the bottom shear force of the upper column at the east side (C1 and C3).

$V_{T,C,LE}$ is the top shear force of the lower column at the east side (C1 and C3).

d_b is the center-to-center distance of the beam flanges, i.e., height of the panel.

d_c is the center-to-center distance of the column flanges, i.e., width of the panel.

All the shear force and moment are in the global coordinate system.

3.6.5.4 Spine force

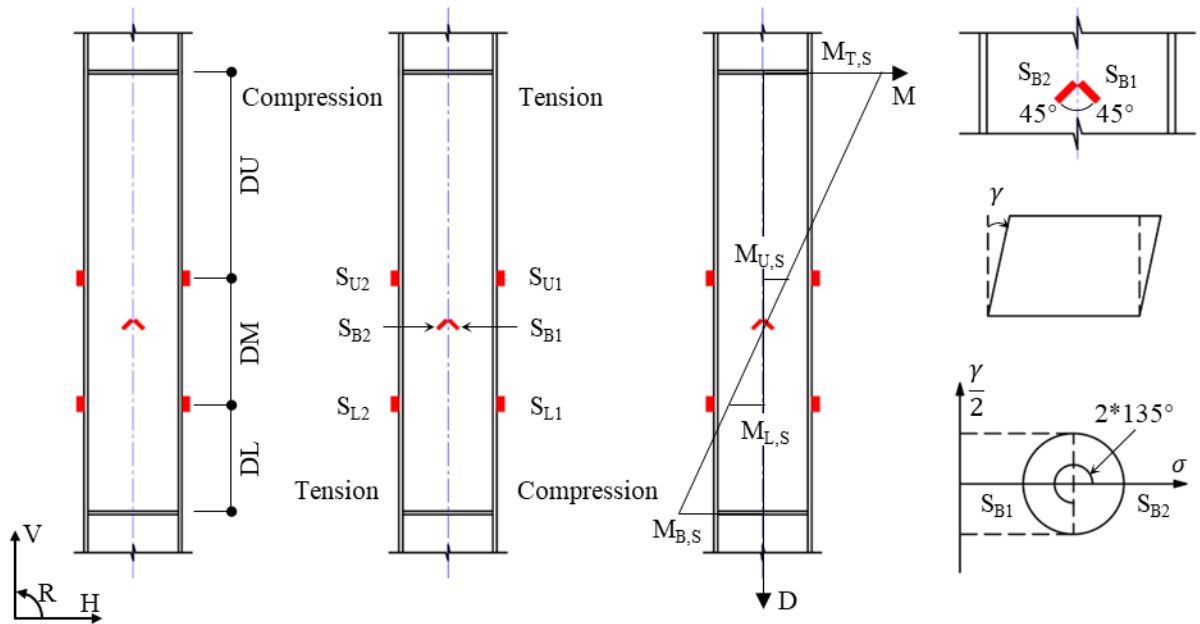


Figure 3.19 Strain and internal force pattern of the spines

Two measurements can be used for the calculation of spine forces. The uniaxial strain gauges can provide the same results as the columns, namely axial force, moment, and shear force. The biaxial strain gauges, calibrated by the load cell measurement, can provide shear force of the spines. Similar to the columns, it is assumed that axial force is constant within one story and taken as the average of two uniaxial gauge sections for the upper stories and load cell vertical force for the 1st story. Figure 3.19

illustrates the scheme of spine force measurement.

Using the uniaxial strain gauges, similar to the calculation of columns, the spine internal forces can be calculated as follows:

$$N_{U,S} = \frac{S_{U1} + S_{U2}}{2} * EA \quad (3.43)$$

$$N_{L,S} = \frac{S_{L1} + S_{L2}}{2} * EA \quad (3.44)$$

$$N_{A,S} = \frac{N_{U,S} + N_{L,S}}{2} \quad (3.45)$$

$$M_{U,S} = \frac{S_{U1} - S_{U2}}{2} * EZ \quad (3.46)$$

$$M_{L,S} = \frac{S_{L2} - S_{L1}}{2} * EZ \quad (3.47)$$

$$V_{T,S} = \frac{M_{U,S} + M_{L,S}}{DM} \quad (3.48)$$

$$V_{B,S} = -\frac{M_{U,S} + M_{L,S}}{DM} \quad (3.49)$$

$$M_{T,S} = M_{U,S} + \frac{M_{U,S} + M_{L,S}}{DM} * DU \quad (3.50)$$

$$M_{B,S} = M_{L,S} + \frac{M_{U,S} + M_{L,S}}{DM} * DL \quad (3.51)$$

Where,

S_{U1} , S_{U2} , S_{L1} and S_{L2} are strain of the uniaxial gauges, $S_{U1}, S_{L2} > 0$, $S_{U2}, S_{L1} < 0$.

$N_{U,S}$ and $N_{L,S}$ are the axial force at the upper and lower sections. (NOT global coordinate system!)

$N_{A,S}$ is the average axial force of the spine segment. (NOT global coordinate system!)

$M_{U,S}$ and $M_{L,S}$ are the section moment.

$M_{T,S}$ is the moment at the top end of the spine.

$M_{B,S}$ is the moment at the bottom end of the spine.

$V_{T,S}$ and $V_{B,S}$ are the shear force at the top and bottom section, taking the spine as free body.

E is the Young's modulus of the steel.

A is the section area.

Z is the elastic section modulus.

DU , DM and DL are the section distance, as listed previously in Table 3.7.

Due to the lack of uniaxial strain gauges at the 1st story, load cell measurement is used to generate the internal force. Note that the polarity of load cell horizontal measurement is opposite to the coordinate

system.

$$M_{T,S,1} = -F_{LC,H} * H_1 \quad (3.52)$$

$$V_{B,S,1} = F_{LC,H} \quad (3.53)$$

$$V_{T,S,1} = -F_{LC,H} \quad (3.54)$$

$$N_{A,S,1} = F_{LC,V} \quad (3.55)$$

Where,

$M_{T,S,1}$, is the top moment at the 1st story. Bottom moment is zero.

$V_{T,S,1}$ and $V_{B,S,1}$ are the top and bottom shear force at the 1st story.

$N_{A,S,1}$ is the average axial force at the 1st story. (NOT global coordinate system!)

$F_{LC,H}$ and $F_{LC,V}$ are the load cell horizontal and vertical forces.

H_1 is the distance between the center of load cell to the top section of the 1st story spine.

Spine shear force can also be calculated by linear regression of the 1st story shear strain obtained from biaxial strain gauges against the load cell force and then applying the regression factor to the shear strain of the upper stories. Equations of this calculation is as follows:

$$\gamma_i = S_{B1,i} - S_{B2,i} \quad (3.56)$$

$$k = \frac{cov(F_{LC}, \gamma_1)}{var(\gamma_1)} \quad (3.57)$$

$$V_{B,S,i} = k * \gamma_i \quad (3.58)$$

$$V_{T,S,i} = -k * \gamma_i \quad (3.59)$$

Where,

$i = 1,2,3,4$ is the story number.

$S_{B1,i}$ and $S_{B2,i}$ are the strain of the east and west legs of the biaxial gauges at the i-th story.

γ_i is the shear strain of the i-th story.

F_{LC} is the load cell horizontal force at the spine base.

k is the regression factor between the load cell force and shear strain of the 1st story.

$cov(F_{LC}, \gamma_{S,1})$ represents the covariance between the load cell force and shear strain.

$var(\gamma_{S,1})$ represents the variance of the shear strain.

$V_{B,S,i}$ is the shear force of the i-th story at the bottom section.

$V_{T,S,i}$ is the shear force of the i-th story at the top section.

Shear force calculated using the uniaxial and biaxial strain gauges was validated to be consistent. Details of the validation of data consistency can be found in Appendix B.

Considering the better resolution of uniaxial strain gauge and load cell measurement, the spine shear force and moment used for further analysis was selected as the combination of uniaxial gauges for the upper stories and the load cell for the 1st story. Results obtained from the biaxial gauges were used for cross checking only.

All spine force and moment, except for the axial force at the measured sections and their average, are in the global coordinate system. The axial force at the measured sections and their average are defined as “tension positive and compression negative”.

3.6.5.5 MRF-Spine connection force

Force taken by the MRF-Spine connections are calculated using equilibrium at the connection. Both horizontal force and moment can be deduced from spine shear force and moment. Figure 3.20 illustrates the free body of the connections.

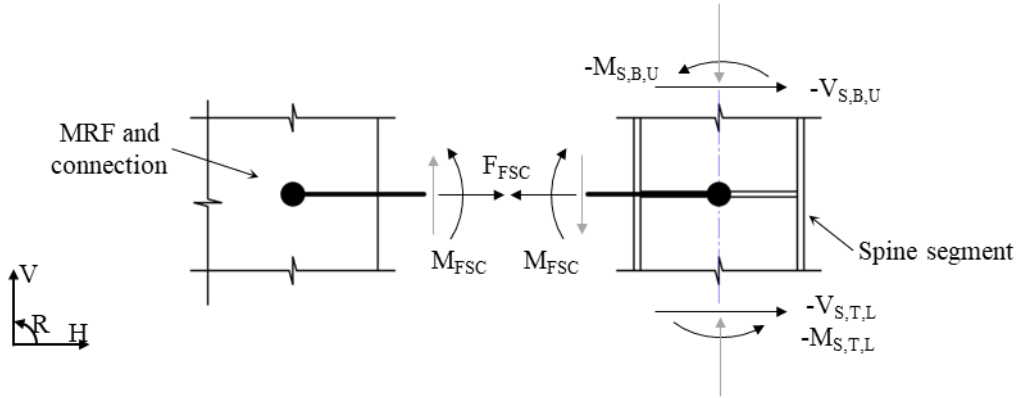


Figure 3.20 Free body at MRF-Spine connections

From the equilibrium of horizontal force and moment of the connection segment, there are

$$F_{FSC} = -V_{S,B,U} - V_{S,T,L} \quad (3.60)$$

$$M_{FSC} = -M_{S,B,U} - M_{S,T,L} \quad (3.61)$$

Where,

$V_{S,B,U}$ is the bottom top shear force of the upper spine.

$M_{S,B,U}$ is the bottom moment of the upper spine.

$V_{S,T,L}$ is the top shear force of the lower spine.

$M_{S,T,L}$ is the top moment of the lower spine.

The force and moment of the connections are in the global coordinate system. Axial force is neglected.

3.6.6 Story shear force

For the MRF, its story shear force is calculated as the sum of shear force of the four columns at each

story. For the spines, shear force can be calculated from 1) biaxial strain rosettes and 2) uniaxial strain gauges at the upper stories and loadcells at the 1st story. The total story shear force is taken as the sum of all columns and spines at each story. In addition, total story shear force can also be deduced from inertia force of each floor. Table 3.12 summarizes the information of three types of story shear force, with the corresponding equations listed afterwards.

Table 3.12 Data sources of story shear force

Data source name	Source of force		
	Column	Spine	Story
SSIF	-	-	Inertia force
SSUU	Uniaxial gauges	Uniaxial gauges and loadcell	Sum of the column and spine shear force
SSUB		Biaxial rosettes	

$$SSIF_i = \sum_{i+1}^4 Inert_i \quad (3.62)$$

$$SSUU_i = \sum S_{column,i} + \sum S_{spine,Uni,i} \quad (3.63)$$

$$SSUB_i = \sum S_{column,i} + \sum S_{spine,Bia,i} \quad (3.64)$$

Where,

i is the number of floor or story, $i = 1, 2, 3, 4$.

$SSIF_i$ is the story shear force deduced from inertia force at the i -th story.

$SSUU_i$ is the story shear force deduced from uniaxial gauges on both MRF and spines at the i -th story.

$SSUB_i$ is the story shear force deduced from uniaxial gauges on the MRF and biaxial gauges on the spines.

$Inert_i$ is the floor inertia force at the i -th floor.

$S_{column,i}$ is the shear force of a single column at the i -th story.

$S_{spine,Uni,i}$ is the shear force of a single spine at the i -th story deduced using uniaxial gauges and loadcells

$S_{spine,Bia,i}$ is the shear force of a single spine at the i -th story deduced using biaxial rosettes.

3.7 GRAVITY LOAD RESPONSE DETERMINATION

3.7.1 General

For the specimen, only force response under gravity load was considered. Deformation was neglected. In addition, since the spines were set after the MRF was tightened to the shake table, their initial force responses are neglected. Thus, force responses under gravity load only considered the MRF members.

3.7.2 First story responses measurement

Force response of the first story columns under gravity load was determined by pre-shake strain measurement. The measurement started when the specimen was fully assembled outside the shake table, and ended after the specimen was fully tightened to the shake table. Figure 3.21 plots the time history of the total axial force of the four columns. The phases on the plot were determined by carefully comparing the time information of data record and photos. Assuming no axial force when the specimen was fully elevated in the air, difference between the reading of “move to shake table” phase (red box) and the “connected to table” phase (blue box) can be deemed equivalent to the strain when gravity load acted on the specimen.

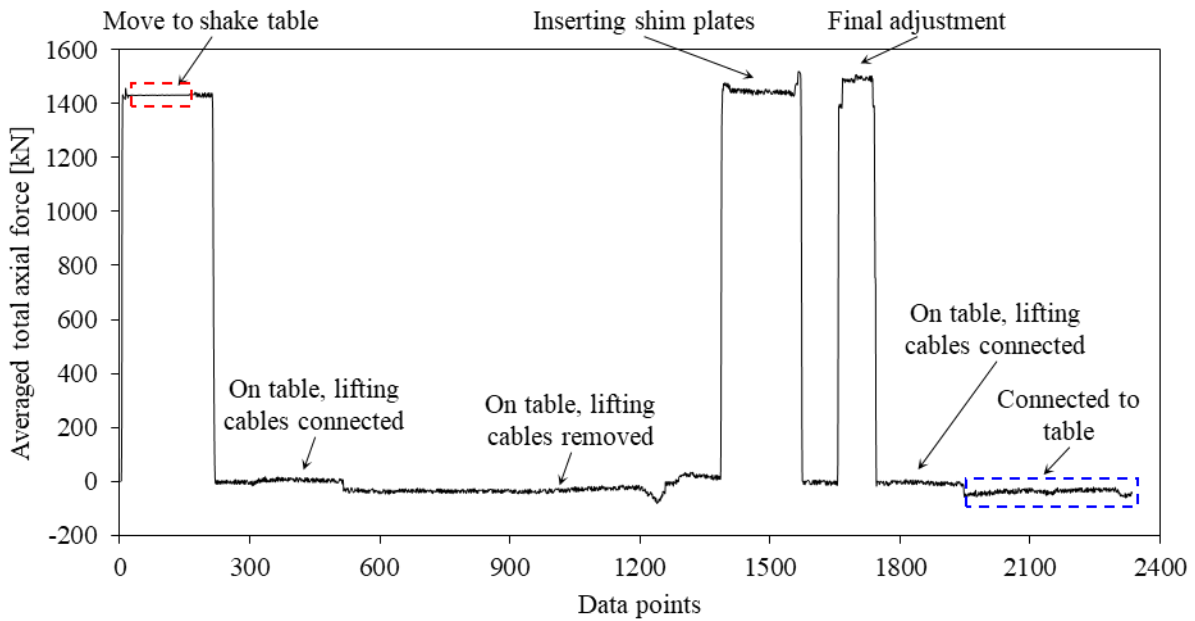


Figure 3.21 Time history of total axial force of the columns

3.7.3 Upper story response calculation

For the upper stories, the force responses of columns and beams were determined using linear static analysis. Considering symmetry of the specimen, one planar frame was used as the calculation model. 3.7.4(a) illustrates the calculation model of the planar frame. The corridor in the real specimen was neglected.

Loads acting on the planar frame were determined by distributing and transferring floor weight. Considering symmetry, half of the floor weight was taken by the planar frame at each floor. 3.7.4(b) illustrates the weight distribution of each floor. The slabs were divided by hinging lines with 45-degree angle from the floor beams. Although the slab division deferred at different floors, dimensions of sections close to the MRF beams were the same for 1FL, 2FL and 4FL but different for 3FL. Slab sections directly connected to the MRF beams (Section-1) transferred their weight as distributed load in the shape of triangle or trapezoid. Slab sections connected to the interior floor beams (Section-2)

transferred half of their weight as concentrated load to the MRF beams. The other slab sections (Section-3), although with different load paths, finally transferred their weight as concentrated load acting on the columns directly, with each column taking half of this part of weight. The load pattern deduced from the weight distribution is illustrated in 3.7.4(c).

Due to the difference between the simplified calculation model and real-world measurement, there was unbalanced force at the first-floor nodes between the calculated upper story responses and measurement first story responses. This unbalanced force was redistributed to the upper stories to reach equilibrium at the first-floor nodes.

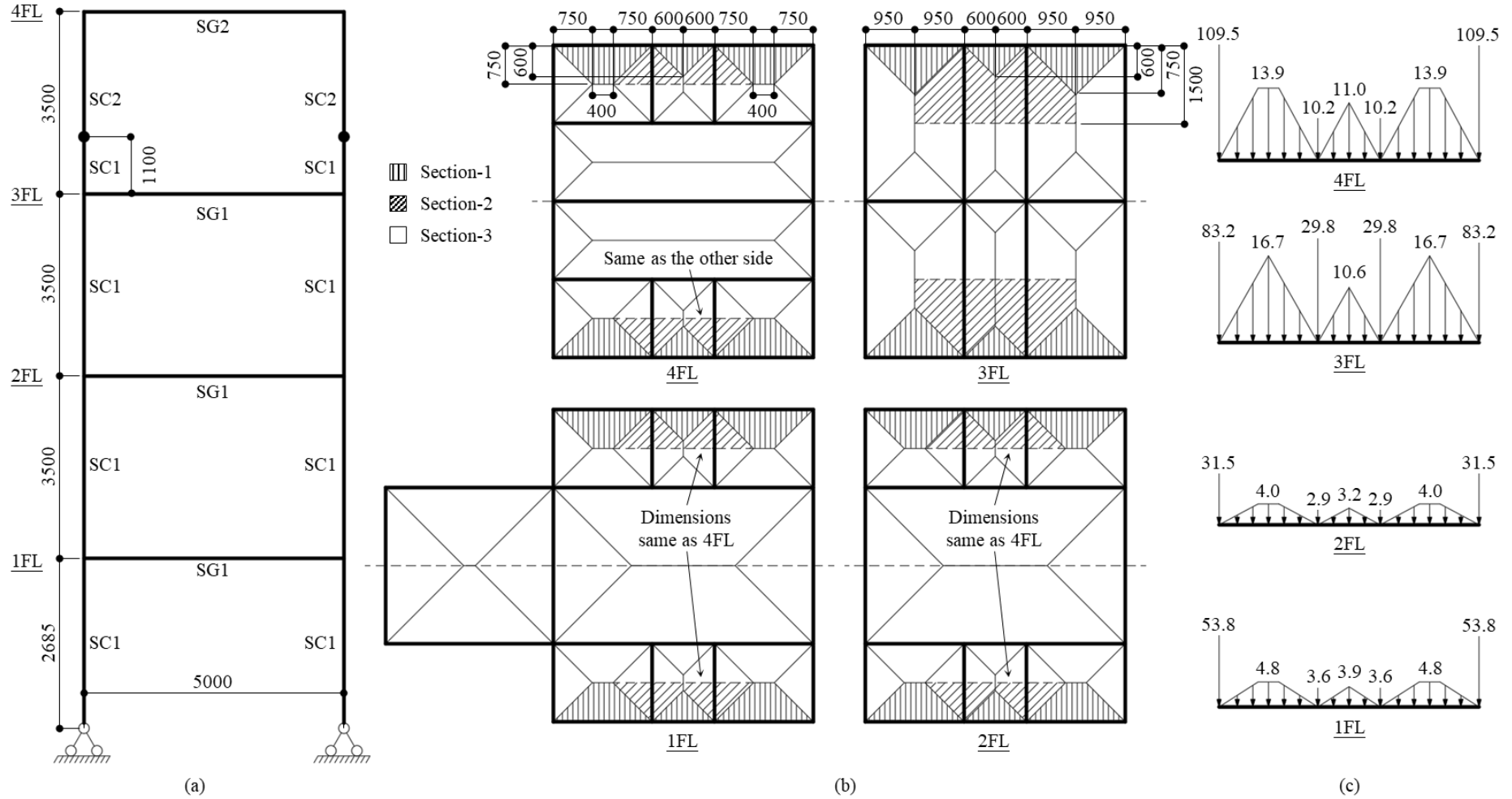


Figure 3.22 Gravity response calculation scheme

(a) Calculation model (Unit: mm); (b) Floor weight distribution patten (Unit: mm); (c) Load pattern on the planar frame (loads not in scale, Unit: kN, kN/m)

3.7.4 Gravity load response at critical sections and nodes

After getting the initial force responses under gravity load, force corresponding to those calculated in the “dynamic” condition can be extracted. These values will be added to the “dynamic” results, getting the “real” force response of the structure. Table 3.13 to Table 3.15 summarize the initial force responses of the columns, beams and panel zones, respectively. The values follow the calculation coordinate system illustrated in Figure 3.1.

Table 3.13 Initial values of column responses under gravity load

Column	Story	Node moment [kN*m]		Section moment [kN*m]				End shear force [kN]		Axial force [kN]
		Lower	Upper	Bottom	Lower	Upper	Top	Bottom	Top	
C1 (South-East)	1	3.65	5.64	3.65	1.38	3.60	5.07	-2.84	2.84	-396.83
	2	3.83	2.03	3.49	2.49	0.44	1.69	-1.67	1.67	-355.94
	3	6.84	14.49	5.62	1.97	8.70	13.27	-6.09	6.09	-357.23
	4	21.36	24.11	18.76	-1.50	12.54	21.47	-12.99	12.99	-358.52
C2 (South-West)	1	1.57	2.96	1.57	0.46	1.96	2.68	-1.38	1.38	-311.07
	2	-4.46	-1.40	-4.12	-3.12	0.19	-1.06	1.67	-1.67	-311.20
	3	-6.89	-14.44	-5.67	-2.01	-8.65	-13.22	6.09	-6.09	-311.10
	4	-21.37	-24.10	-18.77	1.50	-12.54	-21.46	12.99	-12.99	-311.29
C3 (North-East)	1	3.24	3.84	3.24	1.51	2.29	3.41	-2.17	2.17	-271.13
	2	3.98	1.97	3.64	2.62	0.36	1.63	-1.70	1.70	-271.15
	3	6.83	14.47	5.62	1.96	8.69	13.26	-6.09	6.09	-271.13
	4	1.80	1.26	1.80	1.05	0.59	1.07	-0.94	0.94	-271.14
C4 (North-West)	1	21.36	24.11	18.77	-1.50	12.54	21.47	-12.99	12.99	-139.08
	2	-4.45	-1.50	-4.11	-3.09	0.11	-1.16	1.70	-1.70	-139.09
	3	-6.87	-14.44	-5.65	-2.00	-8.66	-13.22	6.09	-6.09	-139.08
	4	-21.37	-24.10	-18.77	1.50	-12.54	-21.46	12.99	-12.99	-139.09

Table 3.14 Initial values of beam responses under gravity load

Frame	Floor	Node moment [kN*m]		Section moment [kN*m]		End shear force [kN]	
		East	West	East	West	East	West
South	1	-9.47	1.18	-7.97	0.10	11.91	8.58
	2	-8.87	8.28	-7.79	7.23	8.59	8.35
	3	-35.85	35.81	-29.74	29.70	48.75	48.74
	4	-24.11	24.10	-19.95	19.94	29.37	29.37
North	1	-8.33	2.13	-6.89	1.00	11.49	9.00
	2	-8.81	8.37	-7.74	7.32	8.56	8.38
	3	-35.84	35.81	-29.73	29.71	48.74	48.74
	4	-24.10	24.10	-19.95	19.94	29.37	29.37

Table 3.15 Initial values of panel zone responses under gravity load

Panel	Floor	Section shear force [kN]			Moment [kN*m]	
		Bottom	Top	West	East	Center
P1 (South-East)	1	18.35	-18.35	-29.46	29.46	7.10
	2	16.25	-16.25	-26.09	26.09	6.29
	3	67.31	-67.31	-108.08	108.08	26.05
	4	44.23	-44.23	-68.58	68.58	17.39
P2 (South-West)	1	-0.11	0.11	0.18	-0.18	-0.04
	2	-14.81	14.81	23.78	-23.78	-5.73
	3	-67.21	67.21	107.93	-107.93	-26.01
	4	-44.22	44.22	68.56	-68.56	-17.39
P3 (North-East)	1	15.88	-15.88	-25.49	25.49	6.14
	2	16.09	-16.09	-25.84	25.84	6.23
	3	67.29	-67.29	-108.06	108.06	26.04
	4	44.23	-44.23	-68.58	68.58	17.39
P4 (North-West)	1	-2.21	2.21	3.55	-3.55	-0.86
	2	-15.02	15.02	24.11	-24.11	-5.81
	3	-67.22	67.22	107.94	-107.94	-26.01
	4	-44.22	44.22	68.57	-68.57	-17.39

3.8 SUMMARY

This chapter introduces the test plan of the full-scale shake table tests on the MRF-Spine-FLC systems. Major contents of this chapter are summarized as follows:

- Layout and details of the test specimen are introduced.
- Material properties of the specimen were obtained from tension coupon tests for steel and cylindrical compression test for concrete. Hysteresis of the FLCs were obtained from static cyclic tests.
- Two ground motions were selected, and six major excitations were conducted I the test.
- Various instruments were set on the specimen to measure as much response as possible.
- Details of data reduction is introduced including equations and key dimensions measured on-site.
- Gravity load in the MRF was obtained from combination of pre-shake measurement and static calculation.

4. FULL-SCALE SHAKE TABLE TESTS: RESULTS AND DISCUSSION

4.1 CONSISTENCY OF GROUND MOTION

Consistency between the target and recorded ground motion is a key issue for shake table tests. Only with accurate reproduction of the target ground motion can we believe in the test results. Figure 4.1 illustrates the pseudo-acceleration (PsA) spectra with 2% damping ratio of the target and recorded ground motions for both Northridge-VA and JMA-Kobe input. The specific spectra values at the two natural periods were summarized in Table 4.1.

For both cases, the target and recorded motion matched well in the majority of the period range. The recorded motion gave larger spectra value in period range lower than 0.5 s. For the predicted 1st natural ($T_1 = 1.2$ s), the spectra acceleration for both ground motions were almost identical for the recorded and target motion. For the predicted 2nd natural period ($T_2 = 0.2$ s), the recorded motions gave larger spectra acceleration for both Northridge-VA and JMA-Kobe ground motions, with an increase of about 28% and 36%, respectively. Considering the above facts, the consistency between the recorded and target motions was satisfactory.

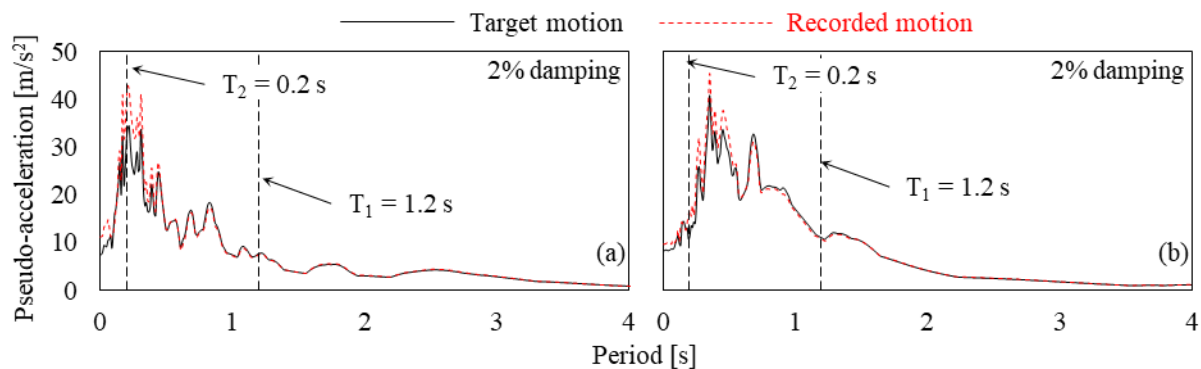


Figure 4.1 Pseudo-acceleration spectra of the target and recorded ground motions
(a) Northridge Sepulveda VA; (b) JMA Kobe NS

Table 4.1 Spectra value at the predicated natural periods [Unit: m/s^2]

Predicted natural periods	Northridge-VA			JMA-Kobe		
	Target	Recorded	Rec. / Tar.	Target	Recorded	Rec. / Tar.
$T_1 = 1.2$ s	7.6	7.4	0.98	10.9	10.7	0.99
$T_2 = 0.2$ s	31.5	40.0	1.27	11.5	17.0	1.48

4.2 NATURAL PERIODS

4.2.1 Transfer function

Natural periods were estimated using the Transfer Function (TF) method with peak-picking on the curves. The TFs were defined as the ratio of the magnitude of the Fourier Spectra of the floor responses

over shake table excitation. This method can only extract period information which is of interest in this chapter. A more comprehensive modal identification will be introduced in following chapters. Since the TFs were computed floor-by-floor, the sum of all TFs was used to represent the “whole” structure. For some peaks that can only be seen on single-floor TFs, the corresponding natural period were determined using the single-floor TF. Figure 4.2 illustrates the summed TF curves of all white noise cases, with peaks indicated by dots for each mode. The floor-by-floor TF curves can be found in Appendix D.

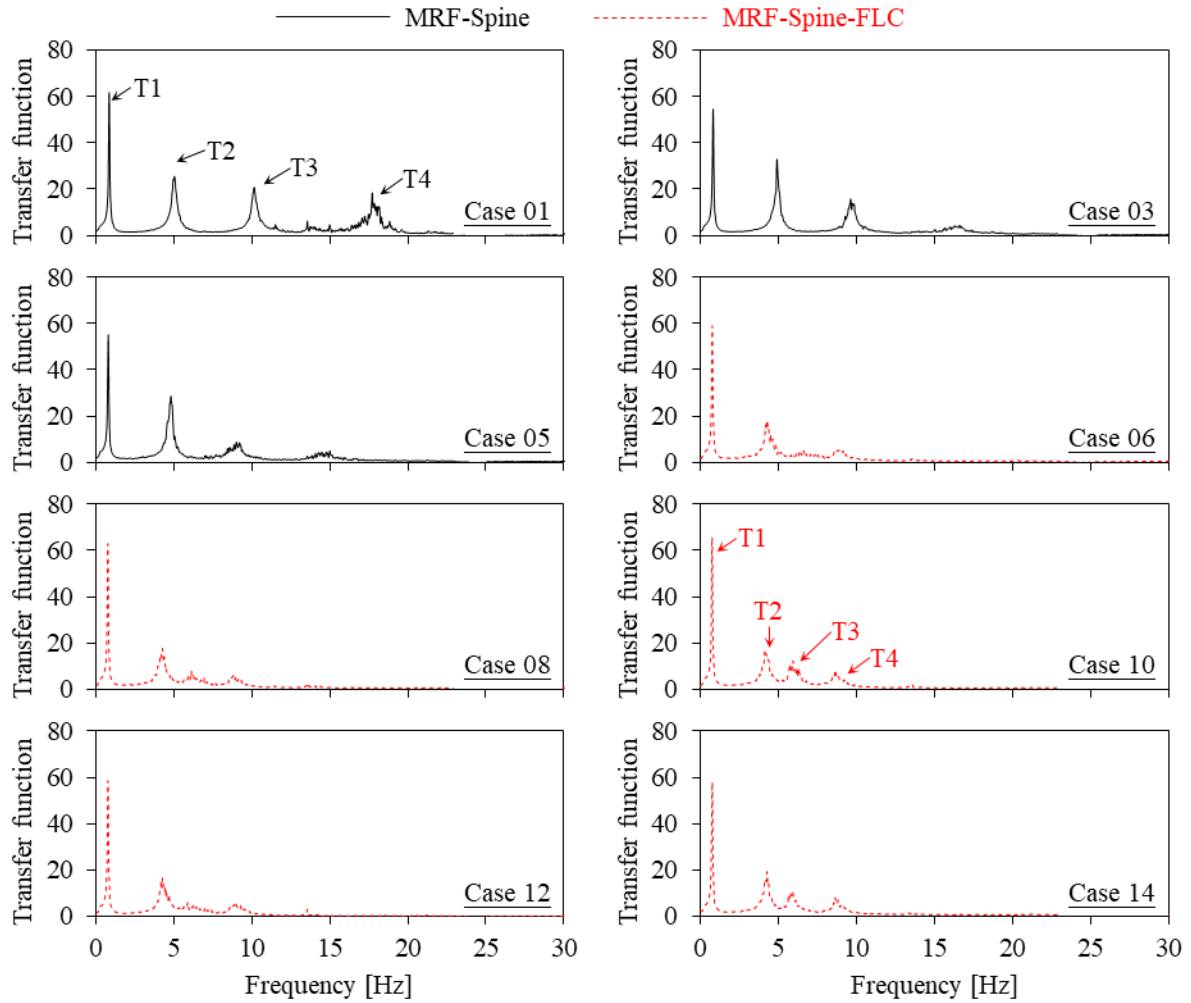


Figure 4.2 Summed Transfer Functions in the white noise excitation cases

4.2.2 Natural period

Figure 4.3 illustrates the transition of natural periods of all white noise excitations during the test. The increase in 1st period after major excitations 1 and 2 is believed to be mainly due to concrete crushing at the beam-to-column connection, with local yielding of steel members. The activation of FLCs caused slight increase in both periods. Aside from these two occasions, the first two periods remained fairly constant throughout the 6 primary excitations. The rather constant periods indicates that although there was yielding in the specimen, degradation of structure members was not significant. The 3rd and 4th periods were also identified and stayed essentially consistent within each structure systems and only

changed at the activation of FLCs.

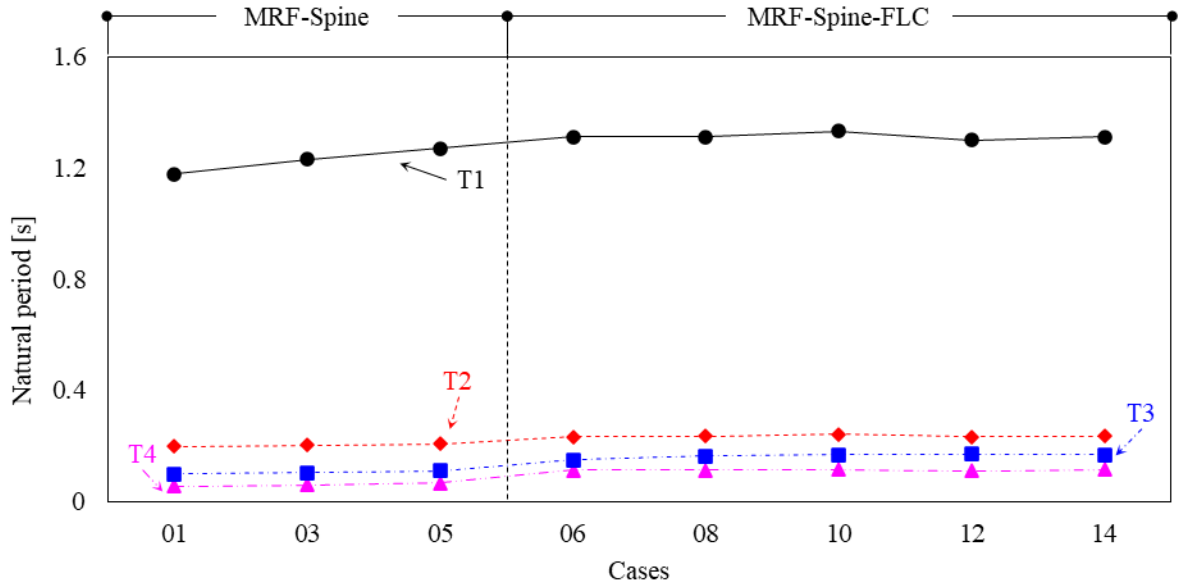


Figure 4.3 Transition of natural periods

4.3 TIME HISTORY RESPONSE OF THE SPECIMEN

4.3.1 Story drift ratio

Figure 4.4 illustrates the story drift ratio (SDR) time history of the 2nd story, comparing the SDR of both systems under two excitation levels. The residual values of each excitation were removed, and sign of the time history of the MRF-Spine-FLC system was inverted to form the comparison. From the plots, it can be seen that difference between the two systems can be found in the time history, with the difference of the 40% excitation larger than that of the 100% excitation. Despite this, the change in SDR was not very significant, especially for the peak values, no matter big or small excitations. Such tendency indicates that the SDR was not very sensitive to FLC activation. Although not shown here, SDR of other stories show similar tendency.

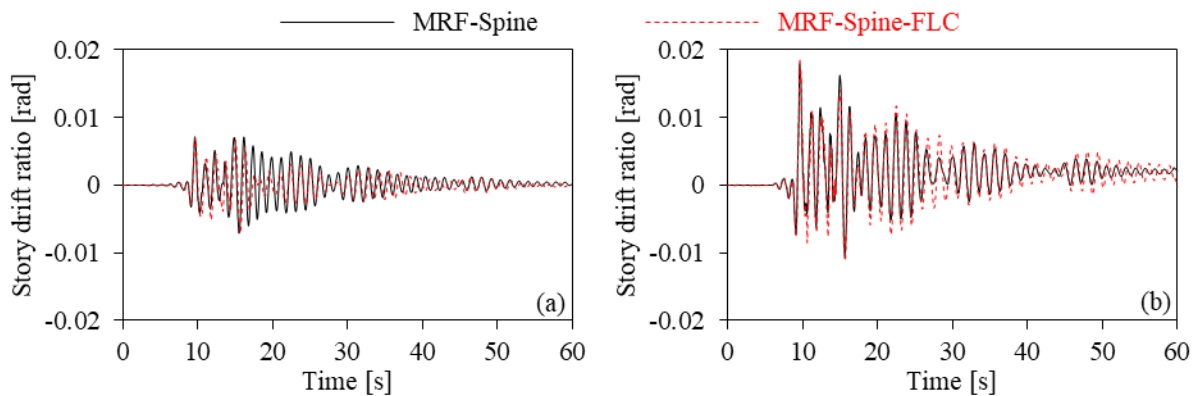


Figure 4.4 Story Drift Ratio (SDR) time history at the 2nd story
(a) Northridge-VA 40%; (b) Northridge-VA 100%

4.3.2 Floor absolute acceleration

Figure 4.5 illustrates the floor absolute acceleration time history at the 1st floor, at which the higher mode responses tends to be stronger. From the plots, it is easy to see that when the FLCs were activated, floor absolute acceleration decreased for both excitation level. Such decrease indicates the effectiveness of the FLCs in controlling floor acceleration in the system. When comparing the two excitation levels, it can be seen that decrease in the yielding cases (100%) was much more obvious than the almost elastic cases (40%). Such difference indicates that the benefits of the FLCs mainly kicked in after the structure started to yield. The plots indicates that, unlike SDR, floor acceleration was more sensitive to FLC activation, especially in big excitations.

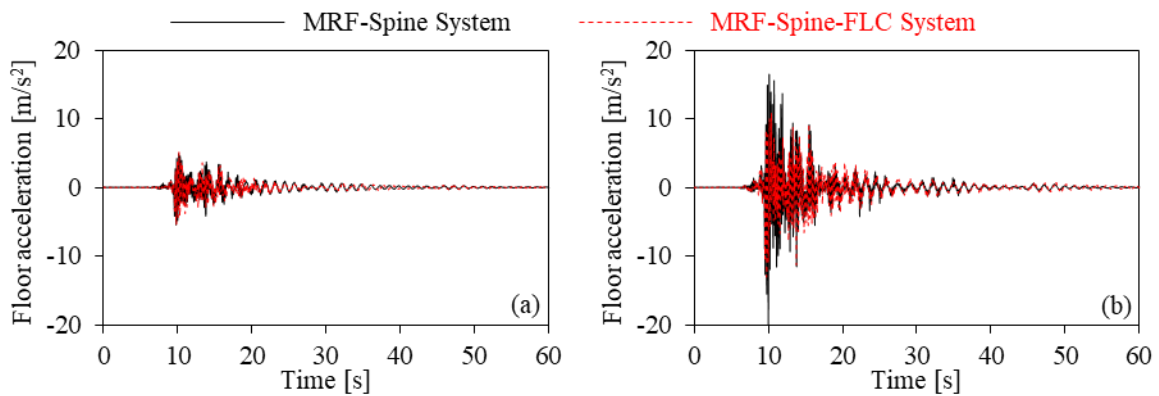


Figure 4.5 Floor acceleration time history at the 1st floor
(a) Northridge-VA 40%; (b) Northridge-VA 100%

4.4 PROFILE OF PEAK GLOBAL RESPONSE

4.4.1 Story drift ratio

Figure 4.6 illustrates the profile of the peak story drift ratio of the selected major excitations.

For all the excitations, the SDR of the 1st story was the largest while that of the 4th story smallest, regardless of the structure system, input ground motion and scale level. For Case 02 and 07, the largest SDR were 0.011 and 0.013 rad, respectively, both within the elastic range. For Case 04 and 09, the largest SDR for both cases were 0.025 rad, indicating yielding at the 1st story. Case 11 gave similar results as Case 09, although the PGA of Case 11 (4.1 m/s^2) was smaller than Case 09 (7.4 m/s^2). The largest deformation occurred in the last excitation Case 13, with the JMA-Kobe 100% input. In this case, the SDR of the lower three stories were larger than all the other cases. The largest SDR occurring at the 1st story was 0.037 rad, indicating a rather severe yielding of this story. The 2nd and 3rd stories also showed large SDR, indicating possible yielding at these stories.

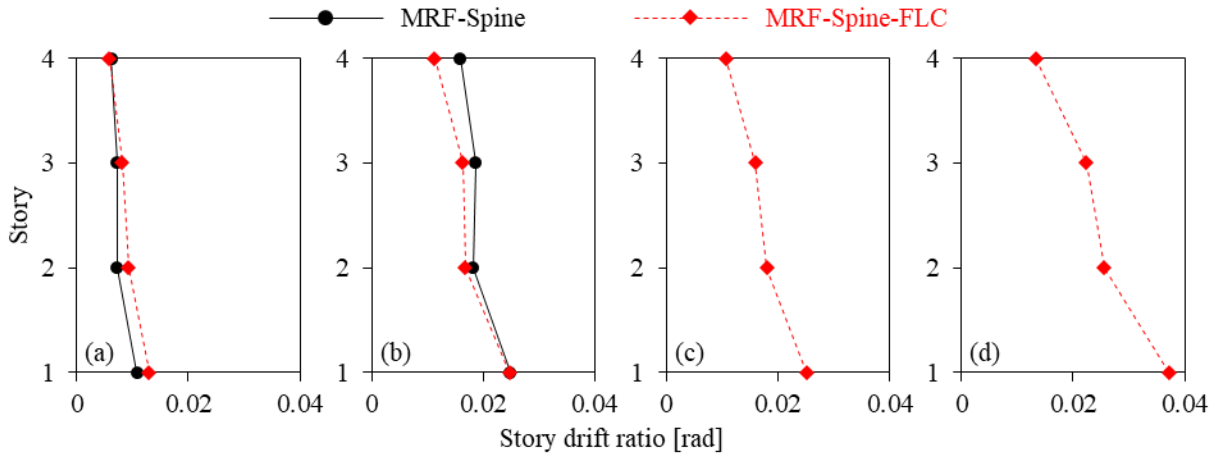


Figure 4.6 Profile of peak story drift ratio

(a) Northridge-VA 40% (Case 02 and 07); (b) Northridge-VA 100% (Case 04 and 09);
 (c) JMA-Kobe 50% (Case 11); (d) JMA-Kobe 100% (Case 13).

It should be noted that the SDR data shown here contained residual drift. These data do not represent the pure deformation in each excitation. Considering this, direct comparison between the corresponding cases, i.e., Case 02 and 07 or Case 04 and 09, is meaningless due to the influence of the residual drift. Further comparison and analyses will be introduced in the following discussion sections.

4.4.2 Floor acceleration

Figure 4.7 illustrates the profile of the peak floor accelerations of the selected major excitations.

For all the major excitations, there was a general tendency that the peak floor acceleration was larger at the 1st and 2nd floors. For Case 02 and Case 13, the largest floor acceleration occurred at the 1st floor, with the value of 5.8 and 12.5 m/s^2 , respectively. For the other four excitations, the largest floor acceleration occurred at the 2nd floor. The 3rd and 4th floor of all the excitations showed smaller peak floor acceleration than the 1st and 2nd floors, with the acceleration of the 4th floor slightly larger than the 3rd floor, except for Case 09. Among all six excitations, Case 04, with the Northridge-VA 100% ground motion input and the MRF-Spine system, gave the largest floor acceleration at all floors. The largest floor acceleration also occurred in this load case, at the 2nd floor, giving the acceleration of 20.3 m/s^2 , almost twice the input acceleration at the table. Although the ground motion of Case 13 was stronger, the peak floor acceleration of this case was still smaller than Case 04.

For cases with the same ground motion and the same structure system, namely Case 02 and 04, Case 07 and 09, and Case 11 and 13, the cases with larger input motion showed larger floor acceleration at all floors. For the two elastic cases with different structure systems (Case 02 and 07), the peak floor acceleration distribution was similar in both the absolute value and the uniformity across each floor. For the 4 yielded cases (Case 04/09/11/13), the case with MRF-Spine system (Case 04) showed floor acceleration with rather uneven distribution, while the other three cases showed quite uniform

distribution.

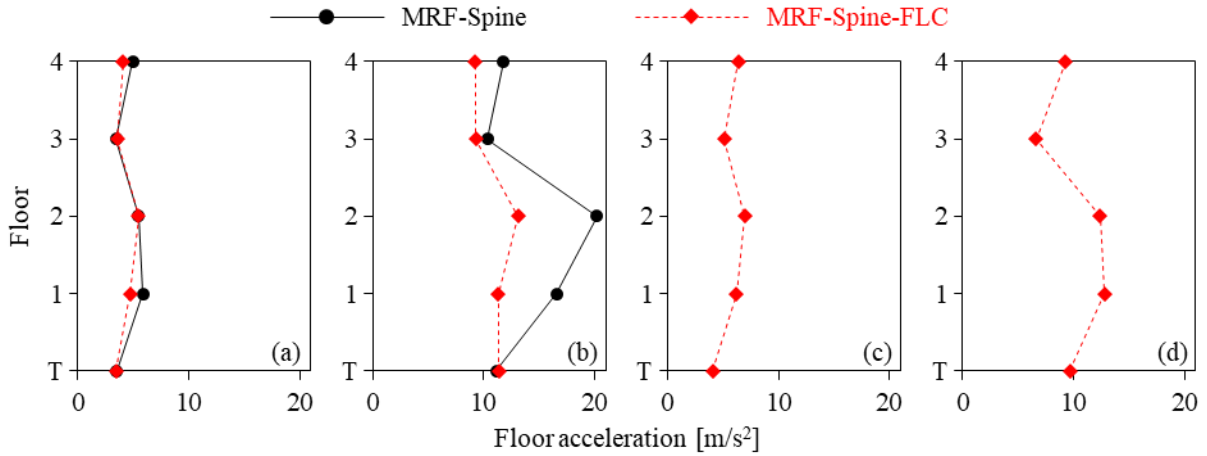


Figure 4.7 Profile of peak floor acceleration (T: shake table)

(a) Northridge-VA 40% (Case 02 and 07); (b) Northridge-VA 100% (Case 04 and 09);
(c) JMA-Kobe 50% (Case 11); (d) JMA-Kobe 100% (Case 13).

4.4.3 Story shear force

As aforementioned in Chapter 3, three types of story shear force can be obtained from measurement:

- 1) From inertia force.
- 2) From uniaxial strain gauges on the columns and the spines, and the load cells at the spine bases.
- 3) From uniaxial strain gauges on the columns and biaxial strain rosettes on the spines.

Consistency of the three types of story shear force was confirmed by checking the linear relationship between each two of them. Details of the checking procedure are introduced in Appendix D. Considering the requirements for a compatible data source for the shear force taken by the MRF and the spines, and the better resolution of uniaxial strain gauges and loadcells set on the spines, the 2nd type of story shear force is used for any illustration and discussion hereafter.

4.4.3.1 Profile of peak story shear force

Figure 4.8 illustrates the profile of the peak structure shear force of all 6 major excitations taken by the whole structure, named as peak story shear force.

For all the six major excitations, the story shear force distribution was similar over each story, with the 1st story having the largest force and the 4th story the smallest. Peak shear force at the 2nd and 3rd stories were different for different cases. In Case 02, 04 and 09, shear force at the 2nd story was smaller than the 3rd story, while for the other three cases the opposite. Among all the six cases, Case 04, with the Northridge-VA 100% input and the MRF-Spine system, showed the largest story shear force at all stories. The largest base shear of the structure also occurred in this case, reaching a value of 794 kN, around half the structure weight (1500 kN).

For cases with the same ground motion and the same structure system, namely Case 02 and 04, Case 07 and 09, and Case 11 and 13, the cases with larger input motion showed larger story shear force at all stories. For the two elastic cases with different structure systems (Case 02 and 07), the story shear force distribution was similar, with slightly smaller story shear force for Case 07 at stories other than the 2nd. For the yielded cases under the Northridge-VA 100% input (Case 04 and 09), the tendency was similar to the elastic cases, but the reduction was larger for Case 09 at all stories. Although the ground motion of Case 13 was stronger, the peak story shear force of this case was still smaller than Case 04.

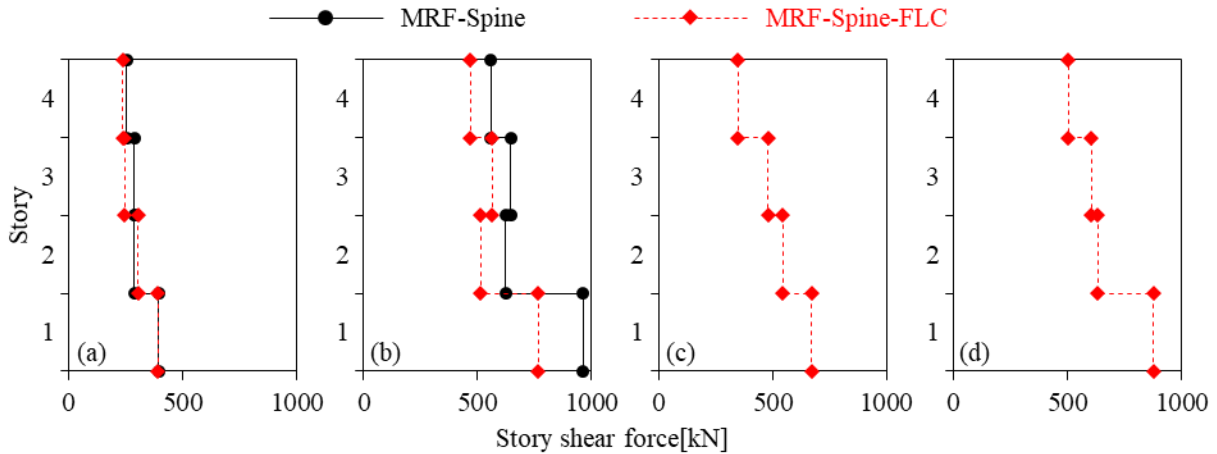


Figure 4.8 Profile of peak story shear force

(a) Northridge-VA 40% (Case 02 and 07); (b) Northridge-VA 100% (Case 04 and 09);
(c) JMA-Kobe 50% (Case 11); (d) JMA-Kobe 100% (Case 13).

4.4.3.2 Profile of peak spine shear force

Figure 4.9 illustrates the profile of the peak structure shear force of all 6 major excitations taken by the two spines, named as peak spine shear force.

For all the six major excitations, the spine shear force distribution was similar over each story. Peak spine shear force at the 1st story was the largest of all stories, while at the 2nd and 3rd stories significantly smaller. Peak spine shear force at the 4th story was larger than that of the 2nd and 3rd stories, but still smaller than the 1st story. The largest shear force taken by the spines occurred in Case 04, at the 1st story, with the value of 561 kN, taking about 70% of the total shear force of the 1st story. The peak spine shear force distribution was similar to the shear distribution of a simply supported beam with several lateral loads working on it, which was consistent with the analog of the spine as a continuous beam.

For cases with the same ground motion and the same structure system, namely Case 02 and 04, Case 07 and 09, and Case 11 and 13, the cases with larger input motion showed larger spine shear force at all stories. For the two elastic cases with different structure systems (Case 02 and 07), the spine shear force distribution was similar, with slightly smaller spine shear force for Case 07 at stories other than the 2nd.

For the yielded cases under the Northridge-VA 100% input (Case 04 and 09), the tendency was similar to the elastic cases, but the reduction was larger for Case 09 at all stories. Although the ground motion of Case 13 was stronger, the peak spine shear force of this case was still smaller than Case 04.

For the MRF-Spine-FLC system (Case 07/09/11/13), the difference between shear force at the 2nd and 3rd stories was smaller than the MRF-Spine system (Case 02 and 04), especially for the three yielded cases (Case 09/11/13). Such distribution was expected since there was no connection between the frame and the spines at the 2nd floor level, which was between the 2nd and 3rd stories. Since the spines were continuous members, the shear force was thus almost identical.

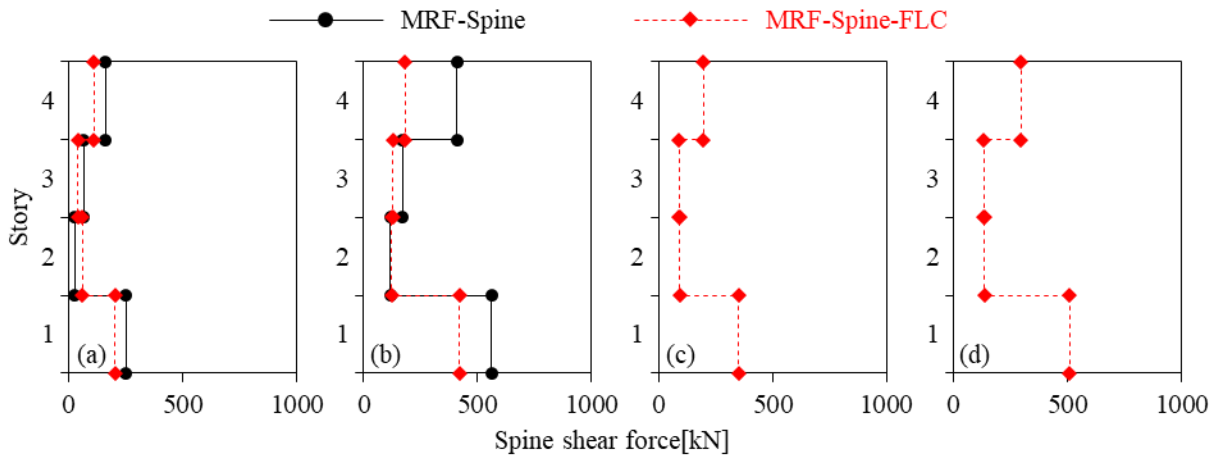


Figure 4.9 Profile of peak spine shear force

(a) Northridge-VA 40% (Case 02 and 07); (b) Northridge-VA 100% (Case 04 and 09);
(c) JMA-Kobe 50% (Case 11); (d) JMA-Kobe 100% (Case 13).

4.4.3.3 Profile of peak MRF shear force

Figure 4.10 illustrates the profile of the peak structure shear force of all 6 major excitations taken by the MRF, named as peak MRF shear force. It should be noted that the summation of the peak MRF shear force and peak spine shear force may not be equal to the peak story shear force, since they may be reached at different time instances or in different directions.

For all the six major excitations, the MRF shear force distribution was similar over each story, which was almost the opposite of the distribution of the peak spine shear force. Peak MRF shear force at the 1st story was the smallest of all stories, while at the 2nd and 3rd stories significantly larger. Peak MRF shear force at the 4th story showed different distribution for the MRF-Spine and MRF-Spine-FLC systems. For the MRF-Spine system, peak MRF shear force at the 4th story was the largest, while for the MRF-Spine-FLC system it was larger than the 1st story but still smaller than the 2nd and 3rd stories. The largest MRF shear force occurred in Case 04, at the 4th story, with the value of 656 kN. This value was even larger than the peak story shear force, which was because that at the time instance of these two peak values, the spine shear force was in the opposite direction and was not at its peak value.

For cases with the same ground motion and the same structure system, namely Case 02 and 04, Case 07 and 09, and Case 11 and 13, the cases with larger input motion showed larger MRF shear force at all stories. For the two elastic cases with different structure systems (Case 02 and 07), the MRF shear force distribution was similar, with slightly smaller MRF shear force for Case 07 at the 3rd and 4th stories. For the yielded cases under the Northridge-VA 100% input (Case 04 and 09), MRF shear force at the 1st story of Case 09 was larger than that of Case 04, but smaller for the upper stories. For the yielded cases under the JMA-Kobe 100% input (Case 11 and 13), due to the yielding of the 1st story, the increase of MRF shear force at the 1st story was smaller than the upper stories. Although Case 13 had larger ground motion input, the MRF shear force at the 1st story (262 kN) was only 4 kN larger than that of Case 09 (258 kN), indicating that the MRF shear force was controlled by the yielding of the frame members.

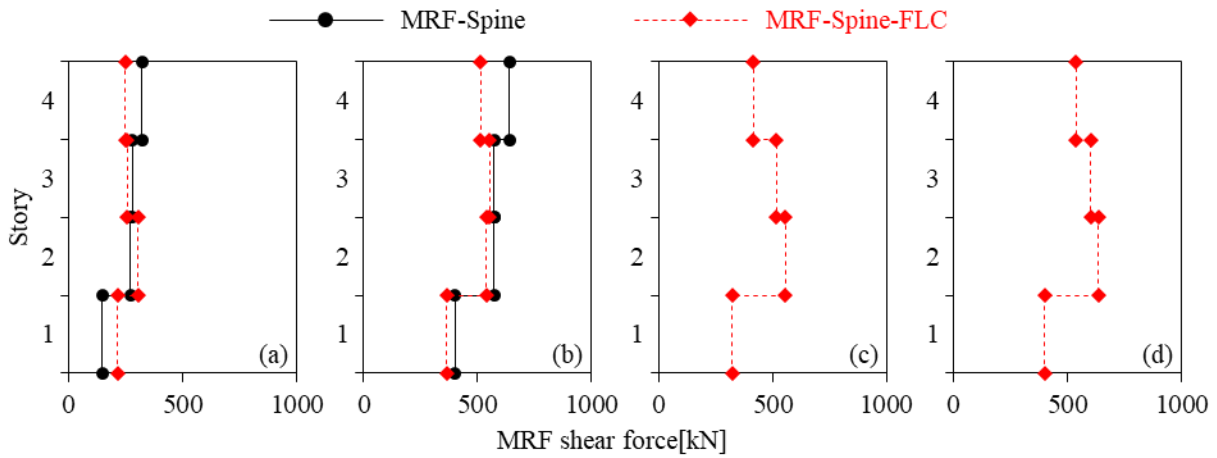


Figure 4.10 Profile of peak MRF shear force

- (a) Northridge-VA 40% (Case 02 and 07); (b) Northridge-VA 100% (Case 04 and 09);
(c) JMA-Kobe 50% (Case 11); (d) JMA-Kobe 100% (Case 13).

4.5 STRUCTURE HYSTERESIS

Hysteresis of the global and local responses were plotted and explained in this section. Illustrated herein were only part of the hysteresis curves. Figures of all the hysteresis curves are included in Appendix D.

4.5.1 Story hysteresis

4.5.1.1 Story hysteresis of the whole structure

Figure 4.11 illustrates the story hysteresis of the whole structure, plotting the relationship between the story shear force and the story deformation. Story shear force was from the 2nd data source aforementioned (uniaxial strain gauges for all members and load cells at spine bases), while the story deformation was the synchronized story drift (SD) and story drift ratio (SDR) from the optical measurement system. Note that hysteresis for the two elastic cases, Cases 02 and 07 were not included here but put in Appendix D. The distortion of the curves was mainly due to the distortion in the measured spine shear force.

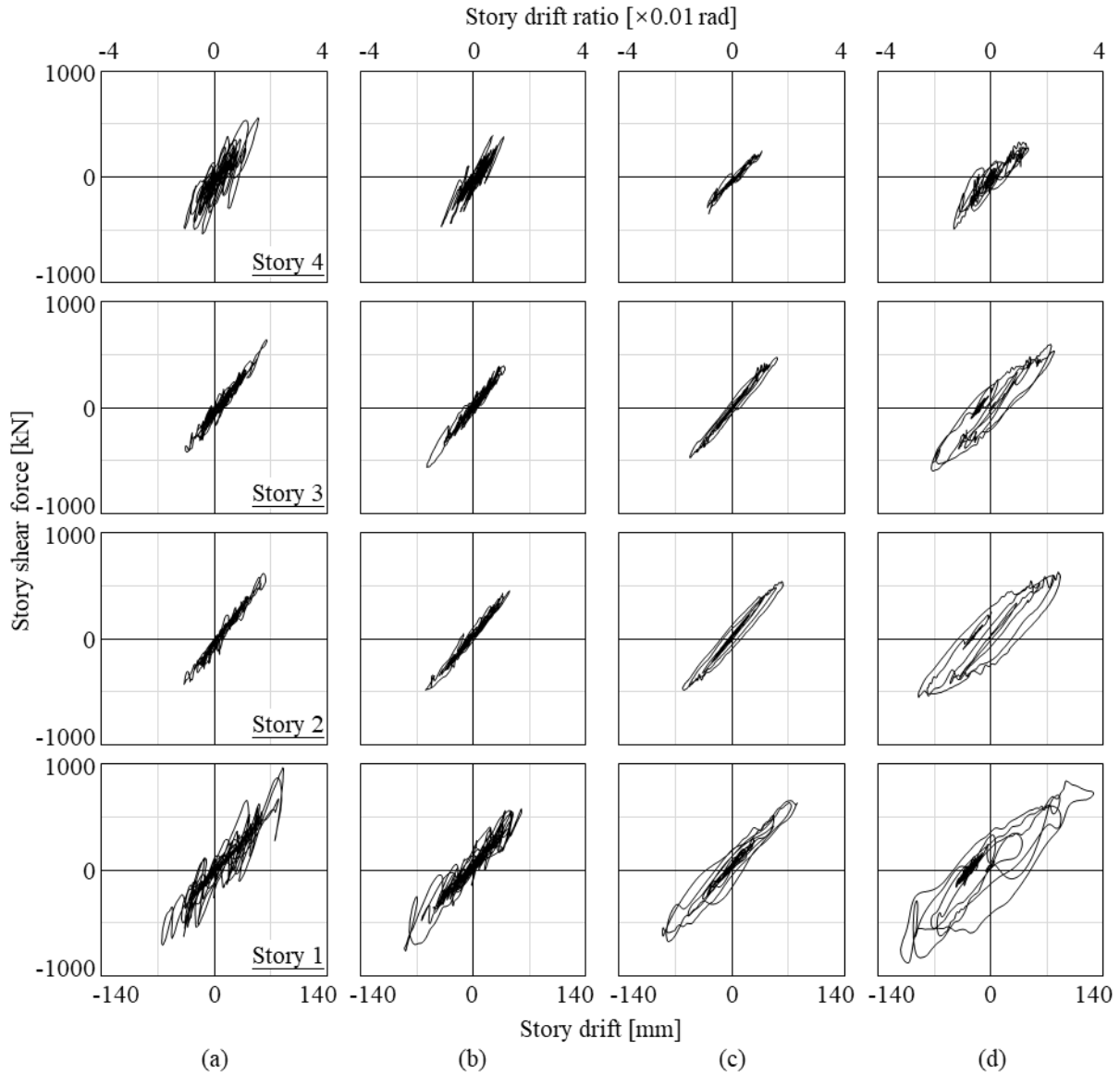


Figure 4.11 Story hysteresis of the whole structure

(a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC; (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

For the four cases shown here, no matter the FLCs were activated or not, the lateral stiffness of each story was quite consistent for each case. Such tendency was especially clear for the 2nd and 3rd stories in Case 04/09/11. For each story, lateral stiffness of the 1st story was slight smaller than the upper stories, which was reasonable due to the pinned base of the MRF columns. The 2nd and 3rd stories showed similar lateral stiffness, which was consistent with the same story height and member sizes of these two stories. The 4th story showed largest lateral stiffness among all stories, which was due to its larger column cross sections compared to the lower stories. Although not shown here, the lateral stiffness of the two elastic cases (Case 02 and 04) was consistent with the above cases.

All stories showed minor yielding in the smaller three cases (Case 04/09/11). In the largest case (Case 13), significant yielding occurred at the 1st to 3rd stories, while the 4th story showed limited yielding

according to the shape of the hysteresis. Despite the rather significant yielding in the last excitation, there was no sign of degradation for the structure, which was consistent with the rather stable natural period results shown in Section 4.2.2.

4.5.1.2 Story hysteresis of the MRF

Figure 4.12 illustrates the story hysteresis of the MRF only, plotting the relationship between the story shear force taken by the MRF and the story deformation. Story shear force was from the aforementioned 2nd data source (uniaxial strain gauges for all members and load cells at spine bases), while the story deformation was the synchronized story drift (SD) and story drift ratio (SDR) from the optical measurement system. Note that hysteresis for the two elastic cases, Cases 02 and 07 were not included here but put in Appendix D.

In general, the hysteresis curves of the MRF were similar to those of the whole structure. But since distortion from the spine shear force measurement was not included, the curves shown here had better shape and can better illustrate the story properties. For the MRF alone, lateral stiffness of the 1st story was the smallest, which was due to the pins set at the column base, making the effective length of the 1st story columns twice that of the 2nd and 3rd story columns. Largest lateral stiffness occurred at the 4th story, which was due to the larger cross sections of the columns at this story. The 2nd and 3rd stories showed intermediate and identical lateral stiffness, with the boundary condition and section properties being same in these two stories. The difference between the 1st story stiffness and the upper stories for the MRF was larger than the whole structure cases, indicating that the spines helped to even the lateral stiffness of each story, thus helping limit the formation of soft stories. Although not shown here, the lateral stiffness of the two elastic cases (Case 02 and 04) was consistent with the above cases. Yielding condition of the MRF was almost identical to that of the whole structure, with significant yielding only occurred in the last excitation and at the lower three stories.

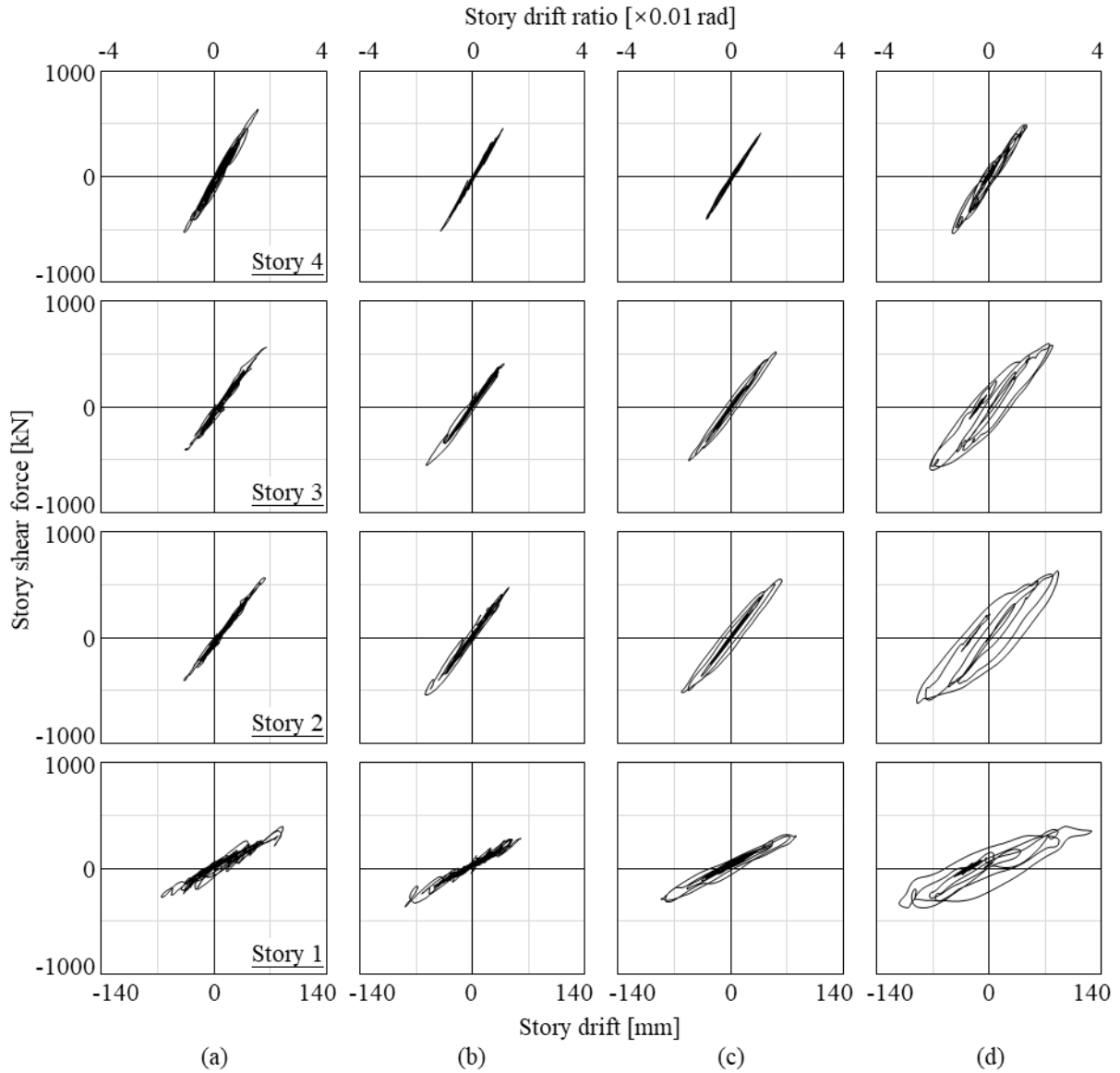


Figure 4.12 Story hysteresis of the pure MRF

(a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC; (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

4.5.2 MRF-Spine connection hysteresis

Figure 4.13 illustrates the hysteresis of the MRF-Spine connections of the south frame, plotting the transferred force to the relative displacement at the connections. Note that hysteresis for the two elastic cases, Cases 02 and 07 were not included here but put in Appendix D.

In the MRF-Spine system (Case 04), the relative displacement was no greater than 8 mm. The shape of hysteresis indicates that the relative motion was due to tilting and slipping of the bolts in the rather long connection. Rather sharp peaks near the maximum displacement indicates that large stiffness was realized when the bolts went into bearing. In the MRF-Spine-FLC system (Case 09/11/13), where the bolted connections were released at the 1st ~3rd floors, and FLCs were activated at the 1st and 3rd floors,

the relative displacement reached 24 mm at the 1st floor and 36 mm at the 3rd floor. The relative displacement at these connections was on the order of 20 mm ~ 40 mm, which was not a large value compared with the dimension of the structure and the overall story drift.

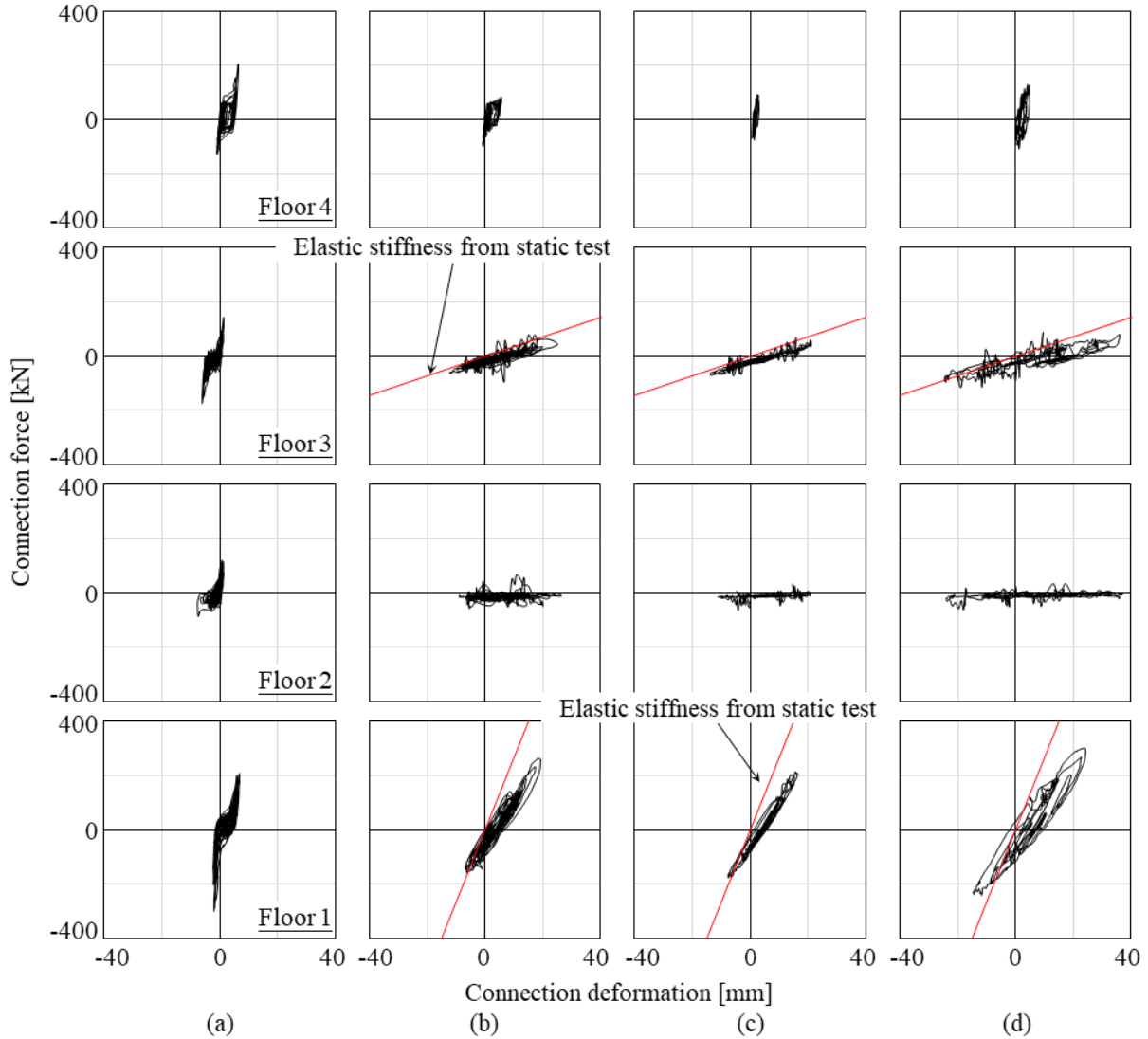


Figure 4.13 Hysteresis of MRF-Spine connections of the south frame

(a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC; (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

The stiffness of the FLCs was consistent between the Northridge-VA 100% and JMA-Kobe 100% excitations (Case 09/11/13). The relative displacements at the 1st to 3rd floors were greater from the JMA-Kobe excitation. For both excitations, the T-FLCs at the 1st floor was worked to smaller deformation demands compared to the U-FLCs at the 3rd floor but transmitted larger forces. The transferred force in the MRF-Spine-FLC system was smaller than in the MRF-Spine system at the 3rd and 4th floors, but somewhat larger at the 1st floor, and near zero at the 3rd floor where only the sliding bearing and tie plates remained. As intended, at the 1st and 3rd floors, the connection was significantly softer in the MRF-Spine-FLC system than in the MRF-Spine system, of which the U-FLCs at the 3rd

floor was even softer than the T-FLCs at the 1st floor.

The plots also included elastic stiffness of the FLCs obtained from static component tests, which were doubled from the static test results to represent the two devices at one connection. Compared to the static stiffness of the FLCs, stiffness of the U-FLCs in the shake table tests matched well, but stiffness of the T-FLCs in the shake table tests were smaller. The stiffness loss of the T-FLC connections was attributed mainly to the fact that two T-FLCs at one connection setup did not work in sync. Instead, it was observed from the test video that there was rocking of the pin connecting the two T-FLCs to the spine, suggesting that the two T-FLCs had relative movement. Thus, the total stiffness of the connection is smaller than two-times the static stiffness. Using linear regression on the test results from Case 07 (elastic test), the stiffness in the shake table test was about 70% of the static stiffness.

As for the energy dissipation at the FLCs, it did not appear that substantial energy dissipation took place at the FLCs. Some level of yielding occurred, especially in Case 13 with the JMA-Kobe 100% input, but the hysteresis curves were not developed to a large area. In addition, no degradation was observed at these MRF-Spine connections.

4.5.3 Panel zone hysteresis

Figure 4.14 illustrates the hysteresis of east panel zones of the south frame (i.e., C1), plotting the relationship between the panel zone shear force and shear angle. In all cases shown here, the panel zone at the roof floor (Floor 4) did not yield, with the shear angle of the roof floor panels no larger than 0.005 rad. It should be noted that the shear capacity of the panel zones at the roof floor was calculated using the nominal strength of the steel due to lack of coupon test results. Since the possible overstrength was neglected, the value in the figures tends to be lower than the real capacity. Considering this, exceedance of the plotted capacity did not necessarily indicate yielding.

Comparing the cases of Case 04 and 09, the panel zones of the MRF-Spine system and MRF-Spine-FLC system yielded to a similar extent at the same locations when subjected to the 100% Northridge-VA excitation. Significant yielding occurred in Case 11, JMA-Kobe 50% excitation at the 1st and 2nd floors. In the last major excitation of JMA-Kobe 100% input, yielding at the 1st and 2nd floors continued and developed to the 3rd floor. At the 1st and 2nd floors, the maximum shear deformation reached 0.015 rad. As expected by the proportion of components, the panel zones acted as a major source of energy dissipation in the MRF.

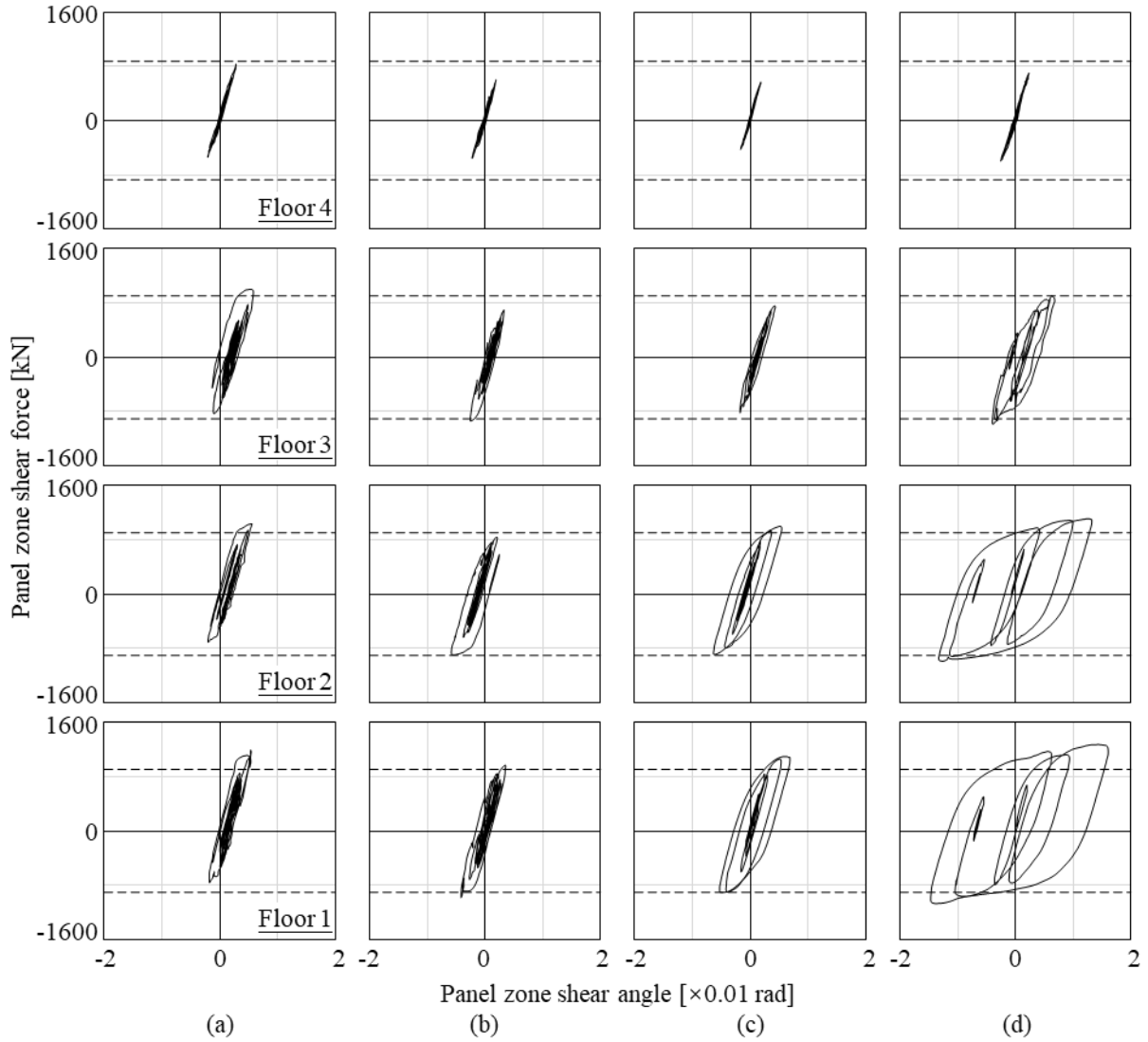


Figure 4.14 Hysteresis of panel zones at the east column of the south frame

(a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC; (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

4.5.4 Column end hysteresis

Figure 4.15 illustrates the hysteresis of east column ends of the south frame (i.e., C1), plotting the relationship between the column end moment and story drift ratio. Since the columns were in a high axial force status, the flexural capacity alone cannot represent yielding condition of the section, and thus is not indicated in the figures. Note that hysteresis for the two elastic cases, Cases 02 and 07 were not included here but put in Appendix D. Selecting SDR as the deformation was due to lack of direct measurement of column end rotation. Column ends in the same story were placed in the same plot. The bottom sections of the 1st story columns were neglected since they were pinned connections.

For all the four cases shown here, the top and bottom section of the 4th story, which were connected to the 4th floor panel and 3rd floor panel, respectively, did not yield. For Case 04, 09 and 11, yielding did

not occur for both MRF-Spine system (Case 04) and MRF-Spine-FLC system (Case 09/11) at most of the sections. Some minor yielding occurred at the top section of the 1st story and the bottom section of the 2nd story that were both connected to the 1st floor panel, but the condition was not severe. Major yielding occurred in the last excitation (Case 13) with JMA-Kobe 100% input and developed up to the top section of the 3rd story. Considering the shape of the curves, there was no sign of the formation of plastic hinges at these sections except for the top section of the 1st story, of which the hysteresis curve formed a plateau with almost constant moment, indicating the formation of plastic hinge at this section. Aside from the above yielding condition, no degradation was observed at these sections. The plastic capacity of the column sections is also illustrated in the figures, shown as black dashed lines. It should be noted that column sections were different for the top and bottom sections of the 4th story, but capacity of the top sections (W10×100) was 734 kN·m, and thus not shown.

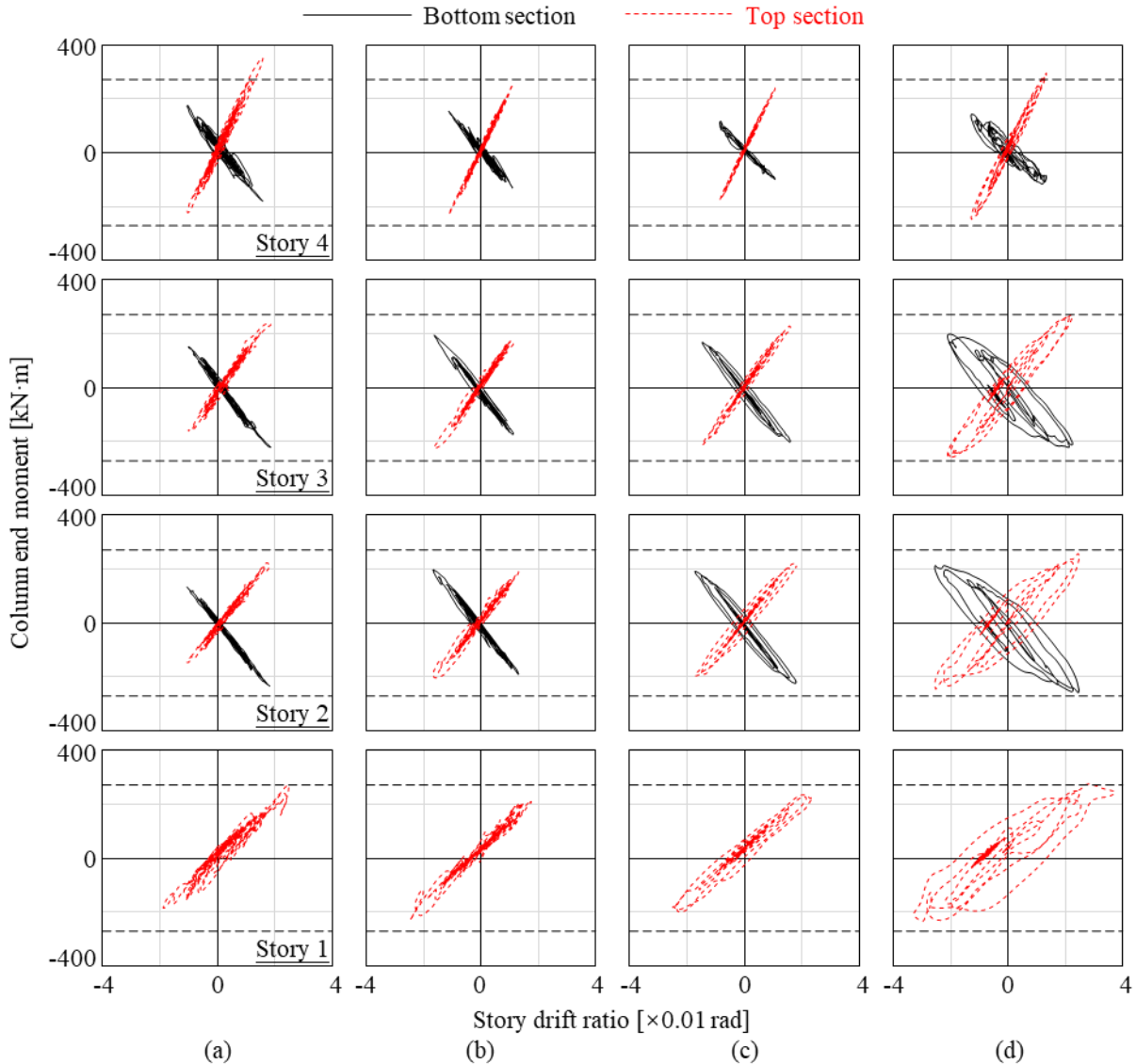


Figure 4.15 Hysteresis of the east column ends of the south frame

- (a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC;
(c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

4.5.5 Beam end hysteresis

Figure 4.16 illustrates the hysteresis of the east beam end of the south frame, plotting the relationship between the beam end moment and rotation angle. The quite fuzzy curves are due to the heavily distorted measurement of the beam end rotation using string pots. Flexural capacity is indicated in the figures by horizontal dashed lines., with 549 kN·m for the 1st to 3rd floors and 414 kN·m for the 4th floor. Note that hysteresis for the two elastic cases, Cases 02 and 07 were not included here but put in Appendix D.

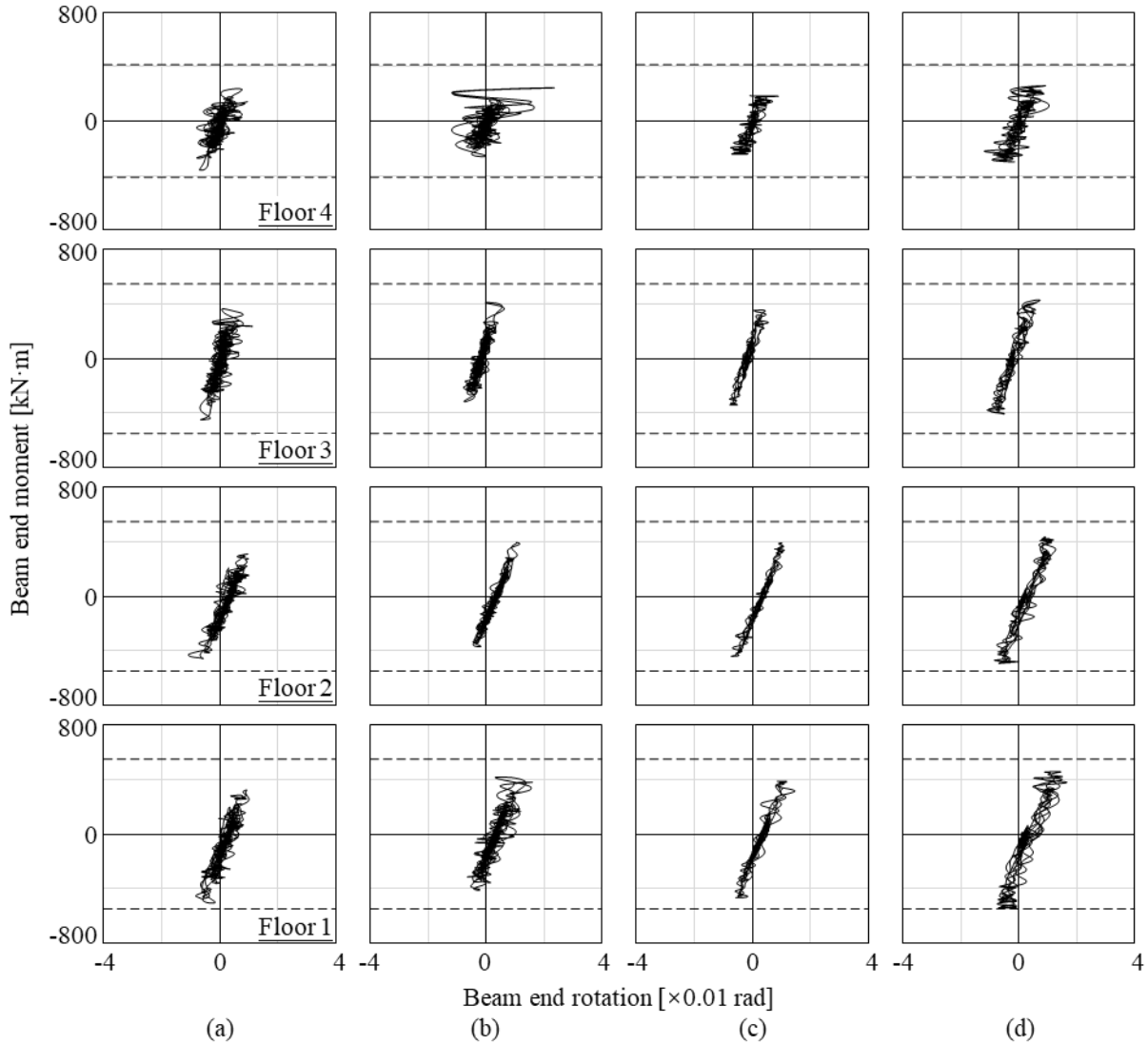


Figure 4.16 Hysteresis of the east beam ends of the south frame

- (a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC;
 (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

For the cases shown here, the beam ends of the lower three floors (Floor 1~3) remained almost elastic, with the beam end moment smaller than the capacity and the hysteresis curves being almost a straight line. For the roof floor (Floor 4), the distortion in rotation was severe, but the general tendency was clear that no significant yielding occurred. Note that the capacity for the roof floor was calculated based on

the nominal strength of the material and thus may be lower than the real condition. In general, as expected by the proportion of the structure members, beams of the MRF remained essentially elastic throughout the whole test.

4.6 DEFORMATION OF THE SPINES

In order to evaluate the deformation of the spines, displacement of the spines at each floor level was deduced from the story drift and the relative deformation of the MRF-Spine connections. It should be noted that for the spines there is no concept of story. The term ‘story’ in this section refers to the segment of the spines corresponding to a certain MRF story.

4.6.1 Story drift ratio compared with base rotation angle

For the spines, the story drift ratio refers to the drift ratio between two MRF-Spine connections. Story drift ratio can represent the deformation of each segment of the spine. Rotation measured at the spine base was taken as the reference to evaluate the spine deformation in each excitation. By conducting linear regression between the drift ratio and the base rotation, the overall relationship between the drift ratio data can be evaluated. If there is no deformation in the spines, the spine drift ratio should be identical with the base rotation angle, with regression factor being unity. On the contrary, if there is deformation in the spines, the regression factor will be away from unity. Since all drift ratio data were compared with the same reference, the base rotation, deformation of each segment can then be compared using the values of the regression factor.

Figure 4.17 illustrates the relationship between spine drift ratio and their base rotation of the four strong excitations of the south spine. Similar plots for all cases and both spines can be found in Appendix D. From the figure, the relationship of the spine drift ratio and the base rotation was consistent among different cases at the same story. The linearity of the regression was quite good, making the regression factors summarized in the table usable for comparison of the spine deformation at each story.

Table 4.2 summarizes the regression factors including the two small excitations. According to the table data, similar tendency of the spine deformation occurred in all the six major excitations. For the roof drift ratio, which was defined as the spine roof displacement divided by the spine height, took about 70% to 80% of the base rotation angle, indicating that the spine was ‘pulled back’ by the MRF compared to the rigid body movement. The rather consistent portion indicates a stable relationship of the stiffness of the spines and the MRF, which will be discussed in the following sections. For the 1st story drift ratio, the regression factor was quite close to 1, indicating that minor deformation occurred at the 1st story segment of the spines. Considering the rather large cross section of the spines, it was reasonable that minor deformation of the spines could produce rather large shear force. Factors of the 2nd story was smaller than the 1st story but larger than the 3rd and 4th stories for all six excitations, indicating that the spines were ‘bent back’ by the MRF from the rigid body movement to the real movement represented

by the roof drift ratio. Story 3 and 4 showed quite close regression factors, indicating the drift ratio at these two stories were close to each other.

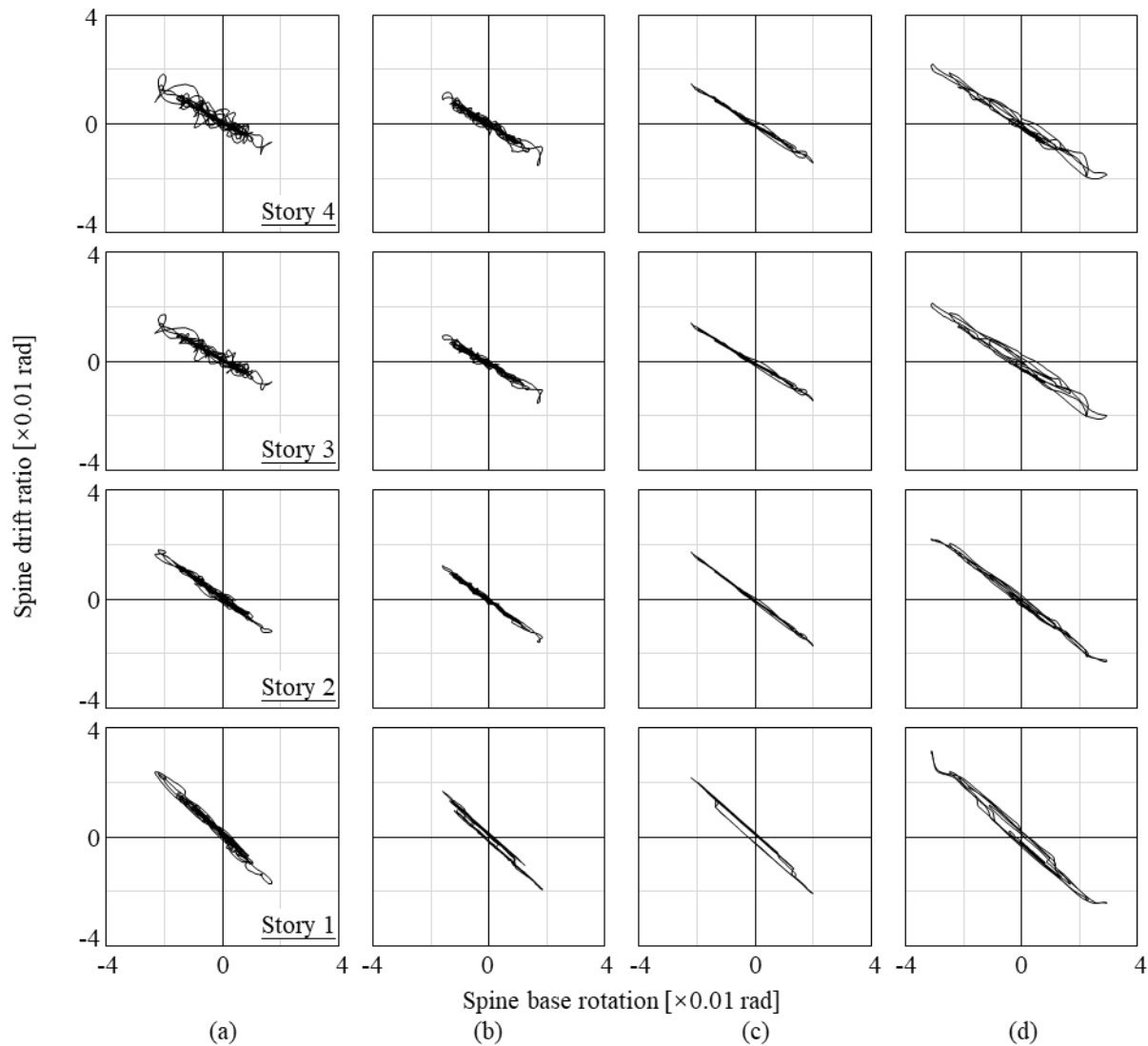


Figure 4.17 Relationship between spine drift ratio and base rotation at the south spine
 (a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC;
 (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

Table 4.2 Regression factors of south spine drift ratio to base rotation

Story	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
Story 4	-0.55	-0.60	-0.65	-0.67	-0.66	-0.73
Story 3	-0.57	-0.60	-0.62	-0.64	-0.64	-0.75
Story 2	-0.78	-0.78	-0.78	-0.79	-0.79	-0.84
Story 1	-0.94	-0.99	-0.94	-0.97	-0.98	-1.05

In all, using the base rotation as a reference, regression factors show that the deformation pattern of the spines were consistent through all six major excitations, with the drift ratio being the largest at the 1st story and decreasing gradually in the upper stories. It should be noted that the spines deformed as a

whole structure member in real world, and using the segmental data for comparison is just a closer look at the spine deformation pattern. The negative slope values are due to definition of positive directions.

4.6.2 Deformation shape at selected time instances

Aside from the overall relationship between the spine drift ratio and base rotation, specific deformation shape at a certain time instance can help illustrate the deformation shape of the spines more intuitively. By plotting the structure height against the spine displacement, deformation shape of the spine can then be illustrated. Time instances of the peak roof displacement (4th floor displacement) and the peak spine shear force were selected to illustrate the spine deformation shape. Figure 4.18 illustrates an example of how the displacement was taken using the peak roof displacement instance in the case of Case 02. For the selected time instances, deformation shape of the assumed rigid body movement, which was calculated using the spine base rotation, was also included. The straight line representing such movement can help with the evaluation of the spine deformation.

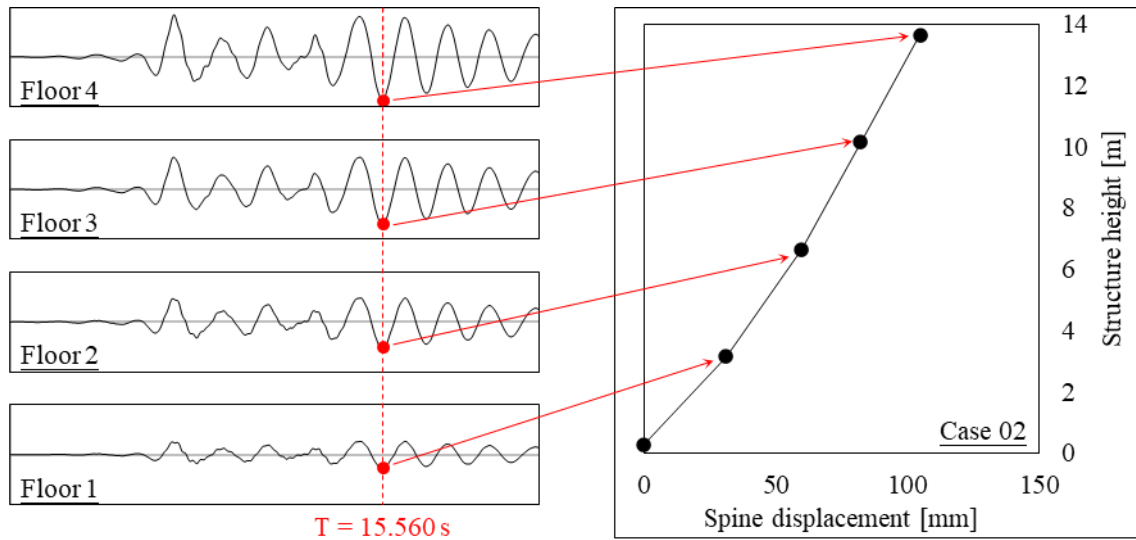


Figure 4.18 Example of taking snapshot of spine deformation (Peak roof displacement, Case 02)

4.6.2.1 Time instance of peak roof displacement

Considering that the spine is a vertically continuous structure member and represents the concept of ‘whole’ structure, and that the connection between the MRF and the spines remained constant through the tests, time instance when the 4th floor achieve peak displacement (peak roof displacement) was taken to illustrate the deformation of the spines when it reached the globally largest displacement. The snapshots of the six major excitations are illustrated in Figure 4.19, with the straight dashed line representing the rigid body movement in each plot. The difference of the displacement at each floor level between the real deformation curve and the rigid body movement is summarized in Table 4.3.

For the selected time instance, the deformation shape for all six major excitations was consistent with the general tendency indicated by the regression factors in Section 4.6.1. For all the cases, displacement

at the 1st floor level between the real shape and the assumed rigid body shape was very close, while the difference was the largest at the roof floor.

In the six major excitations, deformation shape in Case 09, with the MRF-Spine-FLC system under Northridge-VA 100% excitation, was the closest to the rigid body shape. Comparing with Case 04, with the MRF-Spine system under the same excitation, it can be seen that the displacement difference at all floors was smaller in Case 09, indicating that the spine deforms less when the FLCs were activated. However, for Case 02 and 07, although these two cases were also two structure systems under the same excitation, there was not such tendency as Case 04 and 09. The difference between these two pairs of cases was that in Case 02 and 07, there was almost no yielding in the structure, even the FLCs. Difference between these two pairs indicates that benefits of the FLCs mainly took place when the structure yielded.

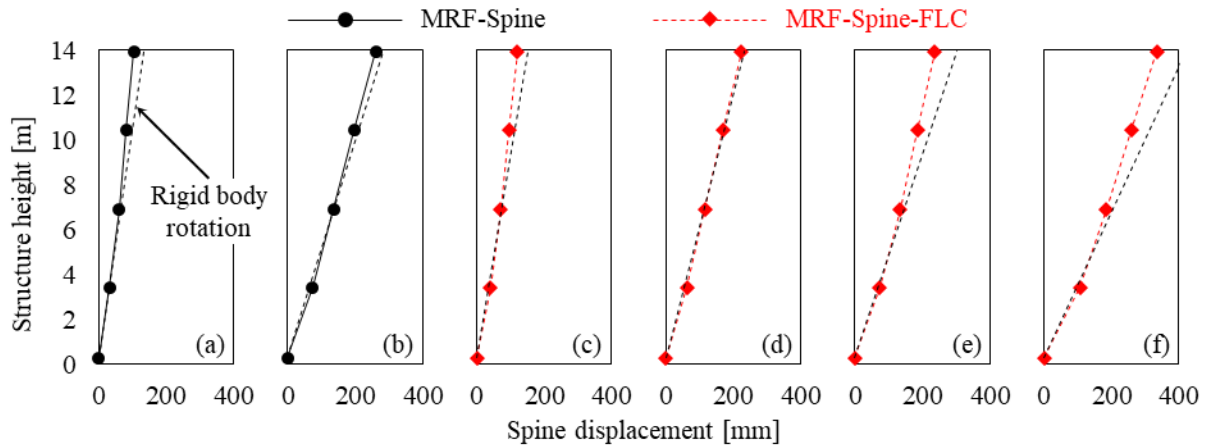


Figure 4.19 Spine deformation at peak roof displacement

- (a) Case 02, $T=15.560s$, Northridge VA 40%; (b) Case 04, $T=9.660s$, Northridge VA 100%;
(c) Case 07, $T=15.675s$, Northridge VA 40%; (d) Case 009, $T=9.670s$, Northridge VA 100%;
(e) Case 11, $T=17.000s$, JMA Kobe 50%; (f) Case 13, $T=15.620s$, JMA Kobe 100%.

Table 4.3 Difference of floor displacement [Unit: mm]

Load cases	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
Floor 4	28.8	20.7	34.1	9.3	64.6	88.1
Floor 3	17.1	13.1	18.7	3.8	38.5	54.9
Floor 2	5.2	1.3	3.8	-3.1	11.6	21.3
Floor 1	-0.4	-7.9	-5.1	-8.3	-4.8	-9.1

4.6.2.2 Time instance of peak spine shear force

Considering that the spine shear force was of concern, time instance when the spine developed the peak shear force was also an interesting condition to be analyzed. The snapshots at this time instance of the six major excitations are illustrated in Figure 4.20, with the straight dashed line representing the rigid body movement in each plot. The difference of the displacement at each floor level between the real deformation curve and the rigid body movement is summarized in Table 4.4.

For the selected time instance, the deformation shape for all six major excitations was also consistent with the general tendency indicated by the regression factors in Section 4.6.1. For all the cases, displacement at the 1st floor level between the real shape and the assumed rigid body shape was very close, while the difference was the largest at the roof floor.

Unlike the time instance of peak roof displacement, the largest difference occurred in Case 04, with the MRF-Spine system under the Northridge-VA 100% excitation. When the FLCs were activated, namely in the case of Case 09, the real deformation shape was closer to the rigid body shape, but still not as close as the previous time instance. For Case 02 and 07, similar to the previous time instance, there was also no such tendency as Case 04 and 09, indicating that benefits of the FLCs mainly took place when the structure yielded. For the cases under JMA-Kobe excitation, the larger excitation of 100% scale level (Case 13) generally gave larger displacement and difference, as expected.

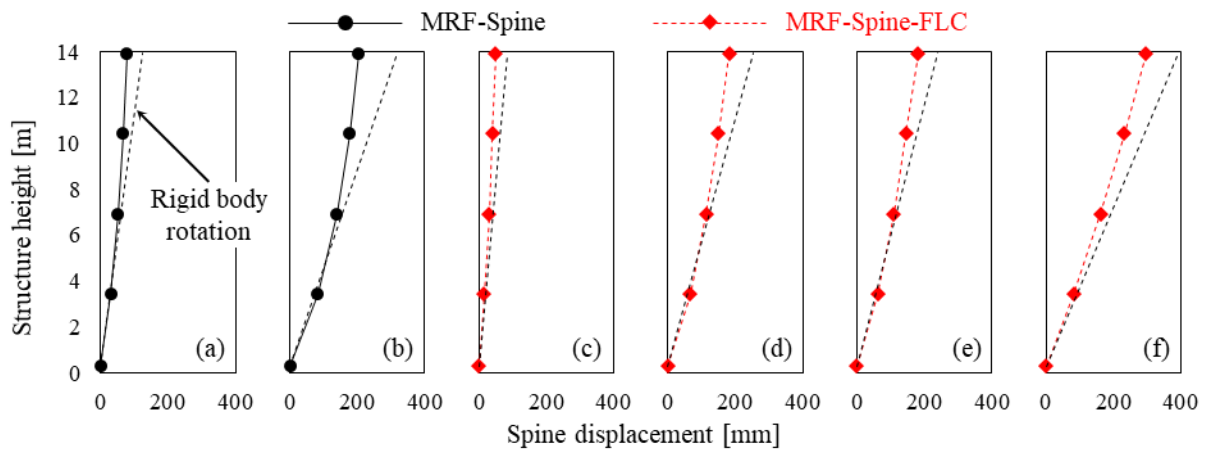


Figure 4.20 Spine deformation at peak spine shear force

- (a) Case 02, $T=9.755s$, Northridge VA 40%; (b) Case 04, $T=9.770s$, Northridge VA 100%;
(c) Case 07, $T=9.770s$, Northridge VA 40%; (d) Case 009, $T=9.770s$, Northridge VA 100%;
(e) Case 11, $T=16.195s$, JMA Kobe 50%; (f) Case 13, $T=15.450s$, JMA Kobe 100%.

Table 4.4 Difference of floor displacement [Unit: mm]

Load cases	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
Floor 4	47.4	114.1	36.3	69.3	57.1	98.1
Floor 3	27.2	60.6	23.5	36.5	31.3	61.4
Floor 2	9.2	15.7	11.0	8.4	7.1	29.5
Floor 1	-0.6	-8.2	3.8	-8.0	-7.3	8.5

4.6.3 Closure

In summary, although the spine deformation was not constant over time, as indicated by the two instances, the general tendency of the spine deformation pattern was quite stable in all the excitations, with the largest drift ratio at the 1st story and gradually decreased drift ratio of the upper stories. Such a general tendency was not significantly affected by the connection types or ground motions.

4.7 STRUCTURE YIELDING DISTRIBUTION

4.7.1 Column interaction curve

4.7.1.1 Interaction curve of the column sections

Considering the large axial force in columns, in addition to hysteresis curves, N-M interaction curve is required to judge the yielding condition of column sections. The N-M interaction curve of a section gives the boundary state of the section. For the HSS section of the 1st to 3rd stories and the I section of the 4th story, their N-M interaction curves can be derived using Eq 4.1 and 4.2, respectively. For both sections, the elastic boundary for m and n , a straight line, can be derived using Eq 4.3.

$$m = \begin{cases} 1 - \frac{4}{3} \times n^2, & n \leq 0.5 \\ \frac{4}{3} \times (1 - n), & n > 0.5 \end{cases} \quad (4.1)$$

$$m = \begin{cases} 1 - \frac{(2 + r_w)^2}{r_w(4 + r_w)} \times n^2, & n \leq \frac{r_w}{2 + r_w} \\ \frac{2 \times (2 + r_w)}{4 + r_w} \times (1 - n), & n > \frac{r_w}{2 + r_w} \end{cases} \quad (4.2)$$

$$m = \frac{Z}{Z_p} (1 - n) \quad (4.3)$$

Where,

$m = M/M_p$: section moment M normalized by the section plastic moment M_p .

$n = N/N_y$: section axial force N normalized by the section yielding axial force N_y .

$r_w = A_w/A_f$: ratio of the web area to the flange area of an I section.

Z : elastic section modulus.

Z_p : plastic section modulus.

Figure 4.21 illustrates the N-M interaction curves of the two column sections of the specimen. For both curves, the red dashed line indicates the boundary of elastic response, while the black solid line indicates the boundary of plastic response. These two lines are denoted as the elastic line and plastic line hereafter, respectively. When a data pair $[m, n]$ of a section falls into region (i), it means the section is elastic. When the data pair falls into region (ii), it means the section yielded but no plastic hinge is formed. When the data pair falls into region (iii), it means plastic hinge is formed at the section.

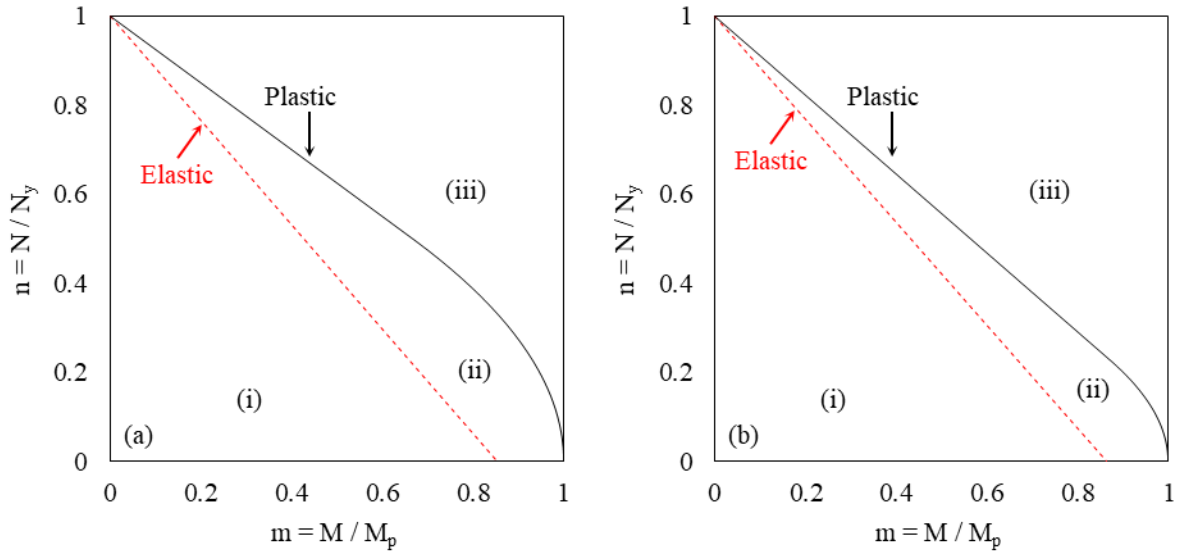


Figure 4.21 N-M interaction curve of the column sections
(a) Section: HSS-250×250×9; (b) Section: W10×100.

4.7.1.2 Interaction in the test

Figure 4.22 illustrates the four-quadrant N-M interaction curves of the east column at the south frame, namely C1. The $[m, n]$ data pairs of the column sections were plotted with the curves. In the plots, the column ends at the same story were plotted in the same figure. Note that curves for the two elastic cases, Cases 02 and 07, were not included here but put in Appendix D.

The N-M curves gave similar but more strict yielding judgement of the column sections compared with the hysteresis curves. For all the four cases shown here, the top and bottom section of the 4th story, which were connected to the 4th floor panel and 3rd floor panel, respectively, did not yield. While the upper section of the 1st story and the lower section of the 2nd story (both connected to the panel of Floor 1) yielded in all the four cases. For Case 04, 09 and 11, yielding developed up to the top section of the 3rd story, but no exceedance of the plastic boundary. In the last excitation, sections up to the top section of the 3rd story exceeded the plastic boundary, indicating possible formation of plastic hinges at these sections. Formation of plastic hinges seemed severe from the P-M curves, but taking the column end hysteresis into account, it can be found that only the top section of the 1st story showed plateau in the hysteresis curves, indicating plastic hinge formation, while the other sections were still in the hardening phase.

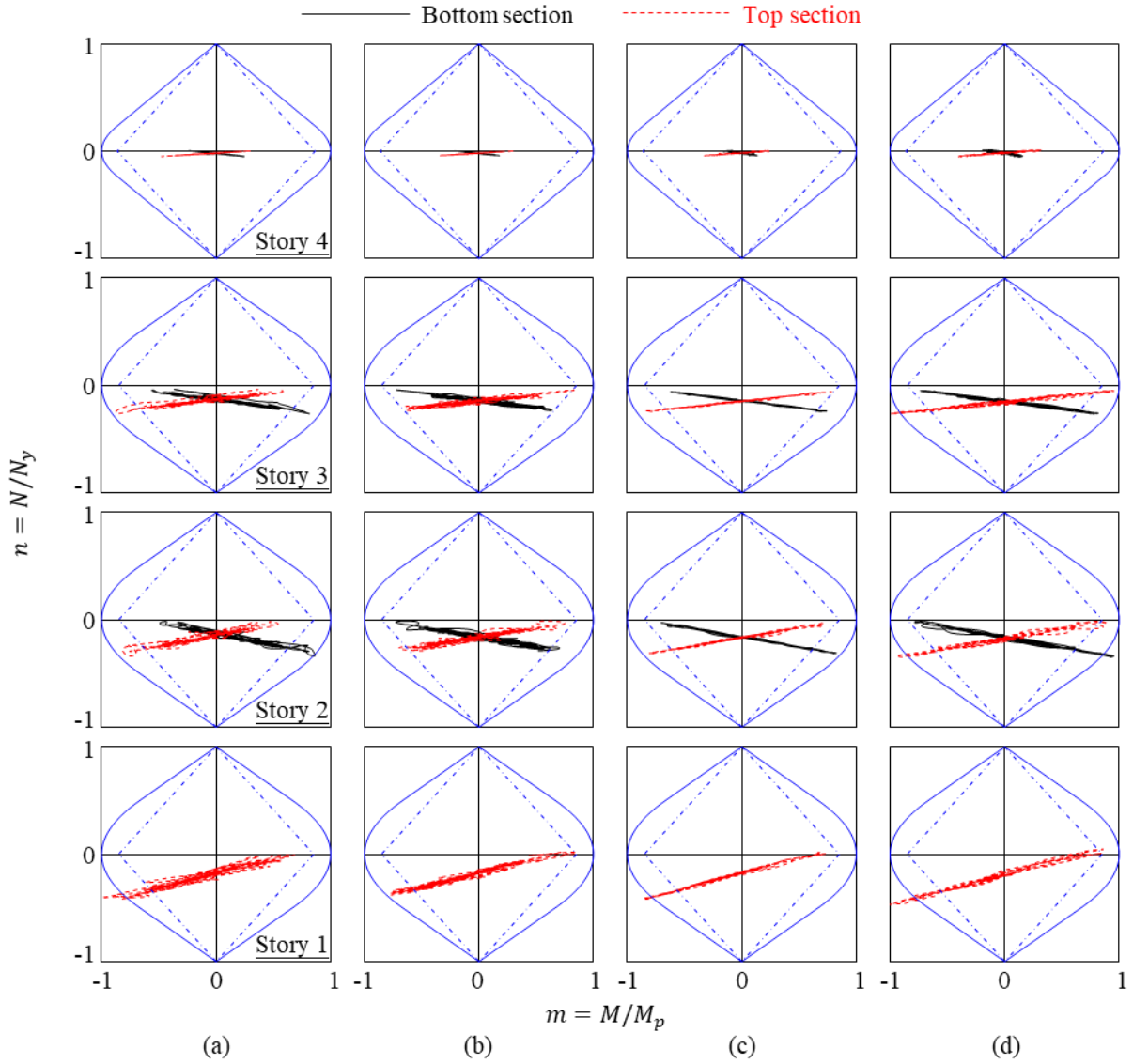


Figure 4.22 N-M interaction curve with column data

- (a) Case 04, Northridge VA 100%, MRF-Spine; (b) Case 09, Northridge VA 100%, MRF-Spine-FLC;
(c) Case 11, JMA Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA Kobe 100%, MRF-Spine-FLC.

4.7.2 Snapshot of moment diagram

According to preliminary numerical simulation and proportion of the structure members, the 1st story was the most vulnerable in the structure, thus the time instance of peak SDR at the 1st story can represent the yielding condition of the structure. In addition, time instances for other key conditions, such as peak spine shear force, peak column moment, peak roof displacement and so on, were close to or the same as this time instance. Thus, yielding distribution at this time instance of each major excitation was selected to evaluate the yielding condition of the specimen, together with the Moment diagram of the south frame and spine.

Figure 4.23 plots the snapshots of moment diagram in the south frame at the aforementioned time instance of the four cases with larger excitations, with the peak moment indicated. Note that the moment

at each beam-column intersection is the nodal moment rather than the column/beam end moment. Moment diagram was plotted at the tensile side of the structure members. Moment diagram of this time instance of all six major excitations of both the south and north frames are included in Appendix D. Although not shown here, the north frame showed similar diaphragm as the south one. For the two elastic cases with Northridge-VA 40% excitation (Case 02 and 07), both frames showed similar diaphragm.

In all the four cases illustrated, the MRF members showed a typical Moment diagram under lateral load. For the two cases with Northridge-VA 100% excitation (Case 04 and 09), no matter the FLCs were activated or not, the moment of the MRF was similar, with quite close peak moment for both columns and beams. For the two cases with JMA-Kobe 50% and 100% excitations (Case 11 and 13), MRF moment of the lower three stories was similar but Case 13 showed larger moment at the 4th story. Such difference was due to the yielding of the lower stories in these two cases. Yielding of the columns and panel zones limited the development of bending moment at the lower stories, but for the elastic top story, moment developed normally when the excitation got stronger.

In all the four cases, the spine showed a Moment diagram similar to a simply supported beam, with almost zero moment at both ends and moment of the same sign over the total height. It can be seen that at each floor level, where there was an MRF-Spine connection, there was a small jump of the spine moment. This jump indicates that the connections also transferred some moment between the MRF and the spine. But considering the limited scale of moment jump, such contribution can be neglected when analyzing the global performance of the structure. The peak moment of the spine occurred at the connection of the 1st floor, which was consistent with the larger spine shear force at the 1st story. The spine moment for the MRF-Spine-FLC system (Case 09/11/13) was significantly smaller than the MRF-Spine system (Case 04). Such difference indicates that the FLCs functioned as expected to control the spine internal forces. Comparing the spine moment of Case 09 and 13, it can be seen that, although the PGA of JMA-Kobe motion was stronger than the Northridge-VA motion (8.2 m/s^2 vs 7.4 m/s^2), the spine moment in Case 13 was smaller than in Case 09. Such difference was reasonable considering the difference between two ground motions. As aforementioned, the Northridge-VA motion was stronger at the 2nd natural period and was easier to excite the higher mode responses in the system. Difference of the spine moment under these two ground motions indicates that contribution of higher modes to the spine forces was important and need to be further analyzed.

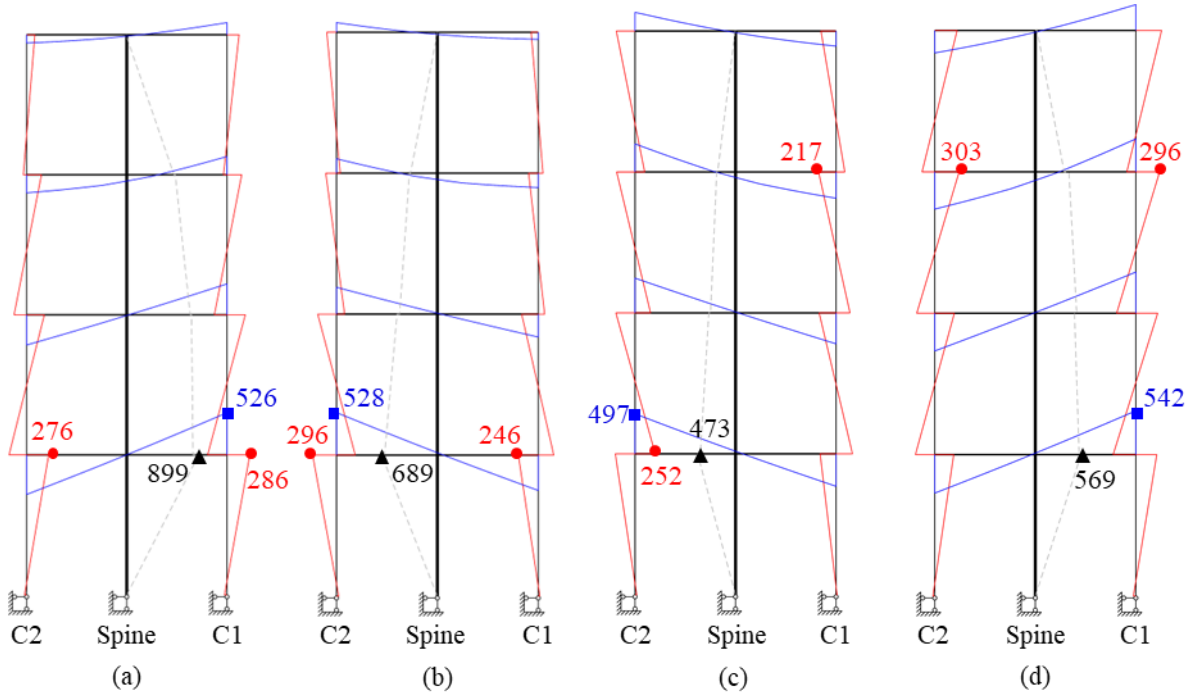


Figure 4.23 Moment diagram of the south frame at peak SDR of the 1st story [Unit: kN·m]

- (a) Case 04, $T=9.760s$, Northridge VA 100%, MRF-Spine;
- (b) Case 09, $T=9.770s$ Northridge VA 100%, MRF-Spine-FLC;
- (c) Case 11, $T=16.350s$, JMA Kobe 50%, MRF-Spine-FLC;
- (d) Case 13, $T=15.625s$ JMA Kobe 100%, MRF-Spine-FLC.

4.7.3 Yielding development of the whole structure

Different components of the structure required different judgement of yielding. For columns, P-M curves and hysteresis were used together for the judgement. For panel zones and FLCs, their yielding was judged using the hysteresis curves. Besides, the beam ends showed no yielding, as aforementioned. Summarizing all these yielding conditions, distribution of the structure yielding can be determined. Figure 4.24 illustrates the case-by-case yielding development of the south frame in the four large major excitations. Cases 02 and 07 were elastic cases with no yielding at all, thus not included.

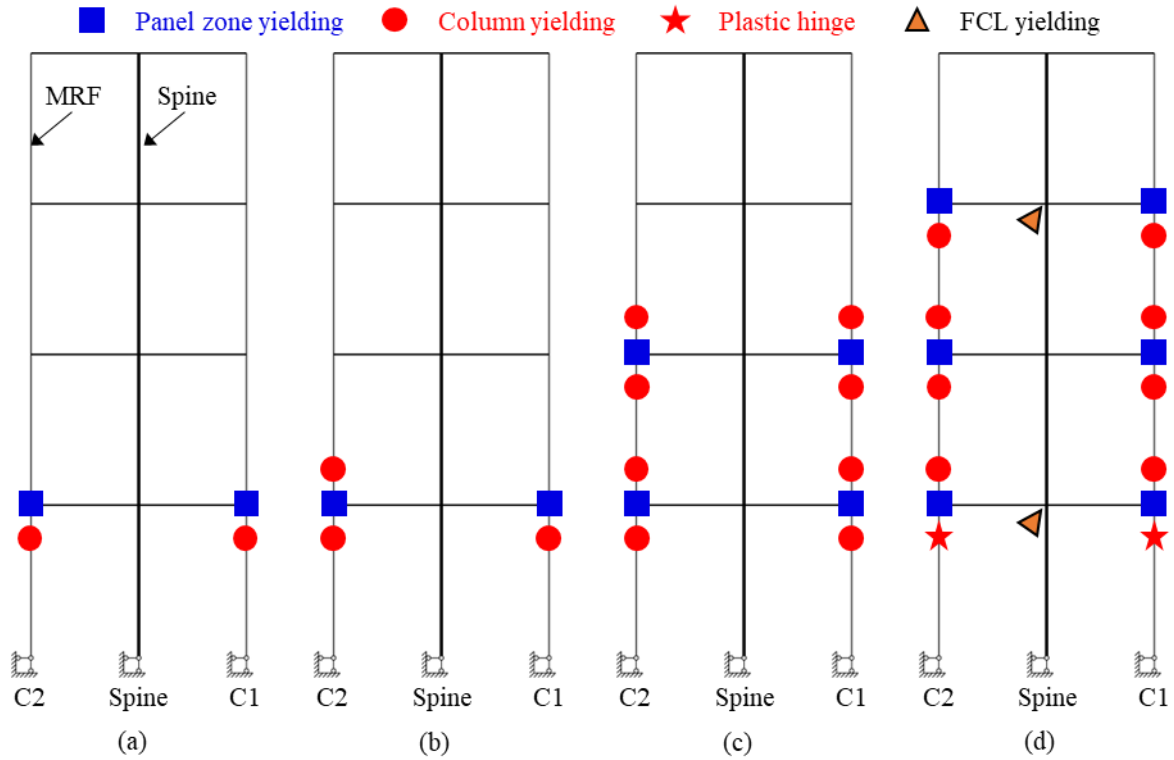


Figure 4.24 Yielding development of the south frame

(a) Case 04, Northridge-VA 100%, MRF-Spine; (b) Case 09, Northridge-VA 100%, MRF-Spine-FLC; (c) Case 11, JMA-Kobe 50%, MRF-Spine-FLC; (d) Case 13, JMA-Kobe 100%, MRF-Spine-FLC.

From the figure, it was clear that yielding developed from lower floors to higher. Case 04 and 09, with the same Northridge-VA 100% input but different structure systems, gave similar yielding distribution, with the panel zones of the 1st floor and column top sections connected to the panels included. In Case 09, the bottom section of the west column at the 2nd story also yielded. In Case 11 with JMA-Kobe 50% input, yielding developed upper to the 2nd floor level, with the yielding of the panel zones at this floor and the columns connected to them included. In Case 13 with JMA-Kobe 100% input, yielding continued to develop to the 3rd floor, counting in the panel zones of this floor and the top section of the 3rd story columns. Also in this excitation, the top sections of the 1st story columns finally developed into plastic hinges. Besides, noticeable yielding of both TFLCs at the 1st floor and UFLCs at the 3rd floor was also observed in this excitation.

The yielding distribution illustrated here is consistent with the hysteresis illustrated previously. Such yielding was inevitable by design because the same beam and column sections were used for the lower three stories of the MRF, which made the capacity smaller compared to the demand at the 1st story. Yielding of the panel zones was inevitable due to the relative strength of the structure members, as aforementioned in Chapter 03. For the four cases above, no matter the FLCs were activated or not, the gradually developed yielding indicates the effectiveness of the spines in distributing damage over the structure height. As shown in the last excitation, all the sections of the lower three stories yielded, forming a quite standard global mechanism. In such case, the full potential of the structure was realized.

4.8 DISCUSSION OF TEST RESULTS

4.8.1 Performance of the spines

4.8.1.1 Effectiveness of the spines

The major function of the spines was to force the frame to deform uniformly and distribute the damage evenly over the structure height. Distribution of Story Drift Ratio (SDR) of each load case over the height can be a measure of the performance of the spines. As aforementioned, the output from the DAQ system contained residual value, making it meaningless to directly compare the SDR of each load case. Considering this, the relative value of the SDR of each load case, namely SDR with initial value removed, was used for the analysis of the effectiveness of the spines. As for the pure MRF, since it was not tested on the shake table, data from numerical simulation were used to form the comparison. Figure 4.25 illustrates the profile of the peak relative SDR of all six major excitations.

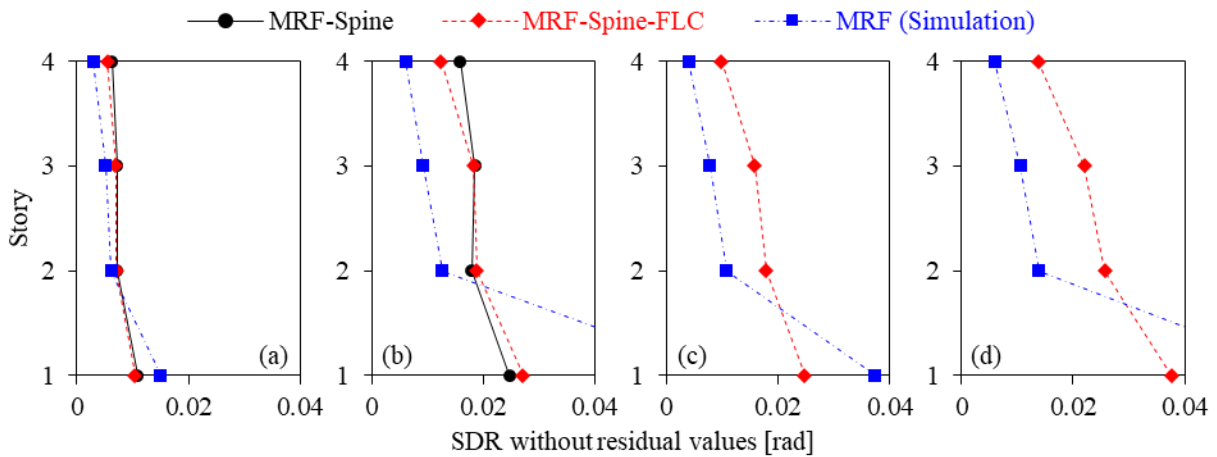


Figure 4.25 Profile of peak relative SDR (SDR larger than 0.04 not plotted)

(a) Northridge-VA 40%; (b) Northridge VA 100%; (c) JMA Kobe 50%; (d) JMA-Kobe 100%.

From the distribution above, it is obvious that the pure MRF formed a significant story mechanism at the 1st story even under the smaller elastic excitations. Under both 100% excitations, SDR of the 1st story exceeded 0.04 rad while the upper stories showed SDR no larger than 0.015 rad. Compared to the smaller excitations, increase at the 1st story was much more significant than the upper stories, indicating obvious drift concentration at the 1st story. Such concentrated deformation drew tremendous demand to the 1st story but utilized rather limited capacity of the upper stories. For the system with spines, the SDR decreased significantly at the 1st story while increased at the upper stories, making the deformation got more uniform at each story and thus the capacity of all the stories can be utilized.

The Drift Concentration Factor (DCF) [5], which took the ratio of the maximum story drift ratio (SDR) to the maximum roof drift ratio (RDR), was selected as the measure of the uniformity of the distribution. When the structure moves as a rigid body, SDR of each story will be identical to RDR, making DCF equal to 1. When there is any concentrated deformation, the maximum SDR will exceed the RDR, and

DCF will be larger than 1. The larger DCF, the more concentrated deformation. Table 4.5 summarizes the DCFs of the major excitations. For all the excitations, regardless the yielding condition of the systems, systems with spines showed much smaller DCF than the system without (the MRF). Largest reduction of DCF was observed from Case 04, the MRF-Spine system, reaching the value of 62%. For Case 09 and 13, in which the FLCs were activated, the reduction was also significant with the rate of 57% and 53%, respectively. These reduced DCF indicates that when spines were attached to the MRF, the uniformity of structural deformation was greatly improved. In addition, for the system with spines, DCF was quite consistent for different ground motions and intensities. The spines functioned as expected in unifying the story deformation of an originally weak-story MRF.

Table 4.5 DCF of considered cases

Case	Sim.	Case 02	Case 07	Sim.	Case 04	Case 09	Sim.	Case 11	Sim.	Case 13
Excitation		Northridge	VA 40%		Northridge	VA 100%		JMA Kobe 50%		JMA Kobe 100%
DCF	2.20	1.45	1.51	3.17	1.34	1.54	2.80	1.48	3.09	1.56

(All Sim. cases refer to MRF system)

Aside from the relative SDR profile, the development of yielding also proved the effectiveness of the spines. As aforementioned in Section 4.7.2, damage developed to the upper stories of the MRF when the input ground motion grew stronger. For the largest input of JMA-Kobe 100% excitation, yielding of the column ends and panel zones even occurred at the 3rd floor. Such development of yielding indicates that the spines functioned effectively to distribute damage and utilize the full potential of the structure, no matter the FLCs were activated or not.

In summary, both the peak SDR profile and the yielding development indicate that the spines functioned satisfactorily during the test. The deformation of the structure was distributed more uniformly, and the total capacity of the structure was thus utilized.

4.8.1.2 Problems drawn by the spines

Aside from the benefits brought by the spines in the deformation domain, there were problems in the force domain, namely floor acceleration and structure shear force. The dimensionless amplification factor, defined as the peak floor acceleration divided by the peak input ground motion, was used for comparison of floor acceleration to deal with different inputs. As for the structure shear force, only the shear force taken by the MRF was compared. Figure 4.26 compares the profile of floor acceleration amplification factors and the MRF shear force of the MRF and MRF-Spine systems under the Northridge VA 40% and 100% excitations.

For the amplification factor, values of the MRF-Spine systems were generally larger at all floor levels in both intensities, especially at the 1st and 2nd floors. The largest increase occurred at the 2nd floor, with an increase from 0.90 to 1.88. In general, the average amplification factor of each floor increased by about 47% from 0.86 to 1.27. The drastic increase of floor acceleration will introduce problems for

the acceleration sensitive non-structural systems, such as ceiling, piping and so on. The damage in these systems will greatly reduce the resilience of the building structure, which was opposite the purpose of utilizing the spines. In addition to the general tendency, increase in the 40% excitation was smaller than the 100% excitation, suggesting that problems brought by the spines may be more significant in post-elastic systems.

For the MRF shear force, shear force in the MRF-Spine system at the 1st story decreased by about 37%, while the upper stories showed much larger shear force. At the 4th story, the MRF shear force after the spines were inserted increased from 312 kN to 656 kN, more than twice the shear force in the pure MRF. In general, the average shear force taken by the MRF increased about 39% from 366 kN to 510 kN. Such significant increase in the shear force taken by the MRF will greatly increase the demand on the structure. If the MRF members remain the same after inserting the spines, special consideration will be required to prevent unexpected failure in the upper stories. For the two excitation levels, increase at the 1st story was similar while much smaller at the upper stories for the 40% excitation. The similar 1st story increase was attributed to the yielding of the story which limited the story shear force, while the larger increase in the upper stories indicates that problems brought by the spines may be more severe in post-elastic systems, same as the trend of amplification factors.

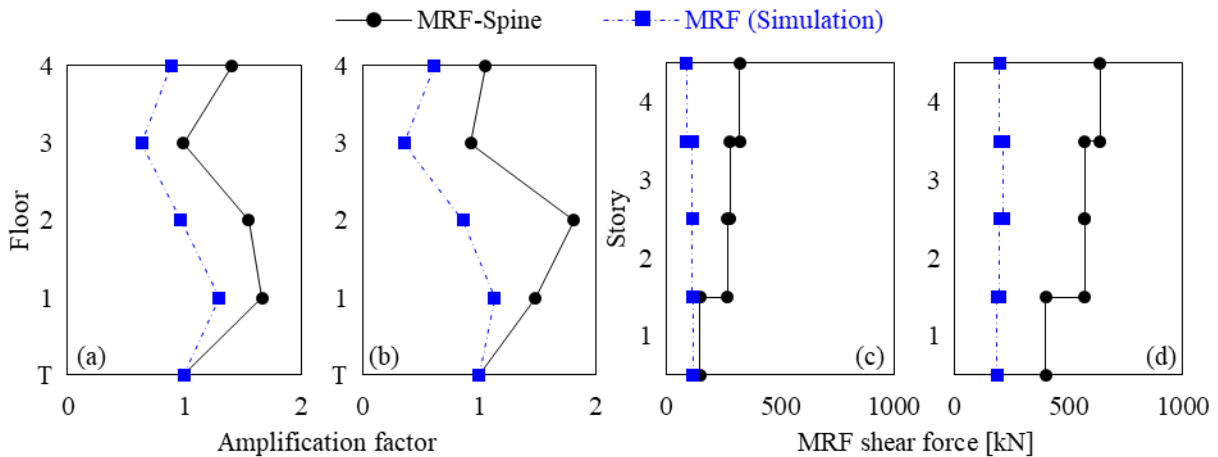


Figure 4.26 Profile of acceleration amplification factor and MRF story shear force
 Amplification factor: (a) Northridge VA 40%; (b) Northridge VA 100%;
 MRF shear force: (c) Northridge VA 40%; (d) Northridge VA 100%.

4.8.1.3 Closure

From the discussion above, it can be concluded that insertion of the spines can effectively control the story mechanism formed in the MRF, no matter the FLCs were activated or not. After the insertion of the spines, distribution of relative SDR was significantly more uniform and the concentration of SDR at the 1st story was limited to a reasonable level. Despite their effectiveness, the spines also brought problems in the force domain. Both floor acceleration and story shear force increased when the spines were inserted. Solving such problems was the purpose of introducing FLCs into the MRF-Spine system,

as discussed in the following section.

4.8.2 Performance of the FLCs

As aforementioned in the previous section, insertion of the spines greatly increased the floor acceleration and MRF shear force. The purpose of introducing the FLCs was just to deal with this shortcoming by allowing some relative displacement between the MRF and the spines. Both the floor acceleration and story shear force are used to evaluate the effectiveness of the FLCs, while the distribution of relative SDR was used to evaluate the cost of using FLCs.

4.8.2.1 Distribution of floor acceleration amplification factors

In order to compare the floor acceleration profile with different input, the amplification factor was used, taking the ratio of the peak floor acceleration of the upper floors to that of the shake table, as illustrated in Figure 4.27.

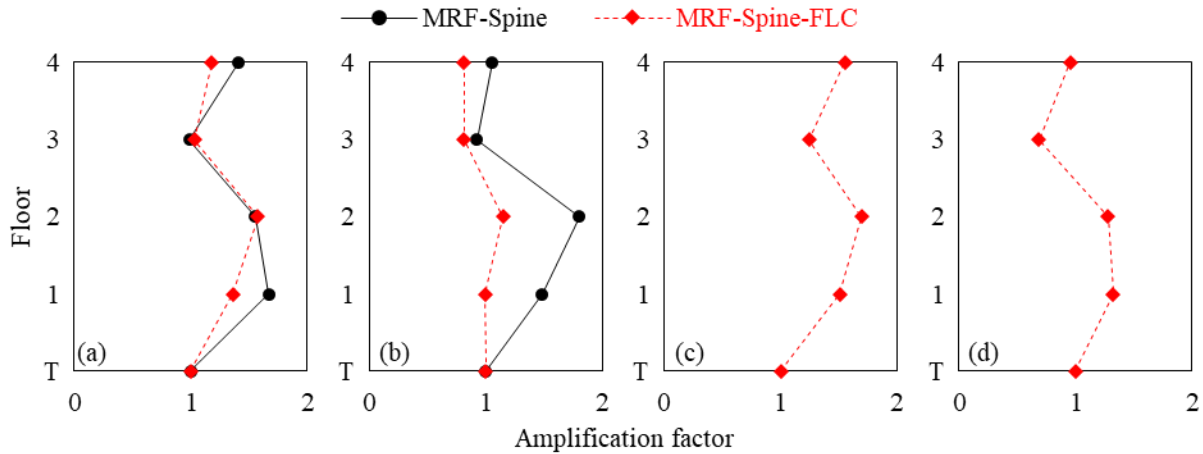


Figure 4.27 Profile of amplification factor

(a) Northridge-VA 40% (Case 02 and 07); (b) Northridge-VA 100% (Case 04 and 09);
(c) JMA-Kobe 50%; (d) JMA Kobe 100% (Case 13)

Comparing the amplification factors of Case 02 and 07, in which the structure remained almost elastic, difference was minor at all floors or the mean value of each floor, with the largest difference of about 19% took place at the roof floor (Floor 4) and the difference of mean value of about 10%. Comparing the factors of Case 04 and 09, in which the structure yielded, difference was generally larger than the elastic cases. The largest reduction occurred at the 2nd floor with the value of 41%, and the reduction of the mean value was 30%. The different reduction level of the mean amplification factor in the elastic and yielded cases (10% vs 30%) indicates that the benefits of the FLCs was much better utilized when the structure started yielding.

Besides, when comparing the three case pairs with different scale levels (Case 02 vs 04, Case 07 vs 09 and Case 11 vs 13), there was a common tendency that the larger the input ground motion, the smaller

the amplification factor at each floor. The reduction of the mean amplification factors of these three pairs were 9%, 30% and 33%, respectively. Since stronger ground motion would cause heavier yielding, such similar tendency indicates that yielding of the structure alone can help reduce the floor acceleration. The larger reduction for FLC activated cases (9% vs 30% and 9% vs 33%) clearly showed the contribution of the FLCs in such reduction.

4.8.2.2 Distribution of story shear force

Aside from floor acceleration, distribution of peak story shear force and spine shear force is also a measure to evaluate the effectiveness of the FLCs. Specific data of the peak shear force distribution have been illustrated in Section 4.4.3, their mean values of each story are summarized in Table 4.6. Comparison of three types of shear force under the elastic and yielded cases (Case 02 vs 07, Case 04 vs 09) are illustrated in Figure 4.28. Note that the force of the spines and the MRF in the table do not add up to the total value, since these values are not taken at the same time instance. The mean values are used to illustrate the general trend of the whole structure.

Table 4.6 Mean values of peak story shear force [Unit: kN]

Case	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
Excitation	NR-40%	NR-100%	NR-40%	NR-100%	JMK-50%	JMK-100%
System	MRF-Spine		MRF-Spine-FLC			
Total	306	698	294	578	512	655
Spine	126	315	102	213	183	269
MRF	255	546	256	492	453	544

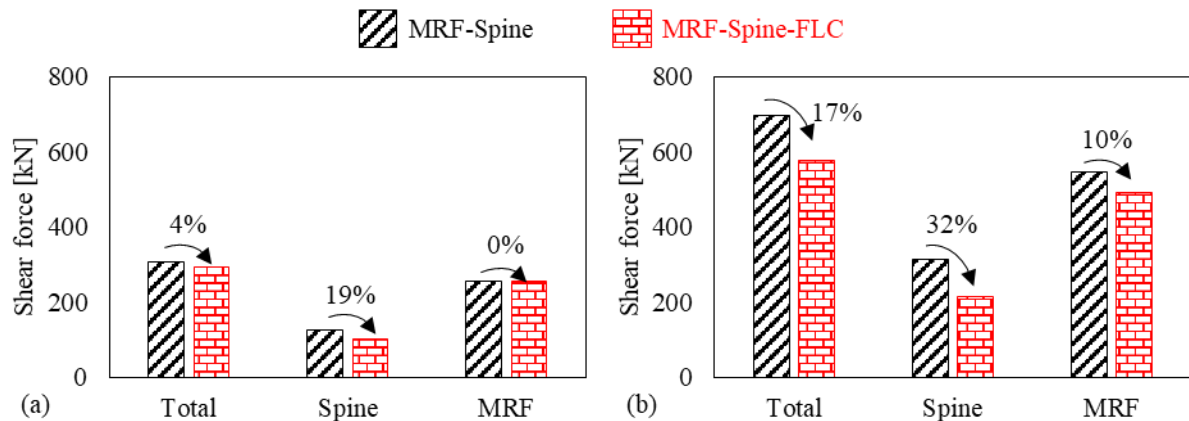


Figure 4.28 Comparison of mean peak story shear force of two structure systems
(a) Elastic cases (Northridge-VA 40%); (b) Yielded cases (Northridge-VA 100%)

Comparing the mean shear force of Case 02 and 07 (black and red, respectively) in Figure 4.28(a), in which the structure was almost elastic, it can be seen that shear force taken by the whole structure, the spines and the MRF all decreased when the FLCs were activated (Case 07). The relative reduction of the total shear force, spine shear force and MRF shear force were 8%, 19% and 4%, respectively. Such reduction indicates that FLCs can help to limit the shear force of the whole structure system, but they

were especially effective in limiting the shear force taken by the spines.

Comparing the mean shear force of Case 04 and Case 09 (black and red, respectively) in Figure 4.28(b), in which the structure yielded, similar tendency as the elastic cases occurred: shear force of the whole structure, the spines and the MRF all decreased, with the relative reduction of 16%, 31% and 9%, respectively. For the yielded cases, the reduction in the spine shear was also the largest, giving the same result that FLCs were more effective in limiting the spine shear force.

Comparing the relative reduction of the aforementioned three types of shear force, it is obvious that for each type, the reduction of the yielded cases (Case 04 and 09) was nearly twice the elastic cases (Case 02 and 07). Such significant difference indicates the same tendency as the floor acceleration distribution that, the benefit of the FLCs mainly occurred after the structure yielded.

Aside from the above comparison, when comparing Case 13 and Case 04, it can be seen that although the input ground motion was stronger in Case 13 (JMA-Kobe 100%), the three types of shear force were still smaller than that of Case 04 (Northridge-VA 100%). This difference also provides solid proof that the FLCs can effectively reduce the structure shear force.

4.8.2.3 Reduced uniformity of peak SDR distribution

Aside from the benefits brought by the FLCs, there were also problems brought by these softer connections, which was mainly in the deformation domain. DCFs and mean SDR of the specimen under the Northridge-VA excitation are summarized in Table 4.7.

Table 4.7 DCF of the specimen under Northridge-VA excitation

Case	Case 02	Case 04	Case 07	Case 09
Scale level	40%	100%	40%	100%
FLC condition	Without	Without	With	With
Mean SDR	0.008	0.019	0.007	0.019
DCF	1.45	1.34	1.51	1.54

Comparing the DCF of the specimen under the same scale level (Case 02 vs 07, Case 04 vs 09), similar tendency occurred that when the FLCs were activated (Case 07 and 09), the DCF increased by 4% and 15%, respectively. The increase for the 40% excitations in which the specimen remained almost elastic, is smaller than that of the 100% excitations in which the specimen yielded. DCFs of the specimen indicate that activation of the FLCs impaired the function of the spines to some extent, and such impairment was heavier when the structure yielded. Although DCF increased when the FLCs were activated, the mean SDR was almost identical for the same excitation level, indicating that the general performance of the structure was not greatly affected.

4.8.2.4 Closure

From the discussion above, it can be concluded that the FLCs worked effectively to reduce the floor acceleration and story shear force in the MRF-Spine-FLC system. Their benefits mainly occurred when the structure yielded. To obtain such benefits, the cost was the reduced uniformity of the structure deformation. The decrease of the DCFs was much larger when the structure yielded, which was just the condition that the FLCs functioned more effectively. However, despite the reduced uniformity, the mean SDR remained almost identical for both elastic and yielded cases. Based on the above analysis, it can be concluded that the FLCs functioned effectively with a reasonable cost in the uniformity of deformation distribution.

4.8.3 Energy dissipation

For a seismic resistant structure, energy dissipation is one of the major concerns for evaluating the structure's performance. The dissipated energy was calculated by adding up the enclosed area of the hysteresis loops of each relevant location. Considering the almost elastic behavior of the beam ends, only the MRF-Spine connections, panel zones and the column ends were considered as source of energy dissipation. Since there was no direct measurement of the column end rotation, the energy dissipated by the columns was deduced using story drift and column shear force, which may overestimate the energy dissipated by the columns since the story drift may contain plastic deformation from panel zones instead of columns.

Figure 4.29(a) illustrates the energy dissipated by the structure, including all major cases and the cumulative value of all cases. Considering the above mentioned issues on column energy dissipation estimation, Figure 4.29(b) illustrates a comparison between the panel zones and MRF-Spine connections only. Values of the dissipated energy and the portion taken by each energy dissipation source are summarized in Table 4.8. It should be noted that energy dissipated by the panel zones were estimated by doubling the values obtained from the panels with measurement (both in the south frame).

In the four large excitations (Case 04, 09, 11 and 13), the panel zones gradually dissipated more energy with the input ground motion getting stronger. In the last excitation (Case 13, JMA-Kobe 100%), although the portion was slightly lower than the previous case, the absolute value grew dramatically. This is consistent with the panel zone hysteresis illustrated in Figure 4.14, that the panel zones developed very stable hysteresis loops in the last excitation at Floor 1~3. Similar trend can be observed from the column ends. Although the column ends showed substantial yielding only in the last and strongest excitation, due to the larger number of column ends, the energy dissipated by column ends was still slightly more than the panel zones.

For the MRF-Spine connections, FLCs were activated in Cases 09, 11 and 13. The energy dissipated by the connections in Case 04 was mainly due to the friction within the connections, since the connections

were not really rigid and some relative displacement occurred. For the FLC activated cases, the absolute value of dissipated energy was lower than that of the rigid case. This was reasonable because when FLCs were activated, connections at the 3rd floor were totally released and no energy dissipation took place, making the total dissipated energy less in these cases.

In summary, the major energy dissipation source of the specimen was the panel zones, especially in the last excitation. The FLCs dissipated much less energy throughout the tests, although some yielding took place. In fact, the major function of the FLCs was not to dissipate energy but to form softer connections and release the constraints between the MRF and the spines.

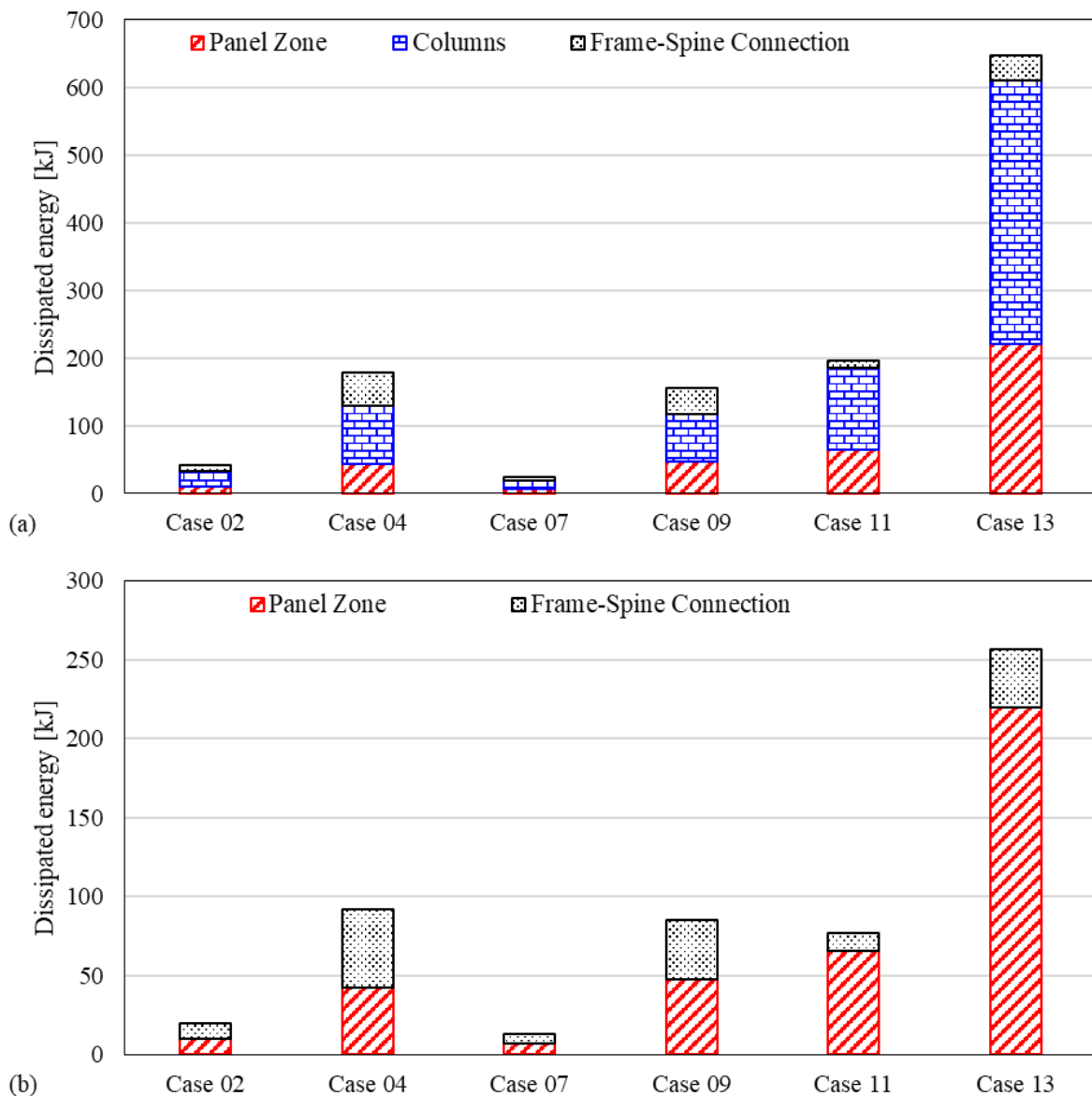


Figure 4.29 Dissipated energy of three major sources
(a) Column contribution included; (b) Column contribution excluded.

Table 4.8 Dissipated energy and the portion of three major sources [Unit: kN·m]

(a) Column contribution included

Cases	MRF-Spine connection		Panel zone		Column		Total dissipation
	Energy	Portion	Energy	Portion	Energy	Portion	
Case 02	9.2	22%	10.2	24%	22.7	54%	42.2
Case 04	49.1	27%	42.6	24%	87.6	49%	179.3
Case 07	6.3	26%	6.9	28%	11.1	46%	24.3
Case 09	37.8	24%	47.2	30%	70.5	45%	155.5
Case 11	11.2	6%	65.3	33%	120.2	61%	196.6
Case 13	36.7	6%	219.8	34%	390.9	60%	647.4

(b) Column contribution excluded

Cases	MRF-Spine connection		Panel zone		Total dissipation
	Energy	Portion	Energy	Portion	
Case 02	9.2	47%	10.2	53%	19.4
Case 04	49.1	54%	42.6	46%	91.7
Case 07	6.3	48%	6.9	52%	13.2
Case 09	37.8	44%	47.2	56%	85.0
Case 11	11.2	15%	65.3	85%	76.5
Case 13	36.7	14%	219.8	86%	256.5

4.8.4 Stiffness ratio between the spines and the MRF

Considering that the spines are vertically continuous structure members, it is better to use a stiffness that can represent the concept of global properties for both the spines and the MRF. In previous research on pin-supported walls, Qu et al [37] suggested to use an analog of a simply supported beam under uniformly distributed loads for the walls, and took the 1st modal stiffness of a Euler beam as the stiffness of the walls. For the MRF, the 1st modal stiffness was used to represent its stiffness.

Taking this concept, the stiffness of the spine, K_s , can be calculated as following:

$$K_s = \frac{\pi^4 E_s I_s}{2H_s^3} \quad (4.4)$$

Where,

E_s is the Young's modulus of the spine.

I_s is the sectional moment of inertia of the spine.

H_s is the total height of the spine.

For the specimen, the parameters were taken as $E_s=205$ GPa, $I_s=0.00278$ m⁴, and $H_s=13.645$ m. Thus, the stiffness of the spine, K_s , was 10926 kN/m.

For the MRF, the first modal stiffness, K_f , can be calculated as following:

$$K_f = \left(\frac{2\pi}{T_1} \right)^2 M_1 \quad (4.5)$$

$$M_1 = \left(\frac{\phi_1^T m \mathbf{1}}{\phi_1^T m \phi_1} \right)^2 \phi_1^T m \phi_1 \quad (4.6)$$

Where,

T_1 is the first mode natural period of the MRF.

M_1 is the first mode participating mass.

m is the mass matrix of the MRF.

ϕ_1 is the 1st mode shape vector of the MRF.

For the specimen, the 1st modal vector and mass matrix are as following illustrated in Eq. 5.6. Note that the modal vector was deduced from test results in the first white noise excitation (Case 01) using Frequency Domain Decomposition (FDD) method [53], while the mass matrix is half of the total lumped mass since only the south frame was considered.

$$\phi_1 = [0.34 \quad 0.60 \quad 0.84 \quad 1]^T \quad (4.7)$$

$$m = \begin{bmatrix} 13.1 & 0 & 0 & 0 \\ 0 & 8.2 & 0 & 0 \\ 0 & 0 & 26.9 & 0 \\ 0 & 0 & 0 & 28.4 \end{bmatrix} \quad (4.8)$$

Considering that $T_1 = 1.54 \text{ s}$, $M_1 = 73.2 \text{ ton}$, there is $K_f = 1219 \text{ kN/m}$.

With the definition of the stiffness of the spine and the MRF, and consider the existence of two spines in the system, the stiffness ratio, β , can thus be defined as following:

$$\beta = \frac{2 * K_s}{K_f} \quad (4.9)$$

Taking the K_s and K_f calculated above, the stiffness ratio for the specimen is then calculated as **18.0**. With such a large stiffness ratio, it was natural that the deformation of the structure was well controlled by the spine. Per the aforementioned paper, the recommended stiffness was 2, which is much smaller than the current case. This drastic difference indicates the necessity of a detailed design of the system.

4.9 SUMMARY

A series of full-scale shake table tests were conducted to evaluate the concept of the MRF-Spine-FLC structure system. Two input ground motions, two scale levels for each motion, and two structure systems were tested, taking 6 excitations in total. This chapter illustrated the test results in detail and discussed several key issues. In general, test results indicate that the spines successfully controlled the story mechanism in the original MRF, and the FLCs acted as expected to reduce the floor acceleration and story shear force demand with a reasonable reduction of the structure deformation uniformity. The key

points of this chapter are summarized in the following:

- The spines functioned satisfactorily in forcing a uniform deformation and distributing damage across the structure height. Compared to the original MRF, the insertion of spines significantly reduced the story drift ratio of the 1st story and increased the uniformity of story drift ratio distribution over the height. Drastic reduction of DCFs after the insertion of the spines gives clear evidence of such tendency. Besides, although the 1st story still showed the largest story drift ratio for the system with spines, the damage was no longer concentrated but developed to upper stories, while on the contrary, the original MRF showed concentrated damage at the 1st story with the capacity of the upper stories unused.
- The spines caused increased floor acceleration and story shear force. Compared with the original MRF, the peak floor acceleration and story shear force increased when the spines were attached, especially when the spines were rigidly connected to the MRF. Although the spines strengthened the whole structure, force demand also increased in the MRF members and the spines. The large floor acceleration requires extra attention on the acceleration-sensitive non-structural components, and the large shear force may cause trouble in the design of the structure elements.
- The Force Limiting Connections (FLCs) worked as expected in limiting both floor acceleration and story shear force. When the FLCs were activated, the peak floor acceleration was reduced at almost all the floors, limiting the amplification factor no larger than 1.5 of all load cases. The peak story shear force showed similar tendency as the acceleration. Both the total shear force and the shear force taken by the spines decreased after the activation of FLCs. Besides, the reduction in acceleration and shear force was much more significant for cases in which the structure yielded (Case 09/11/13), indicating that benefits of FLCs mainly come into effect after the structure yielded.
- The FLCs also brought problems of compromising the function of the spines. Compared to the pure MRF-Spine system, the system with FLCs showed less uniform distribution of peak story drift ratio across the height, which was indicated by a relatively larger DCF. Despite this increase of DCF, the mean SDR of the cases with and without FLCs remained almost identical, indicating that the cost of introducing the FLCs was reasonable in the tested cases. With the coexistence of both the benefits and problems of the FLCs, it is of great importance to balance the use of FLCs, taking the advantages without heavily impairing the function of the spines.
- Moment distribution of the structure members was as expected, with the MRF showing a typical distribution under lateral load and the spines showing that of a simply supported beam. No obvious difference was observed for different structure systems or different ground motion excitations.
- The connections between the MRF and the spines, either bolted connections or FLCs, transferred

primarily lateral force during the tests. Although jump of spine moment observed at the connections indicated that transfer of moment did exist at these locations, such connection moment was minor compared to the peak level of spine moment and can be neglected when analyzing the global system.

- Damage was not concentrated at the 1st story but developed upwards to the other stories, forming an expected global mechanism. Major yielding occurred at the panel zones and column ends of the lower three stories. No yielding observed at beam ends of all stories and column ends or panel zones at the top story. Yielding of the columns and panels both occurred firstly at the 1st story and then developed upwards. Initial yielding of both types of locations occurred at an early stage at the 1st story. With different cases, yielding gradually developed to upper stories. At the end of the tests, all column ends and panel zones of the 1st~3rd stories yielded.
- For the energy dissipation of the structure systems, the column ends and panel zones of the lower three stories and floors served as the major source of energy dissipation with their stable hysteresis. The FLCs showed some yielding, but not much energy was dissipated at these devices. Although not the major concern in the tests discussed herein, it would be helpful if the FLC devices can serve both purposes of limiting force and dissipating energy simultaneously.
- Despite the Strong-Beam-Weak-Column of the structure, global performance was not heavily influenced, indicating that the systems with spines are less sensitive to the proportion of columns and beams, which allows more freedom in structure design.

5. VIBRATION PROPERTY ANALYSIS USING SHAKE-TABLE TEST DATA: SELECTED TESTS AND METHODS

5.1 GENERAL

Given that quite some efforts have been made to investigate and unveil the mechanism of different modes in the systems with spines, and that some practical design methods have considered the influence of higher modes, there is still a lack of knowledge directly based on real world data. As far as the author has noticed, most of the work by far used numerical or analytical ways to investigate the modal mechanism inside the system. Hence, analysis using real-world data would be beneficial to deepen the understanding of the vibration properties of systems with spines.

Shake table tests with full- or large-scale specimens were considered suitable for such analysis considering their controlled excitation and close-to-reality dynamic responses. Considering the availability of test data, four shake-table tests were selected to form comparison with the MRF-Spine test introduced previously. System identification methods mainly used in structural health monitoring were adapted to extract modal parameters from the test results, including an output-only frequency domain method and two input-output time domain methods. In addition, a wavelet-based output-only method and a spectral method were also used for the analysis on ground motion excitation cases. Acceleration data at the upper floors and the shake table were used for all the analysis methods.

This chapter introduces details of the selected tests, data preparation and analysis methods. Results and discussion will be in the following chapter.

5.2 SELECTED SHAKE-TABLE TESTS

5.2.1 General information

The tests used for the analysis were selected so that they can form comparison with the major test, and that the specimens were close to reality. Table 5.1 summarizes the selected shake table tests. All the Japanese tests were conducted at the National Research Institute for Earth Science and Disaster Resilience (NIED) of Japan, with 07EDF and 08EDF at E-Defense and 21TKB at the Tsukuba Large-Scale Earthquake Simulator. The 14UCSD test was conducted at the large high-performance outdoor shake table at University of California, San Diego (UCSD), which is part of the Network for Earthquake Engineering Simulation (NEES). Information on the 21TKB test was obtained from the archive of the author's research group. Information of the 07EDF and 08EDF tests was obtained from the E-Defense data archive (ASEBI, <https://asebi.bosai.go.jp/>). Information of the 14UCSD test was obtained from the DesignSafe-CI Data Depot (<https://www.designsafe-ci.org/>).

The 21TKB and 07EDF tests were selected to form the comparison between systems with and without

spines, from which influence of structure scale can also be investigated. The 08EDF test was selected to provide information of higher modes in high-rise buildings, forming comparison of the properties of higher modes. The 14UCSD test was conducted on a totally different structure system and was selected to provide comparison outside the range of steel structures and moment-resisting-frames. Details of the 20EDF test have been introduced in the previous chapters. Details of the other four tests will be introduced hereafter.

Table 5.1 Selected shake table tests

Test name	Facility	Structure system	Excitation directions	Scale	# of stories
21TKB	Tsukuba	Low-rise steel MRF	1D	40%	4
07EDF	E-Defense	Low-rise steel MRF	3D	Full	4
08EDF	E-Defense	High-rise steel MRF	2D	Full	21
14UCSD	UCSD	RC Flat-slab and rocking wall	1D	50%	4

5.2.2 2021 Tsukuba low-rise steel MRF test (21TKB)

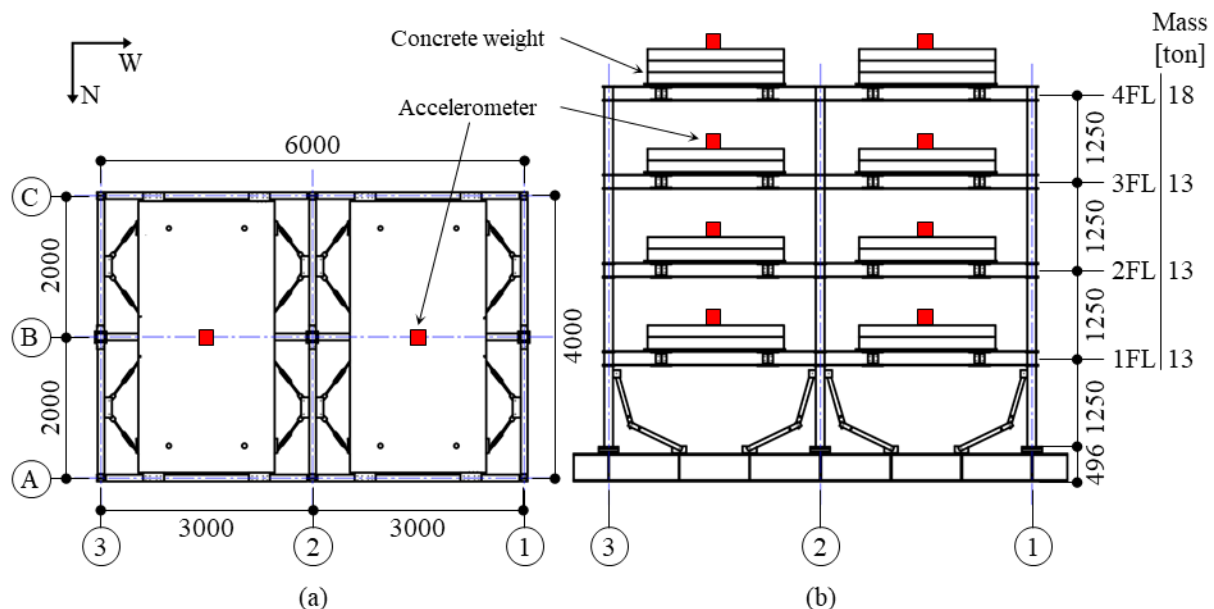


Figure 5.1 Specimen of the 21TKB test

(a) Plan view, (b) Elevation view

This test was conducted in 2021 at the NIED Tsukuba facility. Its objective was to observe and record damage propagation in low-rise steel MRFs. The specimen was a scaled, 2-span, 4-story, planar MRF. 2 parallel gravity frames were set to provide out-of-plane restraint to the MRF and share the weight of the masses. Figure 5.1 illustrates the plan and elevation view of the specimen. The specimen was subjected to uniaxial excitations in the EW direction. The NS component of the JMA Kobe record from the 1995 Kobe earthquake was applied. As listed in Table 5.2, 7 major cases were conducted with increasing intensity. The skipped case numbers represent white noise excitations and other small cases. During the test, fracture occurred at two beam-to-column connections, and afterwards, developed large story drift near 0.15 rad and residual story drift of 0.1 rad at the 1st story, but the specimen did not

collapse. Refer to [74,75] for further information.

Table 5.2 Major excitations in the 21TKB test

Case #	Excitation	Intensity
02	JMA Kobe NS	10%
04	JMA Kobe NS	25%
06	JMA Kobe NS	50%
09	JMA Kobe NS	100%
10	JMA Kobe NS	100%
11	JMA Kobe NS	100%
12	JMA Kobe NS	100%

5.2.3 2007 E-Defense low-rise steel MRF collapse test (07EDF)

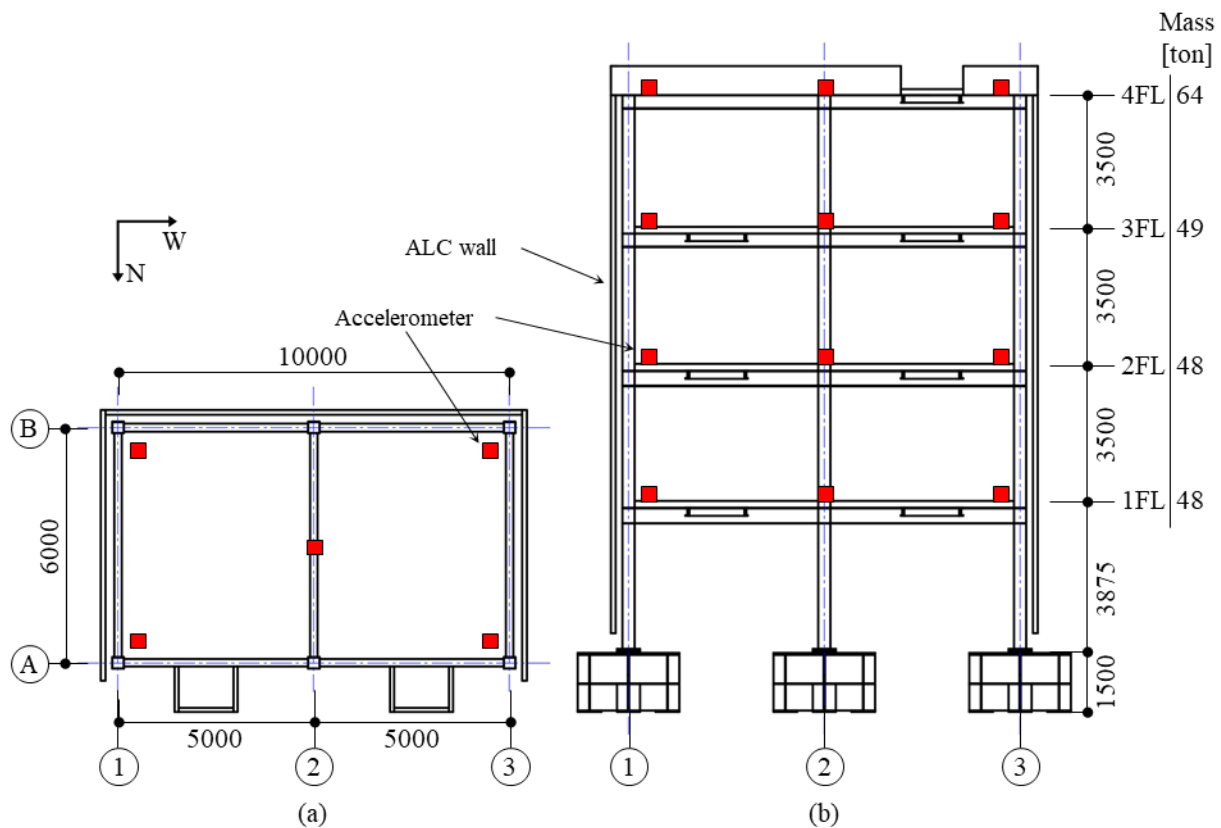


Figure 5.2 Specimen of the 07EDF test
(a) Plan view, (b) Elevation view

This test was conducted in 2007 at the NIED E-Defense facility. Its objective was to quantify the safety margin against collapse of low-rise steel MRFs. The specimen was a full-scale 2-by-1-bay 4-story steel MRF. Figure 5.2 illustrates the plan view and elevation view of the specimen. The JR Takatori record from the 1995 Kobe earthquake was used as ground motion input with increasing intensities, totaling 8 major cases, as summarized in Table 5.3. The skipped case numbers represent white noise and other small excitations. Triaxial input was used for all the excitations, with the NS component in the EW direction, EW component in the NS direction and UD component in the upward direction. Collapse at

the 1st story was triggered during the 100% excitation in the EW direction due to local buckling of the column bases. Refer to [2,76] for more information.

Table 5.3 Excitations in the 2007EDF test

Case #	Excitation	Intensity
07	JR Takatori	5%
11	JR Takatori	10%
12	JR Takatori	12.5%
13	JR Takatori	20%
15	JR Takatori	12.5%
16	JR Takatori	40%
17	JR Takatori	60%
21	JR Takatori	100%

5.2.4 2008 E-Defense high-rise steel MRF test (08EDF)

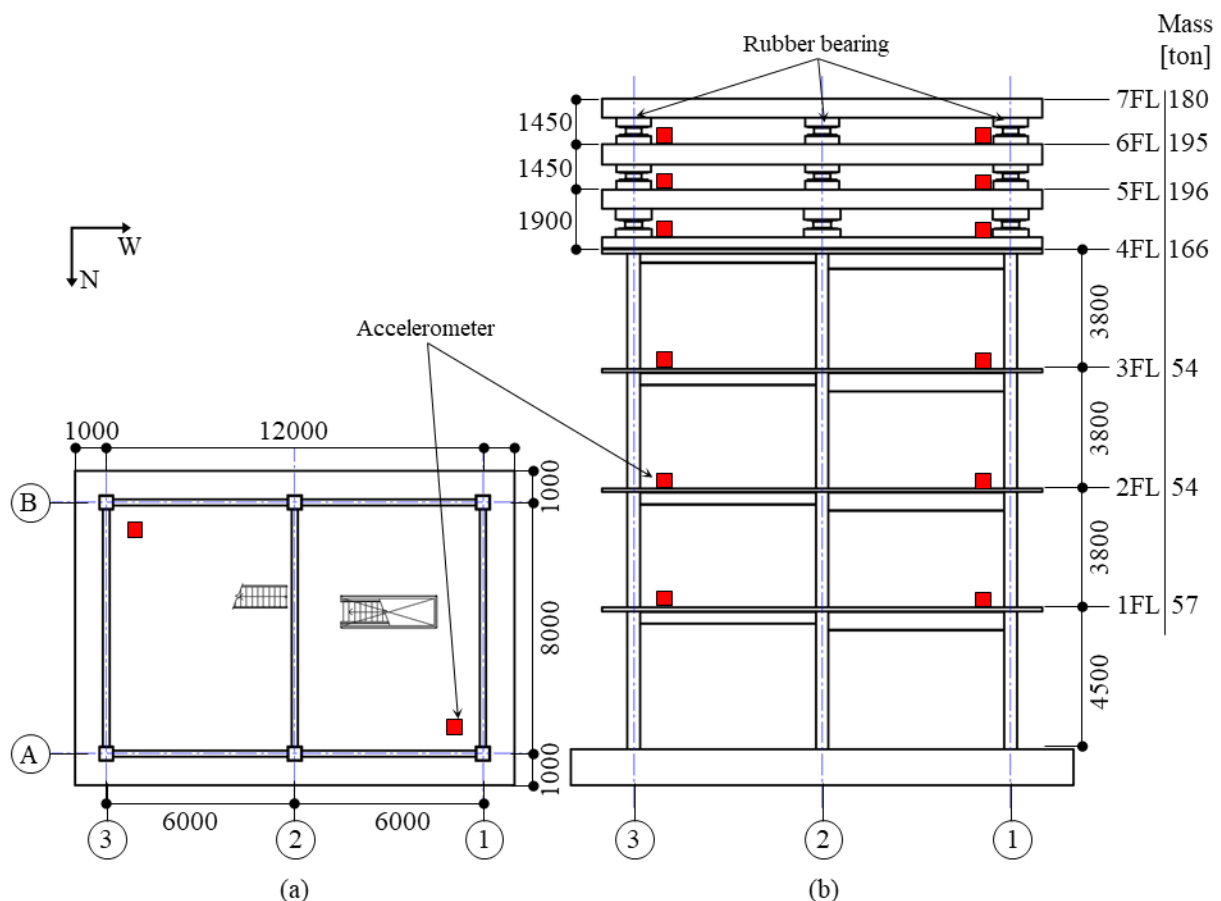


Figure 5.3 Specimen of the 08EDF test
(a) Plan view, (b) Elevation view

This test was conducted in 2008 at the NIED E-Defense facility. Its objective was to investigate the damage in high-rise steel MRFs under long period and long duration ground motion excitations. Figure 5.3 illustrates the plan view and elevation view of the specimen. The specimen was a full-scale 2-by-1-bay steel MRF. 3 levels of large masses were placed above the 4-story steel MRF, connected by rubber

bearings and steel dampers, to represent the 21-story archetype structure. Four ground motions were selected as input motion, with three of them being artificial long duration ground motions generated for specific site conditions. As summarized in Table 5.4, 13 major cases were conducted. The skipped case numbers represent white noise and other small excitations. One recorded ground motion and four artificial long duration ground motions generated for specific site conditions were used in the test. Biaxial excitation in the EW and NS directions were used in cases until 20, with cases 23 and 26 in the EW direction (longitudinal direction). Refer to [77,78] for more information.

Table 5.4 Excitations in the 08EDF test

Case #	Excitation	Intensity
04	Hog	30%
05	Hog	50%
06	Sun	20%
07	Sun	35%
08	El Centro	30%
09	El Centro	50%
12	Tok	60%
13	Tok	100%
14	El Centro	100%
17	Hog	100%
20	Sun	100%
23	Sun	100%
26	Sun	100%

5.2.5 2014 UCSD RC flat-slab rocking-wall test (2014UCSD)

This test was conducted in 2014 at the University of California, San Diego (UCSD) in US. Its objective was to investigate the influence of connections termed as IFAS (Inertial force-limiting Floor Anchorage System) between the flooring system and the lateral force resisting system. Figure 5.4 illustrates the plan view and elevation view of the specimen. The specimen was a scaled RC structure with a Flat-slab system as gravity load resisting system (GLRS) and a rocking-wall as the lateral force resisting system (LFRS). As listed in Table 5.5, 22 major excitations were conducted in this test, among which there were three structural systems, two ground motions, and three scale levels. The skipped case number represents white noise and other small excitations. The first structure system consisted of deformable connections and rocking walls, covered in cases until 24. The second system consisted of rigid connections and rocking walls, covered in cases 27 to 34. The third system consisted of rigid connections and conventional shear walls, covered in cases 327 to 40. Two ground motions selected for the Berkeley (BE) and Seattle (SE) sites were used, which are the 000-component in the Los Gatos Presentation Center record from the 1987 Loma Prieta earthquake and the 140-component in the El Centro Array #5 record from the 1979 Imperial Valley earthquake, respectively. These motions were scaled to different intensities corresponding to the Service level (SVC, 67%), Design Based Earthquake level (DBE, 100%), and Maximum Considered Earthquake level (MCE, 150%). Uniaxial excitation was applied for all the

cases in the EW direction. The specimen was designed to be eccentric so that connections in both directions can be activated under the uniaxial input. Refer to [49] for more information.

Table 5.5 Excitations in the 14UCSD test

Case #	Excitation	Intensity
02	Berkeley	67%
04	Berkeley	67%
05	Seattle	100%
06	Berkeley	100%
09	Berkeley	67%
10	Seattle	100%
11	Berkeley	100%
14	Berkeley	100%
15	Seattle	150%
16	Berkeley	150%
19	Berkeley	100%
20	Berkeley	150%
23	Berkeley	100%
24	Berkeley	150%
27	Seattle	100%
28	Berkeley	67%
29	Berkeley	100%
32	Berkeley	100%
34	Berkeley	150%
37	Berkeley	100%
39	Berkeley	150%
40	Berkeley	150%

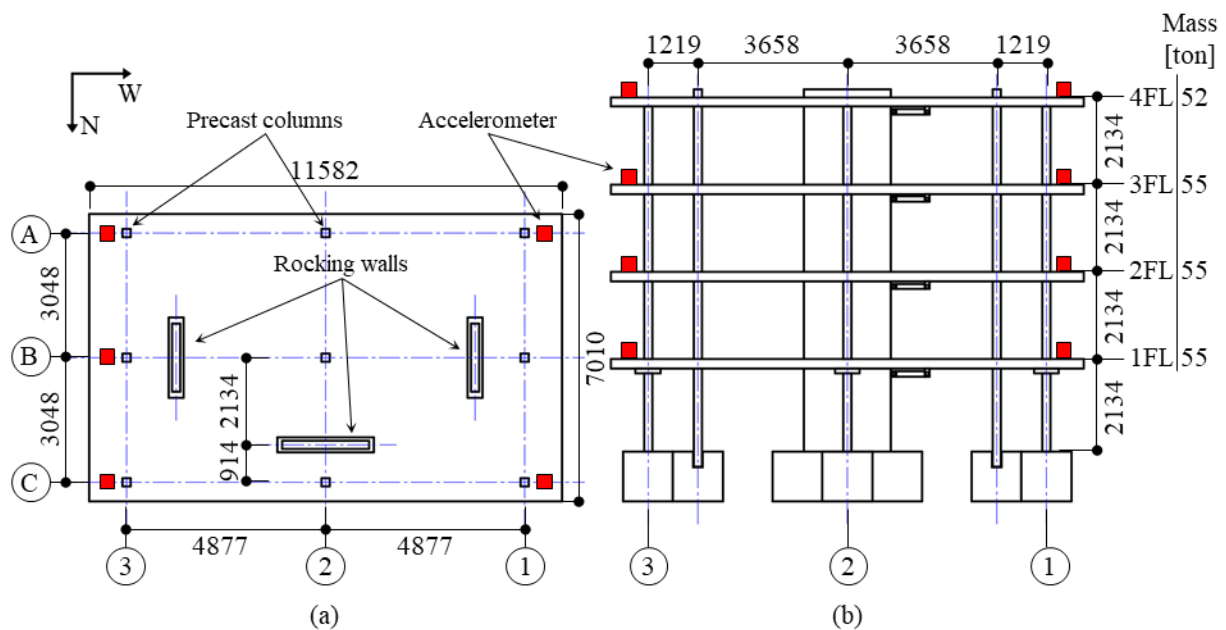


Figure 5.4 Specimen of the 14UCSD test
(a) Plan view, (b) Elevation view

5.3 DATA PREPARATION

5.3.1 General information

Data obtained from each shake table test needs to be prepared to a consistent state for further analysis due to their different initial conditions. Data preparation roughly includes data cleaning for all cases and data segmentation for ground motion excited cases. These procedures will get the raw data ready for analysis using the methods introduced in Section 5.4.

5.3.2 Data cleaning

In general, a three-step procedure was used to clean the acceleration data for all the selected tests. Note that data from the 20EDF test (the primary one) were also re-treated to be consistent with the other tests. However, minor differences were found compared to those treated in previous chapters. In addition to the general treatment for all tests, the 14UCSD test required some special treatment, as introduced in Section 5.3.3.

First, data from relevant sensors were plotted to check if there are obvious problems (out of sync, failure during the excitation, outliers, etc.). The corrupted data was removed or compensated using information from other sensors. Synchronization/trimming was needed only for some cases in the 14UCSD test.

Second, data from effective sensors on the same floor in the same direction were averaged to generate the floor response. The sensors were selected to be symmetric about the corresponding excitation directions so that the averaged value can represent response on the centerline of this direction, avoiding possible amplification due to rigid body motion of the floor diagram. The sampling frequencies of the floor responses of each test were unified to 200 Hz except for the 14UCSD test. For the 21TKB test, data were interpolated linearly.

Third, the floor responses were further treated. Outliers were removed and replaced by linear interpolation. The data were further processed through a Butterworth bandpass filter with consistent filter parameters. Bi-direction filtering was conducted to avoid phase distortion. It should be noted that, due to the bi-directional filtering, the real order applied to the data was 40.

Figure 5.5 illustrates some examples of data cleaning from the 08EDF test. Table 5.6 summarizes the filter parameters of all tests.

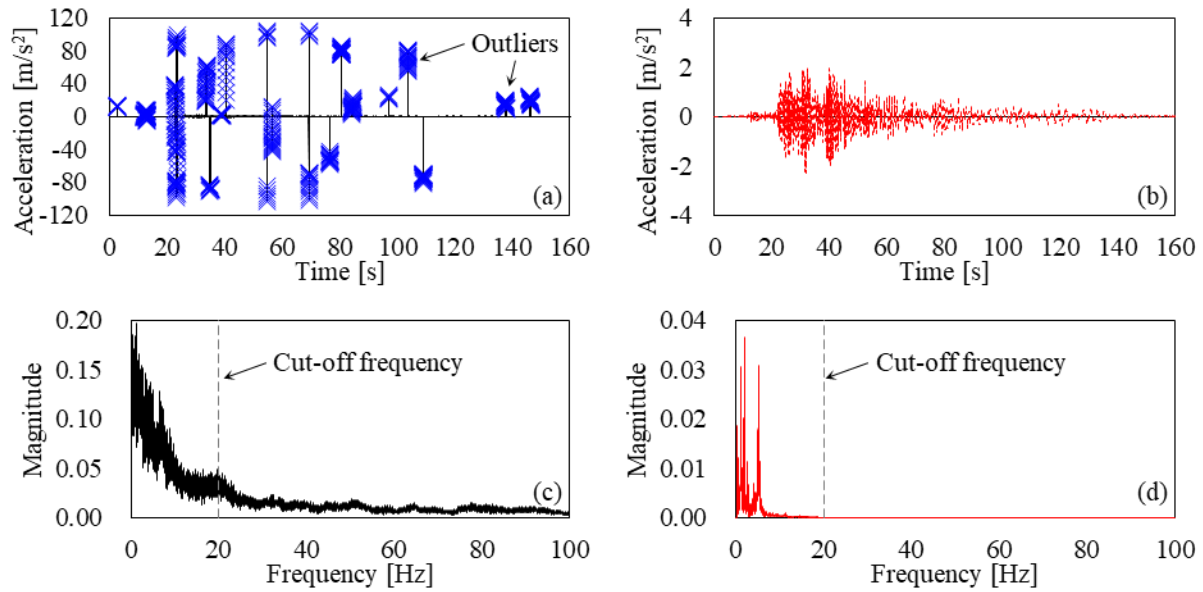


Figure 5.5 Examples of data cleaning (Floor 4, Case 12X, 08EDF)

(a) Original signal; (b) Cleaned signal; (c) Original Fourier spectrum; (d) Cleaned Fourier spectrum

Table 5.6 Filter parameters for each shake table test

Test name	Filter passband [Hz]		Filter order	Sampling frequency [Hz]	
	Low	High		Original	Treated
20EDF	0.2	20	20	200	200
21TKB	0.5	20	20	100	200
07EDF	0.2	20	20	200	200
08EDF	0.2	20	20	200	200
14UCSD	0.2	24	20	240	240

5.3.3 Special treatment for the 14UCSD test

In addition to the aforementioned treatment, data from the 14UCSD test required some special treatment:

1) synchronization and trimming of duplicated and incomplete sensor signals; 2) correction of baseline shift of sensor signals; 3) correction of floor rotation for calculated floor acceleration responses.

5.3.3.1 Duplicate and incomplete signals

In the raw data, some channels showed duplicate or incomplete signals due to unknown corruption of the recordings. Synchronization and trimming were applied to these signals to make them usable. This process was conducted to raw sensor signals before calculating floor responses. Figure 5.6 illustrates some examples of the treatment. It should be noted that such treatment may shorten the time domain signals, but the influence on modal parameter estimation was minor.

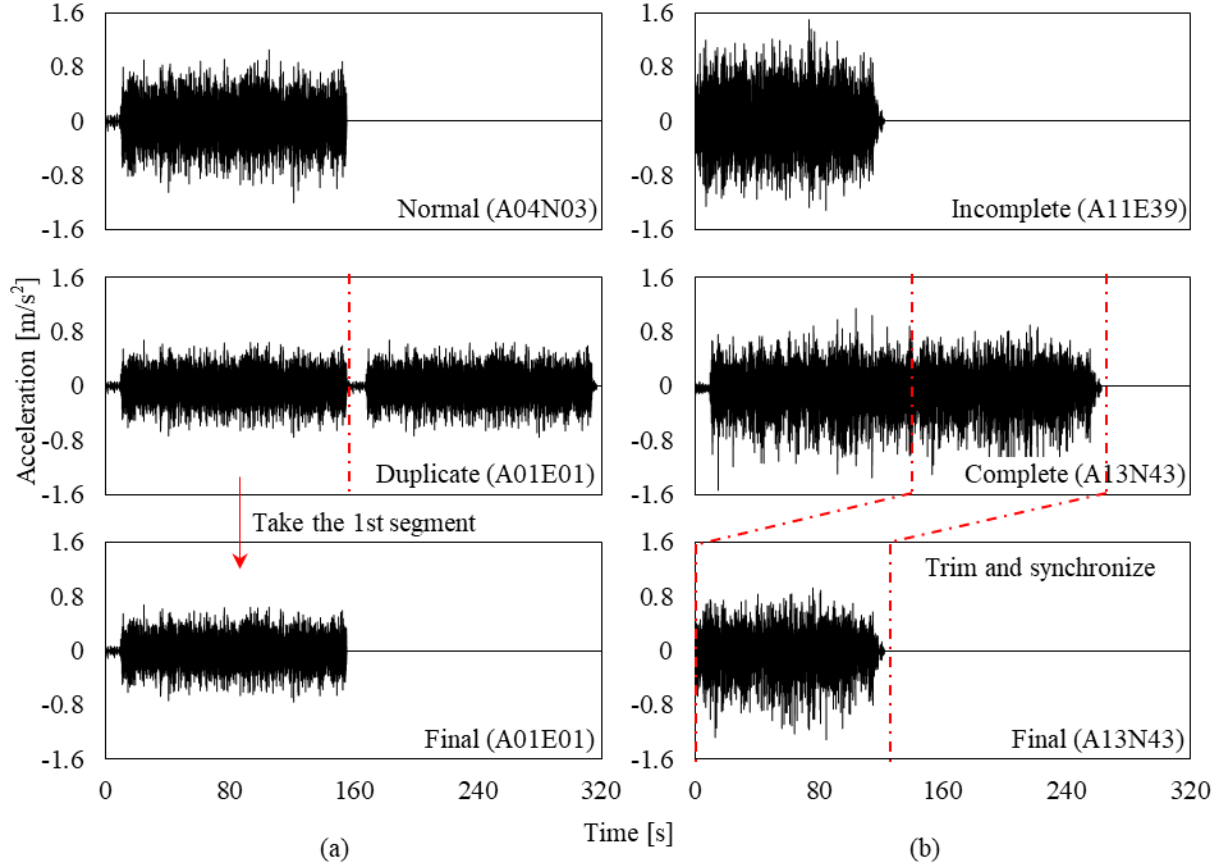


Figure 5.6 Examples of duplicate and incomplete signal treatment of the 2014UCSD test
 (a) Duplicate signal (Case 38); (b) Incomplete signal (Case 33)

5.3.3.2 Baseline shift

Some sensors were found to show baseline shift, i.e., non-zero median values which should not exist in acceleration signals. This phenomenon was attributed to the possible movement of the sensors during the test, which may include in the readings some component of the gravity acceleration. Such shift was considered unreal and needed to be shifted back. Figure 5.7 illustrates the calculation scheme and an example of the correction (Case 18, Channel A44E03).

As illustrated in Figure 5.7(a), when an accelerometer rotated by an angle θ , the horizontal and vertical readings of the accelerometer, $a_{sensor,h}$ and $a_{sensor,v}$, can be calculated by the gravity and horizontal acceleration values, a_g and a_h , using the following equations:

$$a_{sensor,h} = a_g \sin \theta + a_h \cos \theta \quad (5.1)$$

$$a_{sensor,v} = -a_g \cos \theta + a_h \sin \theta - (-a_g) \quad (5.2)$$

In the illustrated condition, the gravity acceleration a_g will give the horizontal and vertical readings, $a_{sensor,h}$ and $a_{sensor,v}$, a baseline shift. Taking the median value of the readings as the baseline value, there are

$$a_{sensor,h,median} = a_g \sin \theta \quad (5.3)$$

$$a_{sensor,v,median} = a_g(1 - \cos \theta) \quad (5.4)$$

Considering that the gravity acceleration a_g is known, the rotation angle θ can then be calculated using either Eq. (5.3) or (5.4). Further, substituting the value of θ and a_g into Eq. (5.1) or (5.2), with the known sensor readings $a_{sensor,h}$ and $a_{sensor,v}$, the corrected horizontal acceleration a_h can be calculated. It should be noted that, although theoretically the θ value calculated using Eq. (5.3) or (5.4) should be identical, this may not be the case for real data. This inconsistency also applied to the a_h calculation using Eq. (5.1) or (5.2). For the processing conducted here, only the horizontal signals were treated using Eq. (5.1) and (5.3).

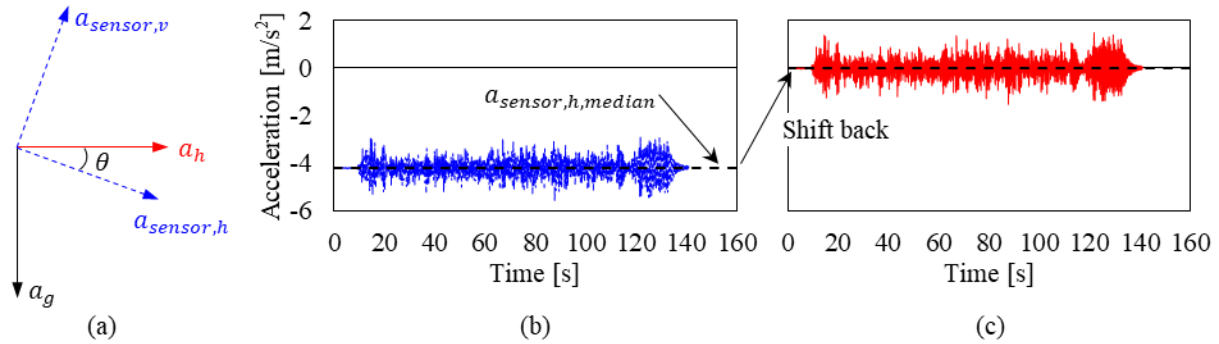


Figure 5.7 Scheme of the baseline shift correction

(a) Calculation scheme; (b) Original horizontal acceleration; (c) Corrected horizontal acceleration

5.3.3.3 Floor rotation correction

Due to the large floor rotation during the test, the axes of the floors no longer coincided with the shake table or each other. To extract the modal parameters in the same direction, the measured acceleration signals at each floor were rotated according to the floor rotation angles to be consistent with the shake table axes. The floor rotation angles were calculated from floor displacement measurement taken at two points per floor. Rotation was only conducted on the floor responses calculated from relevant accelerometers. No rotation was applied to single-sensor signals. Figure 5.8 illustrates the calculation scheme and an example of the correction (Case 10, Floor 4, East direction).

As illustrated in Figure 5.8(a), when the floor had a rotation angle of θ , the acceleration in the table axes, a_E and a_N , can be calculated using the accelerometer readings in the floor axes, $a_{sensor,E}$ and $a_{sensor,N}$, using the following rotation transform:

$$a_E = a_{sensor,E} \cos \theta + a_{sensor,N} \sin \theta \quad (5.5)$$

$$a_N = a_{sensor,N} \cos \theta - a_{sensor,E} \sin \theta \quad (5.6)$$

Where the rotation angle θ was calculated using floor displacement measurement.

From the example shown in Figure 5.8, it is seen that the difference between the original and rotated signals was minor. This also applied to other cases.

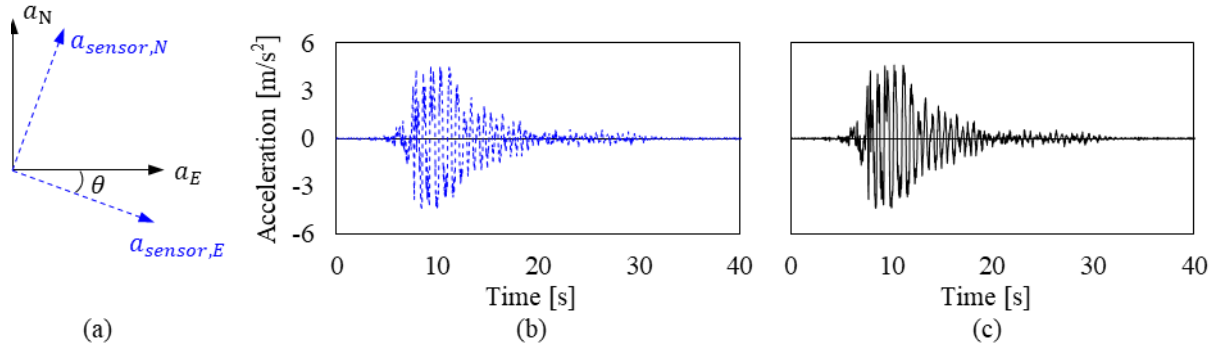


Figure 5.8 Scheme of the floor rotation correction

(a) Calculation scheme; (b) Original East acceleration; (c) Corrected East acceleration

5.3.4 Data Segmentation

For the white noise cases, since the structural systems were considered to be linear time invariant, the whole acceleration signals were used. For the ground motion excitations, the systems were no longer time invariant and the methods except for SSWT and EPA cannot be applied directly. Thus, segmentation was applied to the acceleration signals in these excitations to obtain time variant modal parameters, although with quite limited time resolution. The segmentation used a two-stage method. First, the signal was divided into Time Invariant (TI) and Time Variant (TV) segments, and second, the TV segment was further divided into shorter segments.

The Autoregressive with Exogenous Variables (ARX) method [57] and the Adaptive Forgetting through Multiple Models (AFMM) method [79] were used to identify the TI and TV segments [80] (using the built-in function *segment* of Matlab®). The automatically identified TI-TV-TI segments were then manually checked and adjusted to deal with possible issues such as inconsistent segments within the same case at different floors, obvious wrong segment, and so on. The finally determined TI segments were left unchanged and the TV segment was further processed in the second stage.

In the second stage, the TV segment was further divided manually as following:

- The segment length should be neither too short to ensure reasonable frequency resolution nor too long to ensure reasonable time resolution. The minimum length of each segment was determined as three times the longest 1st natural period identified from the while noise cases. The longest length was determined by the number of sub-segments in the TV segment through trial and error.
- The PGA (Peak Ground Acceleration) point should be placed at the center of a segment.
- The segments should not overlap to exclude information from other segments.

- A certain window function should be applied to each segment to avoid end point problems. A Tukey window was applied to all segments including the TI segments.
- For the same ground motion in the same shake table test, the number of segments should be identical. This can ease the comparison and analysis of the results.

Table 5.7 summarizes the information of the TV segments of each test. Two TI segments were at the beginning and end of the signal, respectively. Figure 5.9 illustrates an example of data segmentation. The segmentation was applied to all methods used in the analysis.

Table 5.7 Information of TV segments

Test	Ground motion	# of data	# of segments	Sampling Freq. [Hz]	Freq. resolution [Hz]
20EDF	Northridge	800	5	200	0.25
	JMA Kobe	800	5	200	0.25
21TKB	JMA Kobe	800	6	200	0.25
07EDF	Takatori	1000	7	200	0.20
08EDF	El Centro	1250	6	200	0.16
	Hog	2500	9	200	0.08
	Tok	2500	11	200	0.08
	Sun	2500	10	200	0.08
14UCSD	Berkeley	800	5	240	0.30
	Seattle	800	5	240	0.30

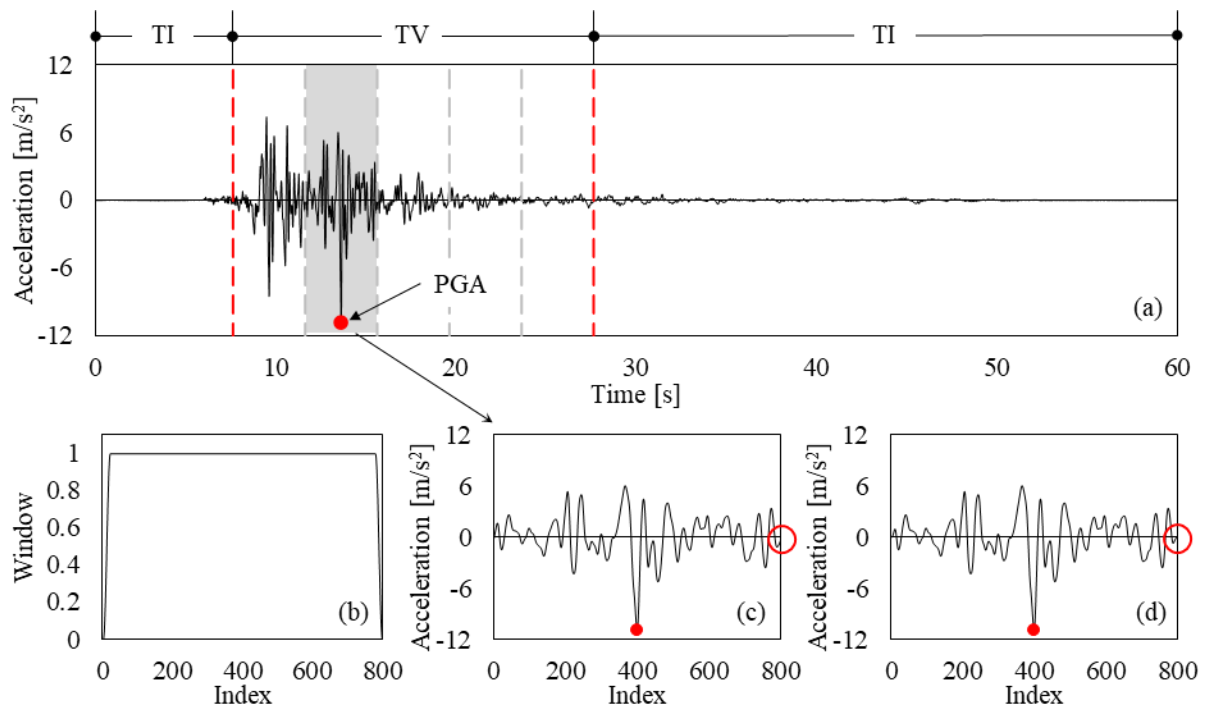


Figure 5.9 Example of data segmentation

- (a) Segmentation of the ground motion (Northridge VA); (b) Tukey window applied to each segment; (c) TV segment without windowing; (d) TV segment windowed

5.4 ANALYSIS METHODS

5.4.1 General information

Table 5.8 summarizes methods used for analysis. All methods used acceleration data prepared as aforementioned. This section introduces details of all the analysis methods. Considering the purpose of this dissertation, instead of introducing the detailed theory of the methods, the introduction in this section will focus more on the practical procedures used in the actual work. Readers can refer to the original papers cited in the following sections if interested in the theoretical background of these methods.

Table 5.8 Analysis methods used in the work

Acronym	Full name	Data structure	Working domain
FDD	Frequency Domain Decomposition	Multiple-Output-only	Frequency
ARX	Auto-Regressive with eXtra input	Single-Input-Multiple-Output	Time
SS	State Space	Single-Input-Multiple-Output	Time
SSWT	Synchro-Squeezed Wavelet Transform	Single-Output-only	Time-frequency
EMPA	Effective Modal Pseudo-Acceleration	Multiple-Output-only	Response spectra

The FDD, ARX and SS methods are modal analysis methods widely used in structural health monitoring and other system identification work. The FDD method works in frequency domain and requires Multiple-Output data. The ARX and SS methods work in time domain and require Single-Input-Multiple-Output data. The SSWT method is a wavelet-based time-frequency analysis method requiring Single-Output (SO) data. The EMPA method is a response spectra method requiring Multiple-Output (MO) data. Due to the theoretical basis of the methods and condition of the analyzed data, none of these methods alone can provide full characterization of the vibration properties of the analyzed systems. Thus, results from these methods will be jointly utilized to obtain a comprehensive understanding of the system properties.

5.4.2 Frequency Domain Decomposition (FDD) method

5.4.2.1 Theory

The Frequency Domain Decomposition (FDD) method [53] is a frequency domain output-only modal analysis method. The original FDD method can only extract modal frequencies and mode shapes. A second generation of the method named Enhanced Frequency Domain Decomposition (EFDD) method [54] can further estimate the modal damping ratio by decomposing each mode in frequency domain and inverse-transforming these individual modes back to time domain, getting the Auto-Correlation Function (ACF). The isolation of modes is done using the Modal Assurance Criterion (MAC) [81]. A third generation of FDD methods was proposed by Zhang et al [82] and further developed by Brincker et al [56], using the so-called “modal filtering” (MF) technique, which is essentially a projection, to decompose each mode in frequency domain to avoid some problems in the EFDD method. Considering

the above condition of the FDD series of methods, the work reported here uses the FDD+MF identification algorithm. This series of methods are based on the assumption of linear invariant system with white noise or ambient excitation. If these criteria are not satisfied, the results will be approximate, for example as introduced in [83].

Same as the other frequency domain methods, the FDD method also starts from the frequency domain relationship between the unknown excitation $x(t)$ and the responses $y(t)$

$$\mathbf{G}_{yy}(\omega) = \mathbf{H}(\omega)\mathbf{G}_{xx}(\omega)\mathbf{H}(\omega)^H \quad (5.7)$$

Where:

ω is the angular frequency.

$\mathbf{G}_{xx}(\omega)$ is the PSD matrix of the excitation.

$\mathbf{G}_{yy}(\omega)$ is the PSD matrix of the responses of the system.

$\mathbf{H}(\omega)$ is the Frequency Response Function (FRF) of the system. The superscript H denotes Hermitian transpose.

For frequency domain methods, modal parameters are included in the FRF $\mathbf{H}(\omega)$. If the excitation and responses are all known, the FRF is determined, and the modal parameters can then be extracted. If the excitation is unknown, the assumption of flat excitation (white noise or ambient) must be adopted, which ensures that $\mathbf{G}_{xx}(\omega)$ is a constant function of ω , and thus the estimation of modal parameters relies only on the response PSD $\mathbf{G}_{yy}(\omega)$. According to the FDD method, the Singular Value Decomposition (SVD) of $\mathbf{G}_{yy}(\omega)$ can give the modal parameters of the system.

Figure 5.10 illustrates the flow chart of the identification algorithm applied in the work reported here, which is roughly a three-phase procedure. Details will be introduced afterwards.

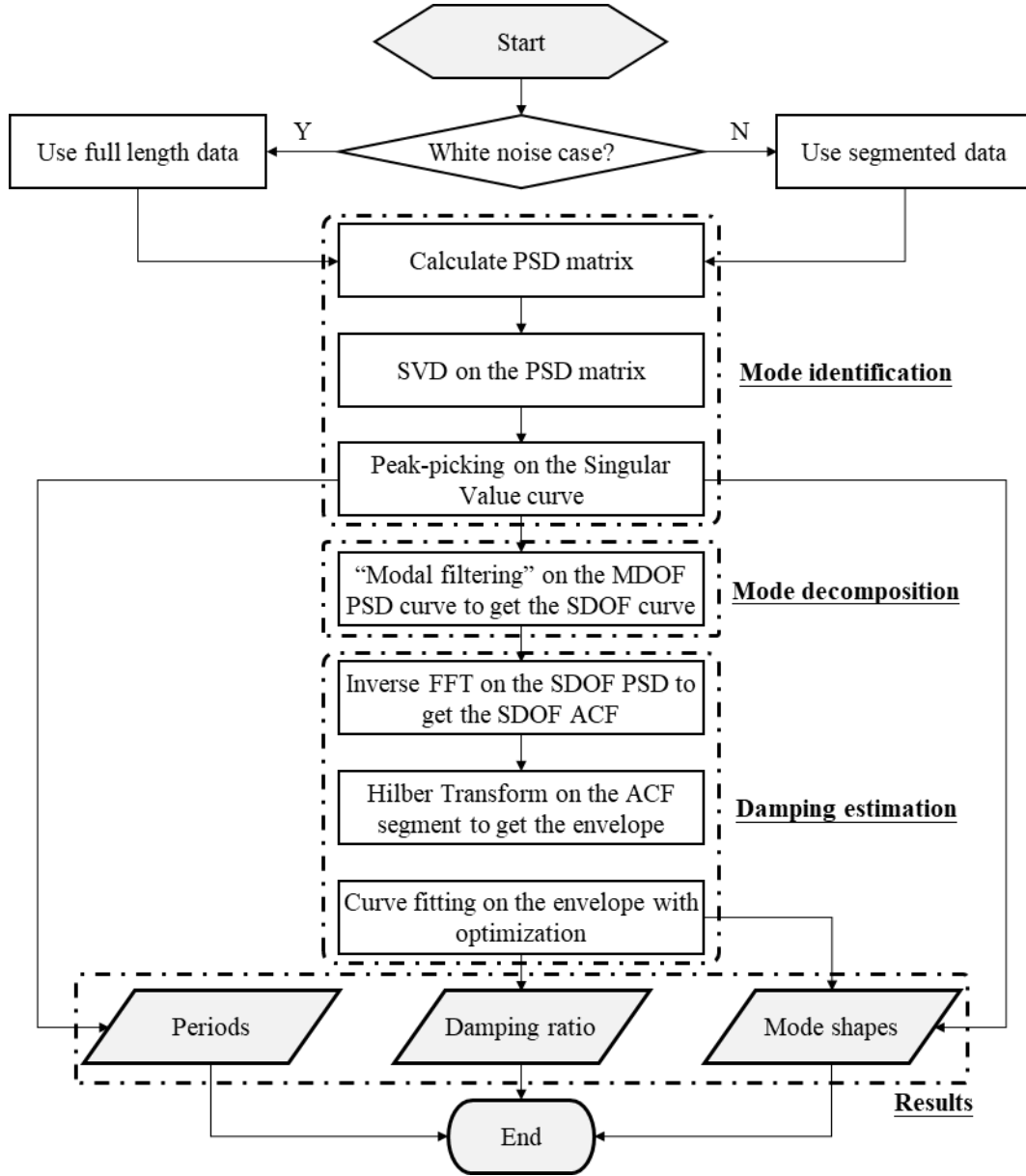


Figure 5.10 Flowchart of the FDD algorithm

5.4.2.2 Mode identification

In the first phase, the FDD algorithm was used to identify different modes of the MDOF system.

First, the PSD matrix of the responses, $\mathbf{G}_{yy}$, was estimated. $\mathbf{G}_{yy}$ is a 3-D matrix with $m \times m \times n$ entries, where m is the number of response signals and n is the number of discrete frequencies. The Welch's method [84] was used to estimate the auto-PSDs (APSD) of each signal and cross-PSDs (CPSD) between any two of them, which, at each ω_k , were the entries of $\mathbf{G}_{yy}(\omega_k)$, as shown in Eq. 5.8

$$\mathbf{G}_{yy}(\omega_k) = \begin{bmatrix} \text{APSD}_{11}(\omega_k) & \text{CPSD}_{12}(\omega_k) & \cdots & \text{CPSD}_{1m}(\omega_k) \\ \text{CPSD}_{21}(\omega_k) & \text{APSD}_{22}(\omega_k) & \cdots & \text{CPSD}_{2m}(\omega_k) \\ \vdots & \vdots & \ddots & \vdots \\ \text{CPSD}_{m1}(\omega_k) & \text{CPSD}_{m2}(\omega_k) & \cdots & \text{APSD}_{mm}(\omega_k) \end{bmatrix} \quad (5.8)$$

Subsequently, Singular Value Decomposition (SVD) was applied at each discrete frequency ω_k , as illustrated in Eq. (5.9).

$$\mathbf{G}_{yy}(\omega_k) = \mathbf{U}(\omega_k)\mathbf{S}(\omega_k)\mathbf{V}(\omega_k)^H \quad (5.9)$$

Where,

$\mathbf{U}(\omega_k)$ and $\mathbf{V}(\omega_k)$ are unitary matrices containing the left and right singular vectors.

$\mathbf{S}(\omega_k)$ is a real diagonal matrix containing all the singular values.

Theoretically, if a certain frequency ω_{md} is close to an eigenfrequency, the 1st column vector of $\mathbf{U}(\omega_{md}) = [\mathbf{u}_1(\omega_{md}), \mathbf{u}_2(\omega_{md}), \dots]$ is an estimate of the mode shape vector, while the corresponding singular value $S_{11}(\omega_{md})$ is the estimate of PSD value at the corresponding PSD peak.

In the work reported here, the eigenfrequencies were selected by plotting the 1st singular values at all frequency points (representing the PSD of a MDOF system) and picking the peaks on the MDOF PSD. Mode shape vectors can then be obtained by simply taking the corresponding columns from the left singular matrix $\mathbf{U}$. Figure 5.11(a) illustrates an example of the plotted MDOF PSD and the selected peaks.

Theoretically, for building structures which have no rigid body motion, if the damping is proportional and there is no noise, the identified mode shapes should be real values. However, this is not the case for real world data and the identified mode shape vectors may contain complex values. Conversion from complex modes to real modes can take various ways. For the work reported here, the real modes were obtained by simply taking the real part of the complex values.

$$\phi_{md} = \text{real}(\mathbf{u}_1(\omega_{md})) \quad (5.10)$$

5.4.2.3 Mode decomposition

In the second phase, the MDOF PSD needs to be decomposed into several SDOF curves. As aforementioned, the decomposition can be realized using different methods, such as using the MAC value as indicators to select segments with mode shapes similar enough to the mode shape vector (like in the EFDD method), or even simply by visual inspection and manual selection. The modal filtering method used in the work reported here was selected because it can take out the SDOF segments automatically without manually determining the threshold of MAC value or selection of curve segments. This section introduces the basic ideas of the algorithm. Refer to [56] for more details.

Since $\mathbf{G}_{yy}(\omega_k)$ is Hermitian and positive definite, matrices $\mathbf{U}$ and $\mathbf{V}$ are equal, and the singular value decomposition can be rewritten as

$$\mathbf{G}_{yy}(\omega_k) = \mathbf{U}(\omega_k)\mathbf{S}(\omega_k)\mathbf{U}(\omega_k)^H \quad (5.11)$$

If deeming all column vectors of $\mathbf{U}(\omega_k)$ as “potential mode shape vectors” and noting $\mathbf{S}(\omega_k)$ contains corresponding modal coordinates, Eq. (5.11) can be explained as modal decomposition based on squared mode shape vectors. Considering that $\mathbf{G}_{yy}(\omega_k)$, PSD of a system, represents square of the frequency spectrum of the system, the system response in frequency domain can therefore be represented as in Eq. (5.12).

$$\mathbf{r}_1(\omega_k) = \mathbf{U}(\omega_k)\mathbf{s}(\omega_k) \quad (5.12)$$

Where,

$\mathbf{s}(\omega_k) = \text{diag}(\mathbf{S}(\omega_k))$ is a vector containing all the singular values.

The system response can also be expressed using the “real” mode shapes vectors as in Eq. (5.13).

$$\mathbf{r}_2(\omega_k) = \Phi_B \mathbf{s}_B(\omega_k) \quad (5.13)$$

Where,

$\Phi_B = [\phi_1, \phi_2, \dots, \phi_p]$ is the mode shape matrix of all identified modes.

$\mathbf{s}_B(\omega_k)$ is the corresponding singular value, also, the modal coordinates corresponding to each mode.

Assuming that the system response can be represented by the identified modes completely, $\mathbf{r}_1 = \mathbf{r}_2$, and therefore, Eq. (5.14) holds true.

$$\mathbf{U}(\omega_k)\mathbf{s}(\omega_k) = \Phi_B \mathbf{s}_B(\omega_k) \quad (5.14)$$

Left-multiplying the inverse of Φ_B to both sides of Eq. (5.14), the singular values, or modal coordinates, of the identified modes can be obtained

$$\mathbf{s}_B(\omega_k) = \Phi_B^{-1} \mathbf{U}(\omega_k)\mathbf{s}(\omega_k) \quad (5.15)$$

The $\mathbf{s}_B(\omega_k)$ obtained from the above equation is a vector with length of number of modes to be decomposed, containing PSD values of SDOF systems equivalent to each identified mode, with each row for an identified mode. $\mathbf{s}_B(\omega_k)$ has significant values only in limited frequency band B, which may vary for different modes.

If all modes are identified, i.e., the mode shape matrix Φ_B is full rank, the real inverse of Φ_B can be used for calculation. If not, its pseudo inverse such as the Moore-Penrose pseudo inverse can be used. If some modes could not be identified, further treatment is needed on the SDOF PSDs. Figure 5.11(b) shows the four decomposed SDOF PSD curves with the corresponding significant frequency band.

5.4.2.4 Damping estimation

In the third phase, damping estimation was conducted using the previously obtained SDOF PSDs. This can be done either in frequency domain using the Half-Power Band Width (HPBW) method or in time

domain using Auto-Correlation Function (ACF) curve fitting. Considering the known problems of the HPBW method such as being sensitive to smoothing of the PSD, and not accurate for lower modes, the ACF curve fitting method was used in the work reported here.

In this method, the SDOF PSDs were inverse transformed from frequency domain to time domain, getting the SDOF ACF of each mode. The ACF is a decaying signal with the same damping properties as the original signal. The Hilbert transform was then applied to the ACF to get its envelope curve. Using this curve, the damping ratio of the corresponding mode can be obtained by curve fitting, as expressed in Eq. (5.16).

$$y = ae^{-\zeta\omega_n t} \quad (5.16)$$

Where,

ζ is the modal damping ratio.

ω_n is the circular frequency of the current mode obtained from the PSD peak picking.

In order to have a better damping estimation result, an optimization algorithm similar to [83] was conducted. This optimization aims to find a segment on the ACF envelope closest in shape to the theoretical exponential curve. The start and end points of the envelope segment were optimized by minimizing the Mean Square Error of the curve fitting. The optimization also required a minimum length of the curve specified as a certain number of peaks and valleys, which were determined through trial and error.

Figure 5.11(c) shows an example of the generated 1st mode ACF, its envelope, the fitted curve, and the optimized fitting range.

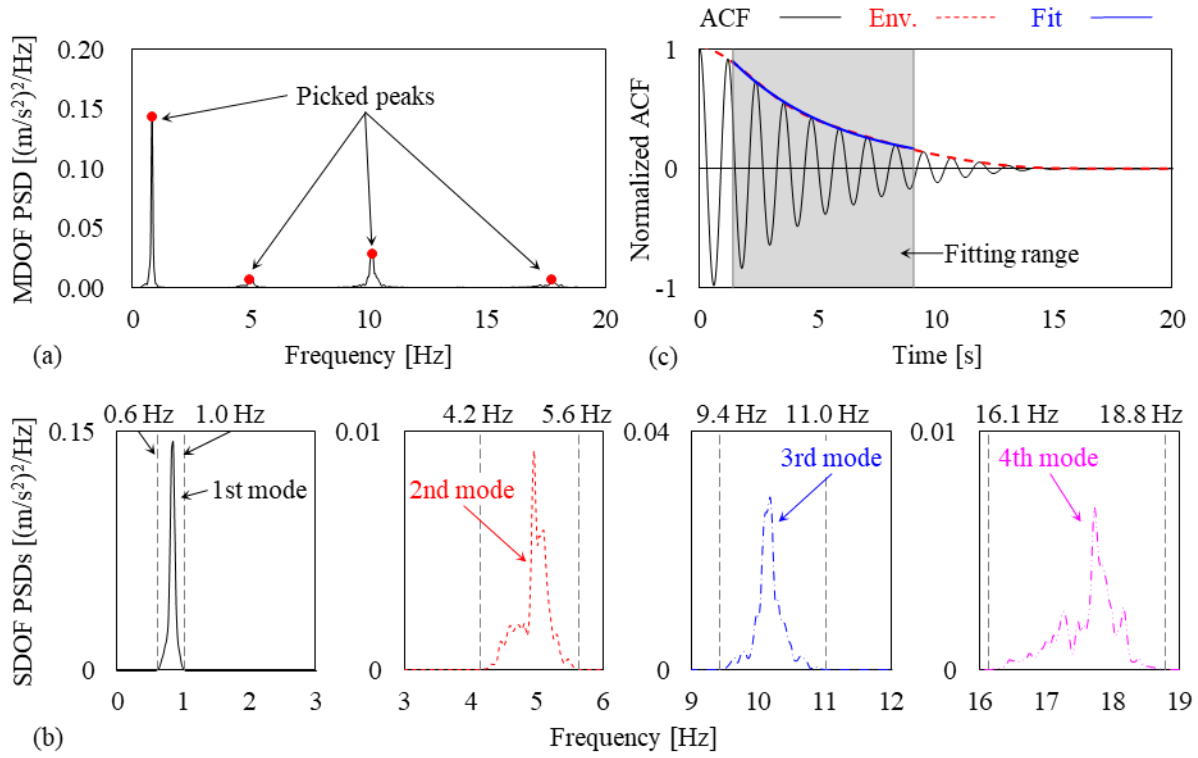


Figure 5.11 Example results of the EFDD algorithm (20EDF, MRF-Spine, white noise)
 (a) MDOF PSD; (b) isolated SDOF PSD; (c) ACF and curve fitting of the 1st mode.

5.4.3 Auto-Regressive with eXtra input (ARX) method

5.4.3.1 Theory

The Auto-Regressive with eXtra input (ARX) method is a time domain input-output method which can estimate a linear system and further deduce its modal parameters. The ARX method is one of the widely used Auto-Regressive (AR) series of methods. This series of methods has been developed and utilized in system identification for decades. Multiple variations have been developed and used for different purposes, such as Auto-Regressive (AR), Auto-Regressive with eXtra input (ARX), Auto-Regressive Moving Average (ARMA), Auto-Regressive Moving Average with eXtra input (ARMAX), and many.

In general, the ARX method uses an auto-regressive polynomial model to model the system responses, i.e., assume that the system responses at a certain time instance are determined by responses at a given number of previous time instances and a given number of excitation steps, while the noise is assumed to be white noise. The common form of the ARX model can be written as follows:

$$\sum_{i=0}^n a_i \mathbf{y}(t-i) = \sum_{j=1}^m b_j \mathbf{x}(t-j) + \mathbf{e}(t) \quad (5.17)$$

Where,

$\mathbf{x}(t)$ and $\mathbf{y}(t)$ are the input and output of the system, which are vectors for single input/output and matrices for multiple inputs/output.

a_i and b_j are the model parameters for the auto-regressive and extra input parts, respectively, which are scalars for single input/output and matrices for multiple inputs/outputs.

m and n are the order of the model for input and output, respectively.

$e(t)$ is the residual error term, usually taken as white noise.

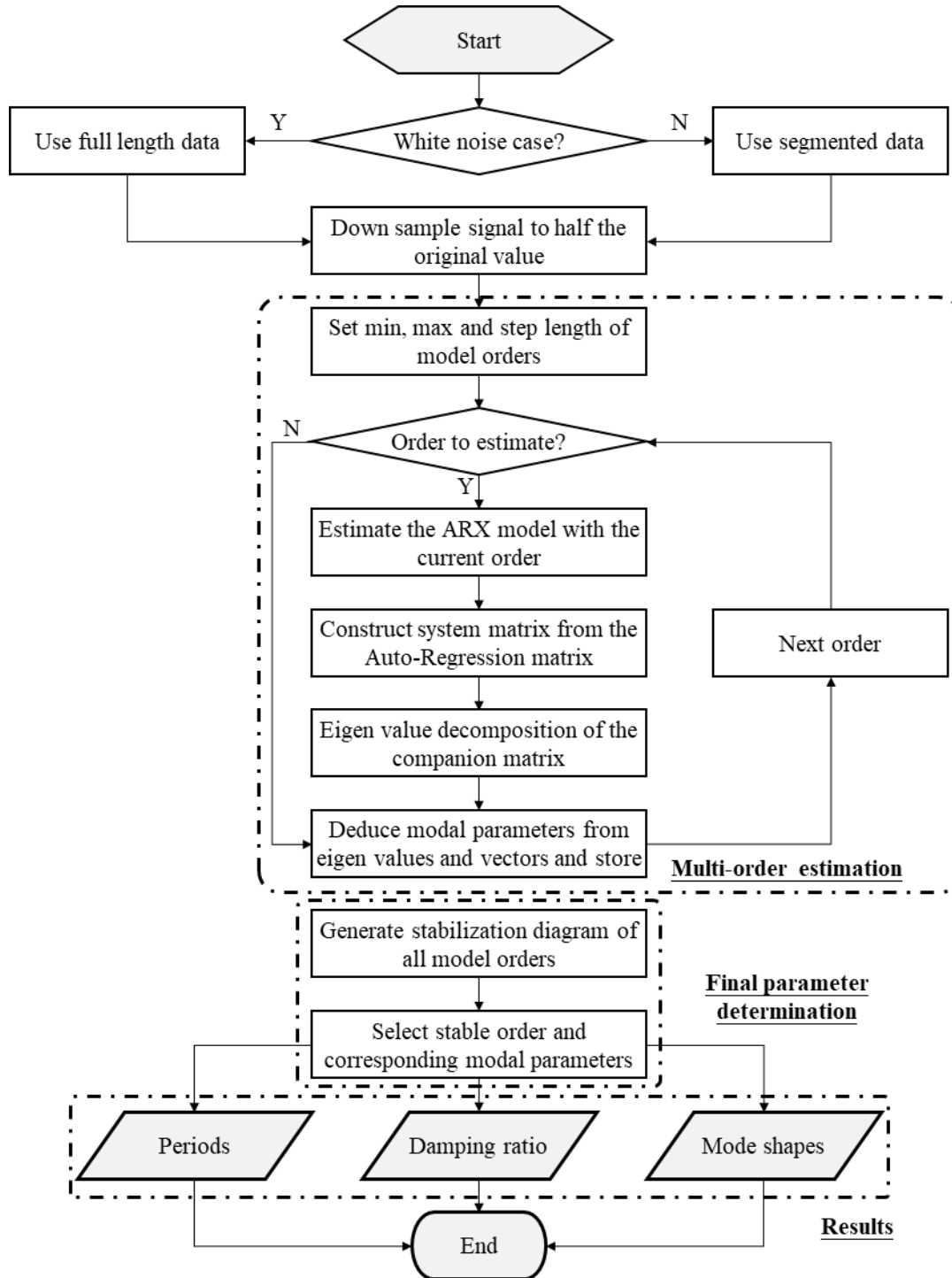


Figure 5.12 Flowchart of the ARX algorithm

Using the ARX method, modal parameters can be estimated directly from the model without transferring between time and frequency domain as in the FDD method. More specifically, once the a_i and b_j

parameters (either scalar or matrices) are determined, the modal parameters are easily deduced. One issue of the ARX method is the determination of the model order. For the application in the work reported here, the stabilization diagram was used to find the stable model order and the corresponding parameters. Details of the ARX method can be found in many of the literatures about system identification and modal analysis, for example in [57]. Figure 5.12 illustrates the flow chart of the ARX identification algorithm used in the current work.

5.4.3.2 Multi-order ARX model estimation

The estimation used multiple ARX models with different orders. The minimum and maximum orders were determined through trial and error considering the balance between computation cost and pole numbers. After the series of orders were determined, the down-sampled signals, either full length or segmented, were sent to each order of model and estimated independently. The down-sampling used a ratio of 0.5, which showed a better result than the original high sampling frequency. A possible explanation for this lower sampling frequency is that this may concentrate more energy on the lower frequency range where most of the modes lay.

The core procedure is the estimation of a single model with a given order. Theoretically, as shown in Eq. (5.17), the order of the input and output can be different. For the application here, they were taken as the same. Rewrite Eq. (5.17) for a Single-Input-Multiple-Output (SIMO) system, assuming the model order is M and there is one input and p outputs, there is

$$\mathbf{y}(t) + \sum_{i=1}^M \mathbf{A}_i \mathbf{y}(t-i) = \sum_{j=1}^M \mathbf{B}_j \mathbf{x}(t-j) + \mathbf{e}(t) \quad (5.18)$$

Where,

$\mathbf{x}(t)$ is the input of the system, which is a $1 \times M$ row vector.

$\mathbf{y}(t) = [\mathbf{y}_1; \mathbf{y}_2; \cdots; \mathbf{y}_p]$ is the output of the system, which is a $p \times M$ matrix, with each row for one output.

$\mathbf{A}_i$ is a $p \times p$ matrix containing all the Auto-Regressive (AR) parameters.

$\mathbf{B}_j$ is a $p \times 1$ column vector containing all the eXtra input parameters.

$\mathbf{e}(t)$ is the residual error term, usually taken as white noise.

Next, construct the system matrix using $\mathbf{A}_i (i = 1 \sim M)$, there is

$$\mathbf{A}_{sys} = \begin{bmatrix} -\mathbf{A}_1 & \cdots & -\mathbf{A}_{M-1} & -\mathbf{A}_M \\ \mathbf{I} & \cdots & \mathbf{0} & \mathbf{0} \\ \vdots & \ddots & \vdots & \vdots \\ \mathbf{0} & \cdots & \mathbf{I} & \mathbf{0} \end{bmatrix} \quad (5.19)$$

Where,

$\mathbf{A}_{sys}$ is a square matrix with both dimensions of pM .

$\mathbf{A}_i$ are the AR factors, $i = 1 \sim M$.

$\mathbf{I}$ is an identity matrix with the dimension of $p \times p$.

$\mathbf{0}$ is a zero matrix with the dimension of $p \times p$.

It should be noted that the arrangement of the submatrices in the system matrix may differ. But the different forms are equivalent only that the extracted parameters correspond to different parts of the matrix. Estimation of the ARX model were realized using the ARX function of Matlab®, which used least square method for the estimation.

Further, conduct eigen value decomposition on the system matrix $\mathbf{A}_{sys}$, getting the eigen vector matrix $\mathbf{V}$ and the eigen value matrix $\mathbf{\Lambda}$:

$$\mathbf{A}_{sys} = \mathbf{V}\mathbf{\Lambda}\mathbf{V}^{-1} \quad (5.20)$$

Where,

$$\mathbf{V} = \begin{bmatrix} v_1 \lambda_1^{M-1} & v_2 \lambda_2^{M-1} & \cdots & v_{pM} \lambda_{pM}^{M-1} \\ v_1 \lambda_1^{M-2} & v_2 \lambda_2^{M-2} & \cdots & v_{pM} \lambda_{pM}^{M-2} \\ \vdots & \vdots & \ddots & \vdots \\ v_1 & v_2 & \cdots & v_{pM} \end{bmatrix} \quad (5.21)$$

$$\mathbf{\Lambda} = \begin{bmatrix} \lambda_1 & 0 & \cdots & 0 \\ 0 & \lambda_2 & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & \lambda_{pM} \end{bmatrix} \quad (5.22)$$

The mode shape vectors are the bottom p rows of $\mathbf{V}$.

Noticing the relationship of continuous-time system frequencies and damping ratios and the corresponding eigen values, there is

$$\mu_i = \ln(\lambda_i) = -\omega_i \zeta_i \pm j\omega_i \sqrt{1 - \zeta_i^2} \quad (5.23)$$

Where,

λ_i is the eigen value in discrete-time system.

μ_i is the eigen value in continuous-time system.

ω_i is the circular frequency in continuous-time system.

ζ_i is the damping ratio in continuous-time system.

From Eq. (5.23), natural frequencies and damping ratios of all modes (regardless of real modes or not) can be deduced as follows:

$$f_i = \frac{\omega_i}{2\pi} = \frac{\text{abs}(\mu_i)}{2\pi} \quad (5.24)$$

$$\zeta_i = \frac{\text{real}(\mu_i)}{\text{abs}(\mu_i)} \quad (5.25)$$

In the above procedure, totally pM modes were identified, which is usually larger than the number of DOFs and contains spurious computational modes. Following the tradition of system identification, these “modes” are termed as “poles” hereafter. Each pole corresponds to a set of modal parameters (f_i, ζ_i, ϕ_i) .

5.4.3.3 Final parameter determination

After getting the modal parameters obtained from ARX models with different orders, it is necessary to determine which poles of which model give the “real” parameters. The stabilization diagram was used to conduct the selection. The idea behind this method is that real physical modes tend to “stabilize” with the increase of model orders, while spurious computational modes will not. Stabilization here means that the relative differences between modal parameters extracted from two models with adjacent orders are smaller than certain thresholds. Practically, determination of final parameters in the work reported here took the following steps.

First, judging the stabilization of modal parameters. Modal frequencies and damping ratios were compared using the absolute relative difference, while the MAC values [81] were used for mode shapes.

$$\left| \frac{f_H - f_L}{f_L} \right| \leq f_{thr} \quad (5.26)$$

$$\left| \frac{\zeta_H - \zeta_L}{\zeta_L} \right| \leq \zeta_{thr} \quad (5.27)$$

$$\text{MAC}(\phi_H, \phi_L) \leq \text{MAC}_{thr} \quad (5.28)$$

Where,

f_H and f_L are modal frequencies from the models with higher and lower orders, respectively.

ζ_H and ζ_L are modal damping ratios from the models with higher and lower orders, respectively.

ϕ_H and ϕ_L are mode shape vectors from the models with higher and lower orders, respectively.

f_{thr} , ζ_{thr} and MAC_{thr} are thresholds for the different parameters, respectively.

It should be noted that, in the practice of the reported work, the above comparison was conducted for all the combination of data pairs. For example, take an f_H value and compare with all frequencies from the lower-order model, and then repeat for all frequencies from the higher-order model. The thresholds for these parameters were set via trial and error. For the work reported here, they were taken as 2%, 10% and 2% for frequencies, damping ratios and MAC values, respectively. In addition to the thresholds, some hard boundaries were also set to screen out unreal modes. The target value ranges for frequencies,

damping ratios and MAC values were set as [0, 25] Hz, [0, 10%], and [0.9, 1], respectively.

Poles stable about all three parameters within the hard boundaries were plotted with frequency as horizontal axis and model order as vertical axis, forming the stabilization diagram.

After getting the stabilization diagram, the proper model order was manually selected first, then the “real” poles from this order was selected, as well as the corresponding modal parameters. As a general rule, the order should be high enough to include as many real modes as possible, but not too high to avoid overparameterization. For the selection of poles of the “right” order, it should be noted that not all modes can be surely detected, and closely spaced or repeated modes were selected by the author. In a word, determination of both model orders and “real” poles relied on not only the stabilization condition, but also the author’s experience. Some trial and error were also inevitable.

Figure 5.13 illustrates an example of the stabilization diagram taken from Case01X of the 07EDF test, with the FRF curve summed from all floors as a reference. It is seen that four physical modes stabilized and formed “columns” on the diagram. An order of 30 is necessary to get all four modes stable.

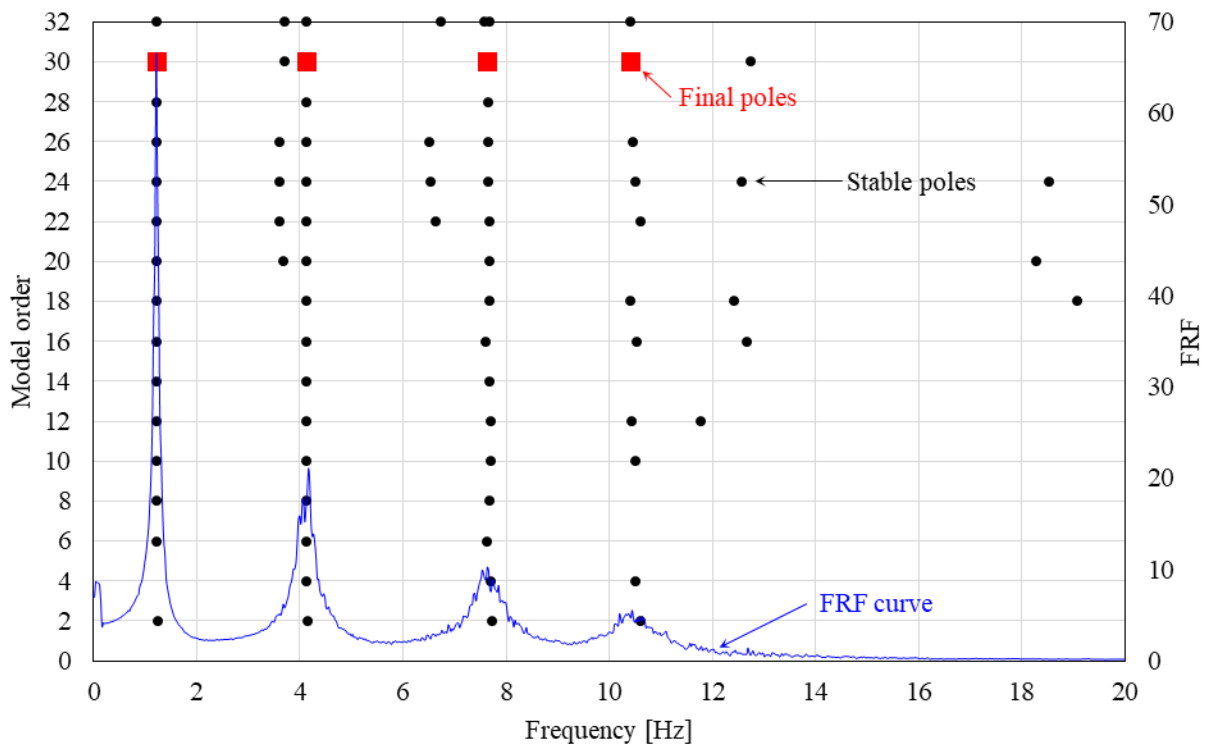


Figure 5.13 Example of stabilization diagram and pole selection

5.4.4 State-Space (SS) method

5.4.4.1 Theory

The State-Space (SS) method is a time domain model describing the dynamic behavior of a linear invariant system. This method has been widely used in system identification and modal analysis, for

example in [57,85,86]. It should be noted that, for the application into modal parameter estimation, the stochastic form should be used. However, considering that the data to be analyzed were all from highly controlled test sources, the deterministic form was used in the work reported here to save computation cost.

For a system with M independent DOFs, the usual representation can be written as a 2nd-order differential equation as follows:

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}_2\dot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{B}_2\mathbf{x}(t) \quad (5.29)$$

Where,

$\mathbf{u}(t)$ is the displacement responses vector in continuous-time form.

$\mathbf{x}(t)$ is the time distribution of excitations in continuous-time form.

$\mathbf{M}$, $\mathbf{C}_2$ and $\mathbf{K}$ are the mass, damping and stiffness matrices of the system.

$\mathbf{B}_2$ is the matrix describing the spatial distribution of excitations.

Introducing the so-called state vector $\mathbf{s}(t) = [\mathbf{u}(t), \dot{\mathbf{u}}(t)]^T$, Eq. (5.29) can be re-written in state form

$$\dot{\mathbf{s}}(t) = \mathbf{A}_c\mathbf{s}(t) + \mathbf{B}_c\mathbf{x}(t) \quad (5.30)$$

Where, $\mathbf{A}_c$ and $\mathbf{B}_c$ are the continuous-time system and input matrices, respectively.

$$\mathbf{A}_c = \begin{bmatrix} \mathbf{0} & \mathbf{I}_M \\ -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C}_2 \end{bmatrix} \quad (5.31)$$

$$\mathbf{B}_c = \begin{bmatrix} \mathbf{0} \\ -\mathbf{M}^{-1}\mathbf{B}_2 \end{bmatrix} \quad (5.32)$$

Assuming that the response vector of the system is $\mathbf{y}(t)$ containing p measurements which can be acceleration, velocity, etc., there is a so-called observation equation written as

$$\mathbf{y}(t) = \mathbf{C}_a\ddot{\mathbf{u}}(t) + \mathbf{C}_v\dot{\mathbf{u}}(t) + \mathbf{C}_d\mathbf{u}(t) \quad (5.33)$$

Where,

$\mathbf{C}_a$, $\mathbf{C}_v$ and $\mathbf{C}_d$ are the output matrices of acceleration, velocity and displacement, respectively.

Using similar form as Eq. (5.30), Eq. (5.33) can be re-written as

$$\mathbf{y}(t) = \mathbf{C}\mathbf{s}(t) + \mathbf{D}\mathbf{u}(t) \quad (5.34)$$

Where, $\mathbf{C}$ and $\mathbf{D}$ are the output and direct transmission matrices, respectively.

$$\mathbf{C} = [\mathbf{C}_d - \mathbf{C}_a\mathbf{M}^{-1}\mathbf{K}, \mathbf{C}_v - \mathbf{C}_a\mathbf{M}^{-1}\mathbf{C}_2] \quad (5.35)$$

$$\mathbf{D} = \mathbf{C}_a\mathbf{M}^{-1}\mathbf{B}_2 \quad (5.36)$$

The above Eq. (5.30) and (5.36) form the state-space model in continuous. For practical usage, the discrete form is needed. Assuming that the sampling time is Δt , the discrete state-space model can be written as

$$\mathbf{s}(k+1) = \mathbf{A}\mathbf{s}(k) + \mathbf{B}\mathbf{x}(k) \quad (5.37)$$

$$\mathbf{y}(k) = \mathbf{C}\mathbf{s}(k) + \mathbf{D}\mathbf{u}(k) \quad (5.38)$$

Where,

$\mathbf{A} = e^{\mathbf{A}_c \Delta t}$ is the discrete-time system matrix.

$\mathbf{B} = [\mathbf{A} - \mathbf{I}]\mathbf{A}_c^{-1}\mathbf{B}_c$ is the discrete-time input matrix.

$\mathbf{C}$ and $\mathbf{D}$ are the same as in Eq. (5.34).

Theoretically, both Eq. (5.37) and (5.38) should be used when using the SS method. For the work reported, only the system matrix $\mathbf{A}$ is needed to extract modal parameters. In fact, the system matrix deduced here is equivalent to the system matrix deduced for the ARX method, as introduced in [58]. Figure 5.14 illustrates the flow chart of the SS identification algorithm used in the current work. Except for the model estimation, the algorithm is essentially the same as the ARX method.

5.4.4.2 Multi-order SS model estimation

Similar to the ARX algorithm, the estimation also used multiple SS models with different orders. The minimum and maximum orders were determined through trial and error considering the balance between computation cost and pole numbers. One thing to notice is that the order step in the SS estimation needs to be set as even numbers considering the structure of the SS models. The estimation of SS models also used down-sampled data with a ratio of 0.5 which also gave better results.

For the estimation of the SS model with a given order, the numerical algorithm for subspace state space system identification (N4SID) [87] was adopted using the built-in function of Matlab[®]. After the SS model was estimated, the system matrix $\mathbf{A}$ can be obtained and the following procedures were the same as the ARX algorithm. Details are omitted here for simplicity.

5.4.4.3 Final parameter determination

The final parameters in the SS model algorithm were also determined using the stabilization diagram method and followed the same procedures as the ARX algorithm. Details are omitted here for simplicity.

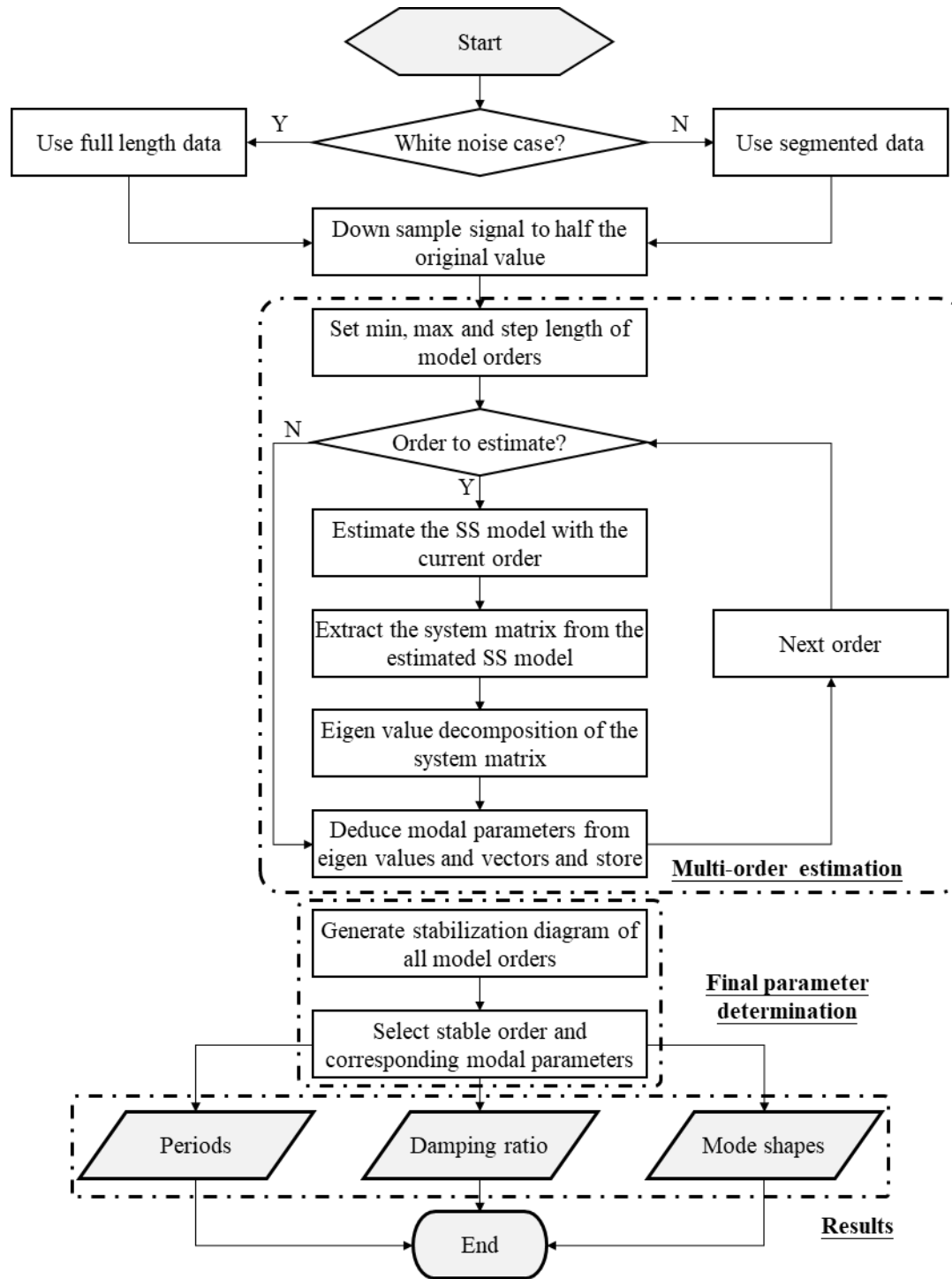


Figure 5.14 Flowchart of the SS algorithm

5.4.5 Synchro-Squeezed Wavelet Transform (SSWT) method

5.4.5.1 Theory

The aforementioned system identification methods are all based on time invariant systems and utilized full length signals or at most segmented signals. These methods can only give approximate results for nonlinear ground motion excited cases. In addition, even with signal segmentation, considering that parameters have to be selected manually and thus there cannot be too many segments, the time resolution

of the variation of modal parameters is quite limited. In order to deal with this deficiency, time-frequency analysis was introduced to the work. Time-frequency illustration of the floor responses can provide estimation of time-dependent modal parameters, especially in nonlinear cases. The purpose of the time-frequency analysis is to get the instantaneous frequencies of the structure during the ground motion excitations.

Among the various methods such as Short-Time Fourier Transform (STFT), Continuous Wavelet Transform (CWT) and so on, the Synchro-Squeezed Wavelet Transform (SSWT) [65] was selected considering its ability to better illustrated the instantaneous modal properties. This method utilizes a synchrosqueezing algorithm [66] to better allocate the energy distribution in the time-frequency plane along the frequency axis to “sharpen” the results. Ridges can then be detected on the sharpened results, from which the instantaneous frequency can be obtained. Once the ridge of a mode is obtained, the corresponding modal component can be obtained by selecting a band centered along the ridge and inverse transform this band from time-frequency domain to time domain. Since the method is signal based, in order to represent the properties of the “whole” structure, the floor responses were summed up before processed by the algorithm. Figure 5.15 illustrates the results of STFT, CWT and SSWT of the same signal (20EDF, MRF-Spine, Northridge VA 100%, summed floor response). It is clear that SSWT gives much more concentrated results than STFT and CWT.

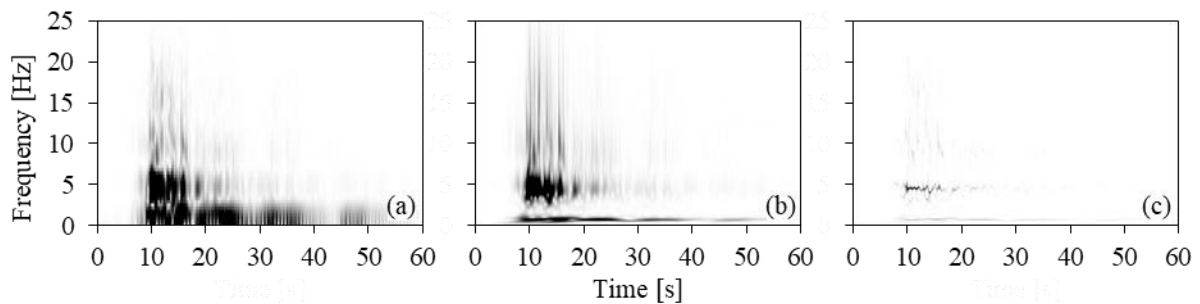


Figure 5.15 Comparison of different time-frequency transform results
(a) STFT; (b) CWT; (c) SSWT

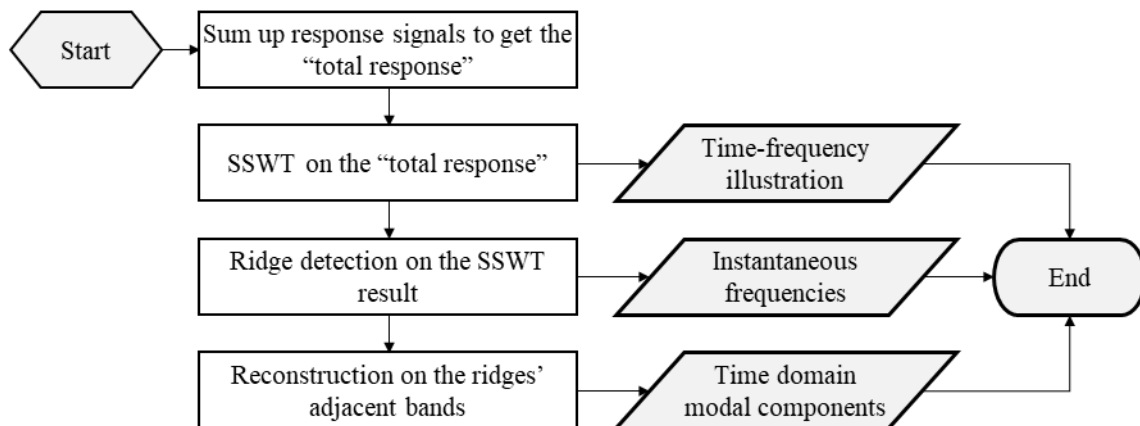


Figure 5.16 Flowchart of the SSWT algorithm

Figure 5.16 illustrates the flow chart of the SSWT algorithm used in the work reported here. Functions from the wavelet toolbox of Matlab[®] were used to conduct the SSWT, ridge detection and reconstruction. The following subsections take some of the equations from the original paper to illustrate the general idea of the algorithm. For details of the algorithm's formulation and proof, readers are referred to the original paper [65].

5.4.5.2 Synchro-Squeezed Wavelet Transform

For simplicity, take a purely harmonic signal as the example:

$$s(t) = A \cos(\omega t) \quad (5.39)$$

Take a wavelet ψ concentrated on the positive frequency axis, CWT of the signal can be rewritten as

$$W_s(a, b) = \frac{A}{4\pi} a^{1/2} \overline{\hat{\psi}(a\omega)} e^{ib\omega} \quad (5.40)$$

According to the synchro-squeezing algorithm introduced in [66], for any (a, b) for which $W_s(a, b) \neq 0$, the instantaneous frequency can be calculated by

$$\omega(a, b) = -i(W_s(a, b))^{-1} \frac{\partial}{\partial b} W_s(a, b) \quad (5.41)$$

For a and ω in continuous form, the synchrosqueezed transform of $W_s(a, b)$ can be written as

$$T_s(\omega, b) = \int_{A(b)} W_s(a, b) a^{-3/2} \delta(\omega(a, b) - \omega) da \quad (5.42)$$

Where $A(b) = \{a; W_s(a, b) \neq 0\}$ and $\omega(a, b)$ is defined in Eq. (5.13) for (a, b) that $a \in A(b)$.

By doing so, energy on the time-frequency plane concentrated near the area of larger wavelet coefficient magnitude. The above illustration is in continuous form, but in practical applications, the frequency variable ω and scale variable a need to be discretized. The *wsst* function of Matlab[®] was used for this step.

5.4.5.3 Ridge detection

After getting the SSWT results, ridge detection was conducted. In this procedure, maximum wavelet coefficient magnitude along the frequency axis was detected at each time instance. These maxima were then connected, getting one ridge on the time-frequency plane. If more than one ridges were needed, which was most likely the case, this detection was conducted iteratively by removing the previously detected ridge from the time-frequency plane and then conducting the detection. As a result, ridges corresponding to each mode can be detected, each representing the instantaneous frequency of a corresponding mode. This procedure was realized using the *wsstridge* function of Matlab[®].

5.4.5.4 Component reconstruction

After getting the ridges, a band along each ridge on the time-frequency plane can be specified. Wavelet coefficients in the bands can then be transformed back to time domain using the reconstruction equations in Eq. (5.15) or Eq. (5.16), getting the transient modal components of each mode. These components are an intuitive means to identify mode strength in time domain. They can be interpreted qualitatively by looking at the signal curves, or quantitatively by using different measures of signals such as peak value, root mean square, etc. The reconstruction was realized using the *iwsst* function of Matlab®.

$$f(t) = C_\psi \int_{-\infty}^{\infty} \int_0^{\infty} W_f(a, b) a^{-5/2} \psi\left(\frac{t-b}{a}\right) da db \quad (5.43)$$

$$f(t) = C'_\psi \int_0^{\infty} W_f(a, t) a^{-3/2} \psi\left(\frac{t-b}{a}\right) da \quad (5.44)$$

Where C_ψ and C'_ψ are constants depending on the wavelet ψ . Selection of the range is not critical since the energy is highly concentrated around the ridges on the time-frequency plane.

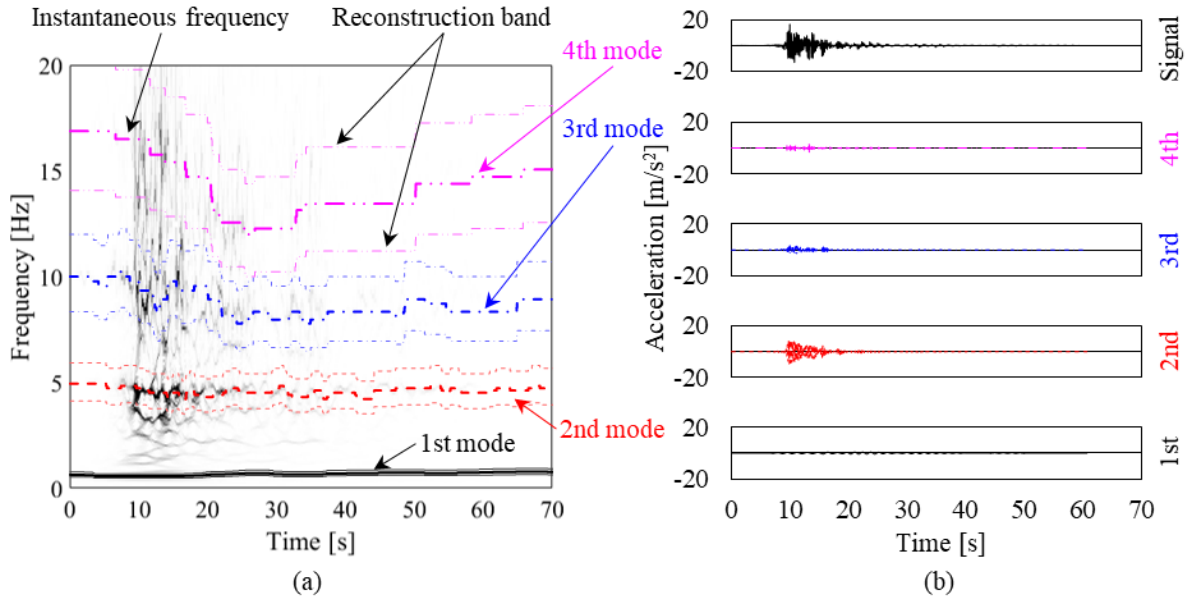


Figure 5.17 Example of the SSWT algorithm

(a) SSWT results with instantaneous frequency and reconstruction bands; (b) reconstructed components

Figure 5.17 shows an example of the SSWT results using “single floor” signal (20EDF, MRF-Spine, Northridge VA 100%, Floor 1). It can be seen that the modal frequency variation can be identified by the instantaneous frequency curves, while the reconstructed time domain components can illustrate the modal strength intuitively. It should also be noted that the summation of the reconstructed components is not necessarily equal to the original signal, since the SSWT method is based on CWT in which the

wavelet bases are redundant, and that the reconstruction does not necessarily use all energy on the time-frequency plane. This inequality is especially obvious when the modes are not well concentrated, but this will not have very significant influence for the work reported hereafter.

5.4.6 Effective modal pseudo-acceleration

In addition to the above methods, the Effective Modal Pseudo-Acceleration (EMPA) [70], can also be used as an indicator to evaluate the transition of modal properties under nonlinear ground motion excitations. The EMPA is an illustration of the modal contribution in the view of response spectra. It is an extension from the elastic range, and has been used in previous work to evaluate the higher mode responses in spine systems, such as [16,71,88]. Computation of the EMPA is briefly introduced hereafter.

For a linear MDOF system, a response $r(t)$ has its modal decomposition as

$$r(t) = \sum_1^N r_n(t) = \sum_1^N r_n^{st} A_n(t) \quad (5.45)$$

Where,

$r_n(t)$ is the n th mode modal component of the total response.

$r_n^{st}(t)$ is the n th mode modal equivalent static response.

$A_n(t) = \omega_n^2 D_n(t)$ is the n th mode modal pseudo acceleration.

Taking this relationship, the total restoring force of a linear MDOF structure can be written as

$$\mathbf{f}_r(t) = \sum_1^N \mathbf{f}_{r,n}(t) = \sum_1^N \mathbf{s}_n A_n(t) = \sum_1^N (\Gamma_n \mathbf{m} \boldsymbol{\phi}_n) A_n(t) \quad (5.46)$$

Where,

$\mathbf{f}_{r,n}(t)$ is the n -th mode restoring force.

$\mathbf{s}_n$ is the n -th mode spatial distribution of external force.

$\Gamma_n = \boldsymbol{\phi}_n^T \mathbf{m} \mathbf{u} / M_n$ is the n -th mode modal participation factor.

$M_n = \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n$ is the n -th mode generalized mass.

$\boldsymbol{\phi}_n$ is the n -th mode shape vector.

For linear systems, left multiplying $\boldsymbol{\phi}_n^T$ to both sides of Eq. (5.46), taking the mass orthogonality, the modal pseudo acceleration can be represented by the known restoring force vector as

$$A_n(t) = \frac{\boldsymbol{\phi}_n^T \mathbf{f}_r(t)}{\Gamma_n M_n} \quad (5.47)$$

For nonlinear systems, the only difference from linear systems is to change the restoring force vector to

nonlinear responses. The Effective Modal Pseudo Acceleration (EMPA) can thus be represented as

$$A_{eff,n}(t) = \frac{\phi_n^T f_r^{NL}(t)}{\Gamma_n M_n} \quad (5.48)$$

Taking the peak value of the $A_{eff,n}(t)$ series, termed as pEMPA hereafter, and plot together with the pseudo acceleration spectrum of the excitation, the strength of each mode can then be evaluated according to the relationship of the pEMPA values.

For the application of EMPA on test data, the restoring force can be obtained from story shear force or inertia force, which are computed using column strain or floor acceleration measurement, respectively. Considering the more reliable acceleration data of the selected tests and the relatively small damping ratio of the systems, the EMPA presented here is computed using restoring force deduced from inertia force. As for the mode shape vectors, those obtained from the white noise cases right before the nonlinear cases were used considering the limited transition of mode shapes throughout the whole tests, as will be illustrated in the next chapter. The periods of each mode were also set as the value from the same white noise case as the mode shapes, which neglected any possible period elongation in the ground motion excitations.

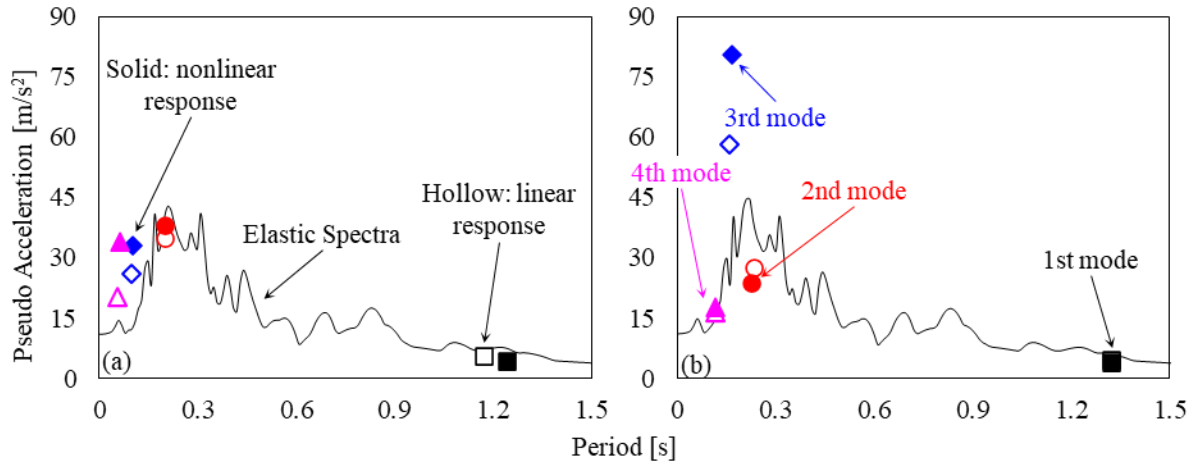


Figure 5.18 Example of EMPA algorithm result
(a) MRF-Spine system; (b) MRF-Spine-FLC system.

It should be noted that, theoretically, when plotted together with Pseudo-Acceleration spectrum having the same damping ratio as the structural system, the pEMPA values taken from elastic cases should lie right on the spectral lines in elastic cases. However, when dealing with real-world test data, it may not be the case due to the existence of noise and the inconsistency of damping ratio between the spectrum and the structure, but the trend can still be captured. Figure 5.18 illustrates an example of the EMPA method, plotting the pEMPA and pseudo-acceleration spectrum of the two structural systems in the 20EDF test under the Northridge VA 100% excitation. The aforementioned inconsistency in spectral values and trend in mode strength transition can be seen.

5.5 SUMMARY

This chapter introduces the basic information of the vibration property analysis. Major contents are summarized as follows:

- Details of the selected supplementary shake table tests are introduced, including the information of the test specimens, excitations used in the tests, and load cases conducted in the tests.
- Preparation of the raw test data is introduced, including the common treatment applied to all select tests and special treatment applied to the 14UCSD test only.
- Details of the selected analysis methods are introduced. The introduction mainly focuses on the practical application in the reported work instead of the detailed theoretical formulation.

6. VIBRATION PROPERTY ANALYSIS USING SHAKE-TABLE TEST DATA: RESULTS AND DISCUSSION

6.1 GENERAL

This chapter introduces the modal properties, time-frequency properties and spectral properties of the selected tests. Discussion of different influential factors on the structural responses will be organized using these results. In this chapter, only results of the major test, the 20EDF test, will be introduced in detail. Results from other tests will be used to form various discussions. Full results of the vibration property analysis are introduced in Appendices E to I, providing information of the 20EDF, 21TKB, 07EDF, 08EDF and 14UCSD tests, respectively.

The discussion was aimed to give both quantitative and qualitative evaluation of the vibration properties of the spine systems in comparison with different structural systems. Influence of FLCs, spines, excitation properties, structure scale, structure proportion, etc. will be evaluated comprehensively. These discussions can help better understand how different modes behave in the spine systems, and whether their behavior is different from other structural systems, providing a different point of view from conventional structural engineering as introduced in Chapter 4.

6.2 ANALYSIS RESULTS OF THE 20EDF TEST

6.2.1 General information

This section introduces analysis results of the 20EDF test to depict the vibration properties of the structure system with spines as sufficient as possible. Modal parameters obtained using different methods (FDD, ARX, and SS) will be introduced, with their consistency being judged. In addition, time-frequency properties and spectral properties will also be introduced to evaluate the vibration properties from different angles.

6.2.2 Modal parameter transition in white noise excitations

6.2.2.1 Natural periods

Figure 6.1 shows the transition of natural periods in white noise cases through the whole test. Table 6.1 summarizes the corresponding values. Periods estimated using three modal analysis methods are all included. The three methods gave quite consistent results on period estimation. For the natural periods, results from any of the methods can be used.

Similar transition of natural periods can be found as introduced in Chapter 4. The 1st periods elongated in the MRF-Spine system mainly due to local damage occurred during excitations, while remained almost unchanged in the MRF-Spine-FLC system. The 2nd periods showed slight elongation when the

system changed but was consistent within both structural systems. The 3rd and 4th periods showed similar transition trends as the 2nd periods, only that the elongation at system change was more significant than the 2nd period. Transition of higher mode natural periods indicates that the activation of FLCs changed the dynamic properties of the system, while the rather consistent higher mode periods within each structural system indicate that these modes stayed essentially elastic as expected.

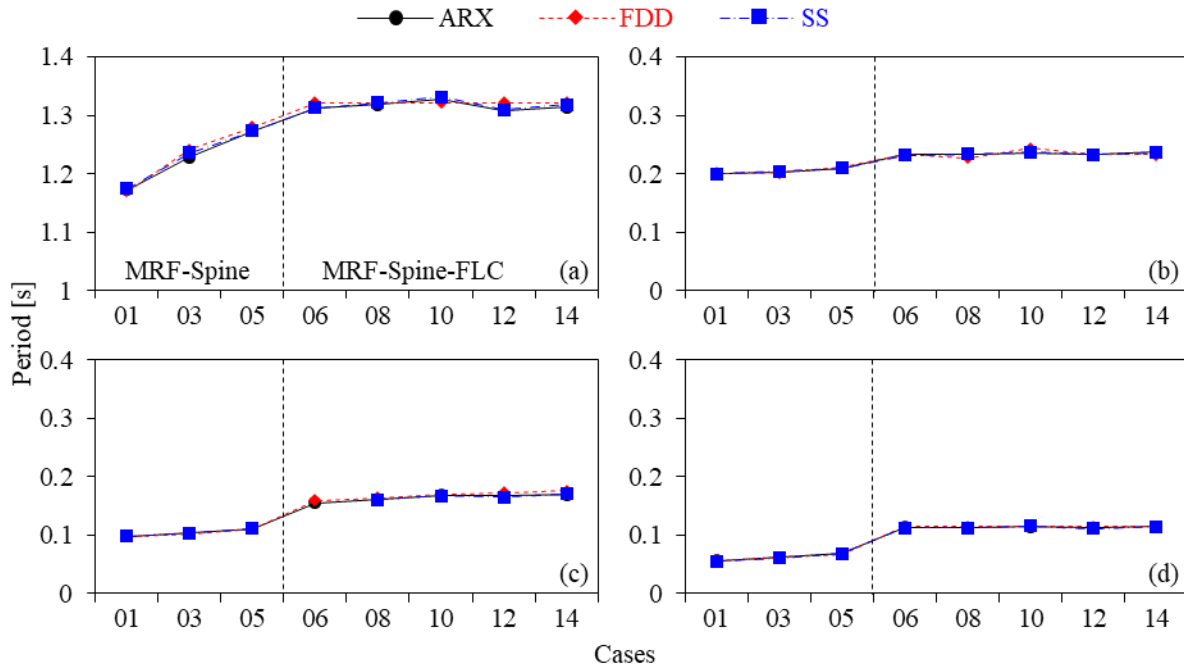


Figure 6.1 Natural periods in white noise excitations
(a) 1st mode; (b) 2nd mode; (c) 3rd mode; (d) 4th mode

Table 6.1 Values of natural periods in white noise excitations [Unit: s]

Case	1st period			2nd period			3rd period			4th period		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	1.17	1.17	1.17	0.20	0.20	0.20	0.10	0.10	0.10	0.06	0.06	0.06
03	1.23	1.24	1.24	0.20	0.20	0.20	0.10	0.10	0.10	0.06	0.06	0.06
05	1.27	1.28	1.27	0.21	0.21	0.21	0.11	0.11	0.11	0.07	0.07	0.07
06	1.31	1.32	1.31	0.23	0.23	0.23	0.16	0.16	N/A	0.11	0.11	0.11
08	1.32	1.32	1.32	0.23	0.23	0.23	0.16	0.16	0.16	0.11	0.11	0.11
10	1.33	1.32	1.33	0.24	0.24	0.24	0.17	0.17	0.17	0.11	0.12	0.12
12	1.31	1.32	1.31	0.23	0.23	0.23	0.17	0.17	0.17	0.11	0.11	0.11

6.2.2.2 Modal damping ratios

Figure 6.2 shows the transition of modal damping ratios in white noise cases through the whole test. Table 6.2 summarizes the corresponding values. Modal damping ratios estimated using three modal analysis methods are all included. It is seen that the ARX and SS methods gave closer results while results from the FDD method showed more difference. This is attributed to the different procedures taken in these methods when estimating damping ratios. In the FDD method, conversion to time domain and manual selection of ACF segment was needed, while in the ARX and SS methods the modal

damping ratios were directly calculated from modal parameters without manual treatment. For the modal damping ratios, results from the ARX and SS methods are more preferable.

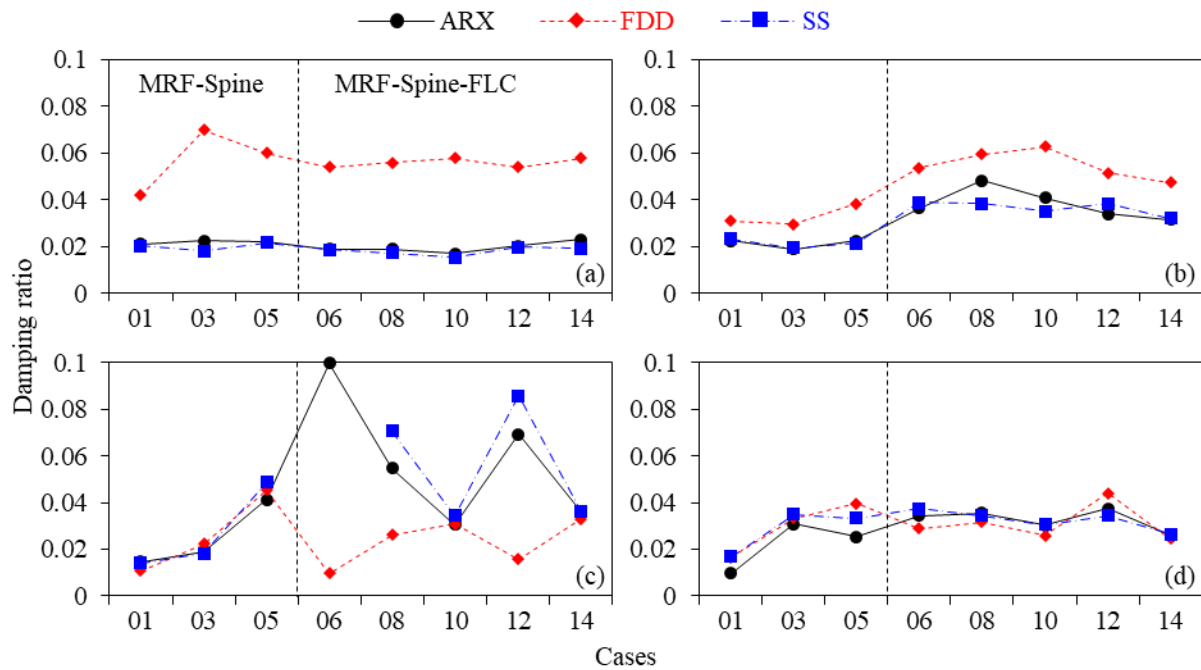


Figure 6.2 Modal damping ratios in white noise excitations

(a) 1st mode; (b) 2nd mode; (c) 3rd mode; (d) 4th mode

Table 6.2 Values of modal damping ratios in white noise excitations

Case	1st period			2nd period			3rd period			4th period		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.02	0.04	0.02	0.02	0.03	0.02	0.01	0.01	0.01	0.01	0.02	0.02
03	0.02	0.07	0.02	0.02	0.03	0.02	0.02	0.02	0.02	0.03	0.03	0.04
05	0.02	0.06	0.02	0.02	0.04	0.02	0.04	0.05	0.05	0.03	0.04	0.03
06	0.02	0.05	0.02	0.04	0.05	0.04	0.10	0.01	N/A	0.03	0.03	0.04
08	0.02	0.06	0.02	0.05	0.06	0.04	0.05	0.03	0.07	0.04	0.03	0.03
10	0.02	0.06	0.02	0.04	0.06	0.04	0.03	0.03	0.03	0.03	0.03	0.03
12	0.02	0.05	0.02	0.03	0.05	0.04	0.07	0.02	0.09	0.04	0.04	0.03

In general, modal damping ratios from all three methods showed different trends for different modes. For the 1st mode, the FDD method gave larger modal damping ratios than the other methods, with some increase in the MRF-Spine system and consistent values in the MRF-Spine-FLC system. The ARX and SS methods gave quite consistent modal damping ratios in both structural systems throughout the whole test with slightly increase in the last excitation. For the 2nd mode, all methods gave similar transition that the modal damping ratios stayed stable in the MRF-Spine system while in the MRF-Spine-FLC system the values first increased then decreased. Similar to the 1st mode, results from the FDD method gave larger values than the other methods while values from the ARX and SS methods were quite close. For the 3rd mode, variation of the damping ratios was rather significant for all methods, with the ARX and SS relatively close. For the FDD method, the estimated damping ratios were slightly smaller in the

MRF-Spine-FLC systems, while the ARX and SS methods gave opposite transition. For the 4th mode, all methods gave similar results with the damping ratios being quite consistent in both structural systems. In general, the estimated modal damping ratios increased after the FLCs were activated, especially for the higher modes. It should also be noted that estimation of modal damping ratios has known difficulties especially for real structures and experiment data. Thus, the estimated damping ratios were more suitable to give qualitative results rather than quantitative ones.

6.2.2.3 Mode shapes

Figure 6.3 shows the mode shapes in Cases 01, 05, 06 and 14, which are the first and last cases of the two structural systems. The mode shapes are normalized using the modal participation factor Γ_n defined as following:

$$\Gamma_n = \frac{\phi_n^T \mathbf{m} \mathbf{u}}{\phi_n^T \mathbf{m} \phi_n} \quad (6.1)$$

Where,

ϕ_n is the mode shape vector.

$\mathbf{m}$ is the mass matrix, which is diagonal.

$\mathbf{u}$ is the excitation influence factor.

Table 6.3 summarizes the Modal Assurance Criteria (MAC) [81] values of the mode shapes in all white noise cases to quantify their similarity compared with the initial case. The closer the MAC value is to unity, the more similar the two mode shape vectors. The MAC values are determined as follows:

$$\text{MAC} = \frac{|\phi_p^T \cdot \phi_q^*|^2}{(\phi_p^T \cdot \phi_q^*)(\phi_p^T \cdot \phi_q^*)} \quad (6.2)$$

Where,

ϕ_p and ϕ_q are two mode shape vectors to compare.

Superscript T and $*$ denotes transpose and complex conjugate.

Table 6.3 MAC values of all modes in all white noise excitations

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
03	1.000	1.000	1.000	1.000	1.000	0.998	0.992	0.995	0.989	0.998	0.999	0.688
05	1.000	1.000	1.000	0.998	0.997	0.996	0.981	0.987	0.975	0.983	0.993	0.976
06	1.000	1.000	1.000	0.991	0.992	0.987	0.989	0.949	N/A	0.998	0.999	0.988
08	1.000	1.000	1.000	0.994	0.994	0.993	0.994	0.990	0.993	0.998	0.996	0.998
10	1.000	1.000	1.000	0.995	0.997	0.994	0.992	0.990	0.989	0.998	0.994	0.995
12	1.000	1.000	1.000	0.997	0.996	0.997	0.995	0.999	0.989	0.999	0.998	0.999

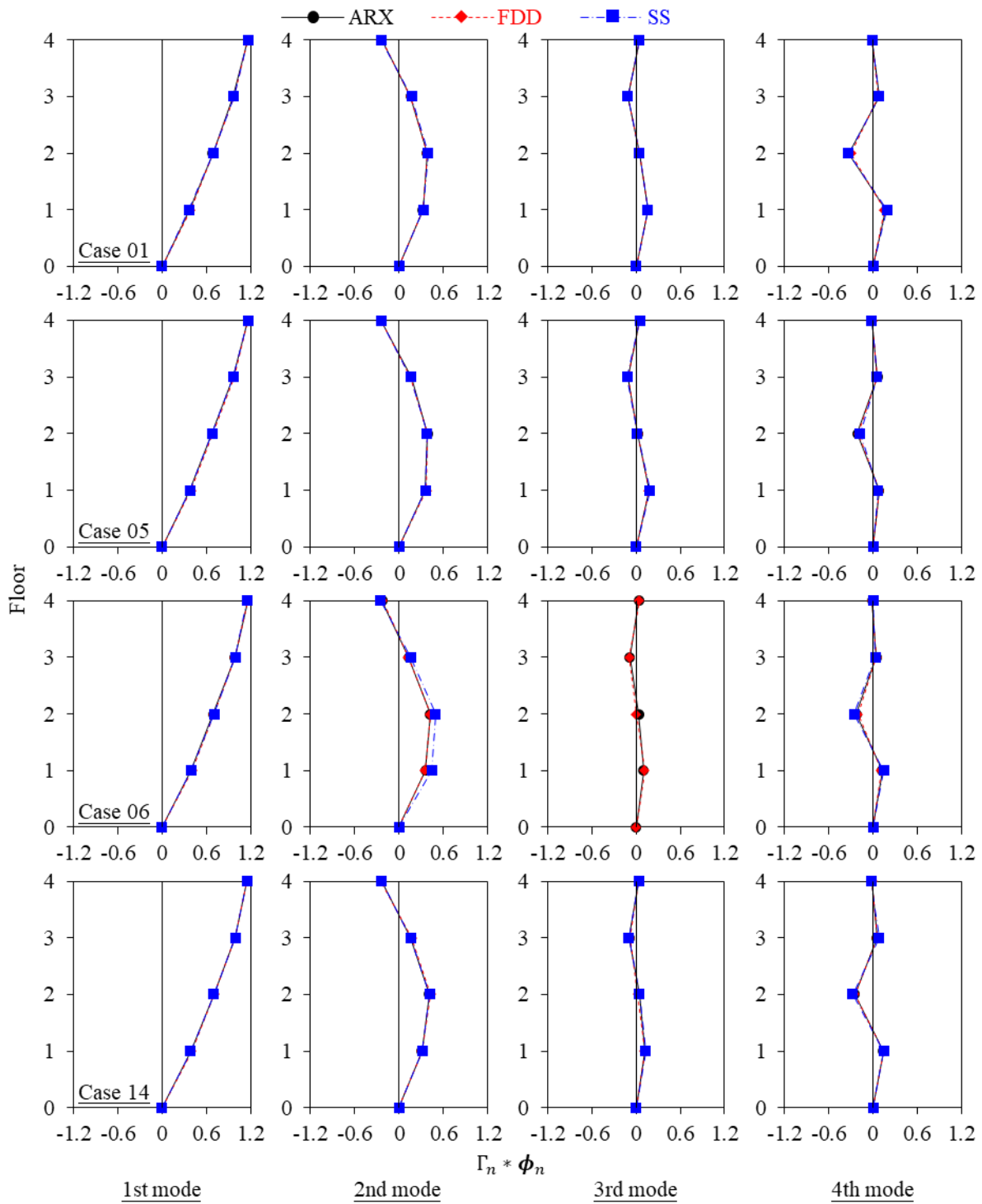


Figure 6.3 Mode shapes in selected white noise cases

It can be seen that all three methods gave consistent estimation on mode shapes of all modes in all white noise cases. For the mode shapes, results from any of the methods can be used.

From the plot and MAC values, it can be seen that the mode shapes did not change much throughout the whole excitation, regardless of the structural systems. The smallest MAC value occurred in Case 03 of the 4th mode estimated by the SS method. The consistent mode shapes indicate that the elastic deformation pattern of the structure did not change much, which can be attributed to the existence of

spines which forced the structure to deform with a consistent shape within the same structural systems. The consistency between the two structural systems suggested that function of FLCs was not significant in elastic vibration. The mode shapes also suggest that different modes had different influence at different floors. For example, the top floor was more influenced by the 1st mode while the 1st floor was influenced almost equally by all modes.

6.2.2.4 Mode strength evaluated by Modal Effective Mass

Different numerical indicators can be used to quantify the strength or contribution of different modes in a structural system. The aforementioned Γ_n is one of the indicators, but it is dependent on how the mode shapes are normalized. For building structures with lumped mass, the Modal Effective Mass (MEM) defined in Eq. (6.3), which is not normalization-dependent, can be used as the indicator. The MEMs can represent the intrinsic mode strength in the system excluding the influence of excitation properties.

$$MEM_n = \frac{(\phi_n^T \mathbf{m} \mathbf{u})^2}{\phi_n^T \mathbf{m} \phi_n} \quad (6.3)$$

Where,

ϕ_n is the mode shape vector.

$\mathbf{m}$ is the mass matrix, which is diagonal.

$\mathbf{u}$ is the excitation influence factor.

n denotes the n -th mode.

Figure 6.4 shows the transition of MEMs of the system in white noise excitations. Table 6.4 summarizes the corresponding MEM values. It can be seen that all methods gave pretty consistent estimation except for the SS method in the 2nd and 4th modes. In general, similar to the transition of mode shapes, the MEM values also did not change much throughout the whole test. The 1st mode was the strongest in both structural systems, with the MEMs in the MRF-Spine-FLC system slightly larger than the MRF-Spine system. The 2nd mode was much weaker than the 1st mode but stronger than the 3rd and 4th modes in both systems, and the MEMs stayed consistent throughout the whole test. The 3rd and 4th modes were the weakest and had similar MEM level, with the MEMs slightly smaller in the MRF-Spine-FLC system. Transition of the MEMs suggests that the mode strength did not change significantly in elastic excitations, indicating that the elastic dynamic properties of the system did not change much even with the activation of the FLCs. This phenomenon also suggests the need to investigate the post-elastic ground motion excited cases, in which the FLCs were expected to be effective. Considering the consistency of methods, MEMs from any of the methods can be used.

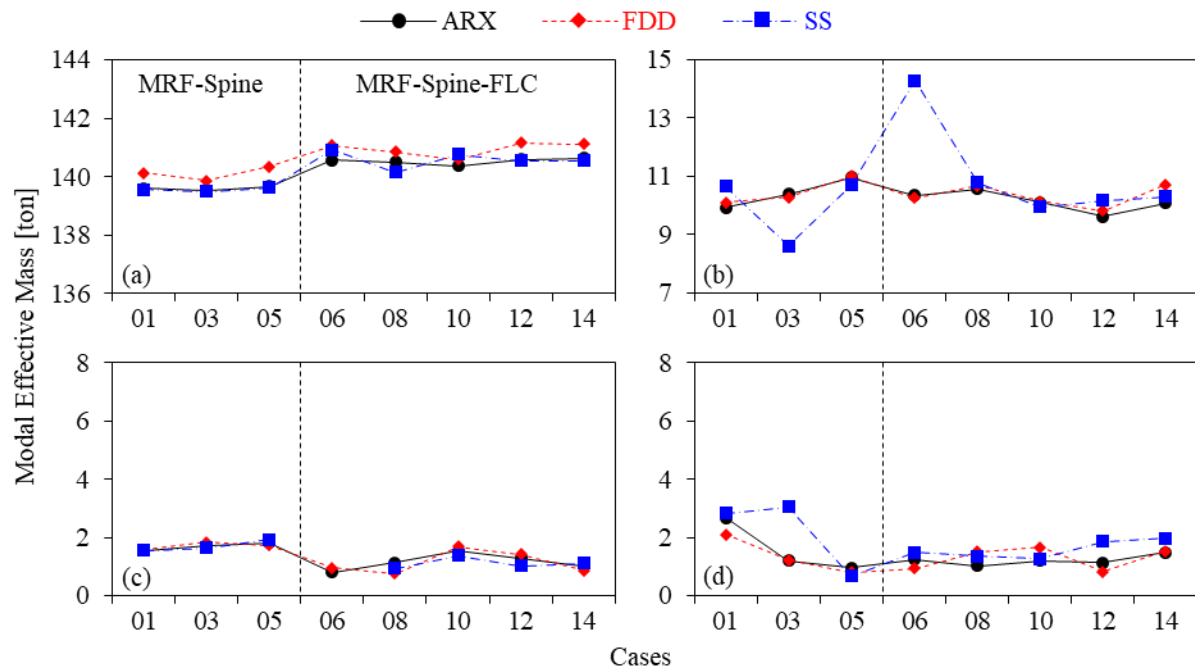


Figure 6.4 Modal Effective Mass (MEM) in white noise excitations

(a) 1st mode; (b) 2nd mode; (c) 3rd mode; (d) 4th mode

Table 6.4 MEM values in white noise excitations [Unit: ton]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	139.6	140.1	139.6	9.9	10.1	10.7	1.5	1.6	1.5	2.7	2.1	2.8
03	139.5	139.9	139.5	10.4	10.3	8.6	1.7	1.8	1.6	1.2	1.2	3.1
05	139.7	140.4	139.6	11.0	11.0	10.7	1.8	1.7	1.9	1.0	0.8	0.7
06	140.6	141.1	140.9	10.3	10.2	14.3	0.8	1.0	N/A	1.2	0.9	1.5
08	140.5	140.9	140.1	10.6	10.7	10.8	1.1	0.8	0.9	1.0	1.5	1.4
10	140.4	140.6	140.8	10.1	10.2	10.0	1.5	1.7	1.4	1.2	1.7	1.3
12	140.6	141.2	140.6	9.6	9.8	10.2	1.3	1.4	1.0	1.1	0.8	1.9

6.2.2.5 Mode strength evaluated by PSD peak values

In addition to the MEM, the FDD method can also estimate the Power Spectral Density (PSD) of the system. The PSD values at the peaks corresponding to each mode can also represent the mode strength from the energy point of view.

Figure 6.5 shows the transition of PSD values at these peaks, with the corresponding values summarized in Table 6.5. The 1st mode was the strongest and stronger in the MRF-Spine-FLC system. The 2nd mode was quite consistent throughout the test, but slightly smaller in the MRF-Spine-FLC system, which is different from the MEM values. The 3rd and 4th modes were even smaller in the MRF-Spine-FLC system, but the 3rd mode was larger than the 2nd mode in the MRF-Spine system which is also different from the MEM values. Differences between the PSD peaks and MEMs can be attributed to the sensitivity of PSD estimation to the influencing factors such as frequency resolution and smoothing, and that the MEMs represent intrinsic properties while the PSD peaks include influence from the excitations. In

general, the difference in white noise excitations is acceptable.

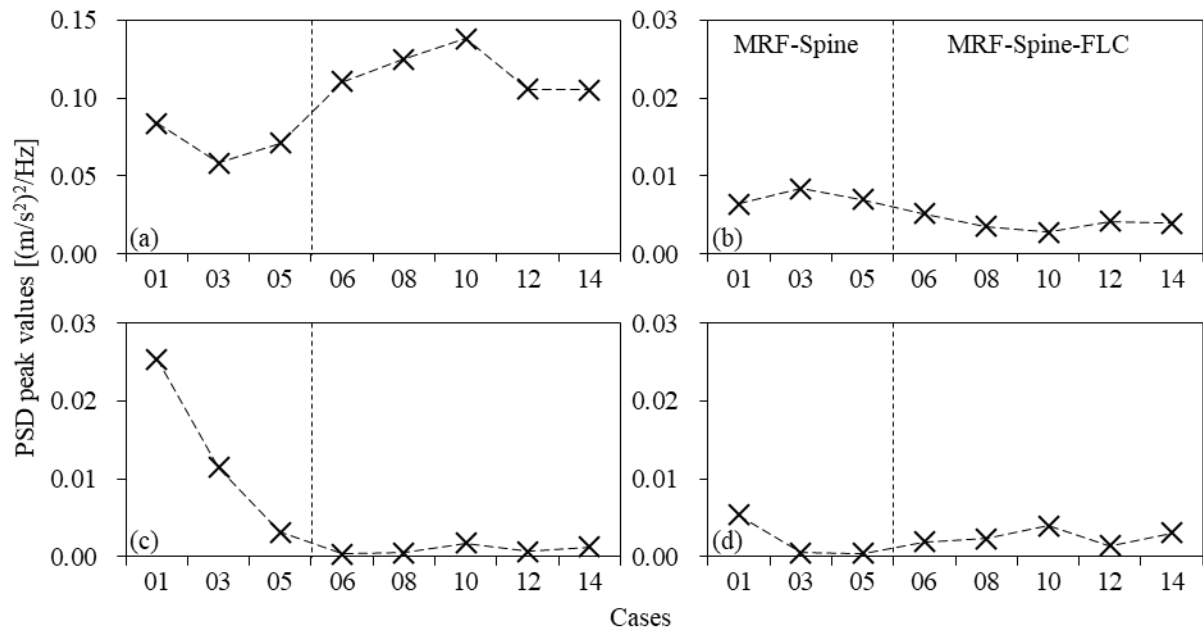


Figure 6.5 PSD peak values in white noise excitations
(a) 1st mode; (b) 2nd mode; (c) 3rd mode; (d) 4th mode

Table 6.5 PSD peak values in white noise excitations [Unit: (m/s²)²/Hz]

Case	1st mode	2nd mode	3rd mode	4th mode
01	0.084	0.006	0.025	0.005
03	0.058	0.008	0.011	0.001
05	0.071	0.007	0.003	0.000
06	0.111	0.005	0.000	0.002
08	0.125	0.004	0.001	0.002
10	0.138	0.003	0.002	0.004
12	0.106	0.004	0.001	0.001

6.2.2.6 Selection of the major estimation method

For the aforementioned four modal parameters, it can be seen that, as expected, the three modal analysis methods gave generally consistent estimates. Considering the consistency of different parameters, and the nature of each method, the ARX method was selected as the major method to form the discussion on white noise excitations. Parameters estimated using the FDD and SS methods will also be used when necessary to provide supplementary information.

6.2.3 Modal parameter transition of ground motion excitations

6.2.3.1 Natural periods

Figure 6.6 shows the transition of natural periods in the three 100% excitations (Case 04, 09 and 13) estimated using the three modal analysis methods. Table 6.6 summarizes the corresponding values of each segment. The table motion signals in each case are also provided to illustrate data segmentation.

It can be seen that all three methods gave better estimation for higher modes than the 1st mode. This is attributed to the fact that the segmentation shortened the signal length for analysis and further reduced the information conveyed by the signal segment, which was more severe for the longer 1st mode periods. For the higher mode periods, the 3 methods gave rather consistent results. For the 1st mode, results from the ARX and SS methods were quite close, but results from the FDD method were not as consistent. This is also attributed to the short segment length which limited the resolution of the 1st mode period, especially for the FDD method which relied heavily on the frequency resolution of PSD curves.

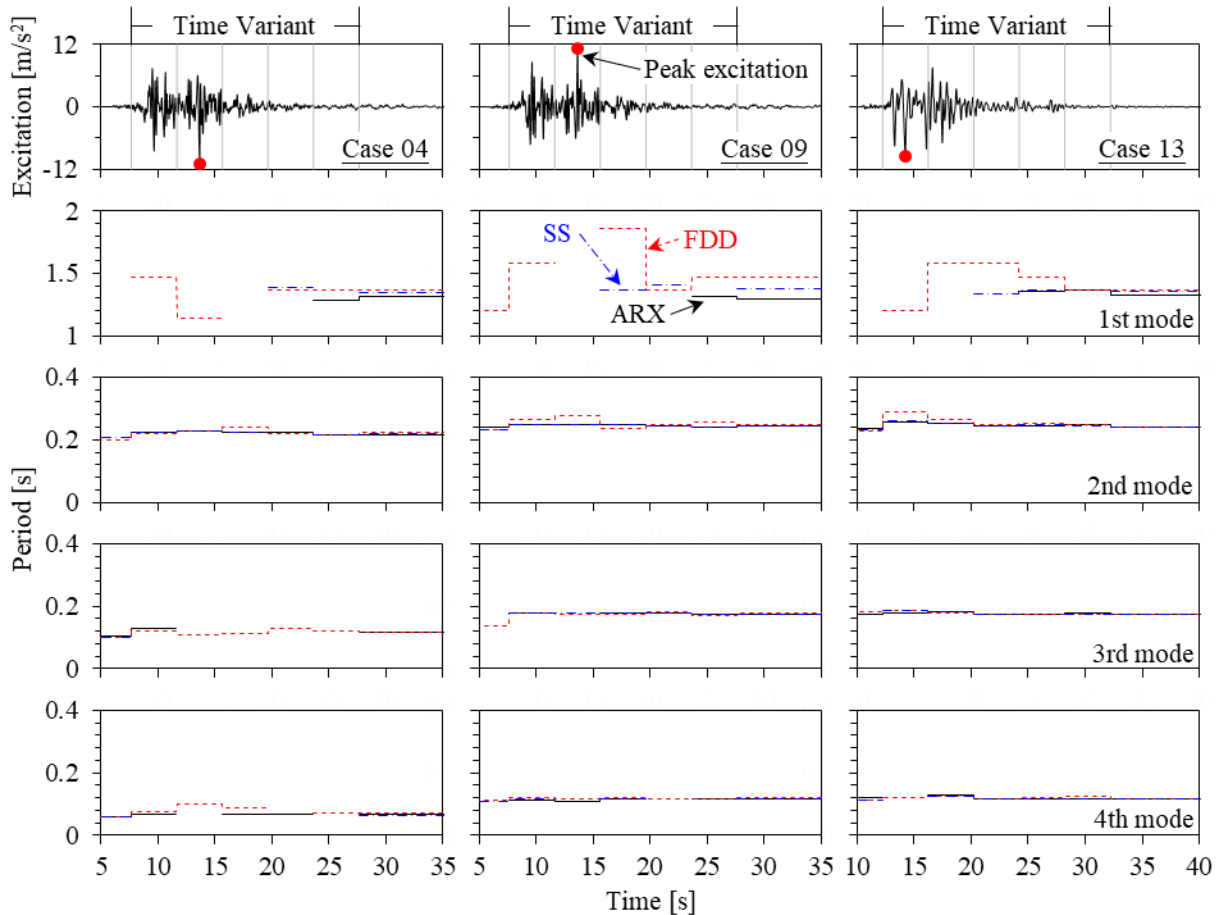


Figure 6.6 Natural periods in selected ground motion cases

For the 1st mode period, values in the “tail” segments in which the vibration decreased were relatively consistent for all the methods and load cases. This is consistent to the observation on white noise cases that the 1st mode periods did not change very much in the two structural systems. The FDD method gave estimates in more segments, which showed some elongation when the excitations were strong. It is known that results from the FDD method were approximate, but they can still give some hint that yielding occurred in the 1st mode when the specimen was loaded several times.

For the higher modes, periods in the MRF-Spine-FLC system (Case 09 and 13) were larger than in the MRF-Spine system (Case 04), which was attributed to the general yielding of the structure including the FLCs. Period elongation can be observed when the excitations were strong, suggesting that the structure

yielded during the excitation, but this may not be observed in white noise excitations in which the specimen returned to elastic range. Elongation was slightly stronger in the MRF-Spine-FLC system, but it is not sure if this can be attributed to the FLCs. More investigation was needed to further the understanding of such phenomena.

Table 6.6 Period values in selected ground motion cases [Unit: s]

Case-Segment	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01					0.20	0.21	0.10	0.10	0.10	0.06	0.06	0.06
04-02		1.46		0.22	0.22	0.22	0.13	0.12		0.07	0.07	
04-03		1.14		0.23	0.23	0.23		0.11			0.10	
04-04				0.22	0.24	0.22		0.11		0.07	0.09	
04-05		1.37	1.39	0.22	0.22			0.13		0.07		
04-06	1.29	1.37		0.22	0.22	0.22		0.12			0.07	
04-07	1.32	1.37	1.35	0.22	0.23	0.22	0.12	0.12		0.07	0.07	0.06
09-01		1.20		0.24	0.23	0.23		0.14			0.11	0.11
09-02		1.58		0.25	0.27	0.25	0.18	0.18	0.18	0.11	0.12	0.12
09-03				0.25	0.28	0.25		0.17	0.18	0.11	0.12	
09-04		1.86	1.37	0.25	0.24	0.25	0.18	0.17	0.18	0.12	0.12	0.12
09-05		1.37	1.41	0.24	0.25	0.25	0.18	0.18	0.18		0.12	
09-06	1.32	1.46		0.24	0.26	0.24	0.17	0.17	0.18	0.12	0.12	
09-07	1.29	1.46	1.37	0.24	0.25	0.24	0.17	0.18	0.17	0.12	0.12	0.12
13-01				0.24	0.23	0.23	0.17	0.18		0.12	0.11	0.11
13-02		1.20		0.26	0.29	0.26	0.18	0.19	0.19		0.12	
13-03		1.58		0.25	0.27	0.25	0.18	0.18	0.18	0.13	0.12	0.12
13-04		1.58	1.33	0.24	0.25	0.24	0.18	0.18	0.18	0.12	0.12	0.12
13-05	1.36	1.46	1.36	0.24	0.25	0.25	0.17	0.17		0.12	0.12	0.12
13-06	1.37	1.37		0.25	0.25	0.24	0.18	0.18	0.17	0.12	0.13	0.12
13-07	1.32	1.37	1.35	0.24	0.24	0.24	0.17	0.17	0.17	0.12	0.12	0.11

It should be noted that none of the 3 modal analysis methods were designed for application in post-elastic ground motion excited cases. The ground motion excitation condition violates the basic assumption of the FDD method which requires flat excitation (white noise or ambient). The possible nonlinearity in post-elastic cases violates the linear system assumption of all methods. However, considering the consistency of results, and that the ARX and SS methods “normalized” the results to the excitation, results from the ARX methods can be used to evaluate the vibration properties of the structural systems under ground motion excitations.

6.2.3.2 Modal damping ratios

Figure 6.7 shows the transition of modal damping ratios in the three 100% excitations (Case 04, 09 and 13) estimated using the three modal analysis methods. Table 6.7 summarizes the corresponding values of each segment. The table motion signals in each case are also provided to illustrate data segmentation.

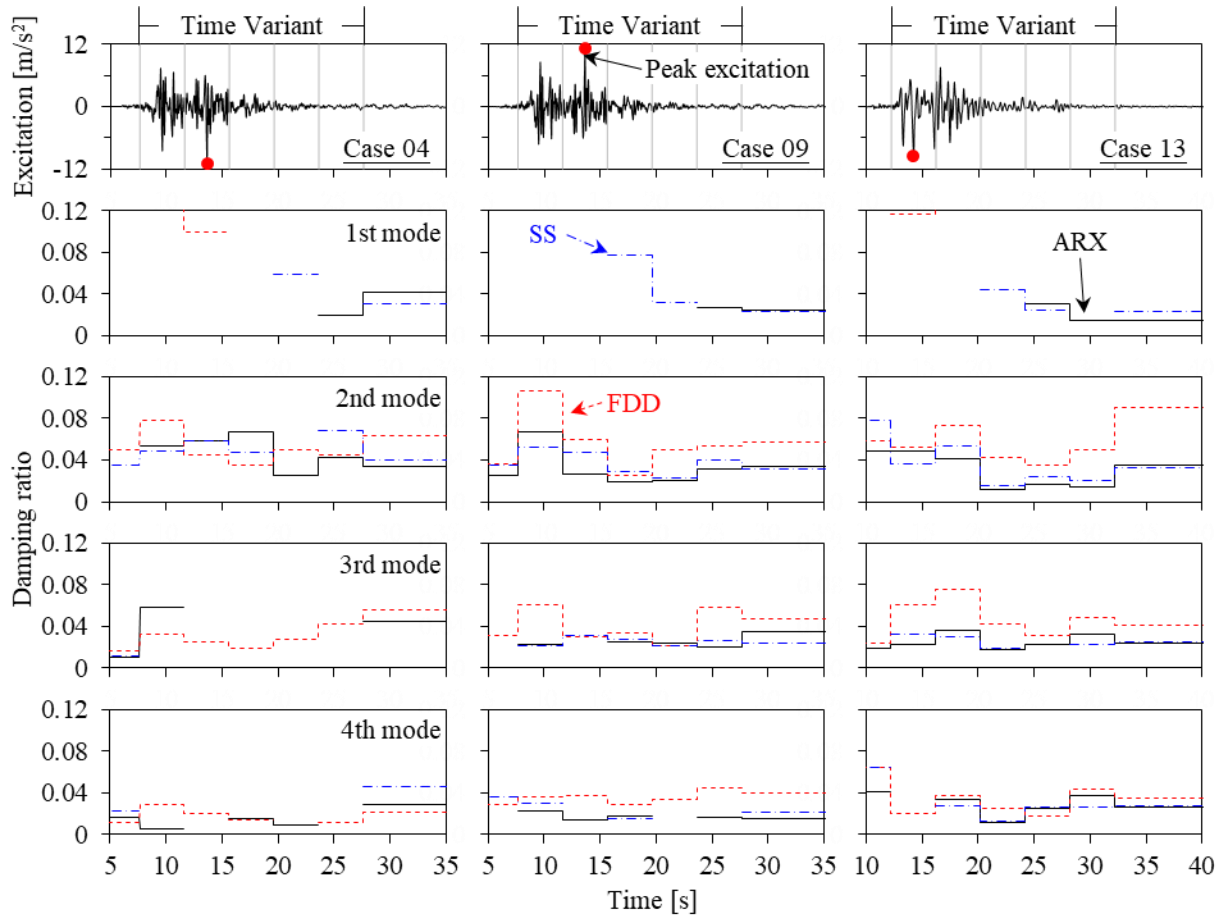


Figure 6.7 Modal damping ratios in selected ground motion cases

For steel structures without supplementary damping, damping ratio larger than 0.1 is usually considered unreasonable. For the 1st mode, the FDD method gave estimates much larger than 0.1, which reached even 0.3 and thus considered unreliable. For the higher modes, the FDD method also gave generally larger modal damping ratios than the ARX and SS methods, which was partly attributed to the fact that curve fitting conducted in the FDD algorithm was heavily affected by the data quality which was not so good in the segmented data. The ARX and SS methods, in the segments where both methods gave estimates, showed relatively close results for all modes, but still not as good as the period estimations.

From the estimated results, there was no obvious trend in the transition of damping ratios. The difference between different modes was also not significant. Thus, comparison in damping cannot help with the evaluation of vibration properties. Considering the less approximation in the ARX and SS methods, results from these methods, although not very meaningful, were preferred.

Table 6.7 Modal damping ratio values in selected ground motion cases

Case-Segment	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01					0.05	0.04	0.01	0.02	0.01	0.02	0.01	0.02
04-02		0.21		0.05	0.08	0.05	0.06	0.03		0.01	0.03	
04-03		0.10		0.06	0.05	0.06		0.02			0.02	
04-04				0.07	0.04	0.05		0.02		0.02	0.01	
04-05		0.20	0.06	0.03	0.05			0.03		0.01		
04-06	0.02	0.20		0.04	0.05	0.07		0.04			0.01	
04-07	0.04	0.20	0.03	0.03	0.06	0.04	0.04	0.06		0.03	0.02	0.05
09-01		0.28		0.03	0.04	0.03		0.03			0.03	0.04
09-02		0.30		0.07	0.11	0.05	0.02	0.06	0.02	0.02	0.04	0.03
09-03				0.03	0.06	0.05		0.03	0.03	0.01	0.04	
09-04		0.16	0.08	0.02	0.03	0.03	0.02	0.03	0.03	0.02	0.03	0.02
09-05		0.20	0.03	0.02	0.05	0.02	0.02	0.02	0.02		0.03	
09-06	0.03	0.20		0.03	0.05	0.04	0.02	0.06	0.03	0.02	0.04	
09-07	0.02	0.21	0.02	0.03	0.06	0.03	0.04	0.05	0.02	0.01	0.04	0.02
13-01				0.05	0.06	0.08	0.02	0.02		0.04	0.06	0.06
13-02		0.12		0.05	0.05	0.04	0.02	0.06	0.03		0.02	
13-03		0.30		0.04	0.07	0.05	0.04	0.08	0.03	0.03	0.04	0.03
13-04		0.22	0.04	0.01	0.04	0.02	0.02	0.04	0.02	0.01	0.03	0.01
13-05	0.03	0.13	0.02	0.02	0.03	0.02	0.02	0.03		0.02	0.02	0.03
13-06	0.01	0.19		0.01	0.05	0.02	0.03	0.05	0.02	0.04	0.04	0.03
13-07	0.01	0.20	0.02	0.03	0.09	0.03	0.02	0.04	0.02	0.03	0.03	0.03

6.2.3.3 Mode shapes

Same as the natural periods and modal damping ratios, mode shapes were not identified for all segments. Considering that the mode shapes were identified in the last segment (Seg 07) of most of the cases, mode shapes of this segment were used as the reference for the comparison of similarities of mode shapes in other segments. The comparison also used the MAC values as the indicator.

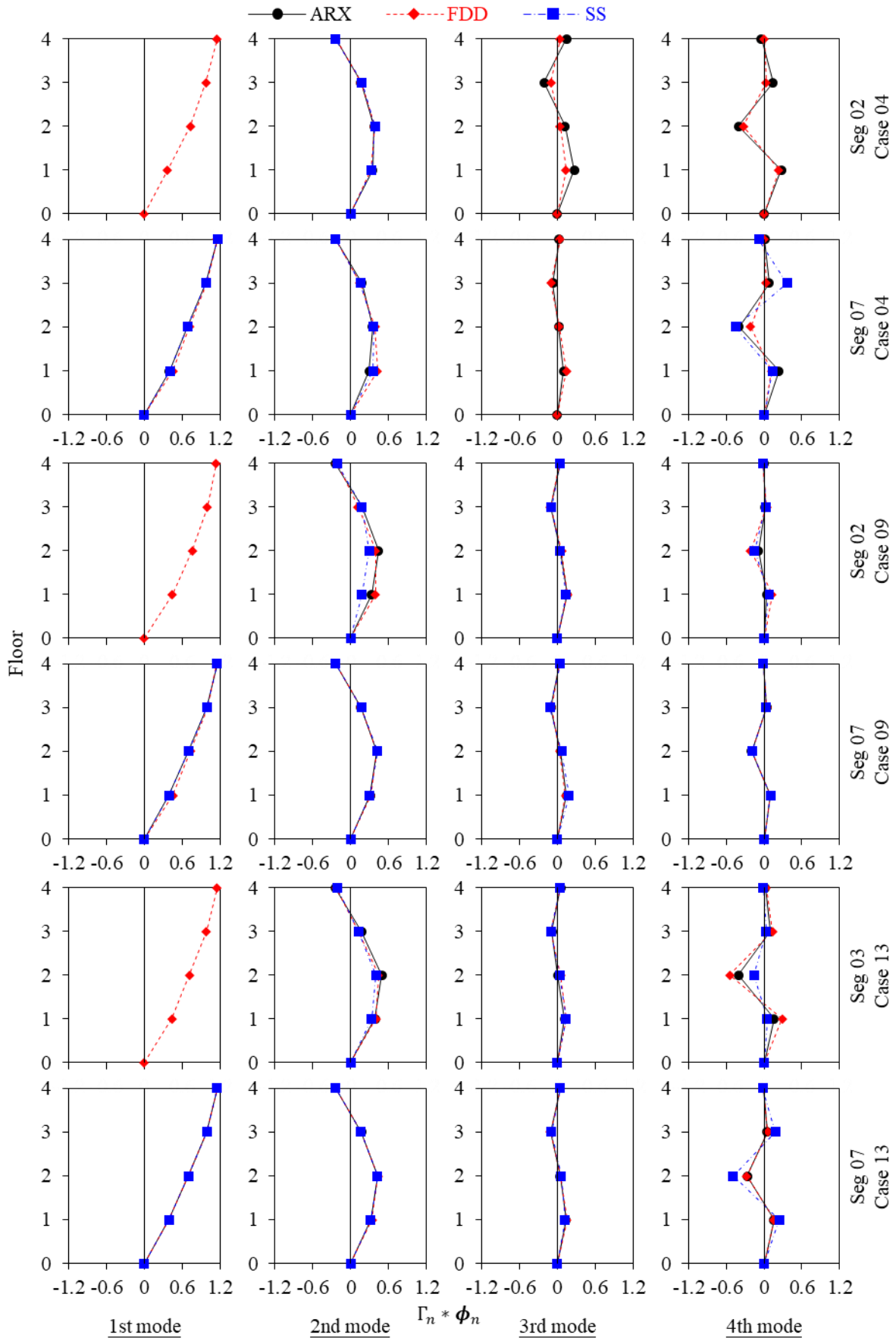


Figure 6.8 Mode shapes in selected ground motion cases

Figure 6.8 shows the mode shape vectors normalized by Γ_n in the 100% excitations, with one from a TV segment when the excitation was strong and the other from the last TI segment (Seg 07 for all cases). Table 6.8 summarizes the MAC values compared with Seg 07 to evaluate the similarity of mode shapes. Specific locations of the segments can be found previously in Figure 6.6 or Figure 6.7.

Table 6.8 MAC values in selected ground motion cases

Case-Segment	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01				0.9823	0.9736		0.9964	0.9980		0.9730	0.9921	0.8150
04-02		0.9975		0.9939	0.9802	0.9961	0.9449	0.9903		0.9727	0.9830	
04-03		0.9954		0.9816	0.9967	0.9985		0.9111			0.9888	
04-04				0.9889	0.9992	0.9685		0.9977		0.9090	0.9969	
04-05		0.9994	0.9987	0.9723	0.9944			0.9676		0.9849		
04-06	1.0000	0.9997		0.9989	0.9987	0.9990		0.9999			0.9795	
04-07	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000		1.0000	1.0000	1.0000
09-01		0.9917		0.9954	0.9804	0.9859		0.9612			0.9952	0.9552
09-02		0.9998		0.9998	0.9747	0.9656	0.9965	0.9845	0.9871	0.9956	0.9963	0.9983
09-03				0.9996	0.9809	0.9900		0.9847	0.9574	0.8504	0.9997	
09-04		0.9823	0.9997	0.9998	0.9860	0.9398	0.9950	0.9967	0.9949	0.9885	0.9965	0.9921
09-05		0.9991	1.0000	0.9997	0.9997	0.9998	0.9778	0.9588	0.9844		0.9902	
09-06	1.0000	1.0000		0.9969	0.9953	0.9969	0.9982	0.9833	0.9888	0.9983	0.9960	
09-07	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
13-01				1.0000	0.9924	0.9061	0.6455	0.9860		0.9961	0.9795	0.9880
13-02		0.9986		0.9776	0.9451	0.8776	0.9804	0.9895	0.9607		0.9771	
13-03		0.9994		0.9966	0.9889	0.9944	0.9384	0.9989	0.9795	0.9692	0.9887	0.9541
13-04		0.9776	0.9964	1.0000	0.9989	0.9986	0.9998	0.9983	0.9860	0.9995	0.9980	0.9441
13-05	0.9999	0.9977	1.0000	0.9999	0.9943	1.0000	0.9972	0.9986		0.9991	0.9944	0.9701
13-06	1.0000	1.0000		0.9998	0.9996	0.9995	0.9979	0.9992	0.9934	0.9960	0.9172	0.9479
13-07	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

From the plotted mode shapes, it can be seen that the three methods generally gave close results on the mode shape vectors. Larger differences can be found for the 4th mode which was possibly due to the weak 4th mode in the system. Mode shapes did not change much in different load cases, indicating that the yielding in the system was relatively uniform, which can be partly attributed to the existence of the spines that enforced a uniform deformation distribution. Comparing the mode shapes in different segments in the same case, from both the plotted shapes and the MAC values, it can be seen that differences were minor, especially for the 1st and 2nd modes. Larger differences can be seen in the 3rd and 4th modes, for example the 3rd mode shape in Segment 01 of Case 13 estimated using the ARX method, but such small MAC values are still not the majority.

From the segmented estimation results, mode shapes did not vary much even when the excitation was strong. Considering that mode shapes are only related to mass and stiffness distribution, and that the lumped mass can be deemed unchanged for building structures, it can be concluded that the story

stiffness distribution pattern did not vary significantly. In addition, from the mode shape point of view, results from the ARX method are preferred for further discussion.

6.2.3.4 Mode strength evaluated by Modal Effective Mass

Same as the white noise cases, the Modal Effective Mass (MEM) was used to quantify the transition of the intrinsic mode strength in the ground motion cases for all three methods. Figure 6.9 shows the transition of MEMs of the three 100% excitations. Table 6.9 summarizes the corresponding values.

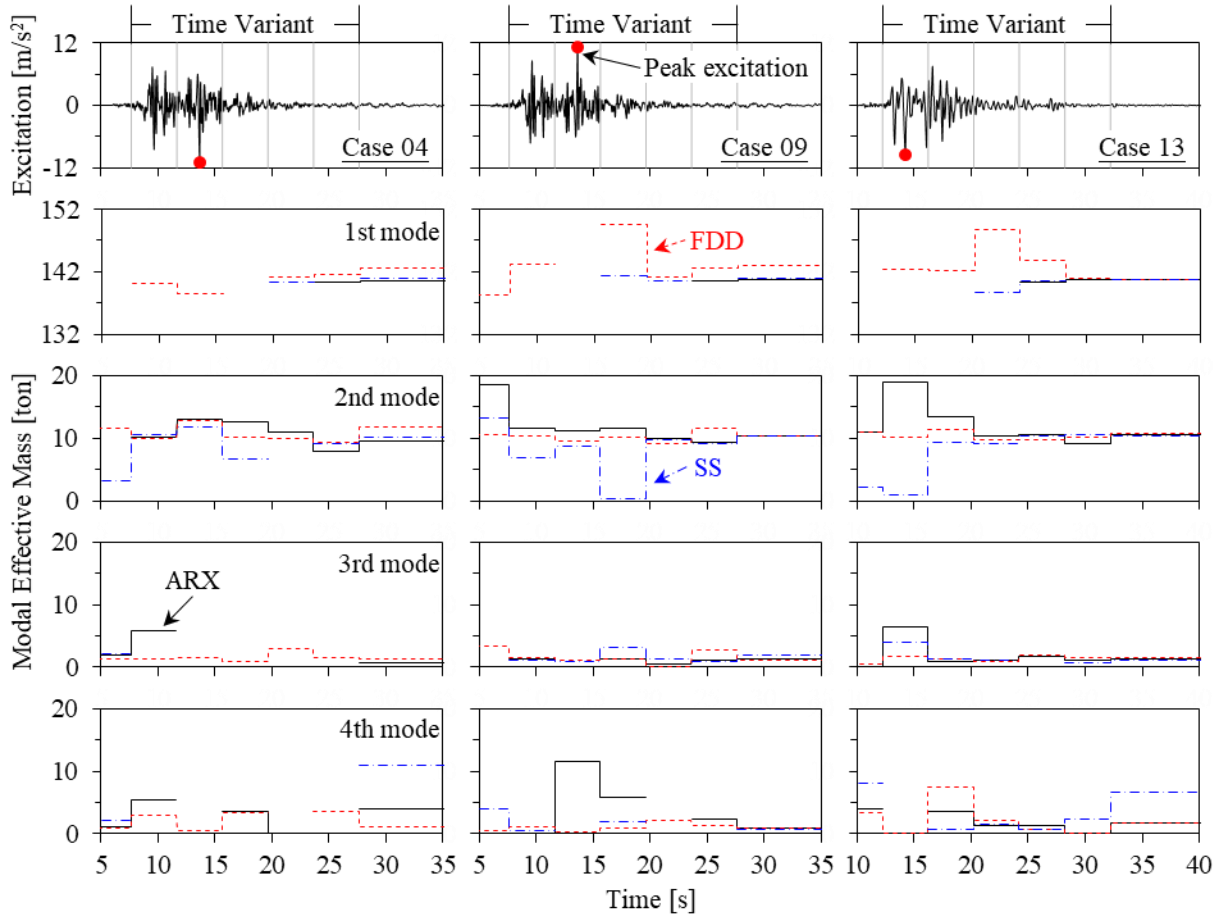


Figure 6.9 Modal Effective Mass (MEM) in selected ground motion cases

From the plots, it can be seen that the three methods did not give very close estimation for all modes. Estimation of the 1st mode MEM was close in the tail segments when the excitation died out, and showed limited variation of the MEM values. Results from the FDD method did show some increase, but considering its approximate nature, such results cannot be deemed as rigid proof of stronger 1st mode. Estimation of the 2nd mode MEMs showed some increase in Case 04 when the excitation was strong, while the increase was not observed in Case 09, which can be partly attributed to the function of the FLCs limiting the 2nd mode strength. However, such function was quite limited from the MEM results. Estimation of the 2nd mode in Case 13 was rather inconsistent when the excitation was strong but “converged” in weak excitation segments, showing limited variation of MEM values. The 3rd mode

MEMs showed similar trends to the 2nd mode, just that the MEM values were much smaller. The 4th mode MEMs showed more obvious variation, but no clear trends can be observed. This is attributed to the relatively weak 4th mode in the system which made the identification less reliable.

Table 6.9 Modal Effective Mass (MEM) values in selected ground motion cases [Unit: ton]

Case-Segment	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01					11.6	3.2	2.0	1.3	2.1	1.0	0.9	2.2
04-02		140.2		10.3	9.9	10.6	5.8	1.3		5.5	3.0	
04-03		138.4		13.0	12.8	11.8		1.6			0.4	
04-04				12.7	10.1	6.8		1.0		3.5	3.4	
04-05		141.1	140.4	10.9	10.0			2.9		0.1		
04-06	140.6	141.7		7.8	9.4	9.2		1.6			3.6	
04-07	140.6	142.6	140.9	9.5	11.7	10.2	0.6	1.3		4.0	1.0	11.0
09-01		138.3		18.6	10.5	13.2		3.4			0.4	4.0
09-02		143.1		11.5	10.3	6.8	1.4	1.6	1.2	0.1	1.2	0.5
09-03				11.2	9.5	8.8		1.2	1.0	11.6	0.3	
09-04		149.5	141.4	11.6	10.1	0.3	1.3	1.4	3.3	5.8	0.9	2.0
09-05		141.1	140.6	10.0	9.0	9.8	0.5	0.1	1.4		2.2	
09-06	140.6	142.5		9.3	11.5	9.2	1.1	2.8	1.0	2.4	1.3	
09-07	140.7	142.9	140.9	10.3	10.4	10.4	1.3	1.0	1.9	0.9	0.8	0.8
13-01				10.9	10.9	2.2	0.1	0.5		4.1	3.3	8.0
13-02		142.5		19.0	10.2	0.9	6.4	1.7	4.0		0.1	
13-03		142.2		13.4	11.5	9.2	0.9	1.3	1.2	3.5	7.4	0.6
13-04		148.8	138.6	10.4	9.8	9.1	1.2	0.9	1.1	1.3	2.0	1.4
13-05	140.3	143.7	140.4	10.7	9.7	10.4	1.6	2.0		1.2	0.7	0.7
13-06	140.7	141.0		9.2	10.1	10.5	1.1	1.5	0.7	0.0	0.1	2.4
13-07	140.8	140.8	140.8	10.5	10.8	10.4	1.3	1.5	1.1	1.7	1.8	6.7

In general, judged from the MEM values, the 1st mode was the strongest while the 2nd mode ranked second. The 3rd and 4th modes were much weaker in both systems. Such results are consistent with the findings from the white noise cases, indicating that the intrinsic modal properties of the structure system itself did not change much with the activation of the FLCs. Reduction of the floor accelerations should have other explanations.

6.2.3.5 Mode strength evaluated by PSD peak values

Figure 6.10 shows the transition of PSD peak values of the three 100% excitations obtained using the FDD method. Table 6.10 summarizes the corresponding values. The transition of PSD peak values was quite different from the MEM values. The 2nd mode PSD peaks were the largest in Case 04 and decreased drastically in Case 09. The 3rd mode strength increased in Case 09. The 4th mode was almost negligible in both systems and under both ground motions. In Case 13, the 1st mode was generally the strongest while the higher modes were smaller. Such differences suggest that the change in higher mode responses was more related to the excitation properties rather than the change of intrinsic modal properties of the structural system.

Table 6.10 PSD peak values in selected ground motion cases [Unit: $(\text{m/s}^2)^2/\text{Hz}$]

Case-Segment	1st mode	2nd mode	3rd mode	4th mode
04-01	0.00	0.00	0.00	0.00
04-02	20.84	99.76	2.46	1.28
04-03	9.22	46.24	1.70	1.44
04-04	0.00	3.36	0.22	0.13
04-05	14.07	1.78	0.17	0.00
04-06	7.39	0.22	0.09	0.02
04-07	1.28	0.02	0.00	0.00
09-01	0.00	0.00	0.00	0.00
09-02	18.96	28.10	19.28	3.20
09-03	0.00	32.89	30.09	2.32
09-04	2.34	5.55	1.17	0.20
09-05	17.80	6.21	0.35	0.13
09-06	9.37	0.53	0.07	0.04
09-07	2.01	0.02	0.01	0.00
13-01	0.00	0.00	0.00	0.00
13-02	36.14	3.44	5.78	0.50
13-03	19.87	28.49	8.80	2.16
13-04	0.25	0.77	0.39	0.03
13-05	0.46	0.48	0.03	0.01
13-06	2.69	0.15	0.02	0.00
13-07	0.23	0.02	0.00	0.00

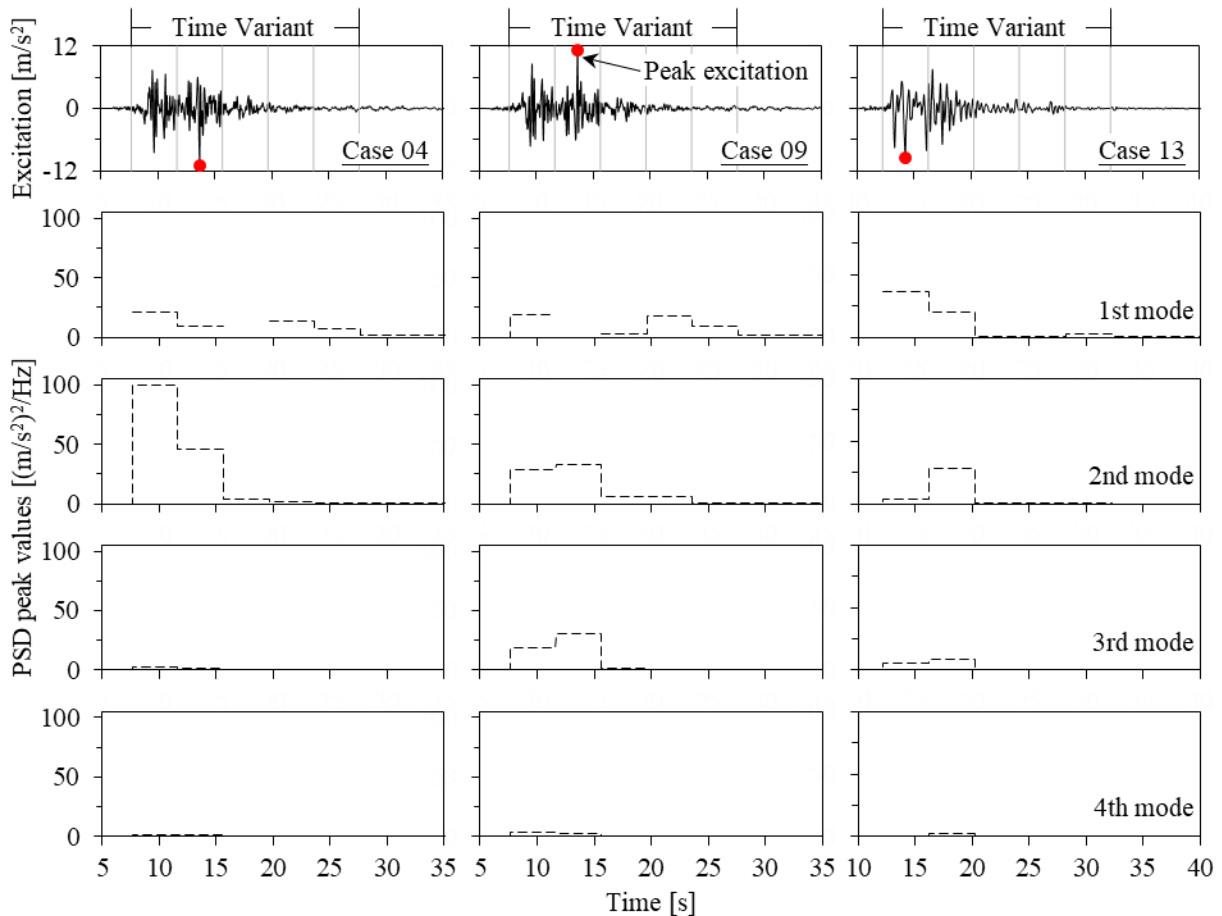


Figure 6.10 PSD peak values in selected ground motion cases

6.2.3.6 Selection of the major estimation method

Considering the reliability and quality of the analysis results, results obtained using the ARX method were selected to form the discussion hereafter for the ground motion excited cases. Using the results from the ARX method also ensures the consistency of data sources with the white noise excited cases. Parameters estimated using the FDD and SS methods will also be used when necessary to provide supplementary information.

6.2.4 Time-frequency properties of ground motion excitations

6.2.4.1 Instantaneous period of the summed response signals

Figure 6.11 shows the instantaneous periods identified using the SSWT method from the three 100% excitations.

The identified 1st mode period showed quite significant variation throughout the whole excitation but without very clear trend. It is difficult to judge whether the variation was due to real changes in structure properties or errors in the identification procedure. The 2nd period stayed quite consistent in Case 04, showing no elongation. Some period elongation can be seen for the 2nd period in the MRF-Spine-FLC system in Case 09 and 13, suggesting some level of yielding when the excitation was strong, which may be introduced by the FLCs and the yielding in the MRF. The 3rd and 4th periods stayed quite stable in all three cases, only that the stable period values increased in Case 09 and 13, which is consistent with the observation made on white noise results. The quite stable 3rd and 4th periods suggested that there were no significant yielding mechanisms corresponding to these two modes in the system even when the FLCs were activated.

The range of axes in the plots are set consistent with those in Figure 6.6 to enable direct comparison. Compared with the results identified using the modal analysis on segmented signals, the transition of 1st mode period identified here did not match well while the higher modes matched acceptably. The difficulties in the 1st mode identification may be due to the relatively low Signal-to-Noise Ratio (SNR) in the low frequency range and the limitation of the methods. More suitable methods need to be considered if further investigation is necessary.

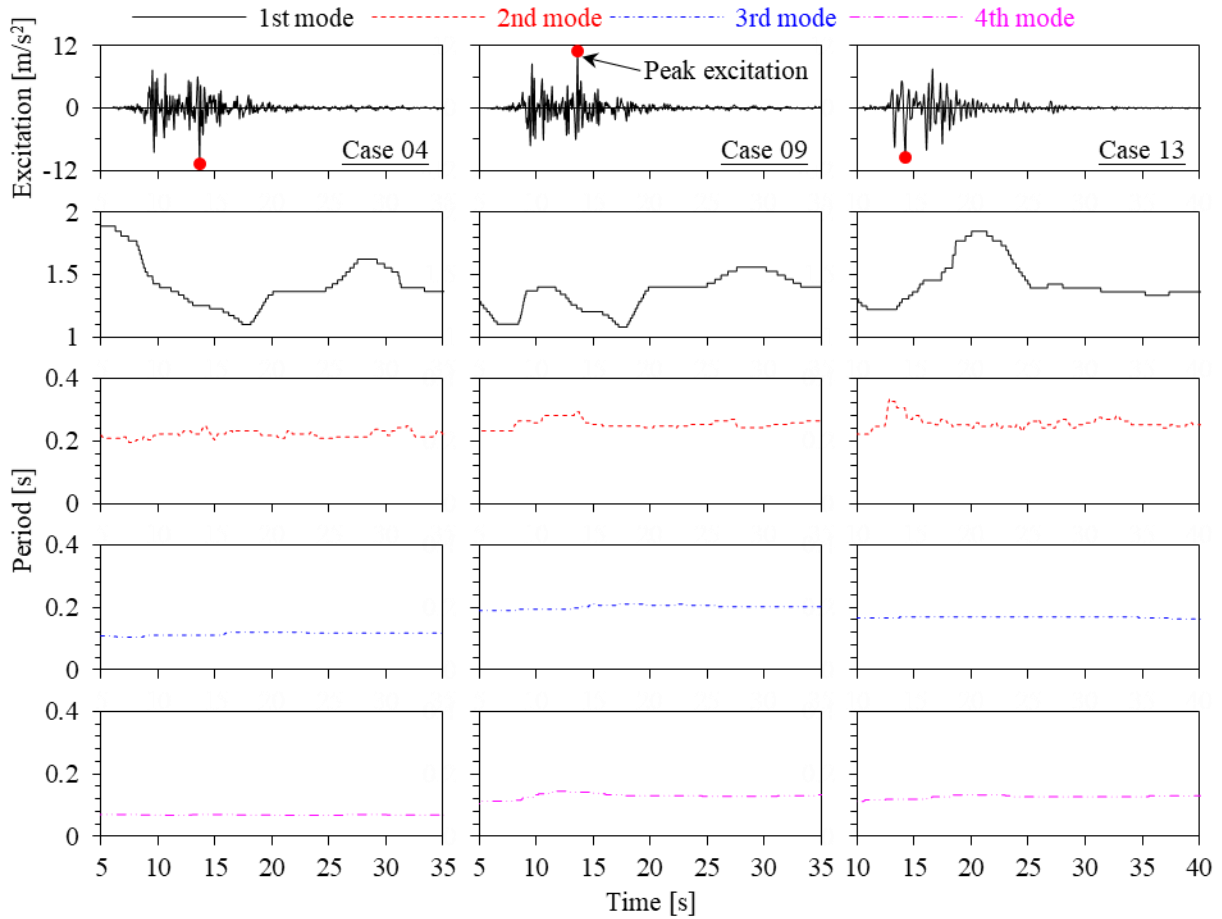


Figure 6.11 Instantaneous periods in selected ground motion cases

6.2.4.2 Magnitude of wavelet coefficients on the ridges

The wavelet coefficients can represent the strength of the signal on the time-frequency plane. Magnitude of the coefficients on the detected ridges, i.e., the wavelet coefficients corresponding to the instantaneous periods discussed in the previous subsection, can be used as a measure of the mode strength in the time-frequency domain. Figure 6.12 shows the magnitude of the ridge wavelet coefficients in the three 100% excitations. Although not physically related, the magnitudes can represent how the energy distributed between different modes and during the excitation procedure.

The 1st mode showed similar maximum magnitudes but different time distribution in all excitations. In Case 04 and 09, the coefficients showed a rather consistent value throughout the whole excitation, while in Case 13 the energy was more concentrated to the time when the excitation was strong. Such difference can be attributed to the different properties of the input excitations. The 2nd mode was the strongest in Case 04 but the magnitude decreased drastically in Case 09, indicating the effectiveness of the FLCs. The magnitude in Case 13 showed similar maximum value as in Case 09, but with more concentration in time. The 3rd mode was almost negligible in Case 04 but increased in Case 09, which can also be attributed to the function of the FLCs. The 3rd mode strength decreased in Case 13 but was still not negligible. In all excitations, the 4th mode was almost negligible.

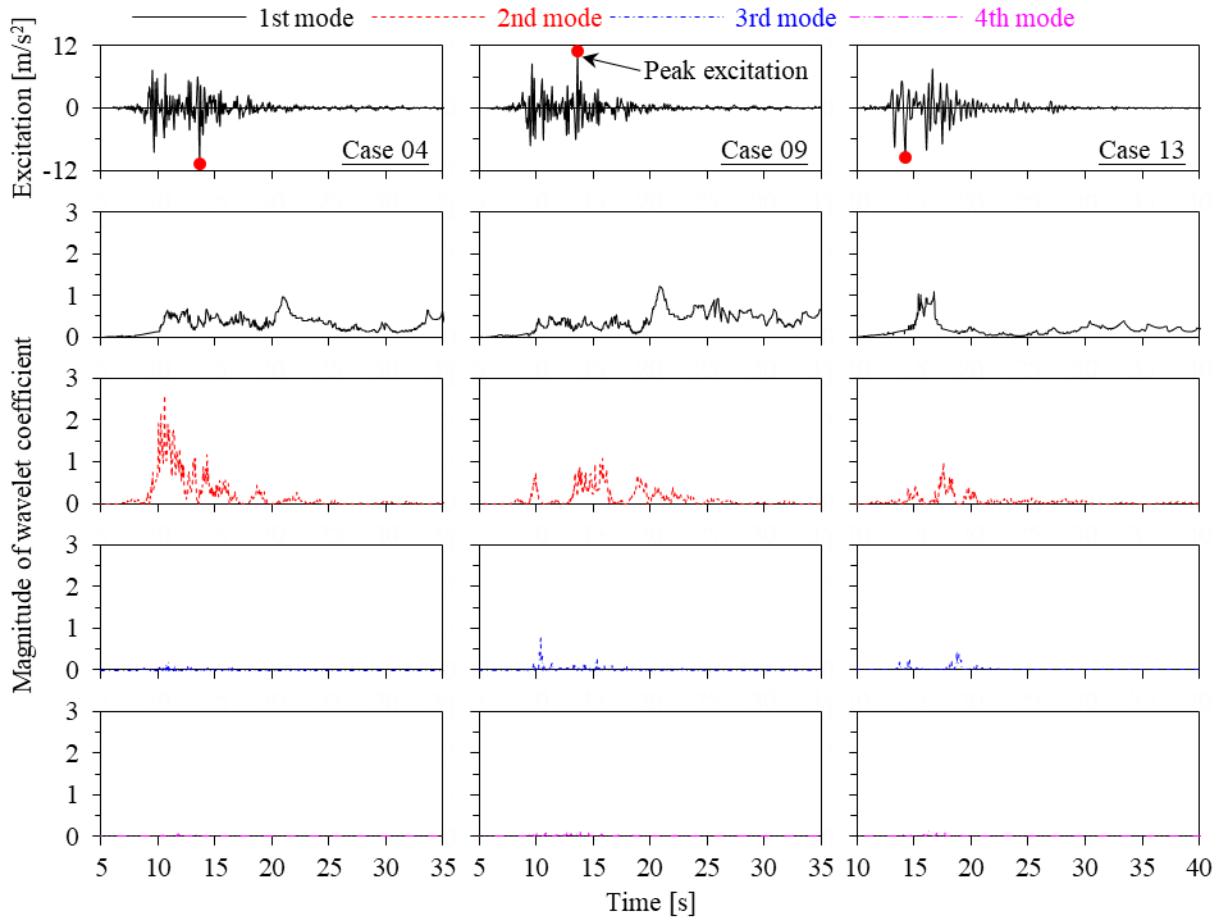


Figure 6.12 Magnitude of ridge wavelet coefficients in selected ground motion cases

In general, the magnitude of the wavelet coefficients roughly matched the PSD peaks obtained using the FDD method. Considering that both methods are output-only, results from here partly support the validity of the application of the FDD method to a ground motion excitation. Also, the difference between “energy-type” indicators (PSD peaks and wavelet coefficient magnitudes) and “mode-shape” indicators (MEMs) suggests the need to further investigate how the FLCs changed the vibration properties.

6.2.4.3 Reconstructed time-domain modal components

As aforementioned in Chapter 5, the SSWT method allows reconstruction of time-domain components using the wavelet coefficients centered along the corresponding ridges on the time-frequency plane. Figure 6.13 shows the reconstructed time-domain components in the 100% excitations.

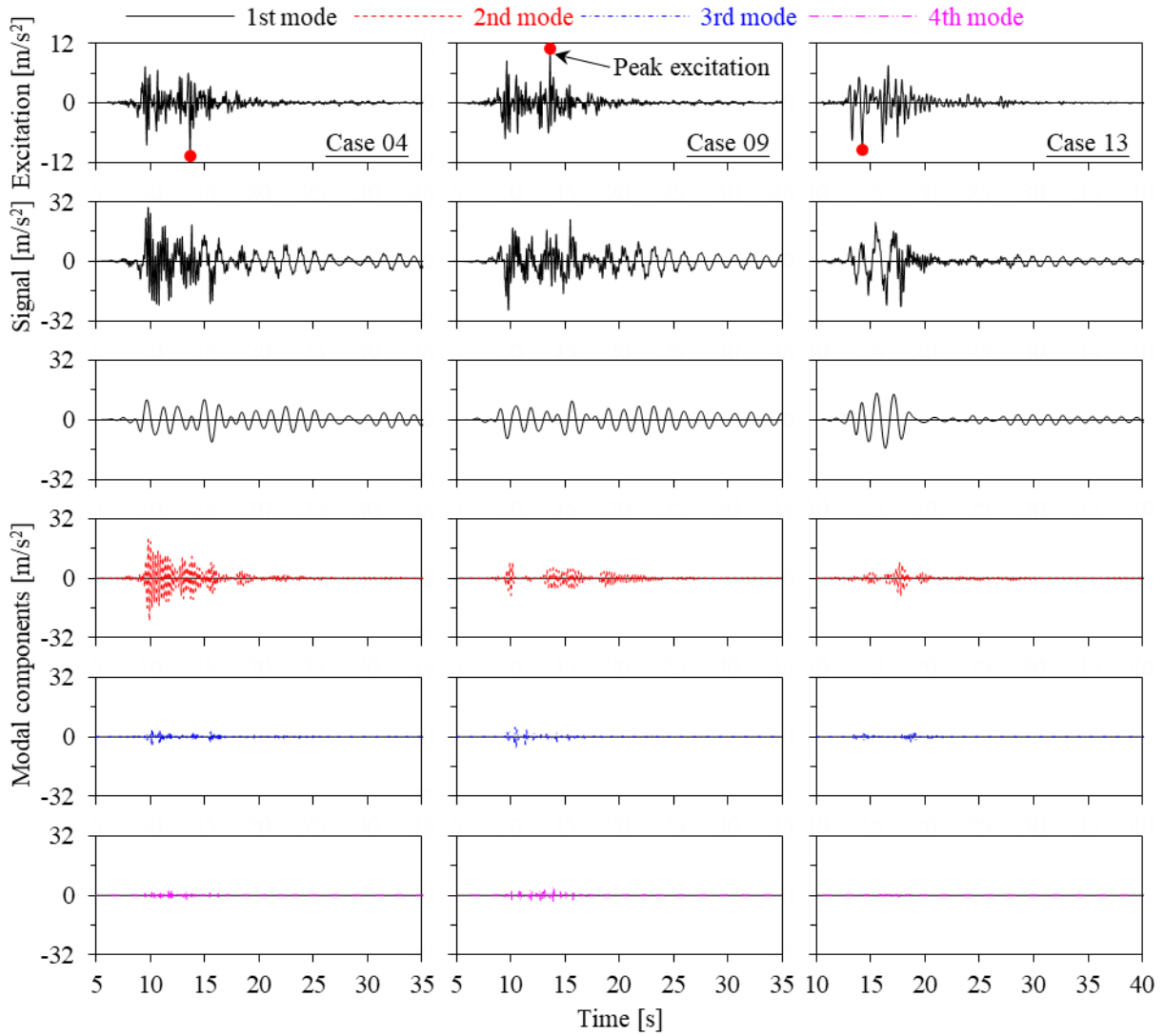


Figure 6.13 Reconstructed modal components in selected ground motion cases

The reconstructed time-domain modal components are counterparts of the wavelet coefficient magnitudes discussed previously. It is a more intuitive way to illustrate which component was stronger in the analyzed signal. Similar to the observations made on the wavelet coefficient magnitudes, the 2nd mode was the strongest in Case 04 and decreased dramatically in Case 09 and 13. The 1st mode stayed with similar strength in Case 04 and 09 but increased slightly in Case 13, which can be attributed to the influence of excitations. Unlike the magnitudes, the 3rd mode did not show significant increase from Case 04 to Case 09, but did show slightly smaller value in Case 13. The 4th mode stayed almost negligible in all cases, as expected.

In order to quantify the strength of each modal component, the Root Mean Square (RMS) defined as follows can be used. This parameter vanishes the time information but can represent the energy of the whole signal.

$$X_{RMS} = \sqrt{\frac{\sum_1^N |x_n|^2}{N}} \quad (6.4)$$

Where,

x_n represents one element in a signal.

N is the number of elements in the signal.

Table 6.11 summarizes the RMS of the whole signal and each modal components in all six major excitations. The variation of the RMS is generally consistent with the plotted signals in Figure 6.13. The 2nd mode, regardless of the system condition (elastic or post-elastic), was generally stronger under the Northridge VA excitation than the JMA Kobe excitation relative to the 1st mode RMS. Reduction of the 2nd mode can be observed in both elastic (Case 02 and 07) and post-elastic (Case 04 and 09) systems, with latter being more significant. For the two cases under the JMA Kobe motion (Case 11 and 13), although the excitation strength was doubled (50 % to 100 %), the 1st mode RMS did not increase so much, which can be attributed to the distributed yielding of the MRF that capped the increase of 1st mode response. Increase of the 2nd and 3rd mode RMS was more significant than the 1st mode, suggesting that it is the nature of the structure system that higher modes are easier to be excited, even with the FLCs activated and under a ground motion not strong near the higher mode periods.

Table 6.11 RMS of the modal components in major excitations [Unit: m/s²]

Singal type	Case02	Case04	Case07	Case09	Case11	Case13
1st mode	1.82	2.78	1.34	2.83	2.25	2.47
2nd mode	0.92	2.49	0.56	1.11	0.26	0.78
3rd mode	0.08	0.35	0.12	0.44	0.07	0.24
4th mode	0.03	0.18	0.05	0.34	0.07	0.08

In general, the reconstructed modal components provided a time-domain point of view on the strength of each mode in the analyzed signal. Since the signal was a linear summation of the upper floor responses, it can represent the behavior of the whole structure. Combined with the PSD peaks obtained by the FDD method and the wavelet coefficient magnitudes, the transition of mode strength in the responses can be assured from time domain, frequency domain, and time-frequency domain. But how this was achieved still requires more investigation, especially when the mode-shape related indicator, the Modal Effective Mass, did not show such a significant transition.

6.2.5 Spectral properties of ground motion excitations

As introduced in Chapter 5, the Effective Modal Pseudo-Acceleration (EMPA) can also illustrate the mode strength in the spectral response point of view. Figure 6.14 shows the peak EMPA values and pseudo-acceleration spectra with 2% damping of the excitations in the three 100% excitation cases. The “nonlinear” values were calculated using the algorithm introduced in Chapter 5, while the “linear”

values were magnified from the smaller excitations in which the structure was believed to be essential elastic.

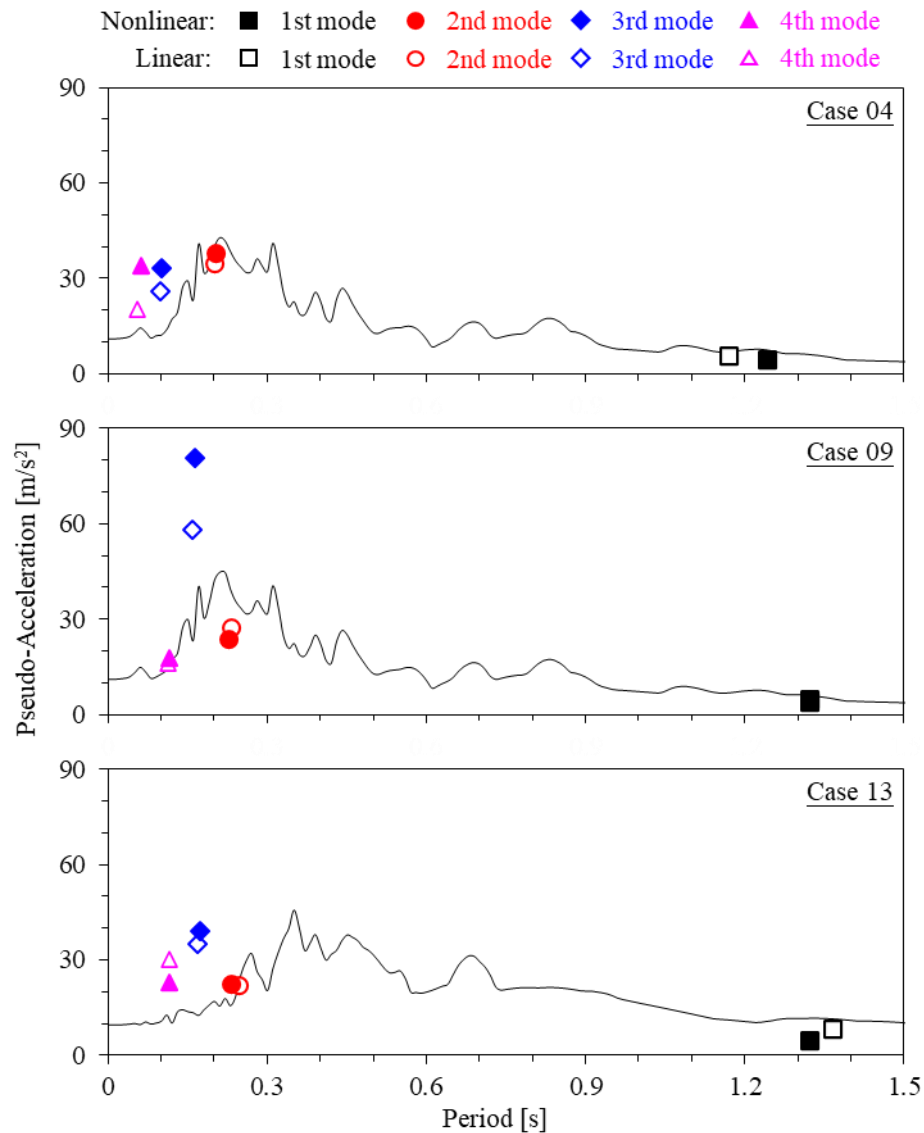


Figure 6.14 Peak EMPA and pseudo-acceleration spectra in selected ground motion cases

In the MRF-Spine system under the Northridge VA excitation (Case 04), the nonlinear EMPAs of higher modes were close to the spectral lines and even larger than the linear responses, indicating that more higher modes were excited in the system, and they were essentially elastic in large excitations. EMPA of the 1st mode was close for both linear and nonlinear responses, indicating that the 1st mode was not affected by the spines.

When the FLCs were activated (Case 09), the 3rd mode was excited significantly large, with the nonlinear response even larger. The 2nd and 4th modes decreased greatly compared to the MRF-Spine system and the linear and nonlinear responses stayed closer, indicating that the 2nd and 4th modes were controlled by the FLCs but the 3rd mode was not. The 1st mode EMPA stayed almost unchanged

compared to the MRF-Spine system, indicating that FLCs did not compromise the ability of the spines to control the 1st mode response.

For the MRF-Spine-FLC system under JMA Kobe ground motions (Case 13), the relationship between linear and nonlinear EMPAs was similar to that under the JMA Kobe motion. Difference between the linear and nonlinear responses was smaller, partly attributed to the developed yielding of the MRF. EMPAs of the 3rd mode decreased for both linear and nonlinear responses, indicating the influence of ground motions on modal responses.

In general, the mode strength illustrated using the peak EMPA values was consistent with those identified using the modal analysis methods and the time-frequency method. The 2nd mode was controlled in the responses but the unexpected 3rd mode was excited when the FLCs were activated. When the ground motion changed, the higher modes were not activated as much. Still, the inconsistency between these indicators and the MEMs need further investigation.

6.2.6 Closure

This section comprehensively introduced the analysis results on the vibration properties of the 20EDF test. For the modal analysis methods, all three methods gave consistent results for white noise cases but not for ground motion cases. The ARX method was considered more reliable and selected as the major method to provide data for further discussion. Results from the FDD and SS methods will be used when necessary. In addition to the modal analysis methods, results from the SSWT and EMPA methods also gave reasonable results on the modal property transition in the system, which was generally consistent with the modal analysis results. One major inconsistency between the methods was the modal strength evaluated using the Modal Effective Mass (MEM) and the other indicators. This will be further discussed in the following sections. In general, the selected methods worked as expected to unveil the vibration properties of the structure system and can provide solid data foundation for further discussion. Although not shown here, results from other tests showed generally similar effectiveness.

6.3 DISCUSSION ON THE ANALYSIS RESULTS

6.3.1 Effectiveness of the FLCs

As shown in Figure 6.5, in the white noise cases, the mode strength represented by PSD peaks showed decrease in the 2nd mode and increase in the 1st mode, indicating that the FLCs slightly controlled the 2nd mode response and did not compromise the function of the spines to enforce the 1st mode deformation in the system. In the ground motion excitations (Case 04 and 09), it is clear that the 2nd mode response decreased after the FLCs were activated. The reduction can be seen from PSD peaks in frequency domain (Figure 6.10), decomposed modal components in time domain (Figure 6.13), magnitude of wavelet coefficients in time-frequency domain (Figure 6.12), and EMPA (Effective Modal

Pseudo-Acceleration) in spectral responses (Figure 6.14). Although not shown here, similar reduction can also be observed in the smaller excitations (Case 02 and 07), but not as significant as in Case 04 and 09. The above observations indicate that the FLCs functioned effectively to reduce the 2nd mode response and thus reduced the whole floor acceleration responses as shown in Chapter 5.

Although the decreased 2nd mode response was observed in different domains, the MEM (Modal Effective Mass) of the system did not change as much. In fact, the 2nd mode response was stronger than the 1st mode in Case 04, but the MEM of the 1st mode was still the largest. Such inconsistency suggests that the FLCs functioned not by changing the intrinsic modal strength of the system but through other routes. It is believed that the FLCs functioned by shifting the modal period which will change the strength of the excitation. The elongated periods may reduce the excitation strength corresponding to the 2nd mode and thus the response was also reduced. Similarly, the increase in 3rd mode response in Case 09 can also be attributed to the increased excitation strength when the modal period elongated. The influence of the excitation properties will be discussed in 6.3.3.

In general, the FLCs worked effectively to control the 2nd mode response which resulted in the reduction of peak floor acceleration in the total responses. The function of the FLCs were achieved mainly by elongation of the modal periods instead of changing the mode shape and intrinsic mode strength.

6.3.2 Influence of the spines

Due to the lack of test results on pure MRF in the 20EDF test, results from the two MRF tests, the 21TKB and 07EDF tests, are used to form the comparison and discussion on the influence of spines.

6.3.2.1 Influence on white noise responses

Figure 6.15 shows the transition of natural periods, MEMs and PSD peaks of each mode of the 21TKB and 07EDF tests. The periods and MEMs were from the ARX method, while the PSD peaks from the FDD method. Note that only representative white noise cases are shown for the 07EDF test. Natural periods of all modes in both tests stayed quite consistent throughout the whole test, which can be attributed to the relatively uniform distribution of member yielding and that the white noise cases were conducted prior to the destructive large ground motion excitation. For the modal responses represented by PSD peaks, the 1st mode was dominant for both tests. The relatively smaller PSD peak in Case 07 of the 21TKB test was due to the incomplete data. The larger PSD peaks in Case 02, 18 and 20 of the 07EDF test were due to the larger excitation level (1.8 m/s^2 for Case 02, 1 m/s^2 for Case 06/08/10/14, 2 m/s^2 for Case 18/20). For the MEMs, the values were quite consistent in all the cases of both tests, indicating that the mode shapes and intrinsic mode strength did not change much during the test. Dominance of the 1st mode can also be observed in the MEMs.

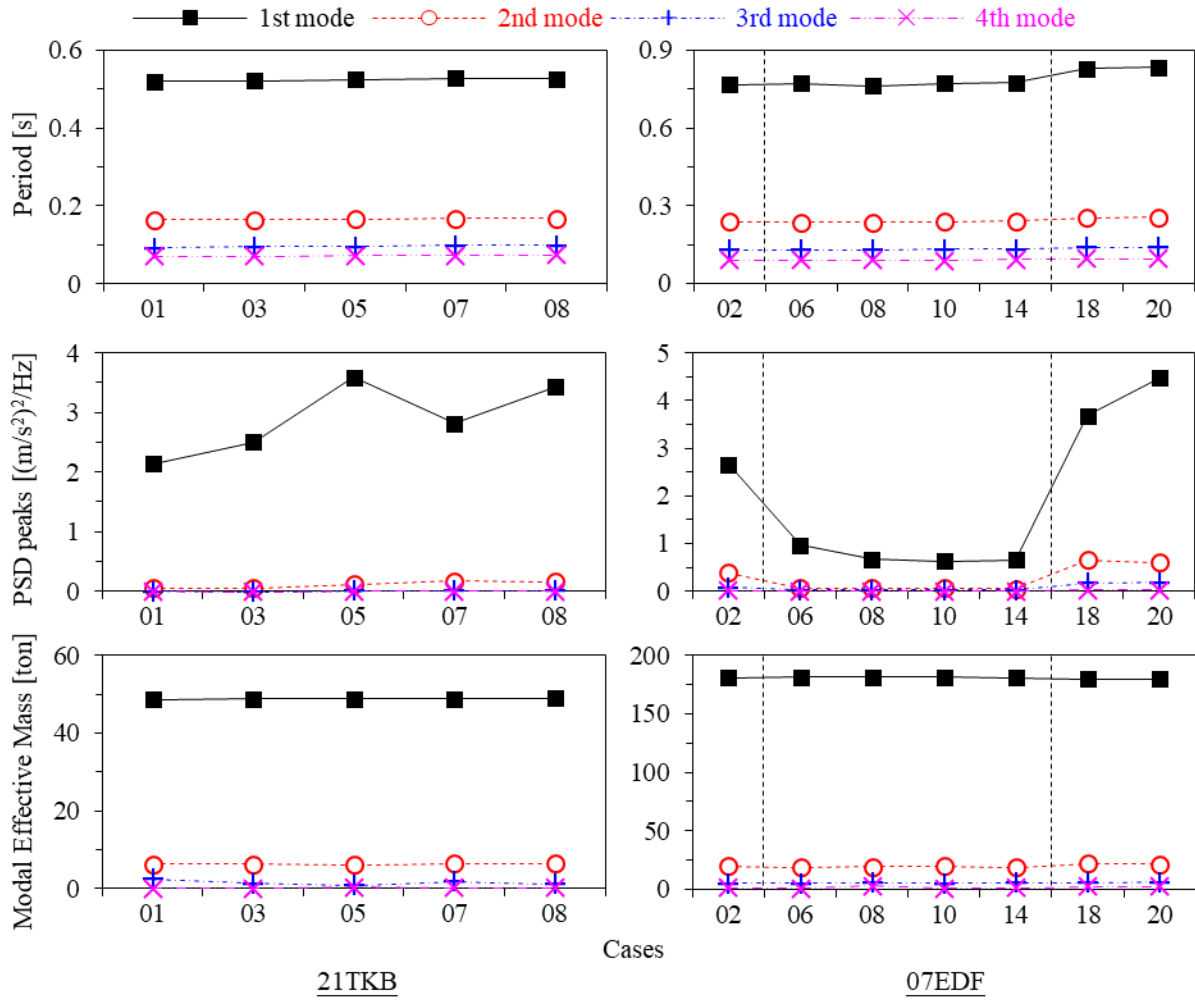


Figure 6.15 Modal parameters in selected white noise excitations of the 21TKB and 07EDF tests

Considering the different absolute PSD peaks of different tests, the values of the higher modes were normalized to the 1st mode to form the comparison between the 20EDF, 21TKB and 07EDF tests. Table 6.12 summarizes the PSD peaks normalized to the 1st mode of the three tests. It can be seen that there was no very significant differences in the higher mode strength in the three tests, indicating that the spine did not cause strong higher mode responses in the white noise cases, which is consistent with previous work that higher modes are not a problem in elastic stage in spine systems.

Table 6.12 Normalized PSD peaks of the selected tests in white noise excitations

Test	Case	1st mode	2nd mode	3rd mode	4th mode	Note
20EDF	01	1.00	0.08	0.30	0.06	MRF-Spine
	03	1.00	0.14	0.20	0.01	
	05	1.00	0.10	0.04	0.01	
	06	1.00	0.05	0.00	0.02	MRF-Spine- FLC
	08	1.00	0.03	0.00	0.02	
	10	1.00	0.02	0.01	0.03	
	12	1.00	0.04	0.01	0.01	
	14	1.00	0.04	0.01	0.03	
21TKB	01	1.00	0.03	0.00	0.00	
	03	1.00	0.02	0.00	0.00	
	05	1.00	0.03	0.00	0.00	
	07	1.00	0.06	0.00	0.00	
	08	1.00	0.05	0.00	0.00	
07EDF	02	1.00	0.15	0.03	0.01	1.8 m/s ²
	06	1.00	0.07	0.03	0.00	1 m/s ²
	08	1.00	0.09	0.04	0.00	
	10	1.00	0.10	0.03	0.01	
	14	1.00	0.08	0.04	0.01	
	18	1.00	0.18	0.05	0.01	2 m/s ²
	20	1.00	0.13	0.04	0.00	

As aforementioned, the PSD peaks represent the mode strength of responses while the MEMs represent the intrinsic properties. Since the specimens in these tests have very different mass, the ratio of the MEM to the total mass of the structure, termed as Modal Effective Mass Ratio (MEMR) hereafter, was used to compare different tests. Table 6.13 summarizes the MEMR of the 20EDF, 21TKB and 07EDF tests in white noise cases. From the MEMRs, it is seen that, similar to the PSD peaks, there were no significant differences in higher mode strength in the system, which was reasonable considering the nature of multi-story building structures with uniform story stiffness. In addition, the 20EDF test even showed larger 1st mode MEMR in both MRF-Spine and MRF-Spine-FLC systems than the MRF systems in 21TKB and 07EDF tests. Considering that mode shapes are a concept in deformation, the larger 1st mode MEMRs indicate that the spines enforced the 1st mode deformation pattern in the system, and the negligible variation between the systems with and without FLCs indicates that activation of FLCs did not compromise the spines function in doing so.

In general, results from the white noise excitation indicate that, in elastic cases, the higher modes in the spine systems were not significantly stronger than in the pure MRF systems. In addition, the spines forced the 1st mode deformation to the MRF and such function was not compromised by the FLCs.

Table 6.13 MEMRs of the selected tests in white noise excitations

Test	Case	1st mode	2nd mode	3rd mode	4th mode	Note
20EDF	01	0.91	0.06	0.01	0.02	MRF-Spine
	03	0.91	0.07	0.01	0.01	
	05	0.91	0.07	0.01	0.01	
	06	0.92	0.07	0.01	0.01	MRF-Spine- FLC
	08	0.92	0.07	0.01	0.01	
	10	0.92	0.07	0.01	0.01	
	12	0.92	0.06	0.01	0.01	
	14	0.92	0.07	0.01	0.01	
21TKB	01	0.87	0.11	0.04	0.00	
	03	0.87	0.11	0.02	0.00	
	05	0.87	0.11	0.02	0.01	
	07	0.87	0.12	0.03	0.00	
	08	0.87	0.11	0.02	0.00	
07EDF	02	0.87	0.09	0.02	0.01	1.8 m/s ²
	06	0.87	0.09	0.02	0.00	1 m/s ²
	08	0.87	0.09	0.02	0.01	
	10	0.87	0.09	0.02	0.00	
	14	0.87	0.09	0.02	0.00	
	18	0.86	0.10	0.03	0.01	2 m/s ²
	20	0.86	0.10	0.03	0.01	

6.3.2.2 Influence on ground motion responses

In order to form the discussion on ground motion responses, Case 13 of the 20EDF test with 100% JMA Kobe excitation, Case 12 of the 21TKB test with 100% JMA Kobe excitation, and Case 17 of the 07EDF test with the 60% JR Takatori excitation, were selected. Case 12 of the 21TKB test was the last ground motion excitation in the test in which major damage occurred. Case 17 of the 07EDF test was the second largest excitation in the test. The 100% excitation of the 07EDF test was not selected because collapse happened in that case and the recorded responses were incomplete. Although the results of Case 13 in the 20EDF test have been shown in previous sections, they are still included in the plots here to form direct comparison. In addition, considering the purpose of the discussion, only results related to mode strength are shown.

Figure 6.16 shows the MEMRs (Modal Effective Mass Ratio) and PSD peaks using segmented data. The MEMRs were calculated using mode shapes obtained from ARX method while the PSD peaks were from the FDD method. The MEMRs were selected to form comparison between different tests. The PSD peaks were not normalized to the 1st mode because the 1st mode was not identified in all segments. From the MEMRs, it is obvious that the 1st mode was dominant in all tests. The rather consistent MEMR values indicate that the intrinsic mode strength did not vary much during the whole shaking. Unlike the MEMRs, the PSD peak values which represents the mode strength in structure responses showed quite different variation trends in the three cases. In Case 13 of the 20EDF test, the 1st mode was stronger in

the PGA segment, but the 2nd mode increased afterwards. With the same input, Case 12 of the 21TKB test showed very different results with the 1st mode being significantly larger than all the other modes. Difference between the 20EDF and 21TKB tests suggests that higher modes in the system with spines are easier to be excited, even when the excitation was not strong near the higher mode periods. Case 17 of the 07EDF showed similar 2nd mode strength to the 21TKB test but much smaller 1st mode strength, which was attributed to the differences of the excitation properties, considering that the PSD peaks were evaluated using the FDD method that did not exclude the influence of the excitation.

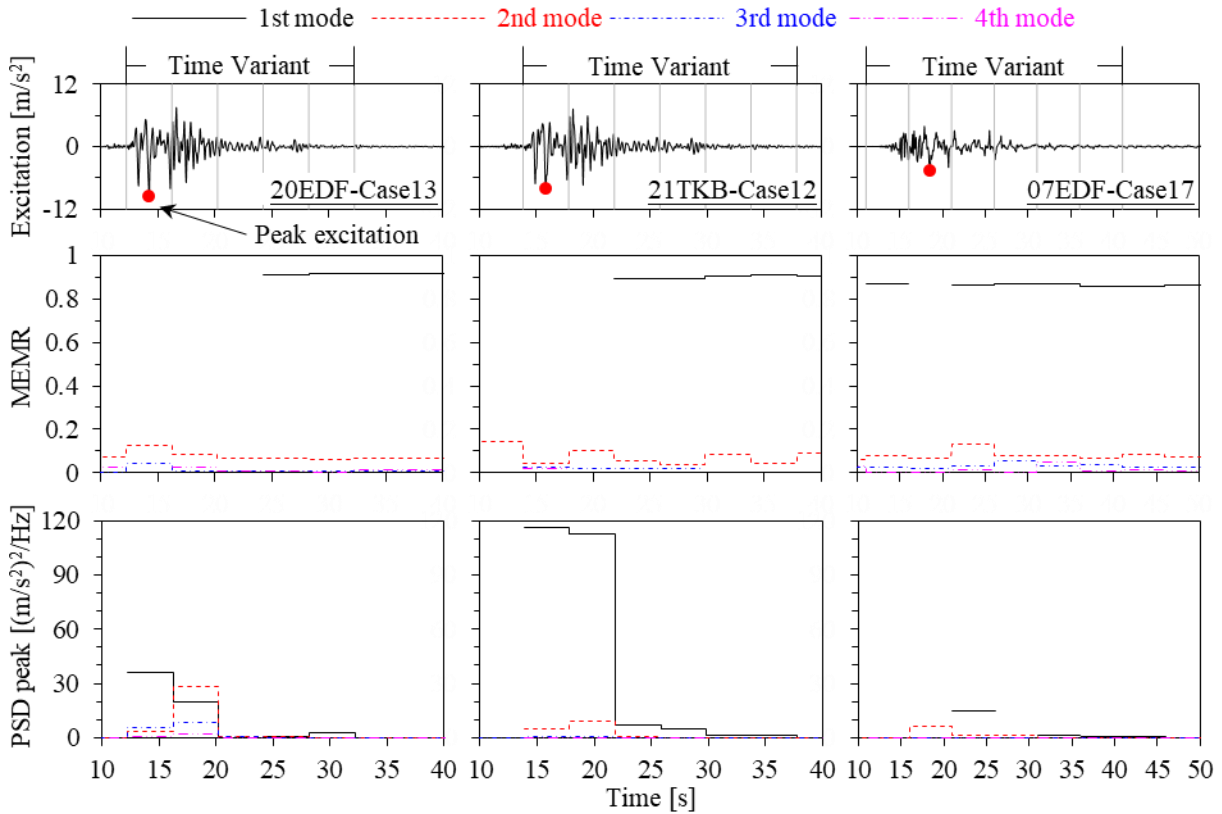


Figure 6.16 MEMR and PSD peaks from selected tests and cases

As another point of view, results from the SSWT analysis can also provide information of the mode strength variation in the structure during a case. Figure 6.17 shows the reconstructed time-domain modal components of the selected cases. RMS values of each component are also provided to quantify the overall strength of each mode. For all three cases, the 1st mode was still the strongest in general. Comparing results from the 20EDF and 21TKB tests, it is clear that the 20EDF test gave relatively stronger 2nd and 3rd mode components, while the 1st mode was significantly stronger in the 21TKB test. Although the excitation was different, the 07EDF test also gave much stronger 1st mode compared to the 20EDF test. The strength of the modal components was consistent with the PSD peaks shown in Figure 6.16. Although not shown here, the transition of magnitude of ridge wavelet coefficients was consistent with that of the modal components.

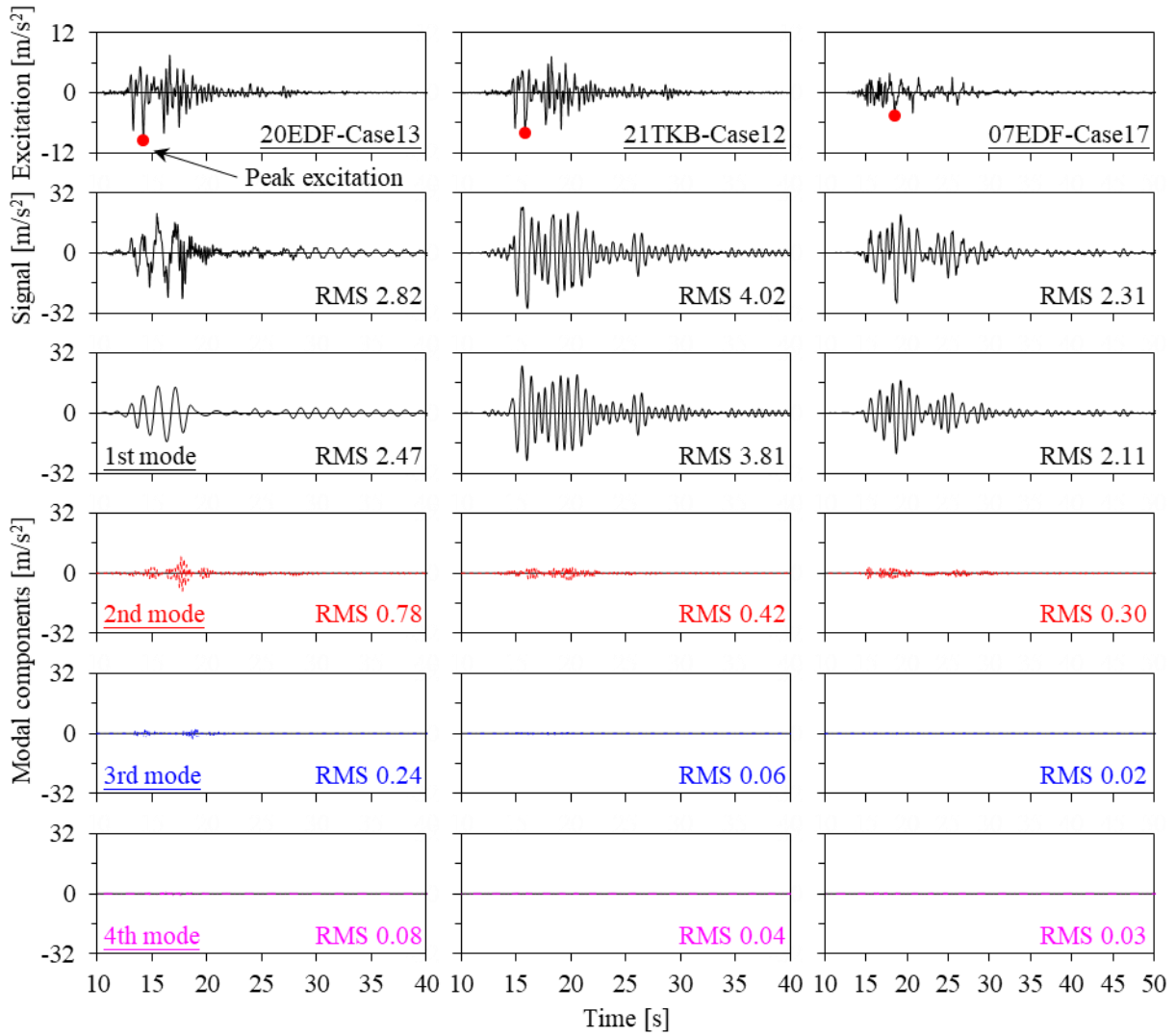


Figure 6.17 Reconstructed modal components from selected tests and cases

In all, the results from ground motion excitations suggest that higher modes are easier to be excited in the system with spines than pure MRF systems, even when the excitation is not strong near the higher mode periods. The stronger higher mode responses may be more prone in post-elastic cases.

6.3.2.3 Closure

From the above results from both white noise and ground motion excitations, influence of the spines can be observed. Although the existence of the spines did not increase the intrinsic strength of the higher modes, they may make the higher modes easier to be excited compared with pure MRF systems, especially in post-elastic cases. The excited higher modes may still be smaller than the 1st mode, but the increased higher mode strength may still exceed the considerations in existing design codes or methods.

6.3.3 Influence of the excitation properties

From the results of the 20EDF test, it can be seen that although the intrinsic mode strength represented by MEMs did not vary significantly, the modal responses represented by PSD peaks, magnitude of ridge

wavelet coefficients and reconstructed modal components all changed significantly. For both the MRF-Spine and MRF-Spine-FLC systems, when excited by the Northridge VA 100% excitation which was strong near the 2nd mode period, the 2nd mode was excited much stronger in the responses than the intrinsic mode strength. In the MRF-Spine-FLC system, when the excitation became the JMA Kobe 100% which was not strong near the 2nd mode period, the 2nd mode response decreased compared to the responses under the Northridge VA 100% excitation. The change in the strength of different modal responses clearly shows the influence of excitation properties.

In addition, the effectiveness of the FLCs, as introduced in Section 6.3.1, may also be partly attributed to the excitation properties. As introduced previously, the mode shapes and corresponding MEMs did not change significantly after the activation of the FLCs, but the mode strength in the responses changed greatly. This can be attributed to the period shift which, due to the shape of the response spectrum of the Northridge VA excitation, changed the excitation intensity greatly. As shown in Figure 6.14, when the FLCs were activated, the 2nd mode period shifted to a low Pseudo-Acceleration Spectrum (PSA) value but the 3rd mode period shifted to a much larger value, which is consistent with the observation of the decreased 2nd mode strength and increased 3rd mode strength.

In order to further look into this phenomenon, PSAs of the Northridge VA and JMA Kobe excitations with 2% damping with the 2nd and 3rd periods are plotted, as shown in Figure 6.18. Periods of Case 01 and 06 were extracted using the ARX method, representing the initial periods of the MRF-Spine and MRF-Spine-FLC systems, respectively. Periods of Case 04, 09 and 13 were the instantaneous periods at the time instance of peak excitation obtained using the SSWT method. With the Northridge VA 100% excitation, when the FLCs were activated, the 2nd mode PSA decreased from around 42 m/s^2 to 34 m/s^2 (Figure 6.18a), while the 3rd mode PSA increased from around 11 m/s^2 to 40 m/s^2 (Figure 6.18b). This change in excitation intensity can explain the decreased 2nd mode strength and increased 3rd mode strength in the acceleration responses discussed previously. With the JMA Kobe 100% excitation, although the 2nd and 3rd periods also elongated compared to the initial conditions (Case 01 and 06), the excitation intensity did not change as much as the Northridge VA excitation (Figure 6.18cd), resulting in the less variation of mode strength in Case 13, as introduced previously.

From the results shown in Figure 6.14 and Figure 6.18, it can be inferred that the effectiveness of the FLCs in the test may be strongly related to the excitation properties, especially with the changed excitation intensity related to the elongated periods. The large 2nd mode intensity of the Northridge VA excitation was deliberately decided to excite more 2nd mode responses, which was successfully realized in the test. But for the reduced responses in Case 09, the reduced 2nd mode PSA was not selected but a natural result. The increased 3rd mode response was also not expected. Although the FLCs “happened to” function well in the 20EDF test, it may not be the case if the excitation was changed. Considering the stochastic nature of ground motions and earthquakes, the robustness of the design of the MRF-Spine-

FLC systems still requires more investigation.

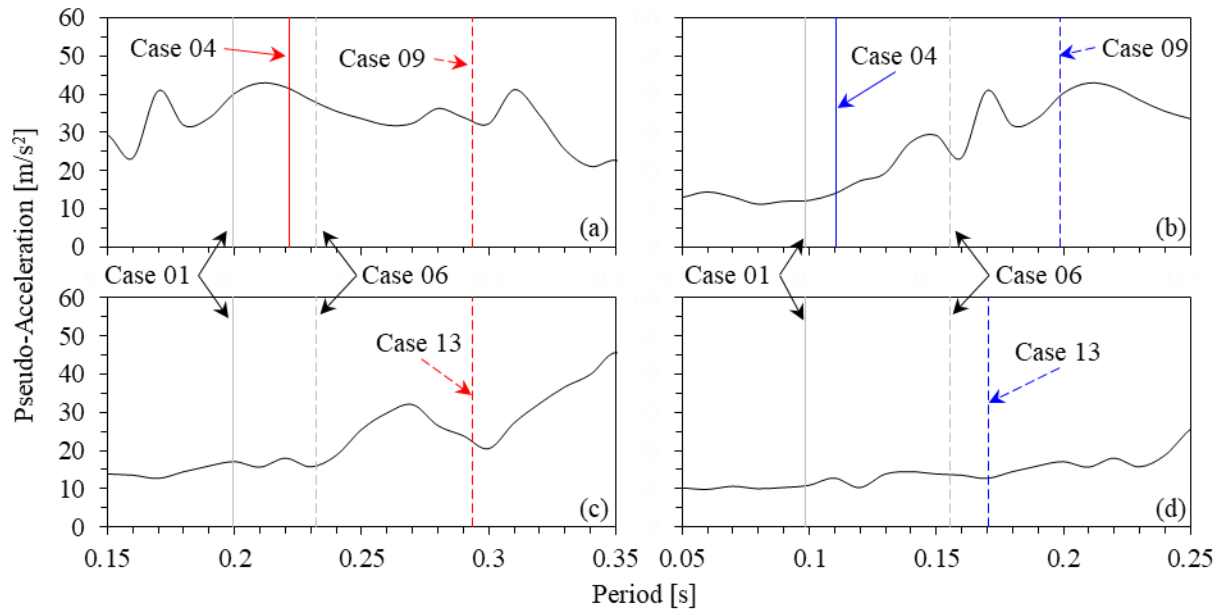


Figure 6.18 Pseudo-acceleration spectra and periods in the 20EDF test Northridge VA 100%: (a) 2nd period; (b) 3rd period. JMA Kobe 100%: (c) 2nd period; (d) 3rd period.

6.3.4 Influence of number of stories

As a common understanding, high-rise buildings may excite stronger higher modes in earthquakes. It is of interest whether the higher mode responses in spine systems are similar to or different from those in the high-rise buildings. Results from the 08EDF test were used to form the discussion. As introduced in Chapter 5, although the specimen of the 08EDF was a 7 story one, its dynamic properties represented a 20-story high-rise residential building.

6.3.4.1 Influence on white noise responses

Figure 6.19 shows the transition of natural periods, PSD peaks and MEMs in selected white noise excitations of the 08EDF test. The periods and MEMs were from the ARX method while PSD peaks from the FDD method. For the natural periods, with the yielding developing in the system, periods of all modes gradually increased, with the 1st mode showing the most obvious increase. The transition of natural periods was similar to that of the low-rise MRFs shown previously (21TKB and 07EDF). For the mode strength in the responses represented by PSD peaks, the 1st mode was the strongest which was the same as the low rise MRF structures (21TKB and 07EDF), but the 5th and 6th modes showed larger strength than the 3rd and 4th modes, and was even stronger than the 2nd mode in the Case 02. This is different from the MEMs which decreased from the 1st mode to the 7th mode. The relative strength of MEMs indicating that the high-rise MRF is still an MRF whose intrinsic mode strength was similar to that of the low-rise ones. But the larger 5th and 6th PSD peaks suggest that higher modes are easier to be excited, as expected.

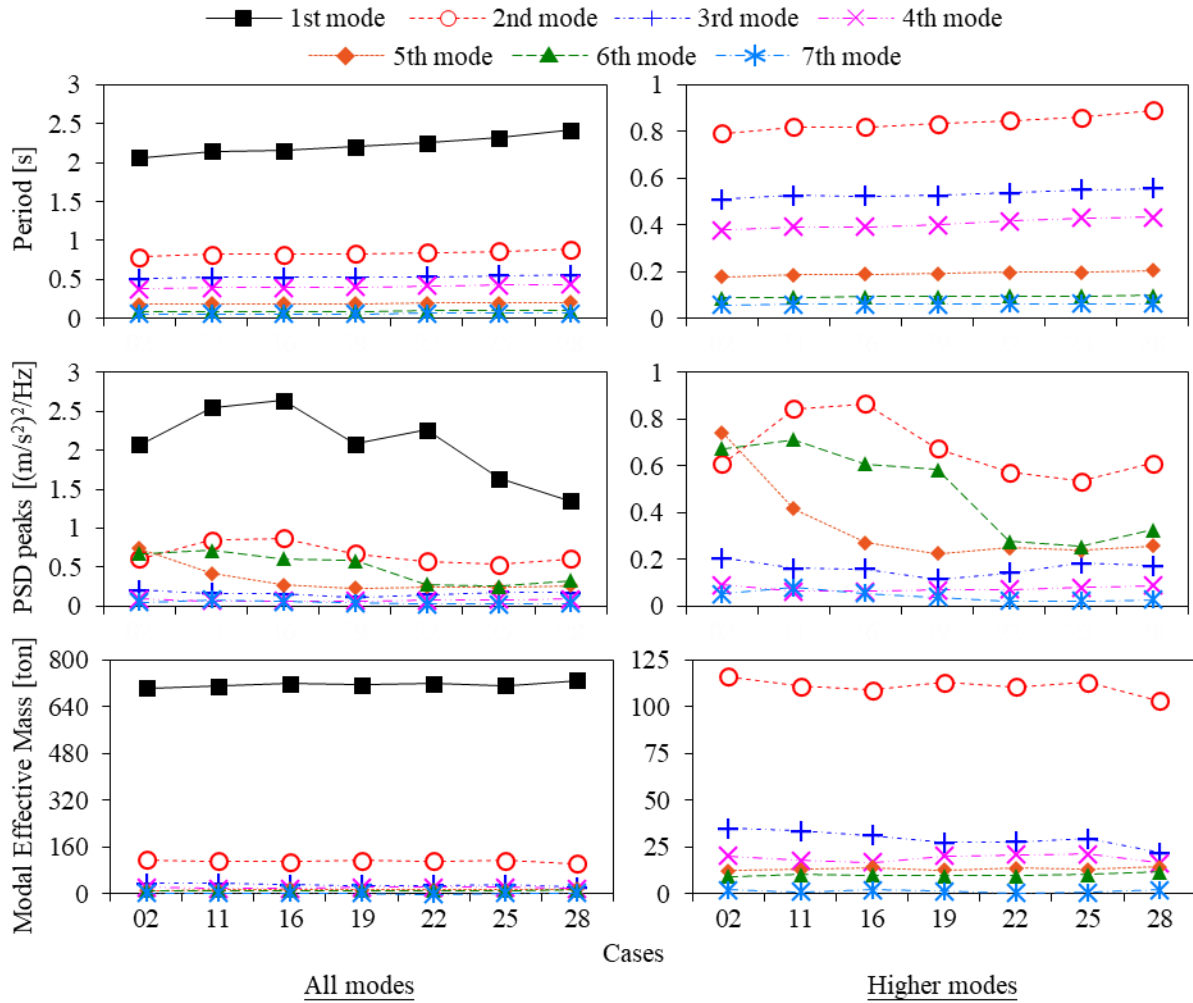


Figure 6.19 Modal parameters in selected white noise excitations of the 08EDF test

Similar to the analysis on the 21TKB and 07EDF tests, PSD peaks normalized to the 1st mode and MEMRs (Modal Effective Mass Ratio) are used to form comparison between the 08EDF and 20EDF tests. Table 6.14 and Table 6.15 summarize the normalized PSD peaks and MEMRs, respectively. The results of the 20EDF test are included to provide easier comparison.

For the PSD peaks, similar to the results shown in previous plots, the 08EDF test showed an even stronger 2nd mode relative to the 1st mode when compared to the 20EDF test. The 5th and 6th modes were also relatively strong. Results here proved that higher modes may be more prevalent in high-rise buildings, and even stronger than the low-rise spine systems. For the MEMRs, the 1st mode was relatively weaker but the 2nd mode stronger in the 08EDF test than the 20EDF test, which is similar to the results of the low-rise MRFs (21TKB and 07EDF). Such results indicate again that the spines forced the 1st mode deformation pattern in the system which may not be the case in “free” MRF systems. In addition, the PSD peaks and MEMRs suggest that the stronger higher mode responses in the high-rise MRFs may be more intrinsic but more forced in the spine systems.

Table 6.14 Normalized PSD peaks in selected white noise excitations of the 08EDF test

Test	Case	Modes							Note
		1st	2nd	3rd	4th	5th	6th	7th	
20EDF	01	1.00	0.08	0.30	0.06				MRF-Spine
	03	1.00	0.14	0.20	0.01				
	05	1.00	0.10	0.04	0.01				
	06	1.00	0.05	0.00	0.02				MRF-Spine-FLC
	08	1.00	0.03	0.00	0.02				
	10	1.00	0.02	0.01	0.03				
	12	1.00	0.04	0.01	0.01				
	14	1.00	0.04	0.01	0.03				
08EDF	02	1.00	0.29	0.10	0.04	0.36	0.33	0.02	
	11	1.00	0.33	0.06	0.02	0.16	0.28	0.03	
	16	1.00	0.33	0.06	0.02	0.10	0.23	0.02	
	19	1.00	0.32	0.06	0.03	0.11	0.28	0.02	
	22	1.00	0.25	0.06	0.03	0.11	0.12	0.01	
	25	1.00	0.33	0.11	0.05	0.15	0.16	0.01	

Table 6.15 MEMRs in selected white noise excitations of the 08EDF test

Test	Case	Modes							Note
		1st	2nd	3rd	4th	5th	6th	7th	
20EDF	01	0.91	0.06	0.01	0.02				MRF-Spine
	03	0.91	0.07	0.01	0.01				
	05	0.91	0.07	0.01	0.01				
	06	0.92	0.07	0.01	0.01				MRF-Spine-FLC
	08	0.92	0.07	0.01	0.01				
	10	0.92	0.07	0.01	0.01				
	12	0.92	0.06	0.01	0.01				
	14	0.92	0.07	0.01	0.01				
08EDF	02	0.78	0.13	0.04	0.02	0.01	0.01	0.00	
	11	0.79	0.12	0.04	0.02	0.01	0.01	0.00	
	16	0.80	0.12	0.03	0.02	0.02	0.01	0.00	
	19	0.79	0.13	0.03	0.02	0.01	0.01	0.00	
	22	0.80	0.12	0.03	0.02	0.01	0.01	0.00	
	25	0.79	0.13	0.03	0.02	0.01	0.01	0.00	

6.3.4.2 Influence on ground motion responses

The reconstructed time-domain modal components are used to illustrate the mode strength in ground motion excitations of the 08EDF test. Case 06, 07 and 20 were selected, with the 20%, 35% and 100% scaled Sun excitation, respectively. Figure 6.20 shows the excitation, original signal (summed from all floor responses) and modal components of the selected cases. The RMS values are also provided for each signal to quantify the strength of the whole signal.

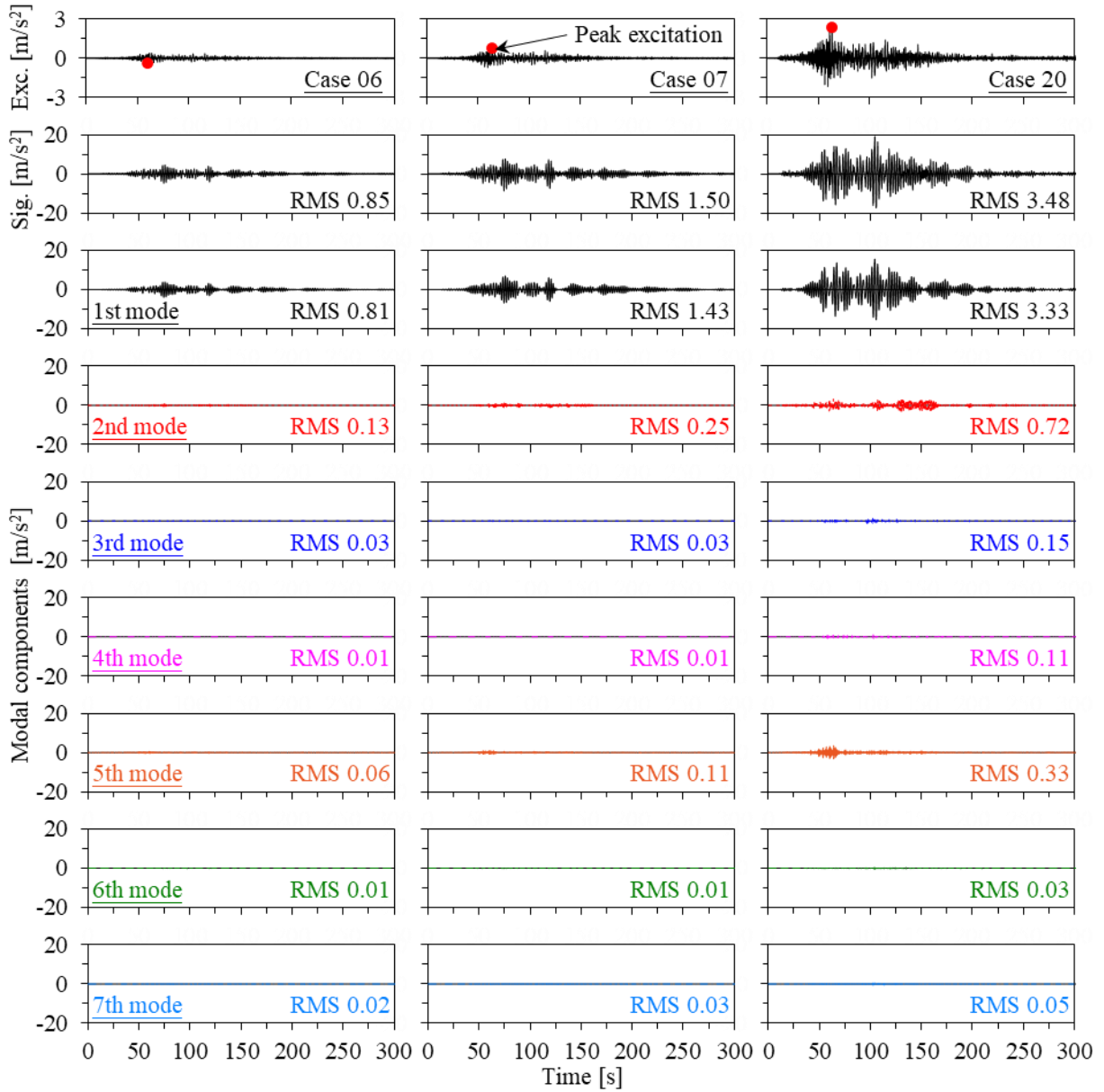


Figure 6.20 Reconstructed modal components from selected cases of the 08EDF test

From the modal components, it is clear that the 1st mode was dominant in the system regardless of the excitation intensities. The dominant 1st mode response was consistent with the low-rise MRFs in the 21TKB and 07EDF tests as shown in Figure 6.17. For the higher modes, the 2nd and 5th modes were stronger than the others, and the pattern did not change in the elastic system (Case 06) and the post-elastic system (Case 20). As introduced in Chapter 05, the upper three floors of the specimen in the 08EDF test were concentrated floors representing multiple floors in the archetype building, while the lower 4 floors were the “real” MRF floors. Thus, the 5th mode was in fact the 1st frame mode, making it understandable to be stronger than the other higher modes.

In order to form a horizontal comparison, the RMS values from different tests and cases of each modal components were normalized to the RMS of the whole signal. Table 6.16 summarizes the normalized RMS values from the 20EDF and 08EDF tests with different excitations and intensities. It can be seen

that, for both the low-rise spine and the high-rise MRF systems, modal strength represented by RMS values were influenced by excitation properties. Different excitations may cause quite different results in the system, for example the Tok and Sun excitations. In both tests, the trend of relative mode strength was not significantly affected by excitation properties, i.e., the strong modes were always strong while the weak ones were always weak. The only difference between different excitations was how strong or weak each mode was when compared to the other modes. These results suggest that there is little difference between the low-rise spine system and the high-rise MRF system when talking about the excitation sensitivity of mode strength in the system.

Table 6.16 Normalized RMS of selected cases and tests

Test	Excitation	Case	Scale	RMS-1	RMS-2	RMS-3	RMS-4	RMS-5	RMS-6	RMS-7
20EDF	Northridge	02	40%	0.88	0.45	0.04	0.02			
		04	100%	0.72	0.64	0.09	0.05			
		07	40%	0.72	0.64	0.09	0.05			
		09	100%	0.79	0.31	0.12	0.10			
	JMA Kobe	11	50%	0.96	0.11	0.03	0.03			
		13	100%	0.87	0.28	0.08	0.03			
	Hog	04	30%	0.96	0.15	0.04	0.01	0.07	0.01	0.02
		17	100%	0.96	0.21	0.04	0.03	0.09	0.01	0.01
08EDF	Sun	06	20%	0.96	0.22	0.06	0.01	0.06	0.01	0.02
		20	100%	0.97	0.12	0.09	0.01	0.04	0.01	0.01
	El Centro	08	30%	0.70	0.50	0.10	0.06	0.27	0.01	0.03
		14	100%	0.60	0.43	0.22	0.09	0.38	0.01	0.02
	Tok	12	60%	0.78	0.44	0.08	0.07	0.30	0.02	0.02
		13	100%	0.70	0.53	0.06	0.06	0.34	0.01	0.02

6.3.4.3 Closure

From the results of the white noise and ground motion excitations, it can be seen that the high-rise building did have stronger higher mode responses excited, regardless of the excitation properties (white noise or ground motion). Based on the analysis, the mode strength in high-rise MRF system was quite similar to that in the low-rise spine systems, suggesting that research in the higher mode issues in high-rise building may be useful for dealing with the higher mode problems in spine systems. It should be noted that although the specimen of the 08EDF test did represent a high-rise building, it had only 7 stories in reality. It is necessary to find results from real high-rise specimens with more stories (also DOFs) to further look into the differences in higher mode responses between the low-rise spine systems and high-rise MRF systems.

6.3.5 Influence of structure proportion

As aforementioned, the FLCs were first used in the project of the 14UCSD test. The specimen of the 14UCSD test was proportioned very differently from that of the 20EDF test. In the 20EDF test, the spine did not provide any overturning moment resistance because of its pinned base, and the MRF resisted all

the overturning moment. On the contrary, in the specimen of the 14UCSD test, the wall provided all overturning moment resistance while the frame was only a gravity frame with negligible overturning moment resistance. As introduced in Chapter 5, the 14UCSD test contains three phases termed as Phase-I, Phase-II and Phase-III, which utilized the Flat Slab-Rocking Wall-FLC, Flat Slab-Rocking Wall, and Flat Slab-Shear Wall systems, respectively. This section discusses the influence of such different proportions on the modal responses of the systems.

6.3.5.1 Influence on white noise responses

Figure 6.21 shows the transition of natural periods, PSD peaks and MEMs in the 14UCSD test. The periods and MEMs were obtained using the ARX method, while the PSD peaks from the FDD method. It should be noted that due to the data quality and characters, only the 1st and 2nd modes were reliable. After checking the consistency of natural periods and mode shapes of the 3rd and 4th modes, parameters from the SS or FDD method were taken to compensate for the missing ones in the results of the ARX method. For the missing PSD peaks, there is no way to compensate for them. Thus, although plotted, parameters of the 3rd and 4th modes are not as reliable as the 1st and 2nd modes. Table 6.17 summarizes the condition of the result compensation.

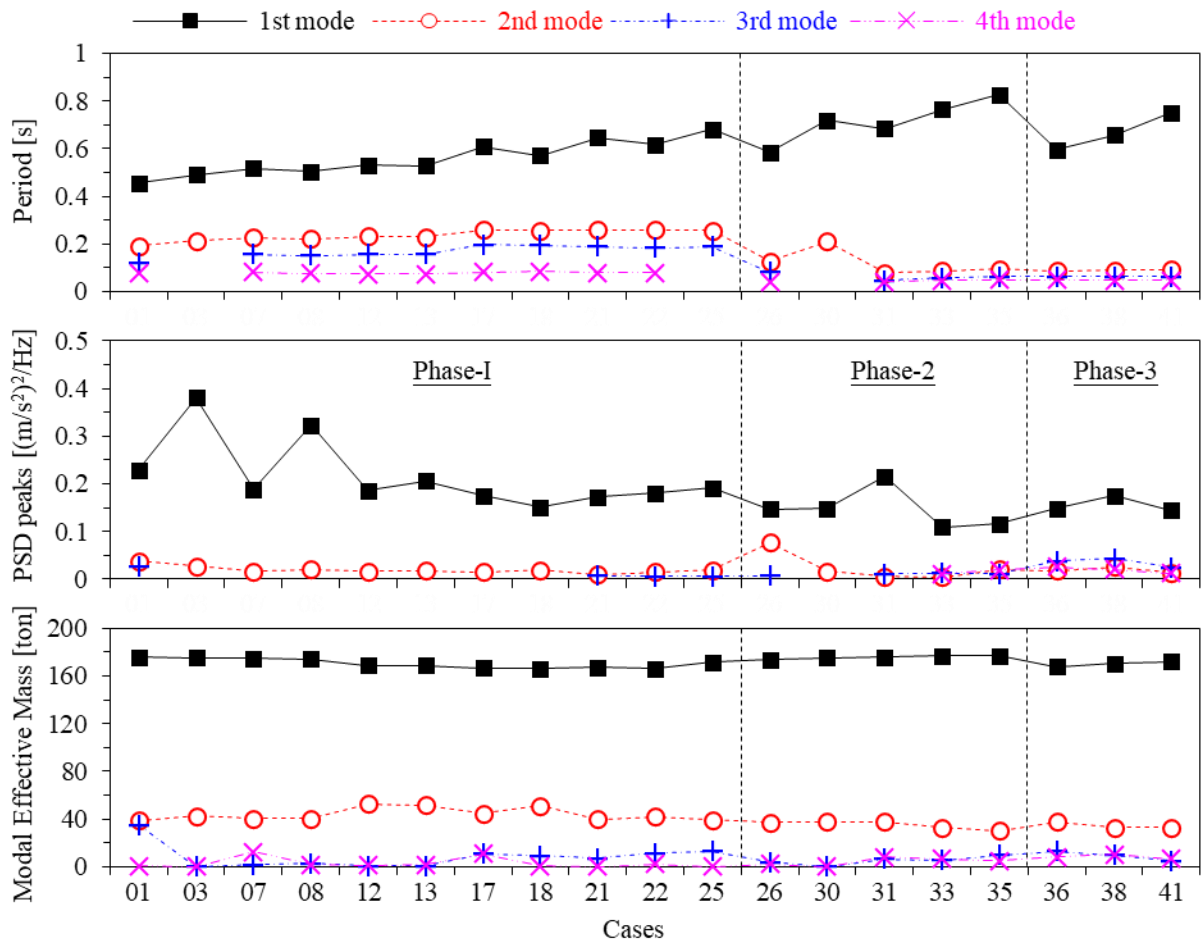


Figure 6.21 Modal parameters in selected white noise cases of the 14UCSD test

Table 6.17 Compensation condition of the 14UCSD white noise modal parameters

Mode	Missing cases	Compensation source	Mode	Missing cases	Compensation source
3	Case03		4	Case17	SS
3	Case30		4	Case18	SS
3	Case35	FDD	4	Case21	SS
3	Case36	FDD	4	Case25	
3	Case38	FDD	4	Case26	SS
3	Case41	FDD	4	Case30	
4	Case03		4	Case31	SS

For the natural periods, the 1st natural periods gradually increased in each phase which was attributed to the gradual yielding in the system due to ground motion excitations. The decreased periods when the phases changed were due to the increase of structure stiffness by changing the connections between Phase-I and Phase-II and fixing the wall base between Phase-II and Phase-III. The identified higher mode periods were generally consistent within each phase, suggesting that the higher modes did not yield significantly. For the mode strength represented by PSD peaks, the 1st mode was dominant in all three phases. Neglecting some jumping points of the PSD peaks which may come from the inaccuracy of the estimation, slight reduction of the 1st mode PSD peaks can be seen. This reduction may be attributed to the yielding that controlled the strength of the 1st mode responses. The MEMs of the 1st and 2nd modes stayed quite consistent throughout the whole test regardless of the phase changes, indicating that the intrinsic mode strength did not change much even when the structure system was changed. This is reasonable considering that all overturning moment resistance came from the wall. Another possible reason for the rather consistent MEMs is that these values were obtained using mode shape vectors which represent the deformation pattern instead of acceleration pattern. The consistent MEMs also suggest that deformation of the system may not be very sensitive to the phase changes. The limited differences between Phase-I and Phase-II also suggest that the FLCs in this test did not work as expected, which is consistent with those reported in previous articles, for example in [49].

6.3.5.2 Influence on ground motion responses

The influence of the structure proportion on modal responses in ground motion excitations can be evaluated using the modal component derived using the SSWT method. Figure 6.22 shows the modal components of Cases 24, 34 and 40 selected from the 14UCSD test. These cases were the last ground motion excitation of each phase, respectively. All cases were excited by the Berkley excitation scaled to 150% of the original record, which was the strongest excitation in the test. The RMS of the signals are also provided to quantify the strength of each signal.

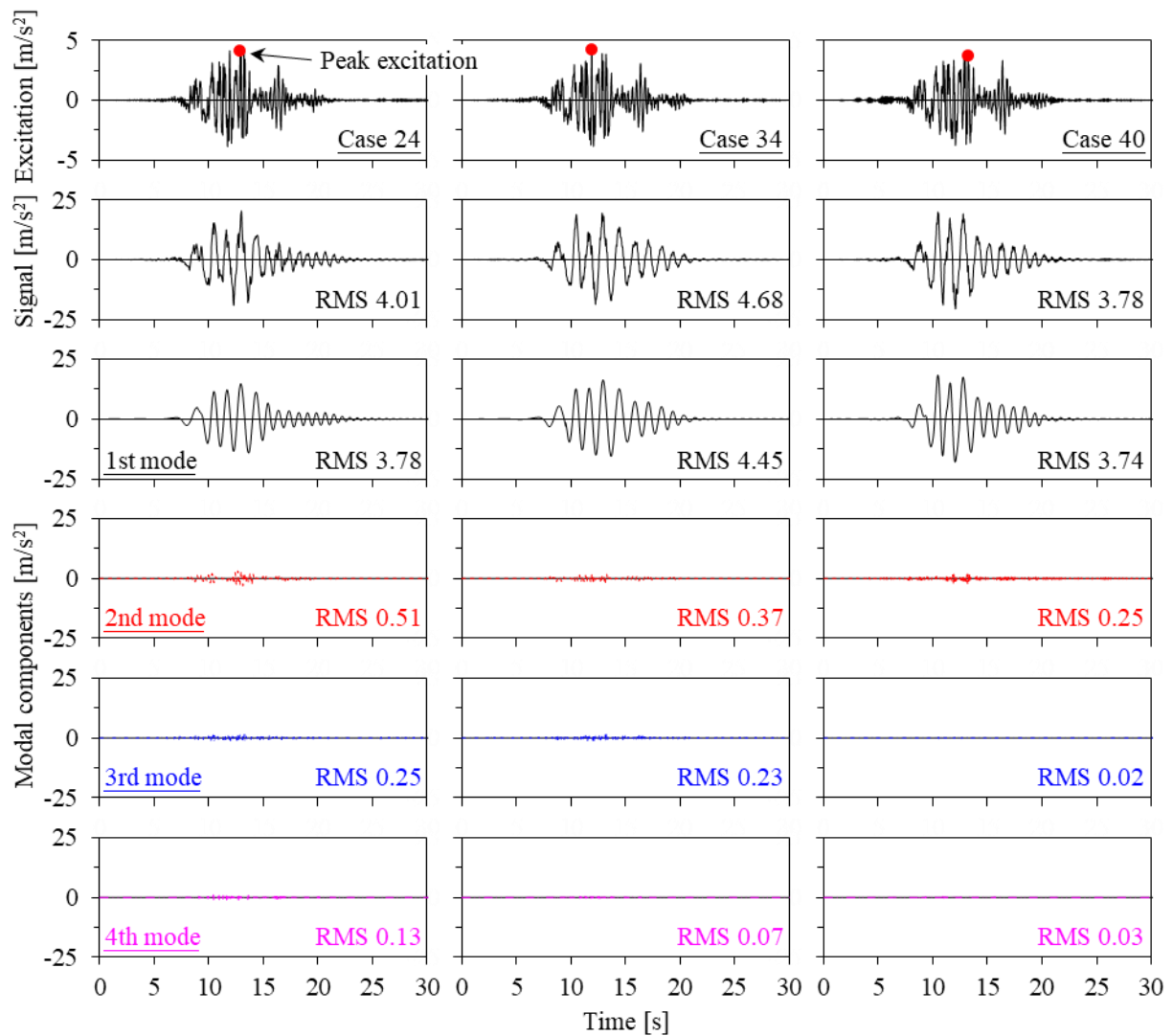


Figure 6.22 Reconstructed modal components from selected cases of the 14UCSD test

From the modal components, clear 1st mode dominance can be observed, regardless of the structural systems in different phases. The 1st mode increased slightly in Case 40, indicating that the shear wall was dominating the vibration of the structure. This is attributed to the fact that the frame provided almost negligible lateral stiffness. For the 2nd mode, a slight decrease can be seen when the system was changed. The 2nd mode was the strongest in Case 24, which is very different from the reduced higher mode response when FLCs were activated in the MRF-Spine system. Reduction of the 2nd mode between Case 34 and 40 may be attributed to the general yielding of the structure. The 3rd mode was very close in Case 24 and 34, suggesting that it was not affected by the FLCs and other variations of the system properties. In Case 40, the 3rd mode became almost negligible. The 4th mode showed a similar trend as the 3rd mode, only that this mode was even weaker in the system. In general, the reduction of higher mode strength in the 14UCSD test was not as significant as in the 20EDF test, which was attributed to the key difference between the two systems that the seismic load resistance came from different part of the structural system.

6.3.5.3 Closure

Comparing the 20EDF and 14UCSD tests using both white noise and ground motion excitations, influence of structure proportion on the modal properties of structural systems was obvious. The relatively stronger higher modes in spine systems are more likely to be excited when the “non-spine” part has seismic load resistance. This is because the stronger the “non-spine” part, the heavier conflict in deformation shapes between it and the spines. If the “non-spine” part has no resistance as in the 14UCSD test, it will just follow however the wall or spine deforms and will not introduce additional constraint and corresponding large responses. The different effectiveness of the FLCs in the two tests suggests that the FLCs may be more effective when the aforementioned “conflict” exists. In addition, design of FLCs may also be sensitive to excitation properties, which should be considered in addition to the structure proportion.

6.3.6 Time-dependence of mode strength

From results of the segmented modal analysis and the time-frequency analysis, time-dependance of mode strength can be observed. Different modes may take the lead at different time instances during the excitation. The strength of a certain mode may also last different time length. Some modes may be strong in amplitude but last only a short time, resulting in smaller strength when evaluated using indicators for a “whole” signal such as the RMS. But the large value, although short in time, may raise different requirements in total demand when designing the structure. Considering this, it may be necessary to consider the time-dependent properties instead of only take the “whole” results when designing the spine systems. Time history analysis may be useful to identify such demands, but this may not be applicable in usual design tasks considering the extra requirements on cost and personnel brought by the more complicated analysis procedures. Thus, proper ways to include the possible large higher mode demands would be necessary.

6.4 SUMMARY

Modal identification and time-frequency analysis were conducted on the MRF-Spine and MRF-Spine-FLC systems. Results from MRF systems are also presented. Linear and nonlinear responses from white noise and ground motion excitations were extracted and analyzed. Major conclusions are summarized as follows:

- FLCs functioned effectively in the MRF-Spine system, especially in the nonlinear cases when structure yields. The 2nd mode was controlled but not all the higher modes. Reduction of total response can be attributed to the reduction of 2nd mode response. The FLCs functioned through changing the natural period of the corresponding modes instead of changing the mode shapes as well as modal effective mass.

- Higher modes are more prevalent in the MRF-Spine system than in MRF systems. This is attributed to the fact that the MRF-Spine system reached a uniform deformation distribution by imposing it using the spines, but in the MRF system by proper design of structure stiffness and strength.
- Ground motion properties have significant influence on the modal responses of the MRF-Spine systems and the effectiveness of the FLCs. Robustness of the FLC design regarding the ground motion properties needs to be further investigated.
- Mode strength in ground motion excited cases showed time-dependence. Indicators that consider only the whole response may not reflect the higher mode response effectively. Methods that can extract time information may be necessary for accurate investigation on the performance of structure systems with spines.
- PSD amplitude derived from the FDD method and modal components decomposed using the SSWT method can be indicators of mode strength in frequency and time domain, respectively. The Effective Modal Pseudo-Acceleration can also be an indicator of modal contribution under nonlinear conditions.
- Linear modal analysis methods such as ARX, FDD and State-Space methods can be used in nonlinear conditions (ground motion excitations, etc.) with signal segmentation. Although the results would be approximate, they can still provide useful information on the modal properties of the analyzed systems with acceptable precision.

7. SUMMARY AND CONCLUSIONS

7.1 SUMMARY

The MRF-Spine systems were expected to control the story mechanisms in building structures by enforcing uniform deformation throughout the height. FLCs were inserted to the system aiming to reduce the large story shear force and floor acceleration brought by the higher mode responses. Properties of the target structure systems were comprehensively investigated. The work reported in this dissertation deepens the understanding of the seismic performance of such systems using real-world data. Major work is summarized as follows:

- A full-scale shake table test was conducted to investigate the performance of MRF-Spine-FLC systems in real world. Two structural systems were tested to evaluate the effectiveness of FLCs in controlling the higher mode responses in such systems. Influence of ground motion properties was also investigated by using different ground motion excitations in the tests. Various sensors were placed on the specimen to capture the behavior of the structural system during excitations.
- Comprehensive analysis was conducted on the measured data. Structural responses, global and local, were extracted from the measurement. These extracted structural responses formed the foundation for better understanding of the properties of MRF-Spine systems.
- Modal properties of the systems were investigated using real-world data. Four more shake table tests were selected to support the investigation, with two low-rise steel MRF, one high-rise steel MRF, and one RC flat-slab rocking-wall. Modal analysis methods, the Auto-Regressive with eXtra input (ARX) method, the Frequency Domain Decomposition method, and the State-Space method, were used to extract modal parameters of the systems. An advanced wavelet method, the Synchro-Squeezed Wavelet Transform, was used to evaluate the time-variant modal properties of the systems.
- Supplementary static test on the T-FLCs used in the shake table test was conducted. Its objective was to obtain the properties of the devices. The devices tested were fabricated in the same batch as tested on the shake table. Elastic stiffness, yielding force and post-elastic stiffness were calibrated from the hysteresis curve obtained from the static test. These parameters can be used in future numerical modeling of the system.

7.2 CONCLUSIONS

Major conclusions of the work are summarized as follows:

- The spines functioned effectively in forcing a uniform deformation and distributing damage across the structure height compared to the simulated MRF responses. Drift concentration at the 1st story

was eliminated in the MRF-Frame system. Although the 1st story still showed largest story drift, damage was not concentrated to this story but developed up to the 3rd story.

- The addition of the spines increased the global floor acceleration and story shear force responses compared to the simulated MRF responses. The increased floor acceleration may affect the acceleration-sensitive non-structural components, while the larger story shear force in both the MRF and the spines may increase difficulties to get a reasonable design of the structure members.
- The Force-Limiting-Connections functioned effectively in reducing the floor acceleration and story shear force responses in the MRF-Spine system. Peak floor acceleration and story shear force were reduced at all floors and stories in the MRF-Spine-FLC system. The reduction was more significant in post-elastic cases, indicating that the benefit of FLCs mainly kicked in after the structure yielded.
- FLCs did compromised the function of the spines in the view of deformation uniformity. Story drift ratio distributed less uniform in the MRF-Spine-FLC system than the MRF-Spine system. This compromise indicates the necessity of a design approach on the FLCs to balance the benefits and loss brought by the FLCs.
- Properties of ground motions affected the response of the MRF-Spine and MRF-Spine-FLC systems. The one stronger to the higher mode periods excited stronger higher mode responses, even with the FLCs activated. Considering the stochastic nature of earthquakes, it is necessary to carefully consider the ground motion selection in the analysis of such structure systems.
- Major source of energy dissipation was the panel zones. Column ends also exhibited substantial yielding in large excitations. FLCs showed some level of yielding but the dissipated energy was not significant. Beam ends remained essentially elastic throughout the tests. The MRF showed a Weak-Column-Strong-Beam proportion, but still caused no collapse with the help of spines, suggesting that structural systems with spines may be less sensitive to beam-column proportion and hinging sequence under strong motion excitations.
- Analysis results on modal properties indicated that the FLCs functioned mainly by reducing the 2nd mode response in the system, which was essentially the purpose of the design. Results also showed that the 3rd mode response increased after the activation of FLCs, suggesting that the FLCs may not be able to control all higher modes, unless a proper tuning approach can be developed.
- The FLCs functioned by shifting the higher mode periods and then changing the excitation intensity. The mode shapes were not significantly changed by the FLCs. The shifted periods were consistent with the reduced 2nd mode response and increased 3rd mode response in the MRF-Spine-FLC system.

- Mode strength of MRF-Spine systems and MRF systems showed that, although the drift distribution was uniform and 1st mode was dominant, and that the intrinsic mode strength represented by Modal Effective Mass did not show much difference, higher modes in the systems with spines are still easier to be excited even with a ground motion not strong near the higher mode periods.
- The higher modes in systems with spines have some similarities with those excited in high-rise buildings. Knowledge about higher modes in high-rise buildings may be useful for the systems with spines, but further investigation is still necessary.
- The proportion of the spines and “non-spine” part also have influence on the modal properties of the systems. The larger the seismic load resistance the “non-spine” part has, the more likely the higher modes are excited in the system.

7.3 FUTURE WORK

The work presented here is a step forward to practical design approaches of the structural systems with spines and deformable connections. All the content is primarily based on real-world data, giving a steady foundation of the discussion but also limited the number of factors that can be investigated. The following work may be necessary to further the study:

- Comprehensive parametric study on different parameters of the systems with spines and deformable connections, including but not limited to proportioning between the substructures, different types of connections, positioning of connections, story height, ground motion properties, etc.
- Code-compliant design approaches.
- Cost-effectiveness and resilience of systems with spines compared to conventional structures.

REFERENCE

- [1] D.G. Lignos, T. Hikino, Y. Matsuoka, M. Nakashima. Collapse Assessment of Steel Moment Frames Based on E-Defense Full-Scale Shake Table Collapse Tests. *Journal of Structural Engineering*, 139, 120–132, 2013. [https://doi.org/10.1061/\(asce\)st.1943-541x.0000608](https://doi.org/10.1061/(asce)st.1943-541x.0000608).
- [2] K. Suita, S. Yamada, M. Tada, K. Kasai, Y. Matsuoka, Y. Shimada. Collapse Experiment on 4-Story Steel Moment Frame: Part 2 Detail of Collapse Behavior. *The 14th World Conference on Earthquake Engineering*, Beijing, China, 2008.
- [3] R. Tremblay. Achieving A Stable Inelastic Seismic Response For Multi-Story Concentrically Braced Steel Frames. *Engineering Journal (New York)*, 40, 111–129, 2003.
- [4] C.-H. Chen, S.A. Mahin. *Performance-Based Seismic Demand Assessment of Concentrically Braced Steel Frame Buildings*. Berkeley, CA, 2012.
- [5] G.A. MacRae, Y. Kimura, C. Roeder. Effect of Column Stiffness on Braced Frame Seismic Behavior. *Journal of Structural Engineering*, 130, 381–391, 2004. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2004\)130:3\(381\)](https://doi.org/10.1061/(ASCE)0733-9445(2004)130:3(381)).
- [6] X. Ji, M. Kato, T. Wang, T. Hitaka, M. Nakashima. Effect of Gravity Columns on Mitigation of Drift Concentration for Braced Frames. *Journal of Constructional Steel Research*, 65, 2148–2156, 2009. <https://doi.org/10.1016/j.jcsr.2009.07.003>.
- [7] S. Kiggins, C.M. Uang. Reducing Residual Drift of Buckling-Restrained Braced Frames as a Dual System. *Engineering Structures*, 28, 1525–1532, 2006. <https://doi.org/10.1016/j.engstruct.2005.10.023>.
- [8] I.F. Khatib, S.A. Mahin, K.S. Pister. *SEISMIC BEHAVIOR OF CONCENTRICALLY BRACED STEEL FRAMES*. 1988.
- [9] P.P. Rossi. A Design Procedure for Tied Braced Frames. *Earthquake Engineering and Structural Dynamics*, 36, 2227–2248, 2007. <https://doi.org/10.1002/eqe.734>.
- [10] Z. Qu, A. Wada, S. Motoyui, H. Sakata, S. Kishiki. Pin-Supported Walls for Enhancing the Seismic Performance of Building Structures. *Earthquake Engineering and Structural Dynamics*, 41, 2075–2091, 2012. <https://doi.org/10.1002/eqe.2175>.
- [11] T. Takeuchi, X. Chen, R. Matsui. Seismic Performance of Controlled Spine Frames with Energy-Dissipating Members. *Journal of Constructional Steel Research*, 114, 51–65, 2015.

- <https://doi.org/10.1016/j.jcsr.2015.07.002>.
- [12] B. Qu, F. Sanchez-Zamora, M. Pollino. Mitigation of Inter-Story Drift Concentration in Multi-Story Steel Concentrically Braced Frames through Implementation of Rocking Cores. *Engineering Structures*, 70, 208–217, 2014. <https://doi.org/10.1016/j.engstruct.2014.03.032>.
- [13] J.-W. Lai, S.A. Mahin. Strongback System: A Way to Reduce Damage Concentration in Steel-Braced Frames. *Journal of Structural Engineering*, 141, 2015. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001198](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001198).
- [14] J. Liu. Strongback Steel-Braced Frames for Improved Seismic Behavior in Buildings. *AISC Engineering Journal*, 54, 297–308, 2017.
- [15] L. Panian, N. Bucci, B. Janhunnen. BRBM Frames: An Improved Approach to Seismic-Resistant Design Using Buckling-Restrained Braces. *Second ATC & SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, 632–643, 2015. www.tippingstructural.com.
- [16] B.G. Simpson. Higher-Mode Force Response in Multi-Story Strongback-Braced Frames. *Earthquake Engineering and Structural Dynamics*, 49, 1406–1427, 2020. <https://doi.org/10.1002/eqe.3310>.
- [17] S. Wu, P. Pan, D. Zhang. Higher Mode Effects in Frame Pin-supported Wall Structure by Using a Distributed Parameter Model. *Earthquake Engineering & Structural Dynamics*, 45, 2371–2387, 2016. <https://doi.org/10.1002/eqe.2766>.
- [18] L. Wiebe, C. Christopoulos. Mitigation of higher mode effects in base-rocking systems by using multiple rocking sections. *Journal of Earthquake Engineering*, 83–108, 2009. <https://doi.org/10.1080/13632460902813315>.
- [19] X. Chen, T. Takeuchi, R. Matsui. Seismic Performance and Evaluation of Controlled Spine Frames Applied in High-Rise Buildings. *Earthquake Spectra*, 34, 1431–1458, 2018. <https://doi.org/10.1193/080817EQS157M>.
- [20] G. Tsampras, R. Sause, D. Zhang, R.B. Fleischman, J.I. Restrepo, D. Mar, J. Maffei. Development of Deformable Connection for Earthquake-Resistant Buildings to Reduce Floor Accelerations and Force Responses. *Earthquake Engineering and Structural Dynamics*, 45, 1473–1494, 2016. <https://doi.org/10.1002/eqe.2718>.
- [21] T. Paulay, M.J.N. Priestly. *Seismic Design of Reinforced Concrete and Masonry Buildings*. Wiley,

1992. <https://doi.org/10.1002/9780470172841>.
- [22] D.G. Lignos, H. Krawinkler, A.S. Whittaker. Prediction and Validation of Sidesway Collapse of Two Scale Models of a 4-Story Steel Moment Frame. *Earthquake Engineering and Structural Dynamics*, 40, 807–825, 2011. <https://doi.org/10.1002/eqe.1061>.
- [23] H. AKIYAMA, M. TAKAHASHI. D_s- VALUES FOR DAMAGE-DISPERSING TYPE MULTI-STORY FRAMES. *Transactions of the Architectural Institute of Japan*, 341, 54–61, 1984. https://doi.org/10.3130/aijsaxx.341.0_54.
- [24] B. Alavi, H. Krawinkler. Strengthening of Moment-Resisting Frame Structures against near-Fault Ground Motion Effects. *Earthquake Engineering and Structural Dynamics*, 33, 707–722, 2004. <https://doi.org/10.1002/eqe.370>.
- [25] Z. Qu, H. Sakata, S. Midorikawa, A. Wada. Lessons from the Behavior of a Monitored 11-Story Building during the 2011 Tohoku Earthquake for Robustness against Design Uncertainties. *Earthquake Spectra*, 31, 1471–1492, 2015. <https://doi.org/10.1193/051813EQS126M>.
- [26] B. Janhunnen, S. Tipping, J. Wolfe. Seismic retrofit of a 1960s steel moment-frame high-rise using a pivoting spine. *Proceedings of the Annual Convention of the Structural Engineers Association of California*, San Diego, 2013.
- [27] G.S. Djojo, G.C. Clifton, R.S. Henry. Rocking steel shear walls with energy dissipation devices 2014 NZSEE Conference. *Proceedings of the 2014 NZSEE Annual Conference*, Auckland, 2014. <http://db.nzsee.org.nz/2014/Orals.htm><https://researchspace.auckland.ac.nz/docs/uoa-docs/rights.htm>.
- [28] E. Casprini, A. Belleri, A. Marini, S. Labò, C. Passoni. Pin-Supported Walls as Seismic Retrofit for Existing RC Frames: Feasibility and Preliminary Design. *Bulletin of Earthquake Engineering*, 20, 5349–5381, 2022. <https://doi.org/10.1007/s10518-022-01405-z>.
- [29] X. Chen, T. Takeuchi, R. Matsui. Simplified Design Procedure for Controlled Spine Frames with Energy-Dissipating Members. *Journal of Constructional Steel Research*, 135, 242–252, 2017. <https://doi.org/10.1016/j.jcsr.2017.04.017>.
- [30] X. Chen, H. Tagawa. Super Elastic System Using Parallel Spine Frames with Steel Braces. *Earthquake Engineering and Structural Dynamics*, 50, 974–994, 2021. <https://doi.org/10.1002/eqe.3384>.
- [31] D. Mar. DESIGN EXAMPLES USING MODE SHAPING SPINES FOR FRAME AND WALL

- BUILDINGS. *Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering*, 25–29, Toronto, Ontario, Canada, 2010.
- [32] B.G. Simpson, S.A. Mahin. Experimental and Numerical Investigation of Strongback Braced Frame System to Mitigate Weak Story Behavior. *Journal of Structural Engineering*, 144, 2018. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001960](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001960).
- [33] M. Palermo, V. Laghi, G. Gasparini, T. Trombetti. Coupled Response of Frame Structures Connected to a Strongback. *Journal of Structural Engineering*, 144, 2018. [https://doi.org/10.1061/\(asce\)st.1943-541x.0002134](https://doi.org/10.1061/(asce)st.1943-541x.0002134).
- [34] J.L. Lin, M.K. Kek, K.C. Tsai. Stiffness Configuration of Strongbacks to Mitigate Inter-Story Drift Concentration in Buildings. *Engineering Structures*, 199, 2019. <https://doi.org/10.1016/j.engstruct.2019.109615>.
- [35] P. Pan, S. Wu, X. Nie. A Distributed Parameter Model of a Frame Pin-Supported Wall Structure. *Earthquake Engineering and Structural Dynamics*, 44, 1643–1659, 2015. <https://doi.org/10.1002/eqe.2550>.
- [36] M. Bosco, E.M. Marino, P.P. Rossi. A Design Procedure for Pin-Supported Rocking Buckling-Restrained Braced Frames. *Earthquake Engineering and Structural Dynamics*, 47, 2840–2863, 2018. <https://doi.org/10.1002/eqe.3112>.
- [37] Z. Qu, T. Gong, X. Wang, Q. Li, T. Wang. Stiffness and Strength Demands for Pin-Supported Walls in Reinforced-Concrete Moment Frames. *Journal of Structural Engineering*, 146, 2020. [https://doi.org/10.1061/\(asce\)st.1943-541x.0002758](https://doi.org/10.1061/(asce)st.1943-541x.0002758).
- [38] M.O. Eberhard, M.A. Sozen. Behavior-Based Method to Determine Design Shear in Earthquake-Resistant Walls. *Journal of Structural Engineering*, 119, 619–640, 1993. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1993\)119:2\(619\)](https://doi.org/10.1061/(ASCE)0733-9445(1993)119:2(619)).
- [39] I. Ghorbanirenani, R. Tremblay, P. Léger, M. Leclerc. Shake Table Testing of Slender RC Shear Walls Subjected to Eastern North America Seismic Ground Motions. *Journal of Structural Engineering*, 138, 1515–1529, 2012. [https://doi.org/10.1061/\(asce\)st.1943-541x.0000581](https://doi.org/10.1061/(asce)st.1943-541x.0000581).
- [40] M. Panagiotou, J.I. Restrepo. Displacement-Based Method of Analysis for Regular Reinforced-Concrete Wall Buildings: Application to a Full-Scale 7-Story Building Slice Tested at UC–San Diego. *Journal of Structural Engineering*, 137, 677–690, 2011. [https://doi.org/10.1061/\(asce\)st.1943-541x.0000333](https://doi.org/10.1061/(asce)st.1943-541x.0000333).

- [41] L. Wiebe, C. Christopoulos. A Cantilever Beam Analogy for Quantifying Higher Mode Effects in Multistorey Buildings. *Earthquake Engineering and Structural Dynamics*, 44, 1697–1716, 2015. <https://doi.org/10.1002/eqe.2549>.
- [42] M.R. Eatherton, X. Ma, H. Krawinkler, D. Mar, S. Billington, J.F. Hajjar, G.G. Deierlein. Design Concepts for Controlled Rocking of Self-Centering Steel-Braced Frames. *Journal of Structural Engineering*, 140, 2014. [https://doi.org/10.1061/\(asce\)st.1943-541x.0001047](https://doi.org/10.1061/(asce)st.1943-541x.0001047).
- [43] L. Wiebe, C. Christopoulos. Performance-Based Seismic Design of Controlled Rocking Steel Braced Frames. II: Design of Capacity-Protected Elements. *Journal of Structural Engineering*, 141, 2015. [https://doi.org/10.1061/\(asce\)st.1943-541x.0001201](https://doi.org/10.1061/(asce)st.1943-541x.0001201).
- [44] B.G. Simpson, D. Rivera Torres. Simplified Modal Pushover Analysis to Estimate First- and Higher-Mode Force Demands for Design of Strongback-Braced Frames. *Journal of Structural Engineering*, 147, 2021. [https://doi.org/10.1061/\(asce\)st.1943-541x.0003163](https://doi.org/10.1061/(asce)st.1943-541x.0003163).
- [45] R. Tremblay, L. Chen, L. Tirca. Enhancing the Seismic Performance of Multi-Storey Buildings with a Modular Tied Braced Frame System with Added Energy Dissipating Devices. *International Journal of High-Rise*, 3, 21–33, 2014.
- [46] R. Fleischman, J. Restrepo, A. Nema, D. Zhang, U. Shakya, Z. Zhang, R. Sause, G. Tsampras, G. Monti. Inertial Force-Limiting Anchorage System for Seismic Resistant Building Structures. *Structures Congress 2015*, 1302–1313, American Society of Civil Engineers, Reston, VA, 2015. <https://doi.org/10.1061/9780784479117.111>.
- [47] G. Tsampras, R. Sause, R.B. Fleischman, J.I. Restrepo. Experimental Study of Deformable Connection Consisting of Friction Device and Rubber Bearings to Connect Floor System to Lateral Force Resisting System. *Earthquake Engineering and Structural Dynamics*, 47, 1032–1053, 2018. <https://doi.org/10.1002/eqe.3004>.
- [48] G. Tsampras, R. Sause, R.B. Fleischman, J.I. Restrepo. Experimental Study of Deformable Connection Consisting of Buckling-Restrained Brace and Rubber Bearings to Connect Floor System to Lateral Force Resisting System. *Earthquake Engineering and Structural Dynamics*, 46, 1287–1305, 2017. <https://doi.org/10.1002/eqe.2856>.
- [49] Z. Zhang, R.B. Fleischman, J.I. Restrepo, G. Guerrini, A. Nema, D. Zhang, U. Shakya, G. Tsampras, R. Sause. Shake-Table Test Performance of an Inertial Force-Limiting Floor Anchorage System. *Earthquake Engineering and Structural Dynamics*, 47, 1987–2011, 2018. <https://doi.org/10.1002/eqe.3047>.

-
- [50] G. Tsampras, R. Sause. Force-Based Design Method for Force-Limiting Deformable Connections in Earthquake-Resistant Buildings. *Journal of Structural Engineering*, 148, 2022. [https://doi.org/10.1061/\(asce\)st.1943-541x.0003456](https://doi.org/10.1061/(asce)st.1943-541x.0003456).
- [51] B. Astudillo, D. Rivera, J. Duke, B. Simpson, L.A. Fahnestock, R. Sause, J. Ricles, M. Kurata, T. Okazaki, Y. Kawamata, Z. Tao, Y. Qie. Modeling Uncertainty of Specimens Employing Spines and Force-limiting Connections Tested at E-defense Shake Table. *Earthquake Engineering & Structural Dynamics*, 52, 4638–4659, 2023. <https://doi.org/10.1002/eqe.3976>.
- [52] D.J. Ewins. *Modal Testing: Theory, Practice and Application*. 2nd ed. John Wiley & Sons, 2009.
- [53] R. Brincker, L. Zhang, P. Andersen. Modal Identification of Output-Only Systems Using Frequency Domain Decomposition. *Smart Materials and Structures*, 10, 441–445, 2001. <https://doi.org/10.1088/0964-1726/10/3/303>.
- [54] R. Brincker, C.E. Ventura, P. Andersen. Damping Estimation by Frequency Domain Decomposition. *IMAC19: A Conference on Structural Dynamics*, 698–703, Hyatt Orlando, Kissimmee, Florida, 2001.
- [55] T. Wang, L. Zhang, Y. Tamura. An Operational Modal Analysis Method in Frequency and Spatial Domain. *Earthquake Engineering and Engineering Vibration*, 4, 295–300, 2005. <https://doi.org/10.1007/s11803-005-0012-0>.
- [56] R. Brincker, L. Zhang. Frequency Domain Decomposition Revisited. *Proceedings of the 3rd International Operational Modal Analysis Conference - IOMAC*, 615, Hale, 2009.
- [57] L. Ljung. *System Identification: Theory for the User (Second Edition)*. Prentice Hall PTR, New Jersey, 1999.
- [58] B. Peeters. *SYSTEM IDENTIFICATION AND DAMAGE DETECTION IN CIVIL ENGINEERING*. , PhD, Katholieke Universiteit Lueven, 2000.
- [59] S.N. Pakzad, G.L. Fenves. Statistical Analysis of Vibration Modes of a Suspension Bridge Using Spatially Dense Wireless Sensor Network. *Journal of Structural Engineering*, 135, 863–872, 2009. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000033](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000033).
- [60] S.R. Ibrahim. Random Decrement Technique for Modal Identification of Structures. *Journal of Spacecraft and Rockets*, 14, 596–700, 1977. <https://doi.org/10.2514/3.57251>.
- [61] S.R. Ibrahim. Fundamentals of Time Domain Modal Identification. J.M.M. Silva, N.M.M. Maia (Eds.), *Modal Analysis and Testing*, 241–250, Springer Netherlands, Dordrecht, 1999.

https://doi.org/10.1007/978-94-011-4503-9_11.

- [62] G.H. James III, T.G. Carne, J.P. Lauffer. THE NATURAL EXCITATION TECHNIQUE (NExT) FOR MODAL PARAMETER EXTRACTION FROM OPERATING STRUCTURES. *Modal Analysis: The International Journal of Analytical and Experimental Modal Analysis*, 10, 260–277, 1995.
- [63] A.V. Oppenheim, R.W. Schafer. *Discrete-Time Signal Processing*. 3rd ed. Pearson, 2010.
- [64] S. Nagarajaiah, B. Basu. Output Only Modal Identification and Structural Damage Detection Using Time Frequency & Wavelet Techniques. *Earthquake Engineering and Engineering Vibration*, 8, 583–605, 2009. <https://doi.org/10.1007/s11803-009-9120-6>.
- [65] I. Daubechies, J. Lu, H.T. Wu. Synchrosqueezed Wavelet Transforms: An Empirical Mode Decomposition-like Tool. *Applied and Computational Harmonic Analysis*, 30, 243–261, 2011. <https://doi.org/10.1016/j.acha.2010.08.002>.
- [66] I. Daubechies, S. Maes. A Nonlinear Squeezing of the Continuous Wavelet Transform Based on Auditory Nerve Models. *WAVELETS in Medicine and Biology*, 1st ed., 527–546, Routledge, 1996. <https://www.taylorfrancis.com/books/9781351404723/chapters/10.1201/9780203734032-20>.
- [67] K. Dziedziech, W.J. Staszewski, T. Uhl. Wavelet-Based Modal Analysis for Time-Variant Systems. *Mechanical Systems and Signal Processing*, 50–51, 323–337, 2015. <https://doi.org/10.1016/j.ymssp.2014.05.003>.
- [68] K. Dziedziech, W.J. Staszewski, T. Uhl. Wavelet-Based Frequency Response Function: Comparative Study of Input Excitation. *Shock and Vibration*, 2014, 2014. <https://doi.org/10.1155/2014/502762>.
- [69] W.J. Staszewski, D.M. Wallace. Wavelet-Based Frequency Response Function for Time-Variant Systems - An Exploratory Study. *Mechanical Systems and Signal Processing*, 47, 35–49, 2014. <https://doi.org/10.1016/j.ymssp.2013.03.011>.
- [70] D.A. Roke. *Damage-Free Seismic-Resistant Self-Centering Concentrically-Braced Frames*. , Lehigh University, 2010.
- [71] T. Akbas. *Seismic Response Analysis of Structures with Nonlinear Mechanisms Using a Modal Approach*. , Lehigh University, 2016. <http://preserve.lehigh.edu/etdhttp://preserve.lehigh.edu/etd/2478>.

- [72] K. Suzuki, A. Watanabe, E. Saeki. *Development of U-Shaped Steel Damper for Seismic Isolation System*. 2005.
- [73] K. Inoue, K. Suita. *Building Steel Structure: Theory and Design*. Kajima Institute Publishing Co.,Ltd., 2007.
- [74] T. Okazaki, M. Somarriba, T. Nagae, T. Matsumiya, K. Asai. Shake-Table Collapse Test of a Scaled 4-Story Steel Moment-Frame Part 1: Test Plan. *AIJ National Conference*, 2022.
- [75] M. Somarriba, T. Okazaki, T. Nagae, T. Matsumiya. Shake-Table Collapse Test of a Scaled 4-Story Steel Moment-Frame Part 2: Test Results. *AIJ National Conference*, 2022.
- [76] S. Yamada, K. Suita, M. Tada, K. Kasai, Y. Matsuoka, Y. Shimada. Collapse Experiment on 4-Story Steel Moment Frame: Part 1 Outline of Test Results. *The 14th World Conference on Earthquake Engineering*, Beijing, China, 2008.
- [77] Y.-L. Chung, T. Nagae, T. Hitaka, M. Nakashima. Seismic Resistance Capacity of High-Rise Buildings Subjected to Long-Period Ground Motions: E-Defense Shaking Table Test. *Journal of Structural Engineering*, 136, 637–644, 2010. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000161](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000161).
- [78] X. Ji, G.L. Fenves, K. Kajiwarra, M. Nakashima. Seismic Damage Detection of a Full-Scale Shaking Table Test Structure. *Journal of Structural Engineering*, 137, 14–21, 2011. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000278](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000278).
- [79] P. Andersson. Adaptive Forgetting in Recursive Identification through Multiple Models. *International Journal of Control*, 42, 1175–1193, 1985. <https://doi.org/10.1080/00207178508933420>.
- [80] M. Gong, J. Sun, L. Xie. Identification of Model Structure Parameters via Combination of AFMM and ARX from Seismic Response Data. *Earthquake Engineering and Engineering Vibration*, 13, 411–423, 2014. <https://doi.org/10.1007/s11803-014-0252-y>.
- [81] R.J. Allemang. The Modal Assurance Criterion – Twenty Years Of Use And Abuse. *Sound and Vibration*, 14–21, 2003.
- [82] L. Zhang, T. Wang, Y. Tamura. A Frequency-Spatial Domain Decomposition (FSDD) Method for Operational Modal Analysis. *Mechanical Systems and Signal Processing*, 24, 1227–1239, 2010. <https://doi.org/10.1016/j.ymssp.2009.10.024>.
- [83] F. Pioldi, R. Ferrari, E. Rizzi. Output-Only Modal Dynamic Identification of Frames by a Refined

- FDD Algorithm at Seismic Input and High Damping. *Mechanical Systems and Signal Processing*, 68–69, 265–291, 2016. <https://doi.org/10.1016/j.ymssp.2015.07.004>.
- [84] P. Welch. The Use of Fast Fourier Transform for the Estimation of Power Spectra: A Method Based on Time Averaging over Short, Modified Periodograms. *IEEE Transactions on Audio and Electroacoustics*, 15, 70–73, 1967. <https://doi.org/10.1109/TAU.1967.1161901>.
- [85] J.-N. Juang. *Applied System Identification*. Prentice-Hall, Inc., 1994.
- [86] Z.-F. Fu, J.-M. He. *Modal Analysis*. 2nd ed. Butterworth-Heinemann, 2001.
- [87] P. Van Overschee, B. De Moor. N4SID: Subspace Algorithms for the Identification of Combined Deterministic-Stochastic Systems. *Automatica*, 30, 75–93, 1994. [https://doi.org/10.1016/0005-1098\(94\)90230-5](https://doi.org/10.1016/0005-1098(94)90230-5).
- [88] D. Roke, R. Sause, J.M. Ricles, N. Gonner. DESIGN CONCEPTS FOR DAMAGE-FREE SEISMIC-RESISTANT SELF-CENTERING STEEL CONCENTRICALLY-BRACED FRAMES. *The 14th World Conference on Earthquake Engineering*, Beijing, China, 2008.

APPENDIX A. STATIC TEST ON THE T-FLCS

A.1 GENERAL

This appendix introduces the static test on the TFLCs. Test design, data reduction scheme, main results, and preliminary numerical modeling will be introduced.

A.2 TEST DESIGN

A.2.1 Test specimen

Figure A.1 illustrates dimension of the TFLC. Two specimens from the same batch as those tested on the shake table were tested. The specimens were fabricated from a special type of high strength steel. Yielding strength and tensile strength of the material are 719MPa and 815MPa, respectively.

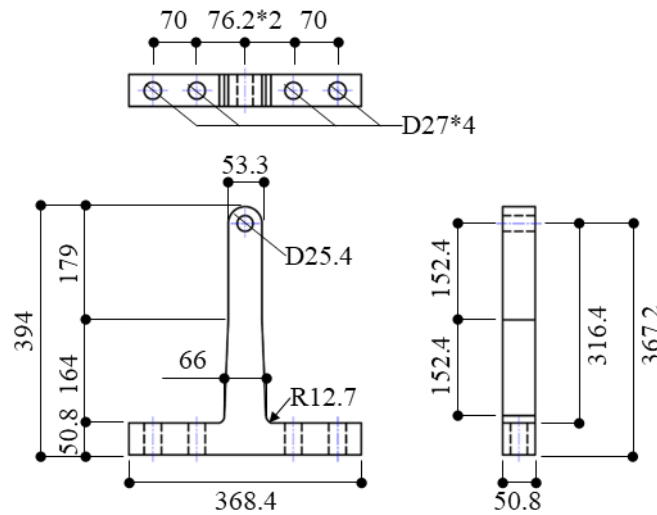


Figure A.1 Dimensions of the specimen [Unit: mm]

A.2.2 Test setup

Figure A.2 illustrates the test setup of the static tests. In general, the whole setup contains the loading system and the lateral bracing system, both mounted to three base girders placed on the lab floor.

The loading system consisted of three reaction columns, one 250 kN oil jack, one 250 kN load cell, and a connector for attaching the jack to the specimen. Braces were set to the reaction columns, forming support from the base girders to the center line of loading system. A 40 mm steel plate was used to connect the oil jack end clevis to the three reaction columns simultaneously. The oil jack, load cell and specimen connectors were connected using M48 threaded rods with lock nuts. At the base of the specimen, three pieces of base plates (two of 40 mm and one of 50 mm) were used to mount it to the base girders, providing sufficient shear and flexural capacity as well as stiffness.

The specimen connector consisted of several pieces of steel plates assembled using high strength bolts.

Connection between the specimen and the connector was realized by using a pin taken from the shake table test specimen with some modification. The center holes on the connectors for the pin were machined to realize a slip fit.

The lateral bracing system consisted of four columns, two channels, two pairs of turn buckles, a set of rollers and their contacting plates. The columns were bolted to the base girders while the channels bolted to the columns. Turn buckles were used to provide better out-of-plane stiffness to the system. The rollers were mounted to the specimen connector, with two on each side. The rollers moved against a pair of contacting plates attached to the channels via adjustable M16 threaded rods. A small gap was left between the rollers and the plates at each side.

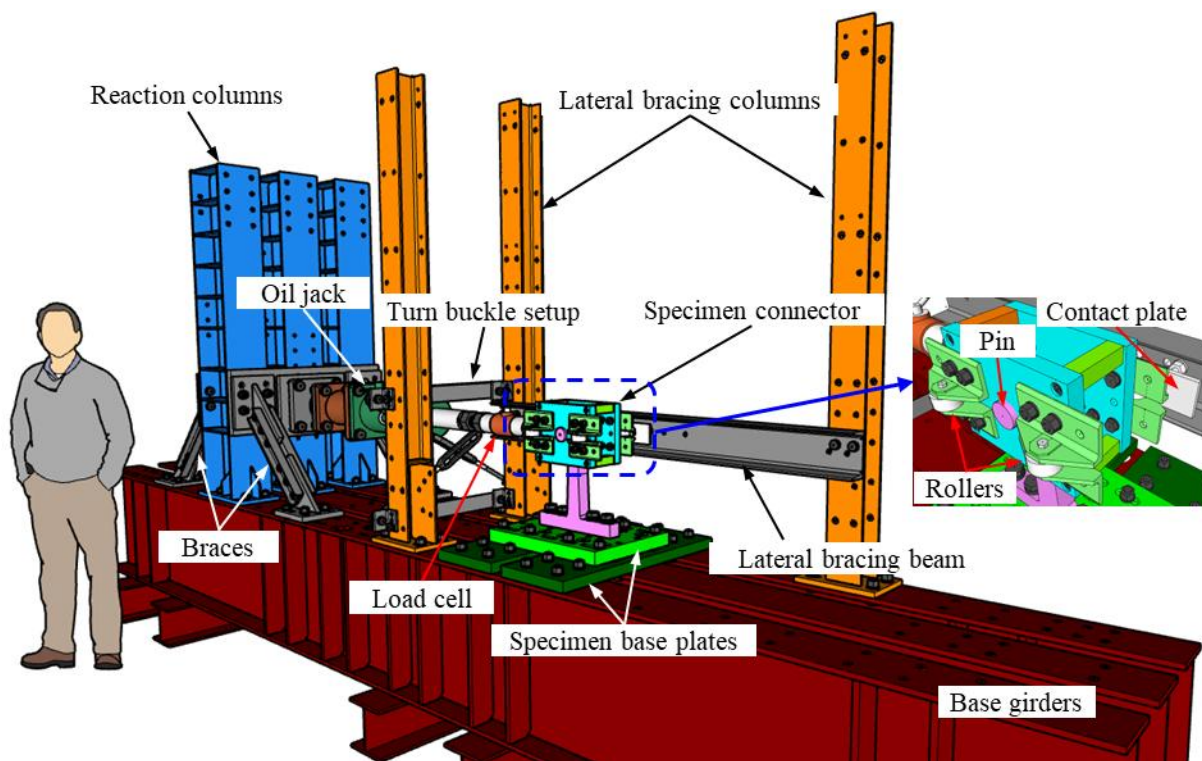


Figure A.2 Test setup

A.2.3 Instrumentation

The instrumentation of the test only uses displacement transducers and a load cell. Table A.1 summarizes information of instruments. Figure A.3 illustrates the layout of the instruments. In order to avoid the possible influence of the base girder deformation, measurement is taken with reference to the lab floor. All instrumentations are products of TML company and are all strain-gauge type sensors.

D1~D3 were used to measure the displacement of the loading point to the floor, with D1 serving the smaller range while D2 and D3 for the whole range. All three transducers are attached at the height of the loading axis to avoid problems when the rotation of the specimen top gets big. These three transducers were attached via a target plate glued to the specimen, with D1 directly pushed against it

while D2 and D3 connected using magnetic hooks and wires. Readings from D2 and D3 were averaged to prevent possible torsion of the specimen (which was not likely to happen). D4 and D5 were used to measure the displacement of the specimen base and base plate to the floor. These two values were further used for calculation of specimen deformation and possible slippage at the specimen base connection. D6 and D7 were used to measure the possible uplift of the specimen base (if any).

The system takes push-out as positive direction of X-axis, and upward as positive direction of Y-axis. Positive of rotation was determined by the right-hand rule, being counterclockwise when looking from the front side of the setup. Polarity of the sensors for calculation was determined by the coordinate system and the sensor polarity together.

Table A.1 Instrument specifications

Tag	Model	Range	Resolution	Polarity for calculation
LC	TCLP-25BS	± 250 kN	0.061 kN	Negative (-)
D1	CDP-25MT	± 12.5 mm	0.002 mm	Negative (-)
D2	SDP-200E	± 100 mm	0.02 mm	Positive (+)
D3	SDP-200E	± 100 mm	0.02 mm	Positive (+)
D4	CDP-25MT	± 12.5 mm	0.002 mm	Negative (-)
D5	CDP-25MT	± 12.5 mm	0.002 mm	Positive (+)
D6	CDP-25MT	± 12.5 mm	0.002 mm	Negative (-)
D7	CDP-25MT	± 12.5 mm	0.002 mm	Negative (-)

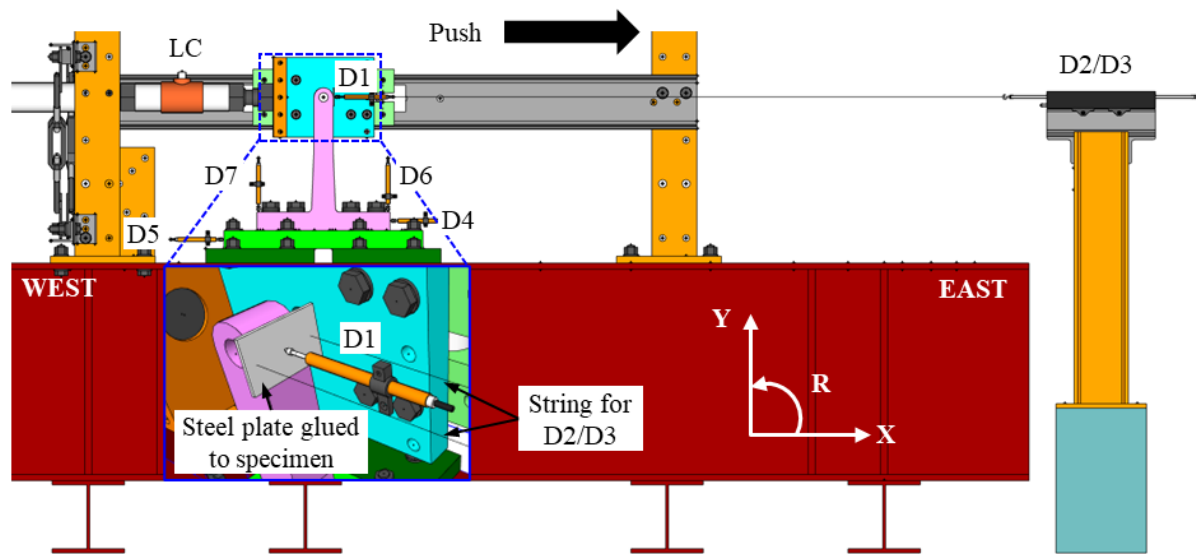


Figure A.3 Instrumentation layout in the test

A.2.3.2 Loading protocol

The T-FLCs tested this time were the two back up devices prepared for the shake table tests. Considering that there was already one T-FLC tested at Lehigh University cyclically prior to the shake table tests, it is not necessary to have both the devices tested cyclically this time. Thus, two types of loading protocols were designed for the two specimens, respectively.

A.2.3.3 Loading protocol of Specimen 1

For Specimen 1, the test used force-displacement hybrid control scheme. Table A.2 summarizes the information of the loading protocol. In total, there were 3 force-controlled levels and 8 displacement-controlled levels. For each of the first 10 levels, 3 cycles were conducted. The last level was a single direction large displacement level trying to reach a limit state of the specimen. Difference of the positive and negative targets in Level 10 was due to the stroke limit of the oil jack.

Table A.2 Loading protocol of Specimen 1

Level	Control scheme	Target force [kN]	Target Displacement [mm]	Cycles
1	Force	± 20	-	3
2		± 40	-	3
3		± 60	-	3
4	Displacement	-	± 6	3
5		-	± 8	3
6		-	± 10	3
7		-	± 20	3
8		-	± 30	3
9		-	± 50	3
10		-	+70/-75	3
11		-	-150	-

A.2.3.4 Loading protocol of Specimen 2

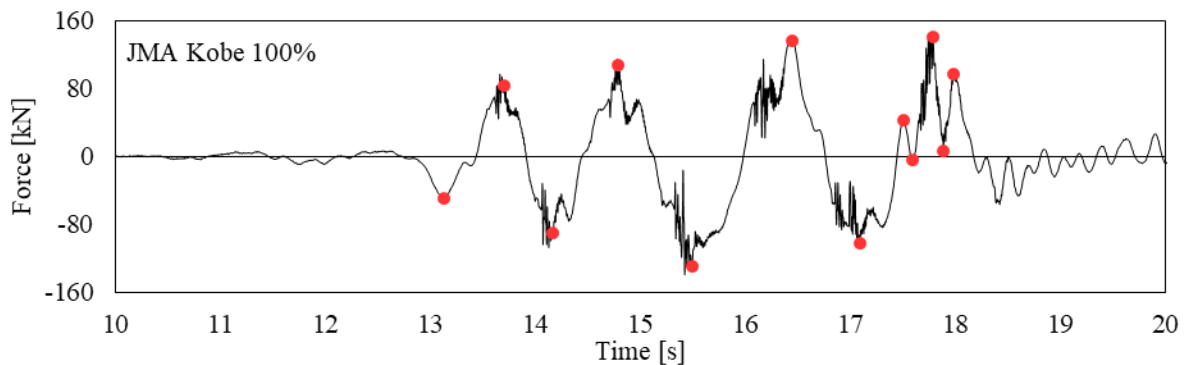


Figure A.4 Target force selection

For Specimen 2, the test used a loading protocol being an analogy of the recorded response history of the T-FLC in the shake table tests. The protocol was divided into 3 phases. The 1st phase was a warm-up phases consisting of two levels of cyclic loading. The 2nd phase was a force-controlled phase with target force selected from the T-FLC force time history in the JMA Kobe 100% excitation. The 3rd phase was a displacement-controlled phase, with the target deformation calculated on-site. The peak displacement values recorded in the 2nd phase were doubled, and then added to the residual deformation of the 2nd phase, forming the target displacement values. Figure A.4 illustrates the selection of target force. Table A.3 summarizes the full loading protocol.

Table A.3 Loading protocol of Specimen 2

Phase	Level	Control scheme	Target force [kN]	Target Displacement [mm]	Cycles
1	1	Force	±20	-	3
	2		±40	-	3
2	1	Force	-50	-	-
	2		83	-	-
	3		-90	-	-
	4		107	-	-
	5		-130	-	-
	6		137	-	-
	7		-102	-	-
	8		44	-	-
	9		-4	-	-
	10		141	-	-
	11		6	-	-
	12		98	(Residual: 5.1 mm)	-
3	1	Displacement	-	-1.1	-
	2		-	16.2	-
	3		-	-7.4	-
	4		-	20.6	-
	5		-	-17.8	-
	6		-	31.8	-
	7		-	-7.9	-
	8		-	12.1	-
	9		-	7.1	-
	10		-	34.4	-
	11		-	16.4	-
	12		-	28	-

A.3 DATA REDUCTION

A.3.1 Calculation equations

A.3.1.1 Specimen deformation

Deformation was calculated by subtracting displacement of the specimen base from the displacement at the loading line. Two pairs of data sources were used. Deformation calculated by D1 and D4 is termed as Δ_1 (in equations) or Deform-1 (in other situations), while D2/3 and D4 as Δ_2 or Deform-2, which are calculated as follows:

$$\Delta_1 = -D_1 - (-D_4) \quad (\text{A.1})$$

$$\Delta_2 = \frac{D_2 + D_3}{2} - (-D_4) \quad (\text{A.2})$$

A.3.1.2 Specimen base slippage

Using the difference between specimen base displacement (D4) and base plate displacement (D5), the slippage at specimen base can be calculated as follows:

$$\delta = (-D_4) - D_5 \quad (A.3)$$

A.3.1.3 Specimen base rotation

Using the two vertical transducers at the specimen base, the rotation can be calculated as follows:

$$\theta = \frac{-D_6 + D_7}{l_{67}} \quad (A.4)$$

Where l_{67} is the distance between D_6 and D_7 and the on-site measurement is 355mm.

A.4 COMPENSATION OF DEFORMATION DATA

Due to the limitation of available range of D1, the large deformation is covered by D2 and D3. However, due to the use of strings instead of direct contact of D2 and D3, some “dead point” can be observed when loading direction is reversed. Such issues were addressed by compensating the deformation data manually. In this section, data of Specimen 1 is used for illustration. Same procedures were also applied to data of Specimen 2.

A.4.1 General policy

The general policy of data compensation is as follows:

- 1) For data within the range of Deform-1, use Deform-1 data.
- 2) For data beyond the range of Deform-1, modify Deform-2 data.
- 3) Connect the two parts of data to get the full range deformation data.

A.4.2 Map between Deform-1 and Deform-2

Figure A.5 illustrates the two data sets and their difference when both Deform-1 and Deform-2 were available.

To get a map between the two data sources, the difference at peaks were located, as shown by the square and circle in the figure. By finding out all the difference values at peaks, the averaged difference at the positive and negative peaks were calculated, being 0.46 mm and -0.24 mm, respectively. The averaged values matched the general trend of the difference values satisfactorily. Such good consistency indicates that it is possible to get an estimate of the peaks of Deform-2 beyond the changing point. For the following peaks, they can be estimated by adding 0.46 or -0.24 to positive or negative peaks, respectively. Since the peaks of Deform-2 and force were close enough, the Deform-2 values used to generate the estimated peak deformation were those at the peak force instead of the real Deform-2 peaks.

For the other data points, they were not modified one by one, but treated as a whole using the following a step-by-step scheme as introduced in the following section.

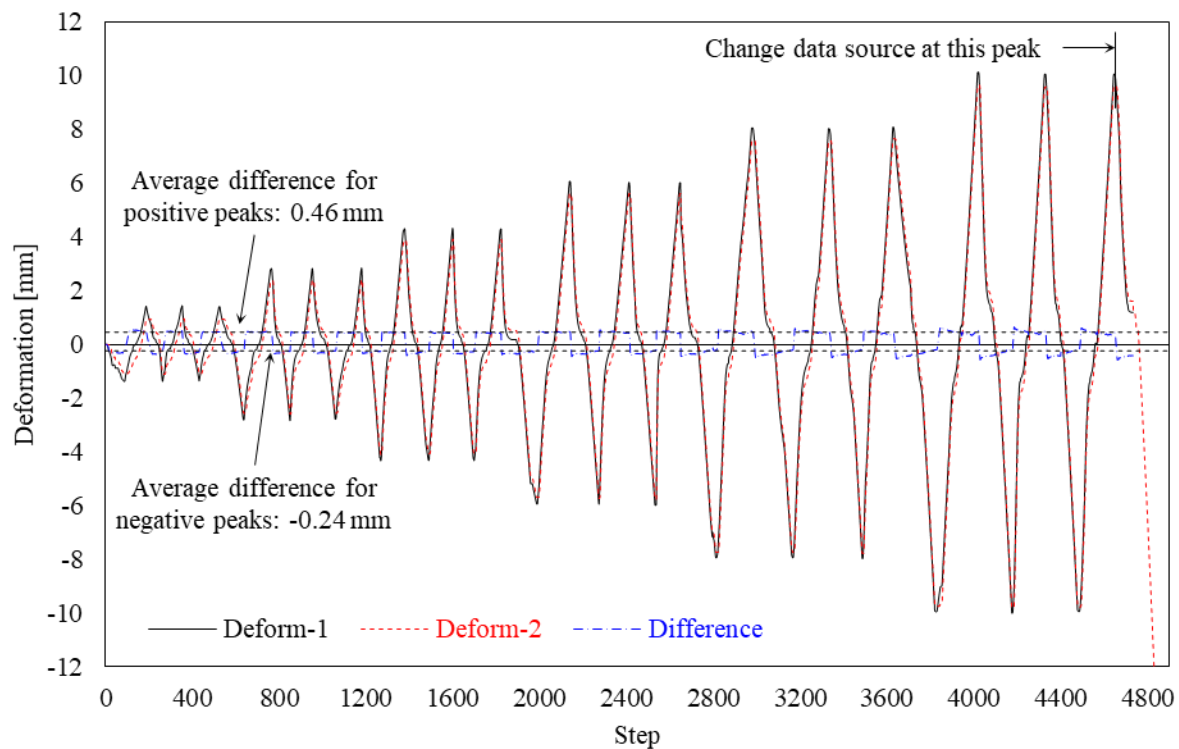


Figure A.5 Comparison of different deformation data

A.4.3 Data segmentation

Compensation was conducted using the hysteresis of Force vs Deform-2. Before compensation, segmentation was needed. The data after the changing point were first divided by the peak and zero-crossing points of the force responses. Each cycle was divided into four segments, namely:

- 1) Segment a: unloading from peak to zero force.
- 2) Segment b: loading to the opposite peak.
- 3) Segment c: unloading from the opposite peak to zero force.
- 4) Segment d: loading to the peak of next cycle.

Each segment was then processed using different algorithms. Processing was conducted cycle by cycle until the end of the data.

A.4.4 Example of data compensation

The cycle right after data source change (step 4644) is used for illustration. Figure A.6 illustrates the segmentation of the cycle, and the detailed procedures are introduced hereafter.

A.4.4.1 Segment a

Segment a was processed as follows:

- 1) Get the estimated target deformation at peak and the corresponding shift values. Use the real difference between Deform-1 and Deform-2 for this cycle and the averaged value for the others.
- 2) Shift the whole segment according to the shift values obtained from 1).

- 3) Divide the segment into the “force drop” portion and the “linear unloading” portion.
- 4) Get the slope of the “linear unloading” portion.
- 5) Get the target deformation of the “force drop” portion using corresponding force values and the slope obtained from 4).
- 6) Shift the “linear unloading” portion according to the end value of the modified “force drop” portion.

A.4.4.2 Segment b

Segment b was processed as follows:

- 1) Get the “initial shift” according to the end value of the modified Segment a.
- 2) Get the “end shift” according to the aforementioned averaged shift (-0.24 mm).
- 3) Distribute the shift values linearly to each data point of this segment.
- 4) Shift the segment according to the shift values obtained in 3).

A.4.4.3 Segment c

Segment c was processed using similar procedures as Segment a.

A.4.4.4 Segment d

Segment d was processed using similar procedures as Segment b, only that the “end” shift was taken as the averaged difference (0.46 mm).

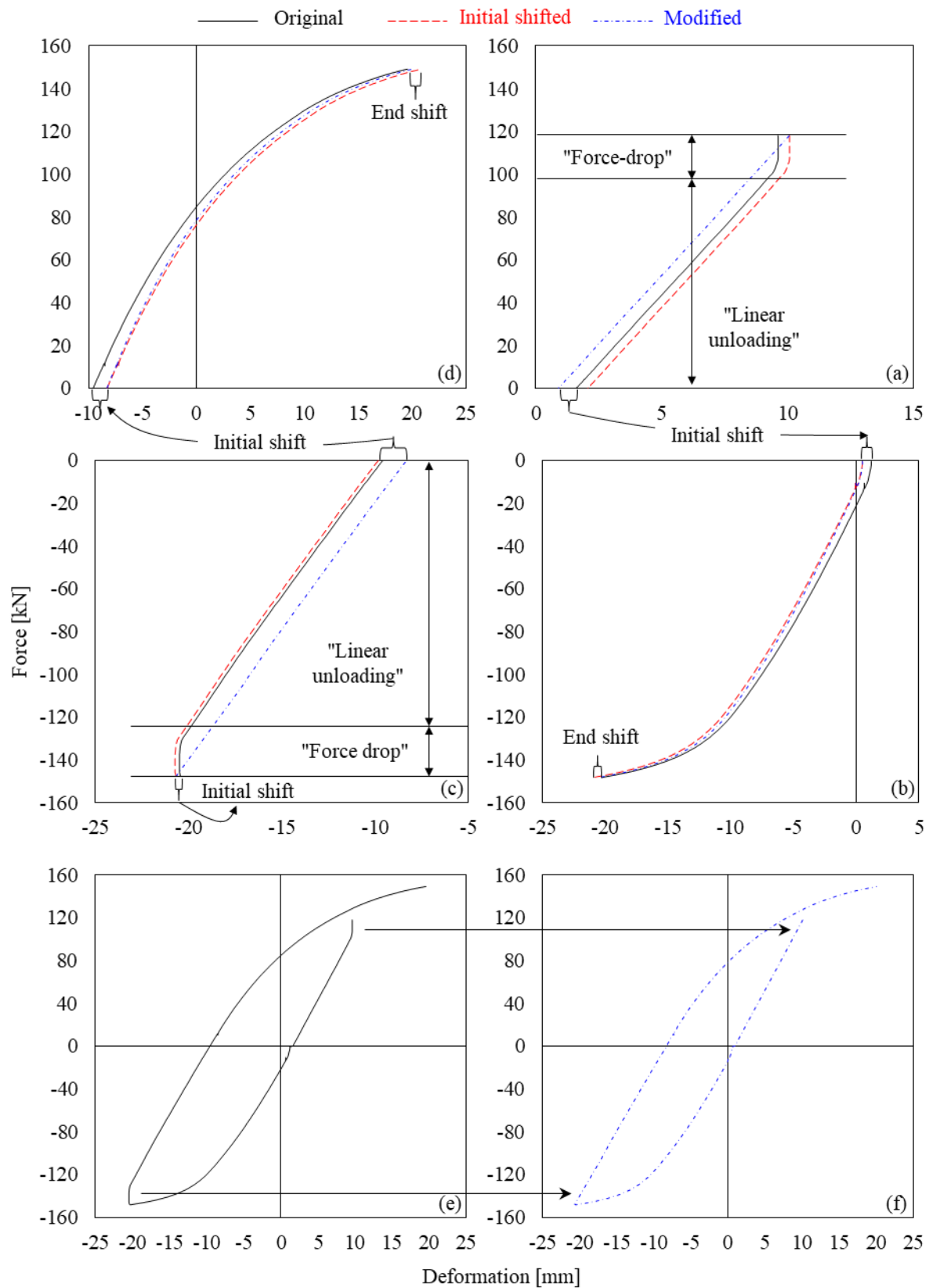


Figure A.6 Example of deformation compensation
 (a) ~ (d) Four segment; (e) Original cycle; (f) Compensated cycle.

A.4.5 Result of compensation

Figure A.7 illustrates the original and compensated hysteresis of Specimen 1. The improvement is obvious.

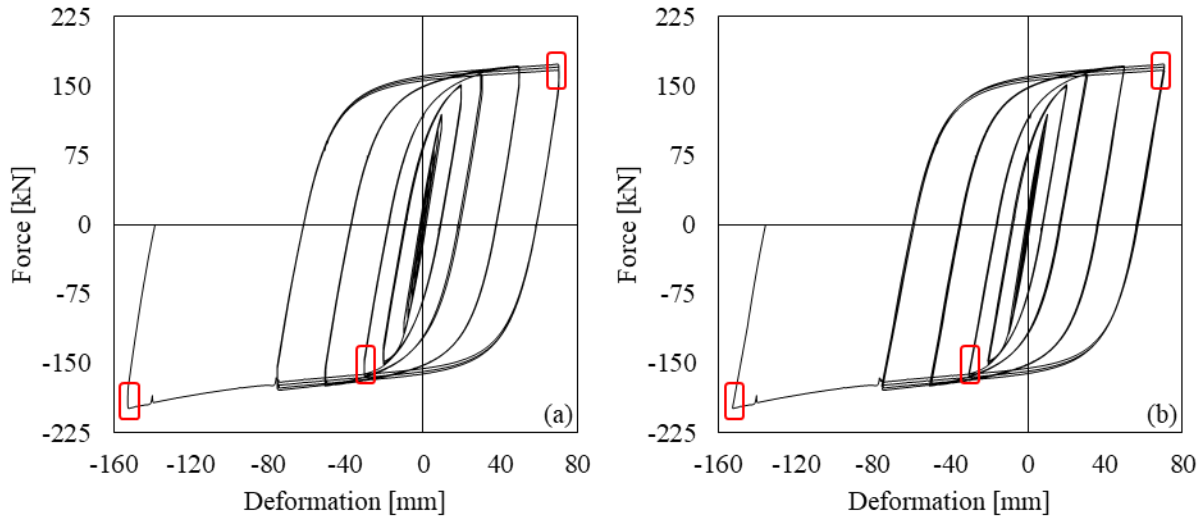


Figure A.7 Comparison of original and compensated deformation
(a) Original hysteresis; (b) Compensated hysteresis.

A.5 TEST RESULTS AND DISCUSSION

A.5.1 Specimen 1

A.5.1.1 Visual observations

In general, the test went as expected. No out of plane movement was observed during the whole loading process. Some relative motion between the specimen and the base plate can be seen from both time lapse and test data, but the movement was minor. Deformation of the base girders can be seen from time lapse, which was believed to be due to the insufficient local stiffness of the girder flanges. Since the deformation of the specimen was taken as the difference between the top and bottom displacement, such movement or deformation of the basement was believed to have no influence on the specimen stiffness.

Figure A.8 illustrates the specimen condition after the test. In the last large deformation loading, necking and minor cracks can be seen at the root of the specimen. Although not loaded reversely, it was believed that this can be deemed as an ultimate state of the specimen, with the deformation going up to 150 mm. After the test, it was found that the pin was bent, and we had to cut it to remove the specimen. Such bending suggested the possibility that the pin was not strong enough in both stiffness and strength, which require further investigation.

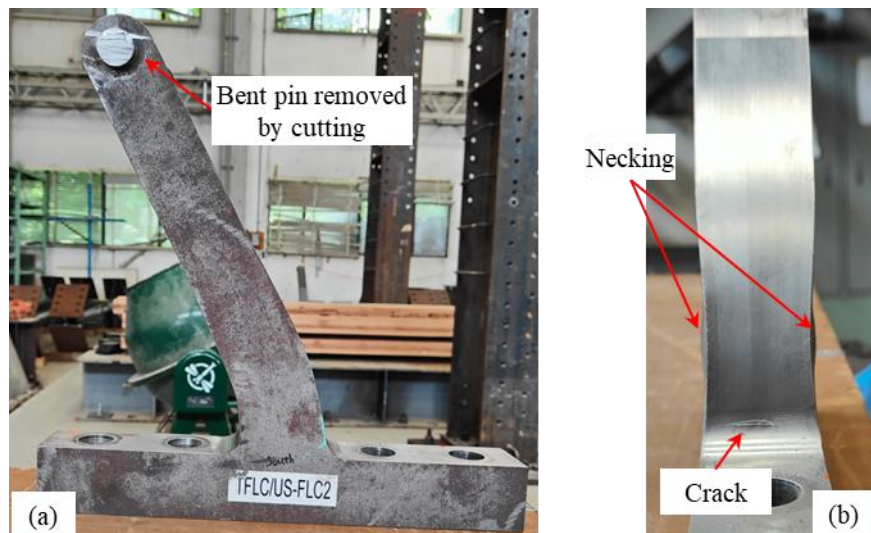


Figure A.8 Specimen 1 after test

A.5.1.2 Specimen hysteresis

Figure A.9 illustrates the hysteresis curve of the specimen, using the modified deformation. The hysteresis was stable throughout the test, showing a typical hysteresis of steel specimen. Isotropic hardening was minor in the tested deformation range. Kinematic hardening can be seen, but also not severe. From the zoomed in vision of the peak parts in the negative direction (pull), it can be seen that degradation occurred until the last level with +70/-75 mm target deformation. Previous levels showed almost no degradation within the same level. In the last strike with large deformation, some hardening can be seen, but still within the general trend of the cyclic loading portion.

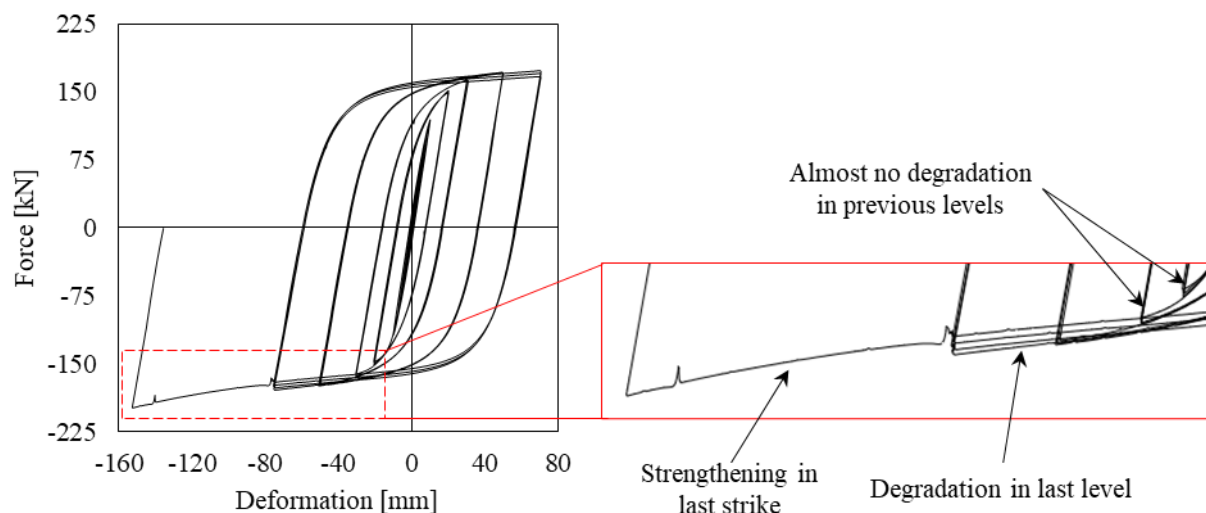


Figure A.9 Hysteresis of Specimen 1

In summary, considering the expected demand in the shake table tests, the T-FLCs showed sufficient capacity to fulfill the design requirements. From the cyclic test result, the T-FLCs can withstand deformation much larger than needed with limited degradation. The minor hardening also made it suitable for the purpose of capping the force transfer at the connections.

A.5.2 Specimen 2

A.5.2.1 Visual observations

General behavior of Specimen 2 was similar to that of Specimen 1. No out of plane movement was observed during the test. Since the target deformation was much smaller in this test, necking and cracking did not occur after the test. Bent of the pin was not observed and it was pulled out pretty easily. Figure A.10 illustrates the residual deformation of Specimen 2, with a direct comparison of Specimen 1.

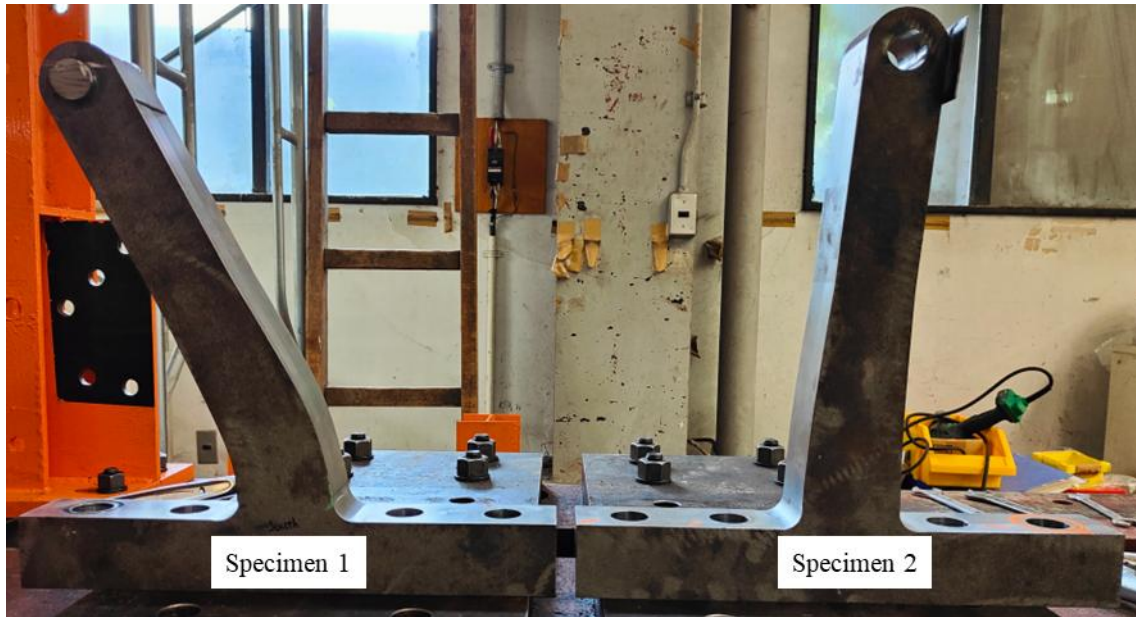


Figure A.10 Final condition of the specimens

A.5.2.2 Specimen hysteresis

Figure A.11 illustrates the hysteresis curves of Specimen 2, with the separated three phases. It can be seen that, regardless of the partial unloading and reloading, the shape of the curve was similar to that of Specimen 1. As aforementioned, the deformation of this specimen was much smaller than the 1st one, reaching about $+35/-18$ mm. In this range, the properties of the T-FLCs were quite stable, ensuring a reliable performance in the shake table tests.

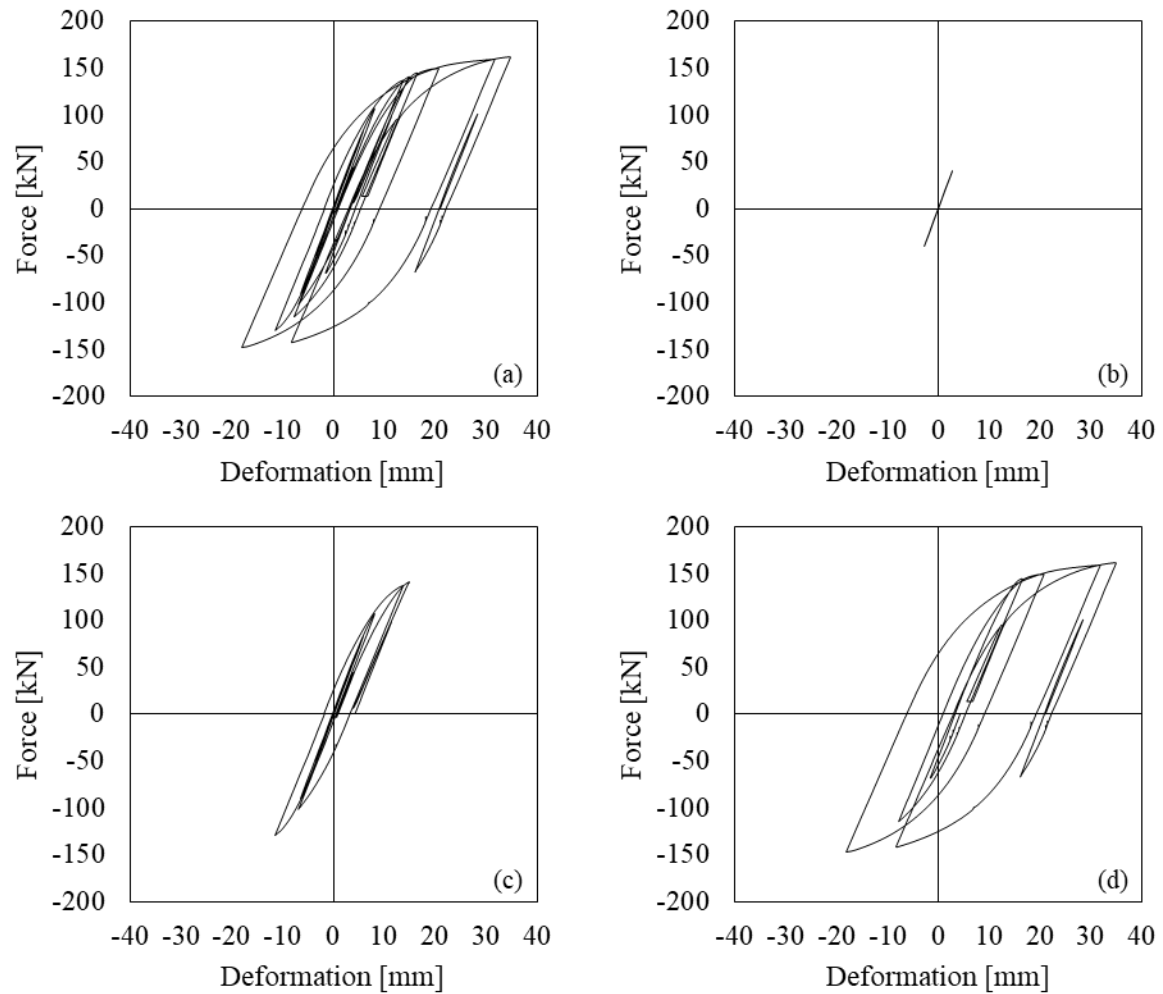


Figure A.11 Hysteresis of Specimen 2
(a) All phases; (b) Phase 1; (c) Phase 2; (d) Phase 3

A.5.2.3 Inconsistency of deformation in shake table and static tests

Table A.4 Comparison of peak deformation [Unit: mm]

Peaks	Shake table	Static	Peaks	Shake table	Static
Peak 1	-5.6	-3.4	Peak 7	-22.6	-6.8
Peak 2	10.5	5.9	Peak 8	-6.2	3.9
Peak 3	-11.7	-6.4	Peak 9	-11.7	0.7
Peak 4	12.7	8.1	Peak 10	20.9	14.9
Peak 5	-30.3	-11.4	Peak 11	9.5	3.9
Peak 6	21.3	13.7	Peak 12	17.2	11.4

Table A.4 summarizes the direct comparison of the T-FLC deformation in the shake table test and the static test. In almost all levels, the deformation in the static test was smaller than that measured in the shake table test. Such inconsistency was attributed to two major reasons:

- 1) Measurement in the shake table test took into account of some deformation of the connecting parts instead of the net deformation of the T-FLC.
- 2) The T-FLC had already yielded before the JMA Kobe 100% excitation, but in the static test it was

loaded from elastic range. Different starting point may result in different equivalent stiffness and thus different deformation.

A.5.3 Estimated parameters of the T-FLC

Using the results of the cyclic test, it is possible extract the mechanical properties of the T-FLCs. Here, a bilinear model calibrated using the backbone curve of Specimen 1 was proposed. Elastic stiffness, yielding force, and kinematic hardening ratio was extracted.

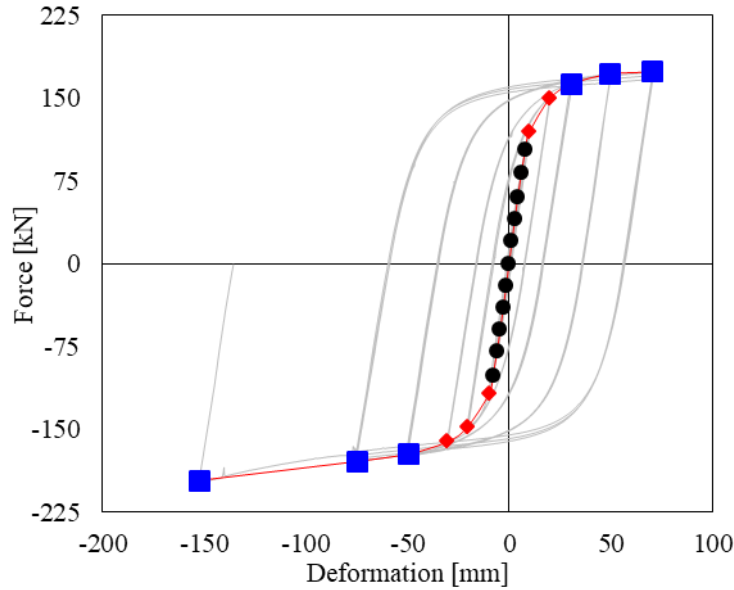


Figure A.12 Backbone curve of Specimen 1

Figure A.12 illustrates the backbone curve extracted using the peak force points of the 1st cycle in each level. The data points in elastic range (black dot) were used to get the elastic stiffness via linear regression. Last three points in the positive and negative directions (blue square) were used to get the post-elastic stiffness. The post-elastic stiffness of both directions was averaged as the post elastic stiffness of the bilinear model. Yielding point was obtained by finding the intersection of the elastic and post-elastic straight lines, while the hardening ratio was taken as the ratio of the two stiffness values.

The same procedure was also applied to the hysteresis from the test conducted at Lehigh University and the results are quite consistent. Table A.5 summarizes the extracted parameters from both tests.

Table A.5 Extracted mechanical parameters using bilinear model

Data source	F_y [kN]	Δ_y [mm]	K_e [kN/mm]	K_p [kN/mm]	b
Hokkaido Univ.	162.9	12.3	13.3	0.26	0.02
Lehigh Univ.	150.2	13.0	11.6	0.37	0.03

For more refined material models, such as the Menegotto-Pinto model, calibration can be done using hysteresis and backbone curves from both tests. Some trial and error procedures will be needed to get an satisfactory parameter set.

A.6 SUMMARY

Major findings from the static test are summarized as following:

- The performance of the T-FLCs was very stable in a pretty large deformation range, ensuring sufficient capacity for the usage under ground motion excitations.
- Deformation measured in the shake table test may contain contributions from other components.
- Some bending of the pin occurred, suggesting the necessity of further investigation on the demand of this type of connections.
- Preliminary mechanical properties of the T-FLCs were extracted from both this test and the Lehigh test. The results match satisfactorily. Such consistent results give confidence that calibration based on these test results can give a reasonable material for future simulation work.

APPENDIX B. SENSOR INFORMATION OF THE SHAKE TABLE TEST

B.1 GENERAL

This appendix provides details of the sensors used in the shake table test. In general, 192 channels were used, with 24 for accelerometers, 60 for displacement transducers, 4 for load cells, and 104 for strain gauges. In addition, 8 channels from the optical measurement system and 23 cameras were also used.

B.2 ACCELEROMETERS

Table B.1 summarizes information of accelerometers used in the test. Figure B.1 illustrates their location on the specimen. Positive directions were set as North for X, West for Y and Up for Z.

Table B.1 List of accelerometers

Channel tag	Direction	Setup location	Sensor location	Floor	Sensor specification
A-B2F-1X	North (X+)	South-West (Column C2)	-	1	Tokyo Keiki TA-25E-10-1 or TA-25J-10-1 ± 98.07 m/s ²
A-B2F-1Y	West (Y+)				
A-B2F-1Z	Up (Z+)				
A-B2F-2X	North (X+)	North-East (Column C3)	-		
A-B2F-2Y	West (Y+)				
A-B2F-2Z	Up (Z+)				
A-B3F-1X	North (X+)	South-West (Column C2)	-	2	
A-B3F-1Y	West (Y+)				
A-B3F-1Z	Up (Z+)				
A-B3F-2X	North (X+)	North-East (Column C3)	-		
A-B3F-2Y	West (Y+)				
A-B3F-2Z	Up (Z+)				
A-BRF-1X	North (X+)	South-West (Column C2)	-	3	
A-BRF-1Y	West (Y+)				
A-BRF-1Z	Up (Z+)				
A-BRF-2X	North (X+)	North-East (Column C3)	-		
A-BRF-2Y	West (Y+)				
A-BRF-2Z	Up (Z+)				
A-BRU-1X	North (X+)	South-West (Column C2)	-	4	
A-BRU-1Y	West (Y+)				
A-BRU-1Z	Up (Z+)				
A-BRU-2X	North (X+)	North-East (Column C3)	-		
A-BRU-2Y	West (Y+)				
A-BRU-2Z	Up (Z+)				

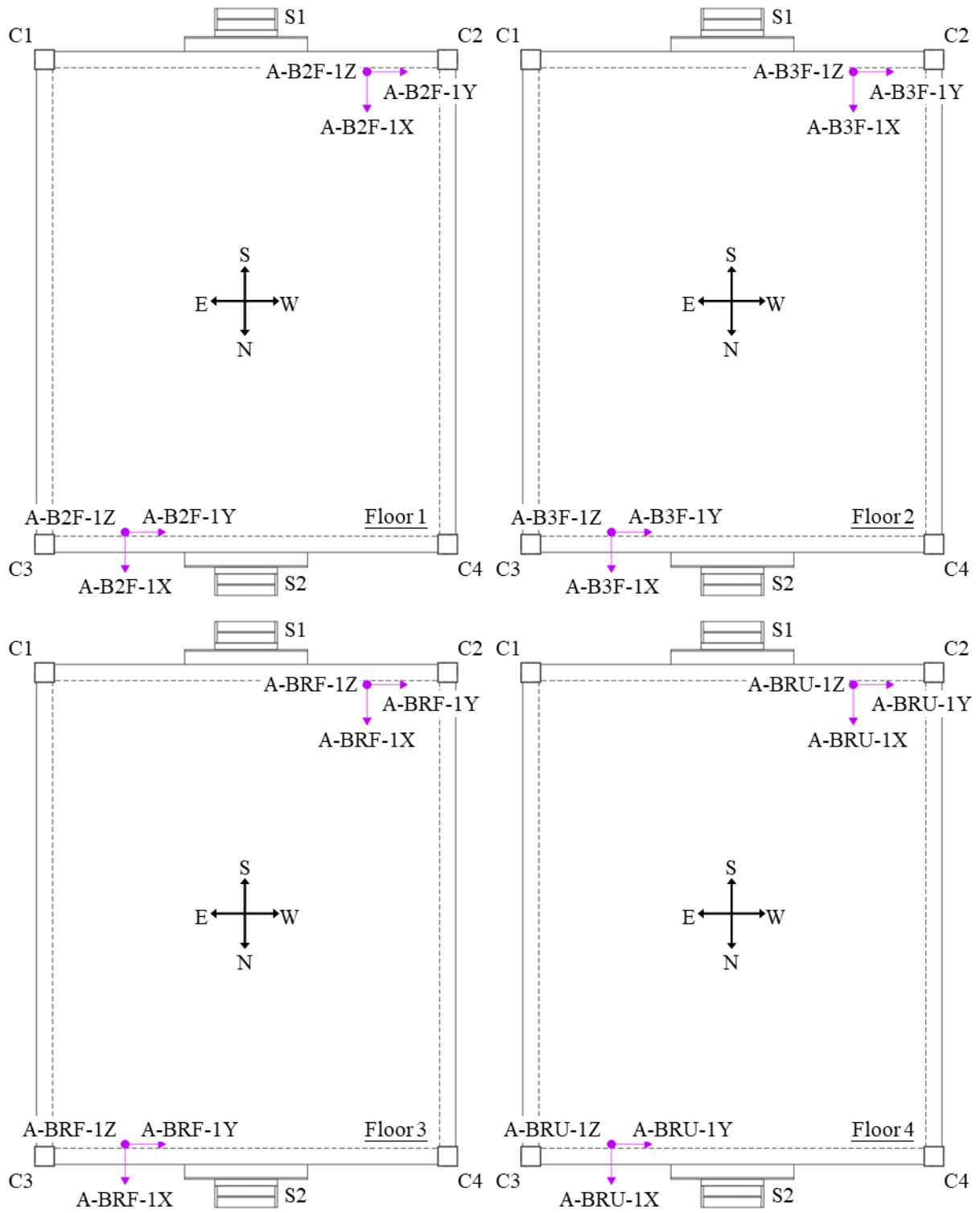


Figure B.1 Channel tags of accelerometers on each floor

B.3 DISPLACEMENT TRANSDUCERS

Table B.2 to Table B.5 summarize the list of displacement transducers used in the test to measure column and spine base rotation, beam end rotation, panel zone deformation, and MRF to spine connection deformation. Figure B.2 and Figure B.3 illustrate tags of displacement transducers on the specimen. For all the transducers, positive direction is set when the target moves away from the transducer.

Table B.2 List of displacement transducers for column/spine base rotation measurement

Channel tag	Direction	Setup location	Sensor location	Floor	Sensor specification
D-BB-C1-E	-	South-East	East	Shake table	KEYENCE LK-500 ± 100 mm
D-BB-C1-W	-	column (C1)	West		
D-BB-C2-E	-	South-West	East		
D-BB-C2-W	-	column (C2)	West		
D-BB-C3-E	-	North-East	East		
D-BB-C3-W	-	column (C3)	West		
D-BB-C4-E	-	North-West	East		
D-BB-C4-W	-	column (C4)	West		
D-BB-S1-E	-	South spine (S1)	East		
D-BB-S1-W	-		West		
D-BB-S2-E	-	North spine (S2)	East		
D-BB-S2-W	-		West		

Table B.3 List of displacement transducers for beam end rotation measurement

Channel tag	Direction	Setup location	Sensor location	Floor	Sensor specification		
D-B2F-B1-1U	-	South-East	Up	1	TML SDP-100CT ± 50 mm		
D-B2F-B1-1L	-	(Column C1)	Low				
D-B2F-B1-2U	-	South-West	Up				
D-B2F-B1-2L	-	(Column C2)	Low				
D-B3F-B1-1U	-	South-East	Up	2		TML SDP-100CT ± 50 mm	
D-B3F-B1-1L	-	(Column C1)	Low				
D-B3F-B1-2U	-	South-West	Up				
D-B3F-B1-2L	-	(Column C2)	Low				
D-B4F-B1-1U	-	South-East	Up	3			TML SDP-100CT ± 50 mm
D-B4F-B1-1L	-	(Column C1)	Low				
D-B4F-B1-2U	-	South-West	Up				
D-B4F-B1-2L	-	(Column C2)	Low				
D-B5F-B1-1U	-	South-East	Up	4	TML SDP-50CT ± 25 mm		
D-B5F-B1-1L	-	(Column C1)	Low				
D-B5F-B1-2U	-	South-West	Up				
D-B5F-B1-2L	-	(Column C2)	Low				

Table B.4 List of displacement transducers for panel zone deformation measurement

Channel tag	Direction	Setup location	Sensor location	Floor	Sensor specification
D-B2F-P1-E	-	South-East	Upper east	1	TML CDP-25 ± 12.5 mm
D-B2F-P1-W	-	(Column C1)	Upper west		
D-B2F-P2-E	-	South-West	Upper east		
D-B2F-P2-W	-	(Column C2)	Upper west		
D-B3F-P1-E	-	South-East	Upper east	2	
D-B3F-P1-W	-	(Column C1)	Upper west		
D-B3F-P2-E	-	South-West	Upper east		
D-B3F-P2-W	-	(Column C2)	Upper west		
D-B4F-P1-E	-	South-East	Upper east	3	
D-B4F-P1-W	-	(Column C1)	Upper west		
D-B4F-P2-E	-	South-West	Upper east		
D-B4F-P2-W	-	(Column C2)	Upper west		
D-B5F-P1-E	-	South-East	Upper east	4	
D-B5F-P1-W	-	(Column C1)	Upper west		
D-B5F-P2-E	-	South-West	Upper east		
D-B5F-P2-W	-	(Column C2)	Upper west		

Table B.5 List of displacement transducers for MRF-Spine connection deformation measurement

Channel tag	Direction	Setup location	Sensor location	Floor	Sensor specification
D-B2F-S1-E	-	South spine (S1)	East	1	KEYENCE LK-500 ± 100 mm
D-B2F-S1-W	-		West		
D-B2F-S2-E	-	North spine (S2)	East		
D-B2F-S2-W	-		West		
D-B3F-S1-E	-	South spine (S1)	East	2	
D-B3F-S1-W	-		West		
D-B3F-S2-E	-	North spine (S2)	East		
D-B3F-S2-W	-		West		
D-B4F-S1-E	-	South spine (S1)	East	3	
D-B4F-S1-W	-		West		
D-B4F-S2-E	-	North spine (S2)	East		
D-B4F-S2-W	-		West		
D-B5F-S1-E	-	South spine (S1)	East	4	
D-B5F-S1-W	-		West		
D-B5F-S2-E	-	North spine (S2)	East		
D-B5F-S2-W	-		West		

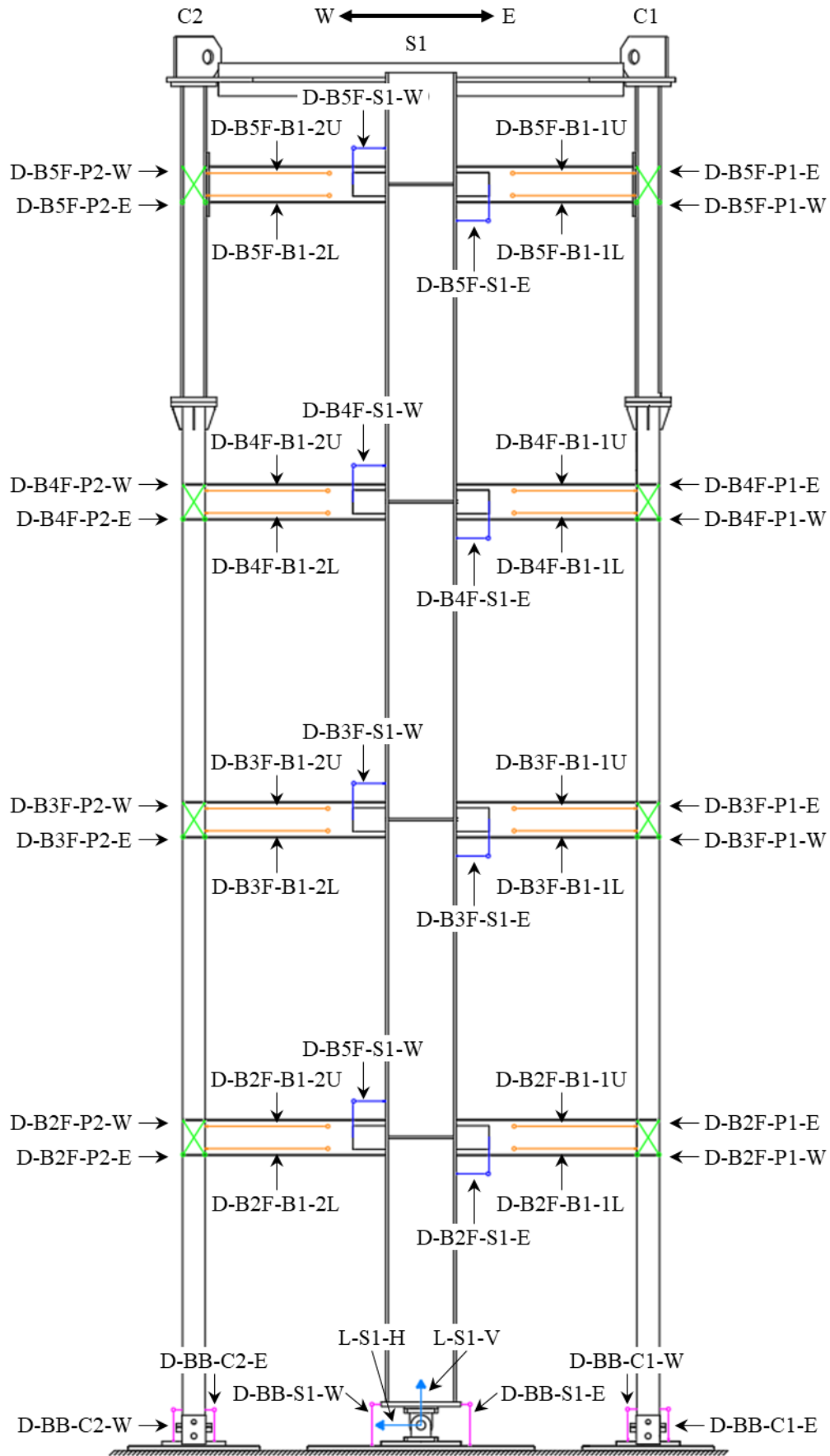


Figure B.2 Channel tags of displacement transducers and load cells at the South frame

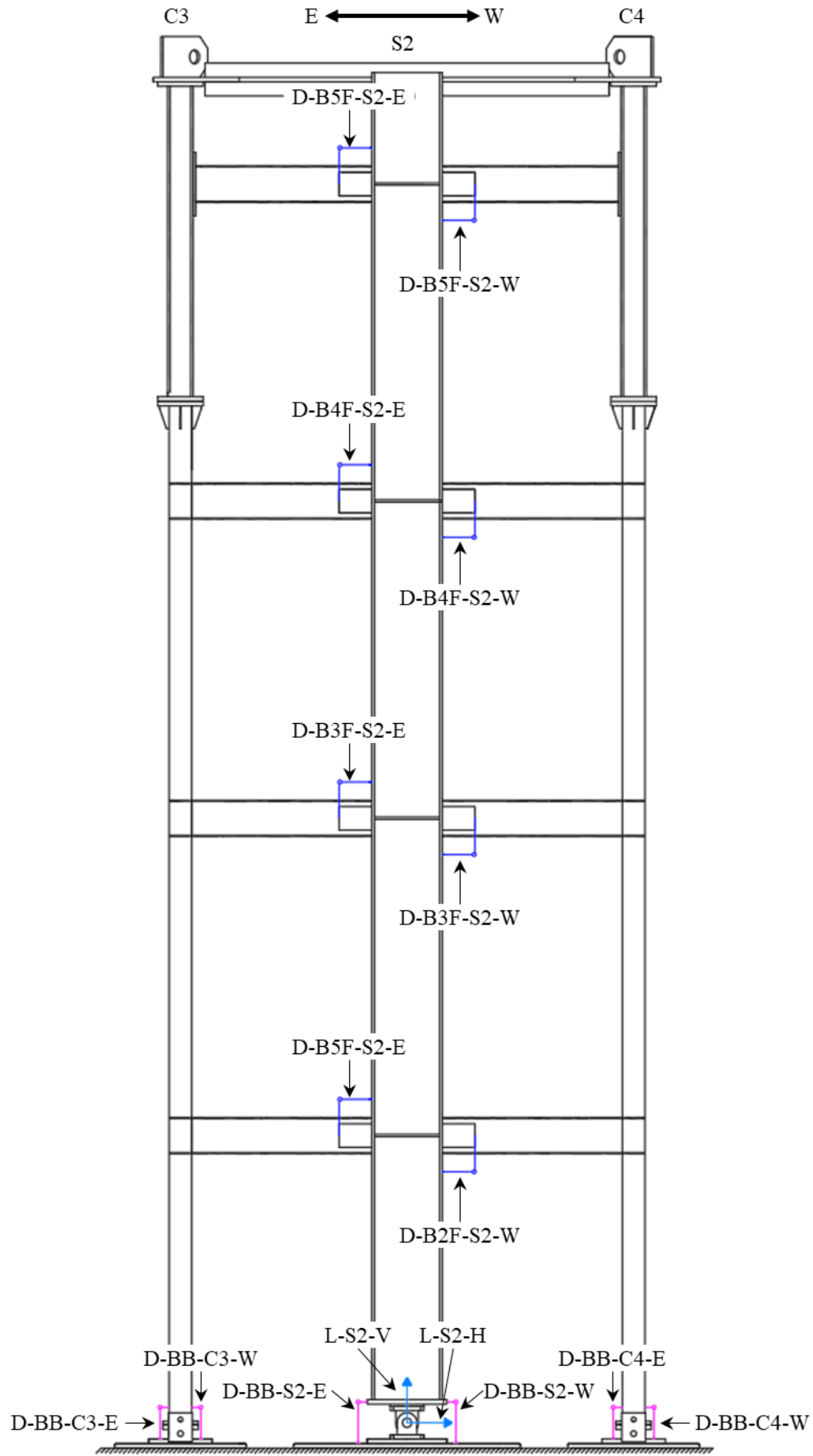


Figure B.3 Channel tags of displacement transducers and load cells at the North frame

B.4 LOADCELLS

Table B.6 summarizes the channels of the load cells set at the spine bases. Figure B.2 and Figure B.3 illustrate tags of loadcells on the specimen. Both load cells were placed with positive directions pointing West and Up for horizontal and vertical forces, respectively.

Table B.6 List of load cells

Channel tag	Direction	Setup location	Sensor location	Floor	Sensor specification
L-S1-H	West (Y+)	South spine (S1)	-	Shake table	TOMOE TORD-PS-750KN ± 750 kN
L-S1-V	Up (Z+)				
L-S2-H	West (Y+)	North spine (S2)	-	Shake table	TOMOE TORD-PS-750KN ± 750 kN
L-S2-V	Up (Z+)				

B.5 STRAIN GAUGES

Table B.7 to Table B.13 summarize the channel tags of strain gauges on the four columns and two spines. Figure B.4 and Figure B.5 illustrate tags of strain gauges on the specimen. For all columns, 4 uniaxial gauges were placed at each story measuring 2 sections. For both spines, 4 uniaxial gauges were placed at the 2nd to 4th stories measuring 2 sections, while 1 biaxial gauge was placed at the 1st to 4th stories measuring shear strain of 1 section.

Table B.7 List of strain gauges on column C1 (South-East)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
S-B1FC1-1L	-	Lower section	East surface	1	TML FLA-5-11
S-B1FC1-2L	-		West surface		
S-B1FC1-1U	-	Upper section	East surface		
S-B1FC1-2U	-		West surface		
S-B2FC1-1L	-	Lower section	East surface	2	
S-B2FC1-2L	-		West surface		
S-B2FC1-1U	-	Upper section	East surface		
S-B2FC1-2U	-		West surface		
S-B3FC1-1L	-	Lower section	East surface	3	
S-B3FC1-2L	-		West surface		
S-B3FC1-1U	-	Upper section	East surface		
S-B3FC1-2U	-		West surface		
S-B4FC1-1L	-	Lower section	East surface	4	
S-B4FC1-2L	-		West surface		
S-B4FC1-1U	-	Upper section	East surface		
S-B4FC1-2U	-		West surface		

Table B.8 List of strain gauges on column C2 (South-West)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
S-B1FC2-1L	-	Lower section	East surface	1	TML FLA-5-11
S-B1FC2-2L	-		West surface		
S-B1FC2-1U	-	Upper section	East surface		
S-B1FC2-2U	-		West surface		
S-B2FC2-1L	-	Lower section	East surface	2	
S-B2FC2-2L	-		West surface		
S-B2FC2-1U	-	Upper section	East surface		
S-B2FC2-2U	-		West surface		
S-B3FC2-1L	-	Lower section	East surface	3	
S-B3FC2-2L	-		West surface		
S-B3FC2-1U	-	Upper section	East surface		
S-B3FC2-2U	-		West surface		
S-B4FC2-1L	-	Lower section	East surface	4	
S-B4FC2-2L	-		West surface		
S-B4FC2-1U	-	Upper section	East surface		
S-B4FC2-2U	-		West surface		

Table B.9 List of strain gauges on column C3 (North-East)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
S-B1FC3-1L	-	Lower section	East surface	1	TML FLA-5-11
S-B1FC3-2L	-		West surface		
S-B1FC3-1U	-	Upper section	East surface		
S-B1FC3-2U	-		West surface		
S-B2FC3-1L	-	Lower section	East surface	2	
S-B2FC3-2L	-		West surface		
S-B2FC3-1U	-	Upper section	East surface		
S-B2FC3-2U	-		West surface		
S-B3FC3-1L	-	Lower section	East surface	3	
S-B3FC3-2L	-		West surface		
S-B3FC3-1U	-	Upper section	East surface		
S-B3FC3-2U	-		West surface		
S-B4FC3-1L	-	Lower section	East surface	4	
S-B4FC3-2L	-		West surface		
S-B4FC3-1U	-	Upper section	East surface		
S-B4FC3-2U	-		West surface		

Table B.10 List of strain gauges on column C4 (North-West)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
S-B1FC4-1L	-	Lower section	East surface	1	TML FLA-5-11
S-B1FC4-2L	-		West surface		
S-B1FC4-1U	-	Upper section	East surface		
S-B1FC4-2U	-		West surface		
S-B2FC4-1L	-	Lower section	East surface	2	
S-B2FC4-2L	-		West surface		
S-B2FC4-1U	-	Upper section	East surface		
S-B2FC4-2U	-		West surface		
S-B3FC4-1L	-	Lower section	East surface	3	
S-B3FC4-2L	-		West surface		
S-B3FC4-1U	-	Upper section	East surface		
S-B3FC4-2U	-		West surface		
S-B4FC4-1L	-	Lower section	East surface	4	
S-B4FC4-2L	-		West surface		
S-B4FC4-1U	-	Upper section	East surface		
S-B4FC4-2U	-		West surface		

Table B.11 List of uniaxial strain gauges on spine S1 (South)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
-	-	-	-	1	TML FLA-5-11
-	-		-		
-	-	-	-		
-	-		-		
S-B2FS1-1L	-	Lower section	East flange	2	
S-B2FS1-2L	-		West flange		
S-B2FS1-1U	-	Upper section	East flange		
S-B2FS1-2U	-		West flange		
S-B3FS1-1L	-	Lower section	East flange	3	
S-B3FS1-2L	-		West flange		
S-B3FS1-1U	-	Upper section	East flange		
S-B3FS1-2U	-		West flange		
S-B4FS1-1L	-	Lower section	East flange	4	
S-B4FS1-2L	-		West flange		
S-B4FS1-1U	-	Upper section	East flange		
S-B4FS1-2U	-		West flange		

Table B.12 List of uniaxial strain gauges on spine S2 (North)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
-	-	-	-	1	TML FLA-5-11
-	-	-	-		
-	-	-	-		
-	-	-	-		
S-B2FS2-1L	-	Lower section	East flange	2	
S-B2FS2-2L	-		West flange		
S-B2FS2-1U	-	Upper section	East flange		
S-B2FS2-2U	-		West flange		
S-B3FS2-1L	-	Lower section	East flange	3	
S-B3FS2-2L	-		West flange		
S-B3FS2-1U	-	Upper section	East flange		
S-B3FS2-2U	-		West flange		
S-B4FS2-1L	-	Lower section	East flange	4	
S-B4FS2-2L	-		West flange		
S-B4FS2-1U	-	Upper section	East flange		
S-B4FS2-2U	-		West flange		

Table B.13 List of biaxial strain gauges on spines S1 and S2 (South and North)

Channel tag	Direction	Section location	Sensor location	Story	Sensor specification
S-B1FS1-1M	-	South spine	East leg	1	TML FCA-5-11
S-B1FS1-2M	-	(S1)	West leg		
S-B1FS2-1M	-	North spine	East leg		
S-B1FS2-2M	-	(S2)	West leg		
S-B2FS1-1M	-	South spine	East leg	2	
S-B2FS1-2M	-	(S1)	West leg		
S-B2FS2-1M	-	North spine	East leg		
S-B2FS2-2M	-	(S2)	West leg		
S-B3FS1-1M	-	South spine	East leg	3	
S-B3FS1-2M	-	(S1)	West leg		
S-B3FS2-1M	-	North spine	East leg		
S-B3FS2-2M	-	(S2)	West leg		
S-B4FS1-1M	-	South spine	East leg	4	
S-B4FS1-2M	-	(S1)	West leg		
S-B4FS2-1M	-	North spine	East leg		
S-B4FS2-2M	-	(S2)	West leg		

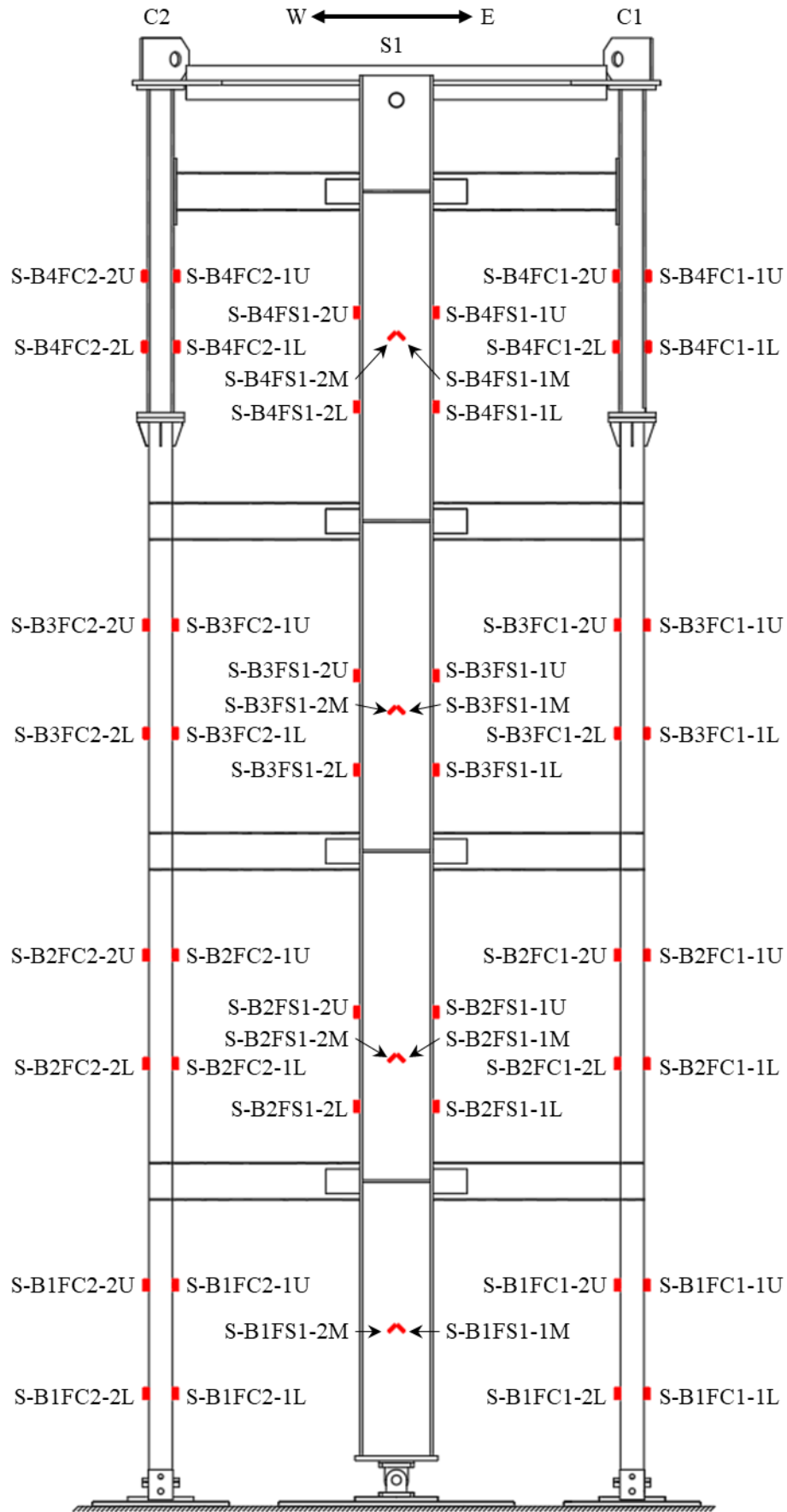


Figure B.4 Channel tags of strain gauges at the South frame

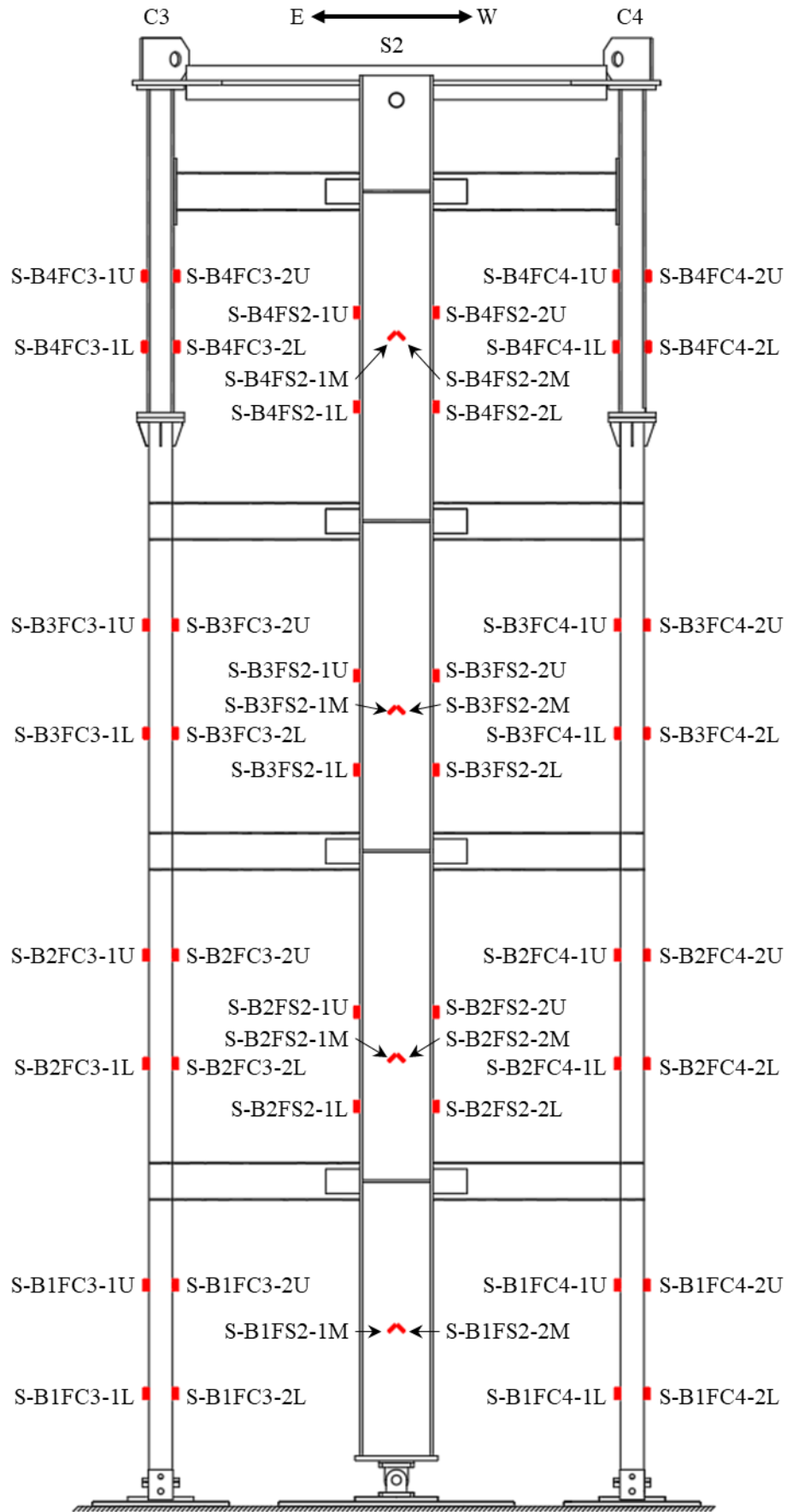


Figure B.5 Channel tags of strain gauges at the North frame

B.6 STORY DRIFT MEASUREMENT

Table B.14 summarizes the channels of the story drift measurement. Figure B.6 illustrates the tags on each floor. This measurement was done using an optical measurement system independent from the E-Defense measurement system. Tags for these channels differ from those specified in the optical system. This was done because each channel of the optical system contains story drift in both X and Y directions. Mapping of the new tags and the optical channels are included in the table. For the optical system, positive directions were defined as when the upper floor moved to the south and east with respect to the lower floor, respectively. Directions in the table are in the E-Defense system, which is opposite to the optical system.

Table B.14 List of optical measurement channels

Channel tag	Direction	Setup location	Original channel	Story	Sensor location
SD-B1S-1X	South (X-)	North-East	Channel 01	1	
SD-B1S-1Y	East (Y-)	(Column C3)	(ch01)		
SD-B1S-2X	South (X-)	South-West	Channel 02		
SD-B1S-2Y	East (Y-)	(Column C2)	(ch02)		
SD-B2S-1X	South (X-)	North-East	Channel 03	2	
SD-B2S-1Y	East (Y-)	(Column C3)	(ch03)		
SD-B2S-2X	South (X-)	South-West	Channel 04		
SD-B2S-2Y	East (Y-)	(Column C2)	(ch04)		
SD-B3S-1X	South (X-)	North-East	Channel 05	3	
SD-B3S-1Y	East (Y-)	(Column C3)	(ch05)		
SD-B3S-2X	South (X-)	South-West	Channel 06		
SD-B3S-2Y	East (Y-)	(Column C2)	(ch06)		
SD-B4S-1X	South (X-)	North-East	Channel 07	4	
SD-B4S-1Y	East (Y-)	(Column C3)	(ch07)		
SD-B4S-2X	South (X-)	South-West	Channel 08		
SD-B4S-2Y	East (Y-)	(Column C2)	(ch08)		

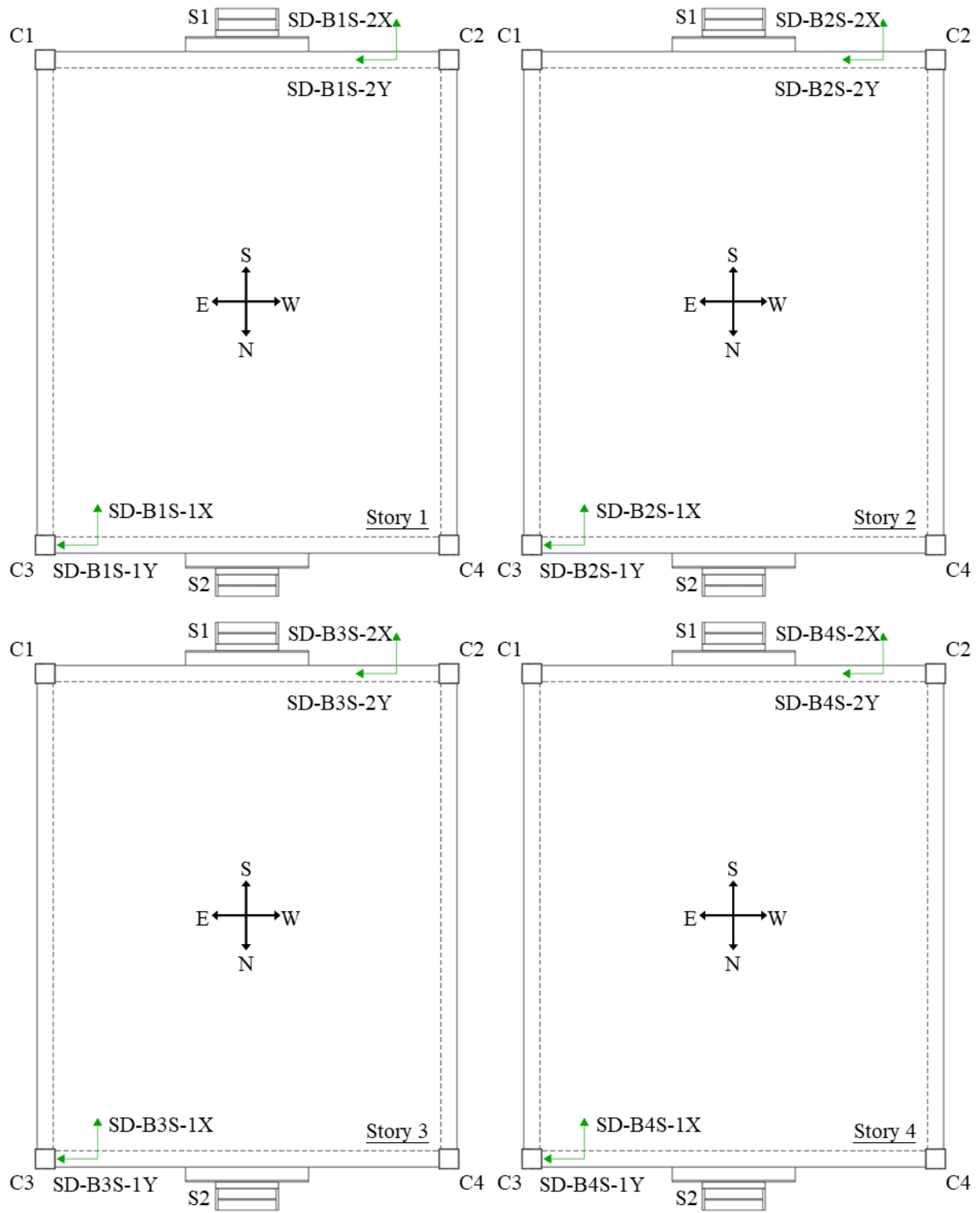


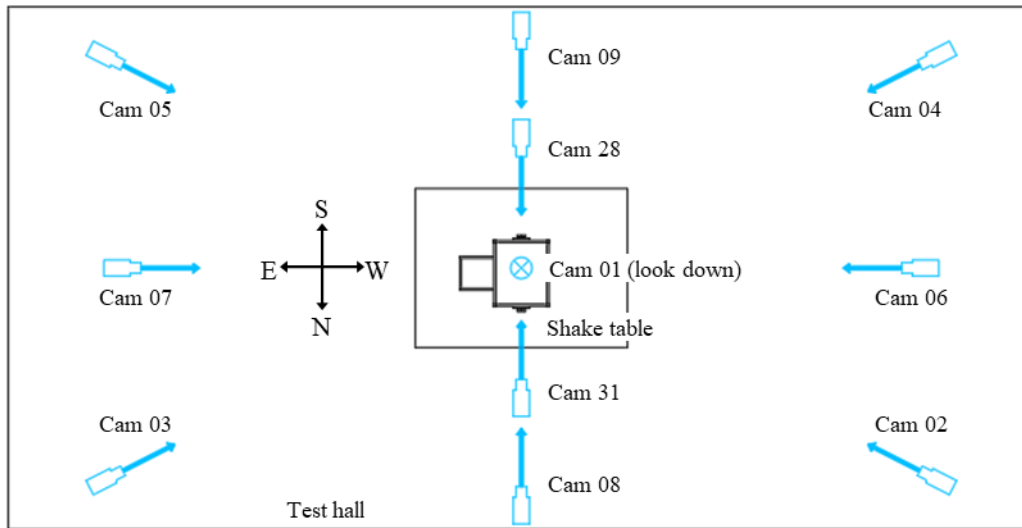
Figure B.6 Channel tags of story drift measurement system

B.7 CAMERAS

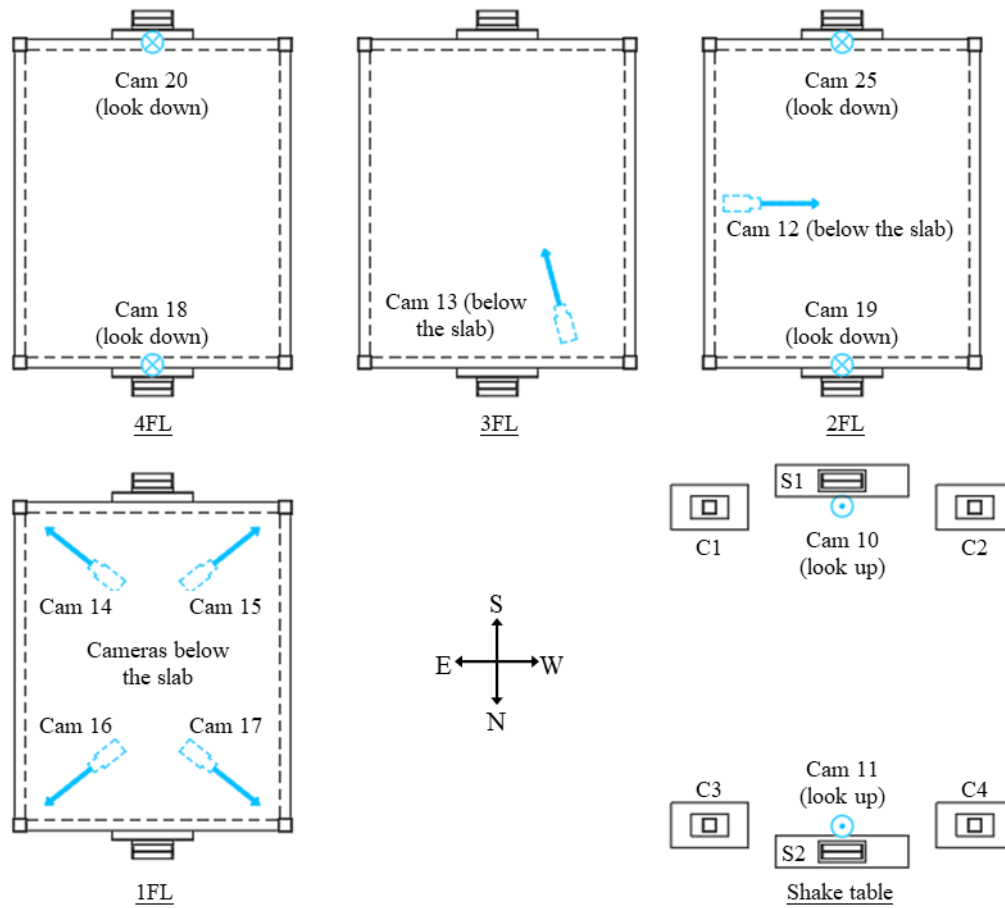
Table B.15 summarizes the information on cameras used in the test. Figure B.7 illustrates the tags in the test hall and on the specimen. The cameras include 9 fixed cameras of the E-Defense test hall, 12 cameras on the specimen, and 2 portable cameras on the floor of the test hall. All test cases were captured.

Table B.15 Information of cameras

Camera tag	Camera type	Camera location	Target scene
Cam 01	Test hall fixed	Ceiling	Specimen global movement
Cam 02		North-West	
Cam 03		North-East	
Cam 04		South-West	
Cam 05		South-East	
Cam 06		West	
Cam 07		East	
Cam 08		North	
Cam 09		South	
Cam 10	On specimen	Spine-1 at shake table level	T-FLC from below
Cam 11		Spine-2 at shake table level	T-FLC from below
Cam 12		East side at Story 2	NICU equipment
Cam 13		North side at Story 3	Dialysis room equipment
Cam 14		South-East under 1FL slab	Panel zone of C1
Cam 15		South-West under 1FL slab	Panel zone of C2
Cam 16		North-East under 1FL slab	Panel zone of C3
Cam 17		North-West under 1FL slab	Panel zone of C4
Cam 18		Spine-2 at 4FL level	U-FLC from above
Cam 19		Spine-2 at 2FL level	T-FLC from above
Cam 20		Spine-1 at 4FL level	U-FLC from above
Cam 25		Spine-1 at 2FL level	T-FLC from above
Cam 28	On floor	South of specimen on floor	South elevation of specimen
Cam 31		North of specimen on floor	North elevation of specimen



(a)



(b)

Figure B.7 Channel tags of camera
(a) Cameras in the test hall; (b) cameras on the specimen.

APPENDIX C. DATA PREPROCESSING FOR REDUCTION

C.1 GENERAL

This appendix introduces data preprocessing to prepare the raw data for response reduction. Data from the measurement systems were first checked channel by channel as time domain signals to find out the problematic channels. Then preprocessing was conducted as necessary. Details are introduced in the following sections.

C.2 ACCELERATION

For the acceleration data, all channels were preprocessed. Outlier removal was applied as necessary while filtering was applied to all channels. In general, outlier removal was conducted prior to filtering to avoid the influence of outliers on the frequency responses. When removing the outliers, the signal was first plotted, then the outliers were manually selected on the plot and further replaced by linear interpolation. For the filtering, the bi-directional Butterworth Low-pass filter was used for all channels to provide flat pass band and avoid phase distortion. The filter was set with the order of 20 and cutoff frequency of 35 Hz for most of the channels, but was also adjusted if not suitable. Figure C.1 illustrates an example using the acceleration data at the 1st floor from Case 09, the Northridge VA 100% excitation in the MRF-Spine system, with the filter parameters set as cut-off frequency of 35Hz and order of 20.

It should be noted that tags of **A-B2F-2Y** (channel 133)) and **A-B2F-2Z** (channel 134) were flipped. They were flipped back before all other processing. These two channels came from a triaxial accelerometer set at the north-east corner (near column C3) on the 1st floor slab.

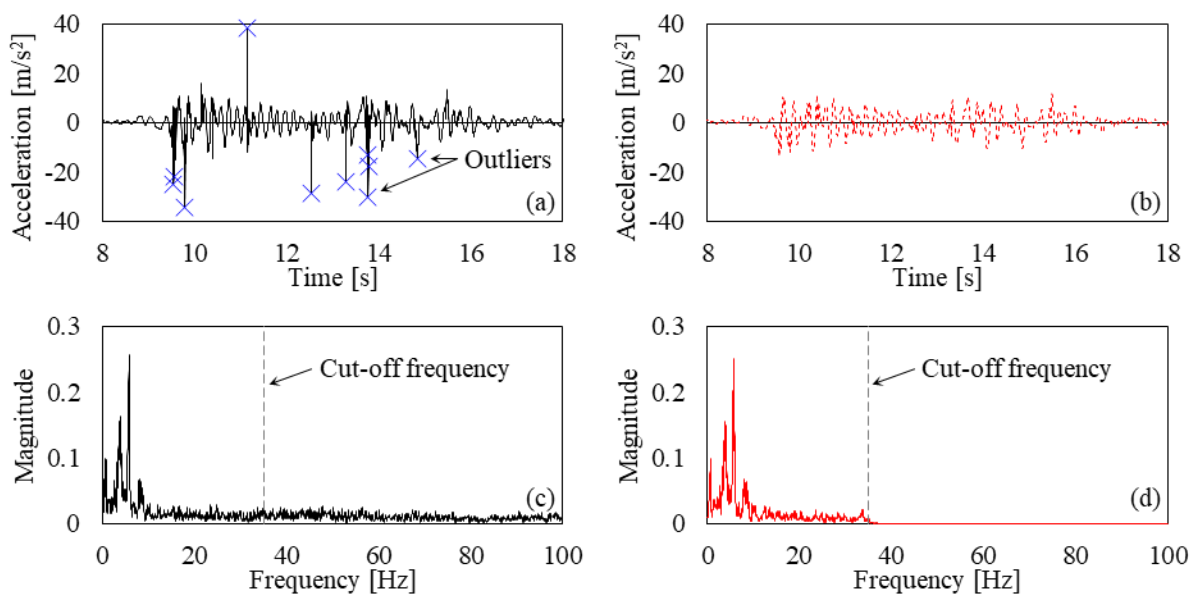


Figure C.1 Example of acceleration preprocessing

(a) Original signal; (b) Processed signal; (c) Original Fourier spectrum; (d) Processed Fourier spectrum

C.3 LOCAL DISPLACEMENT

C.3.1 General processing procedures

For most of the local displacement measurement, since the transducers were not frequency sensitive devices, only outlier-removal is needed. Figure C.2 illustrates an example of displacement processing, using data of the south spine base rotation taken from Case 04, Northridge VA 100% excitation in the MRF-Spine system.

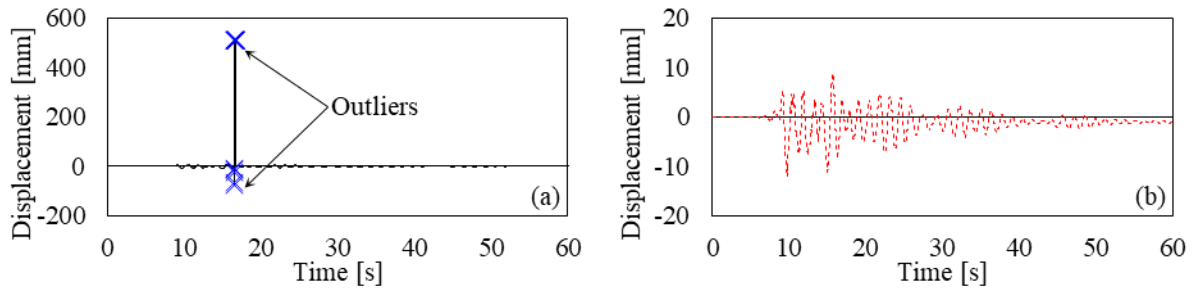


Figure C.2 Example of local displacement preprocessing
(a) Original signal; (b) Processed signal

C.3.2 Special treatment on beam end rotation measurement

For the beam end rotation measurement at the 2nd and 3rd floors, the sensors used were TML SDP-100CT while in the E-Defense system they were mistakenly recorded as TML SDP-50CT. Factors of these two models are 0.02 and 0.01, respectively. Thus, the values from the measurement system were half of the real number. Before any processing, values from related channels were corrected by multiplying 2. Table C.1 summarizes the information of the affected transducers.

Table C.1 Sensors require treatment for beam end measurement

Sensor tag	Location	Floor	Sensor tag	Location	Floor
D-B3F-B1-1U	East-Up	2	D-B4F-B1-1U	East-Up	3
D-B3F-B1-1L	East-Low		D-B4F-B1-1L	East-Low	
D-B3F-B1-2U	West-Up		D-B4F-B1-2U	West-Up	
D-B3F-B1-2L	West-Low		D-B4F-B1-2L	West-Low	

Besides, due to the configuration of measurement setup, large distortion occurred in the beam end measurement. Aggressive filtering was conducted on these data to make them better (although not perfect) to use. The filtering also utilized bi-directional Butterworth lowpass filter with the order of 8 and cutoff frequency of 20 Hz. Figure C.3 illustrates an example of the preprocessing of beam end measurement

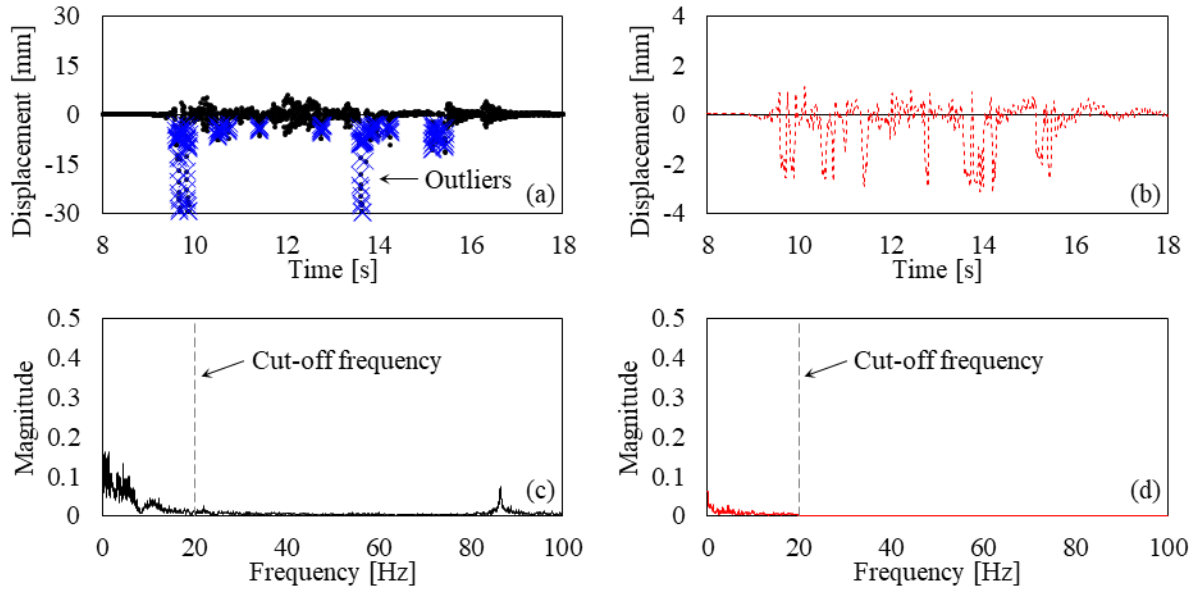


Figure C.3 Example of beam end rotation preprocessing

(a) Original signal; (b) Processed signal; (c) Original Fourier spectrum; (d) Processed Fourier spectrum

C.4 STORY DRIFT

Since the story drift was measured by the optical system, data from this system need to be synchronized with those from the E-Defense data acquisition (DAQ) system. The synchronization used the story drift measurement from the optical system and those obtained from double-integration using the acceleration data of the E-Defense system. Figure C.4 illustrates a flowchart of the synchronization.

For the data from the optical system, to take full advantage of the high sampling frequency of the E-Defense DAQ system (200 Hz), they were up-sampled from 50 Hz to 200 Hz using linear interpolation. For the DAQ system data, the floor acceleration data were firstly filtered using Butterworth bandpass filter to remove the long period components, then integrated into velocity data. The velocity data were again bandpass filtered and integrated, getting the floor displacement data. The filter applied to both acceleration and velocity data was a Butterworth bandpass filter with cutoff frequency from 0.5 Hz to 30 Hz and order of 6. The displacement data were further calculated into story drift data for synchronization. Synchronization was conducted using the Matlab® function ‘finddelay’, which can give the delay between two input signals. After determination of the delay, the optical data were then trimmed and compensated to get the same length of the DAQ data.

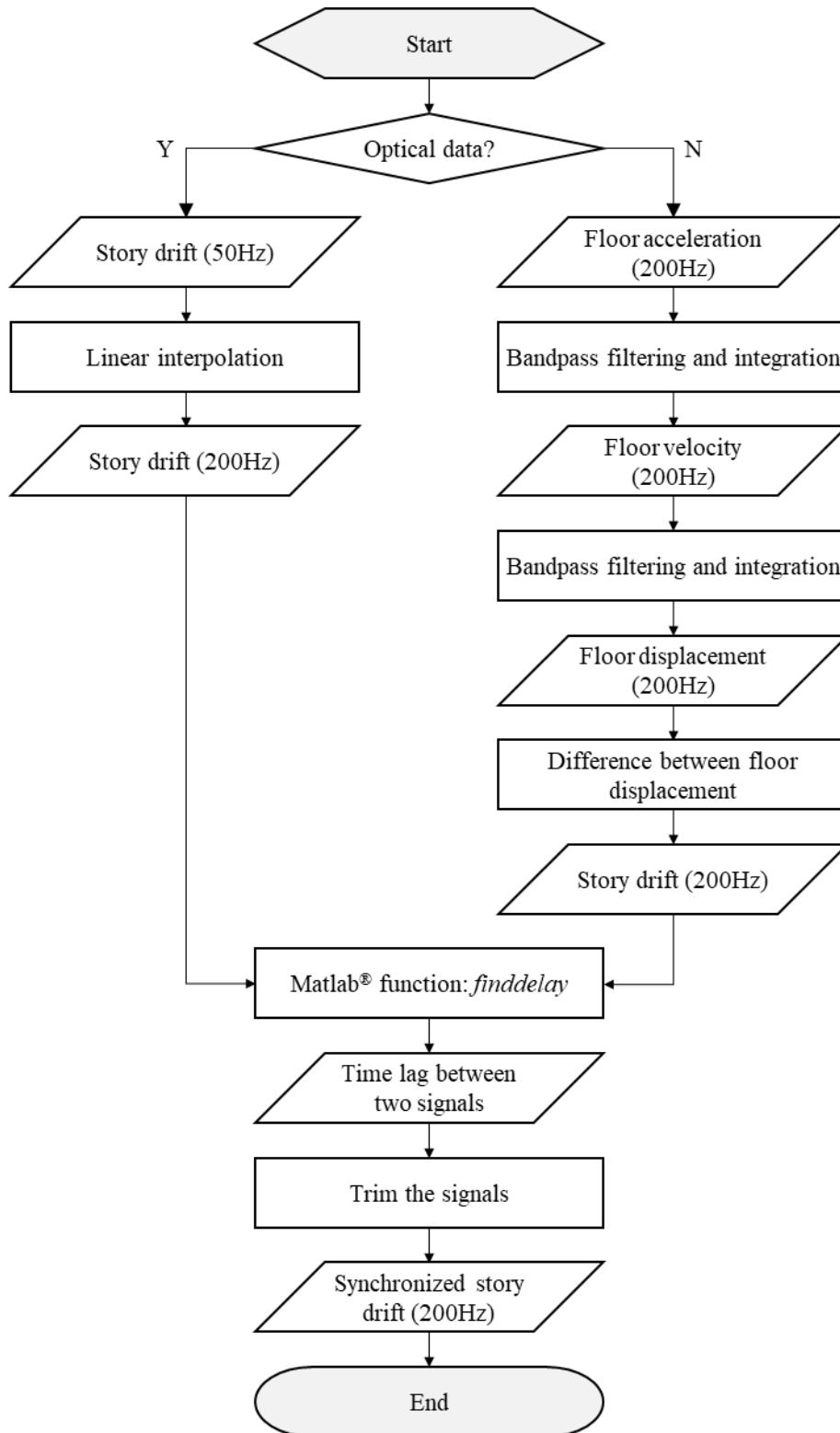


Figure C.4 Flowchart of story drift synchronization

Since the integrated story drift had almost no residual values but the directly measured data had, and ‘finddelay’ function cannot deal with the signals with too much difference, only the segment without much residual values of the story drift data from the optical measurement system were used for synchronization. In accordance with this, data from the DAQ system were also trimmed to the similar range by eyeballing. This data selection was applied to Case 13, the last strong motion excitation, as

illustrated in Figure C.5.

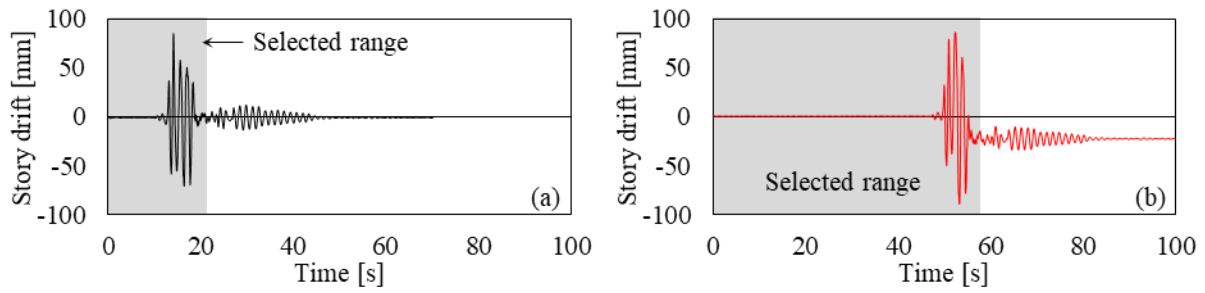


Figure C.5 Data selection for synchronization of Case 13

(a) E-Defense DAQ system (integration from acceleration); (b) Optical measurement system

Figure C.6 illustrates some of the synchronization results, taken from the 2nd story of the primary excitations). It can be seen that the synchronization worked satisfactorily. Since all the data from the E-Defense DAQ system were naturally synchronized, these up-sampled and synchronized optical data can then be used together with whichever data necessary.

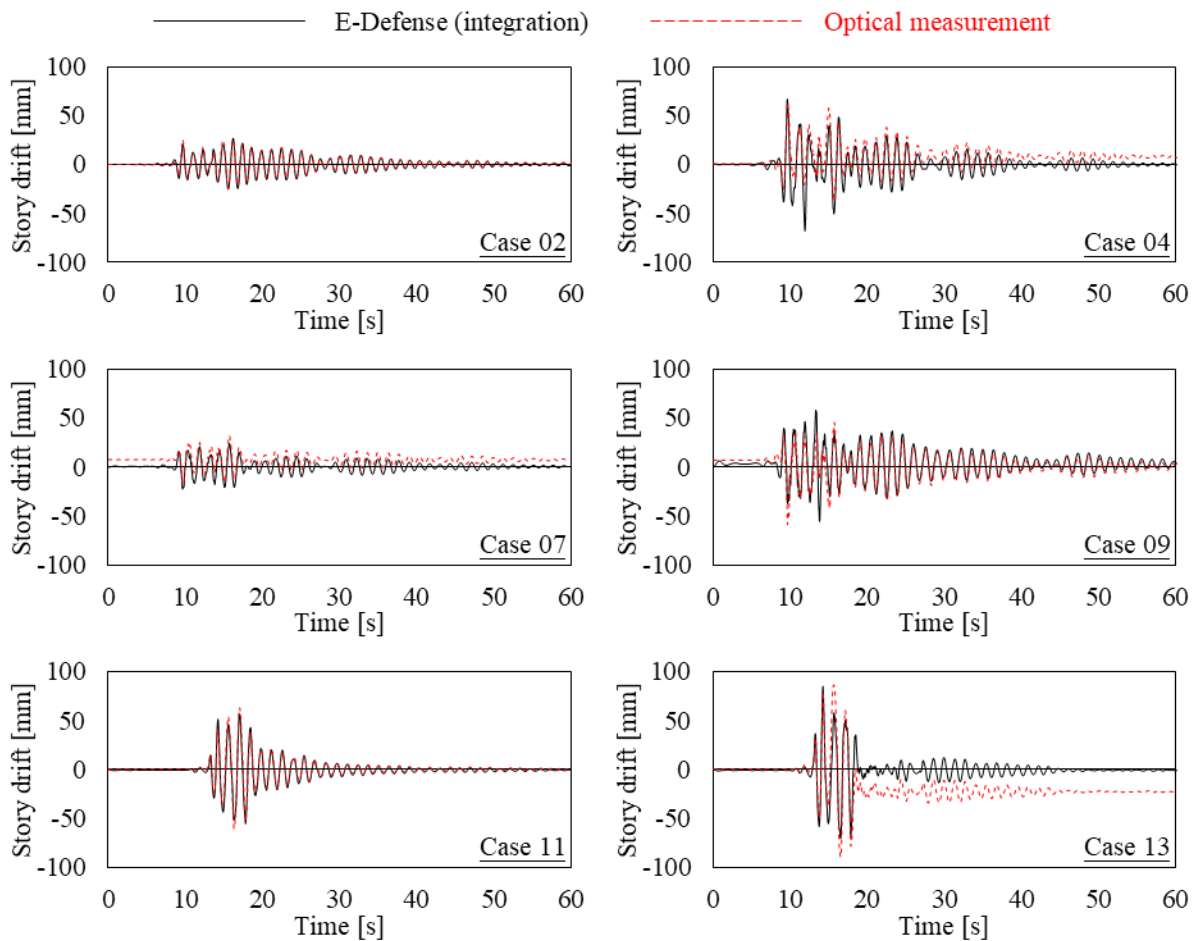


Figure C.6 Example results of story drift synchronization at the 2nd story

C.5 STRAIN

C.5.1 General

Since the strain gauges were not frequency sensitive devices, filtering was conducted only when necessary. In addition, some strain gauges exceeded the elastic limit or failed during the test. These strain gauges were compensated using information from other sensors and some basic assumptions. Table C.2 summarizes the strain gauges with problems. The compensation procedures are introduced in the following sections. The yielding gauges, although with reliable readings, cannot provide information on how elastic and plastic strain were proportioned even in the small excitations. Thus, these gauges were deemed the same as those failed gauges since the first time yielding occurred. Considering this, compensation for all the gauges was conducted for the cases after their initial yielding or failure.

In addition to the general case, tags of strain gauges S-B1FC3-1U (channel 146) and S-B1FC3-2U (channel 147) were flipped. They were flipped back before any processing was conducted. Tags in the table represent the corrected tags. These two gauges were set at the upper section of the north-east column at the 1st story, with “1” referring to east side of the column and “2” west side. Details of the tagging can be found in Appendix B.

Table C.2 Strain gauges with problems

Tag	Column/Spine	Gauge location	Story	Problem	Affected cases
S-B1FC1-2U	South-east	Lower west	1	Yielded	Case 04 and after
S-B1FC2-1U	South-west	Upper east	1	Yielded	Case 04 and after
S-B1FC3-2U	North-east	Upper west	1	Yielded	Case 04 and after
S-B1FC4-1U	North-west	Upper east	1	Yielded	Case 04 and after
S-B2FC2-2L	South-west	Lower west	2	Yielded	Case 13 and after
S-B2FC3-1L	North-east	Lower east	2	Yielded	Case 13 and after
S-B1FC3-2L	North-east	Lower west	1	Corrupted	Case 01 and after
S-B4FC1-2L	South-east	Lower west	4	Corrupted	Case 11 and after
S-B3FS2-2M	North (spine)	West leg (rossete)	3	Corrupted	Case 01 and after

C.5.2 Strain compensation

Compensation for the lost response was categorized into several groups considering different conditions of the strain gauges and rosettes. The compensation utilized information from other sensors and some assumptions, as introduced in the following sections.

C.5.2.1 Category-1

Basic information:

- Strain gauges: S-B1FC1-2U, S-B1FC2-1U, S-B1FC3-2U, S-B1FC4-1U.
- Excitations: Case 04~14

Assumption:

- Reading of strain gauges without initial values at similar locations (i.e., east/west side of different columns) has a stable linear relationship in all cases.
- Symmetricity of the specimen about the East-West direction.
- Uniform shear force in the column.

Data availability:

- Strain measurement at both sides of the columns in Case 01~03.
- Strain measurement at the outer sides of the columns in Case 04~14.

Procedure:

- 1) In Case 01~03, linear regression of the upper east (SU1) and upper west (SU2) strain between each two of all columns, respectively.
- 2) In Case 02~03, take the average of regression parameters of different data pairs as the estimation parameters (neglecting Case 01 due to low R^2 value).
- 3) In Case 04~14, use reading of the east gauges of C1 and C3 to estimate the corresponding reading of C2 and C4 (two estimations per column), and take weighted average to get the final estimation.
- 4) In Case 04~14, use reading of the west gauges of C2 and C4 to estimate the corresponding reading of C2 and C4 (two estimations per column), and take weighted average to get the final estimation.

Table C.3 summarizes the compensation information of this category. The weight factors were determined considering that influence from the column in the same frame (south or north) should be more reliable in the linear relationship. Figure C.7 plots the linear relationship of strain measurements in Case 02 (Northridge VA 40%). Figure C.8 plots some compensation results in Case 04 (Northridge VA 100%, MRF-Spine, initial values included).

Table C.3 Compensation information of Category-1

Reference gauge	Target gauge	Case	Weight	Slope	Interception
S-B1FC2-2U	S-B1FC1-2U	04 ~ 14	0.7	1.87	1.20×10^{-06}
S-B1FC4-2U	S-B1FC1-2U		0.3	1.96	9.78×10^{-06}
S-B1FC1-1U	S-B1FC2-1U		0.7	2.13	-3.71×10^{-05}
S-B1FC3-1U	S-B1FC2-1U		0.3	2.02	-1.06×10^{-04}
S-B1FC2-2U	S-B1FC3-2U		0.3	1.94	-7.47×10^{-05}
S-B1FC4-2U	S-B1FC3-2U		0.7	2.05	-6.53×10^{-05}
S-B1FC1-1U	S-B1FC4-1U		0.3	2.08	5.94×10^{-06}
S-B1FC3-1U	S-B1FC4-1U		0.7	1.98	-6.07×10^{-05}

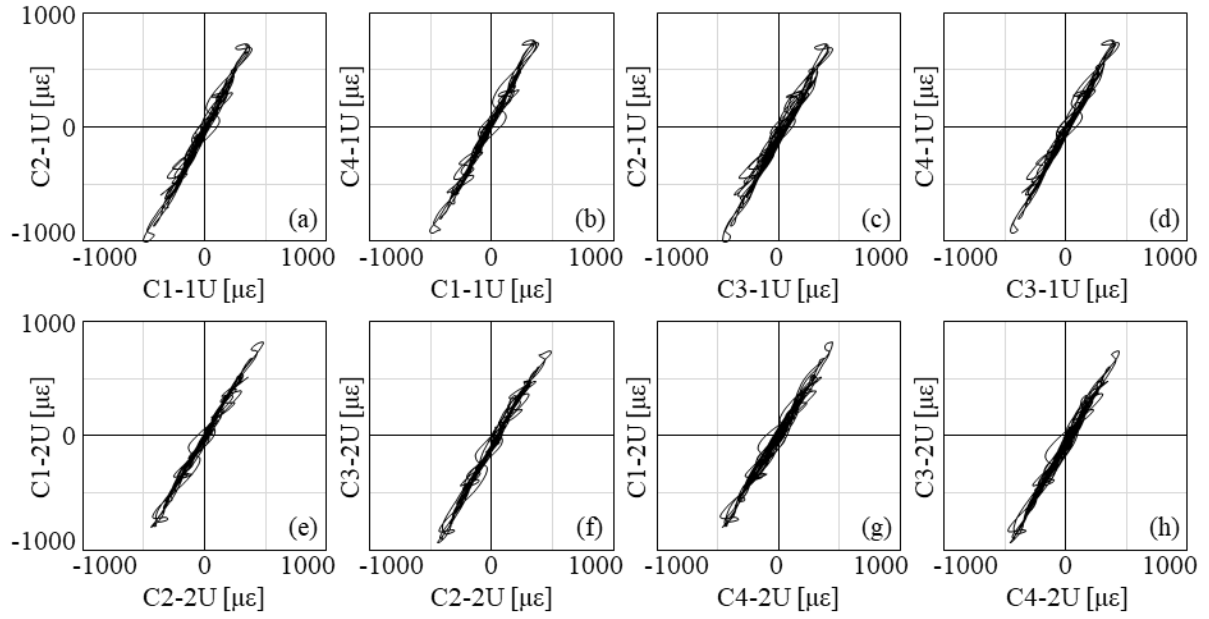


Figure C.7 Linear relationship of strain measurements of Category-1 in Case 02

Strain of the upper east gauge (SU1): (a) C1 vs C2; (b) C1 vs C4; (c) C3 vs C2; (d) C3 vs C4.
Strain of the upper west gauge (SU2): (e) C2 vs C1; (f) C2 vs C3; (g) C4 vs C1; (h) C4 vs C3.

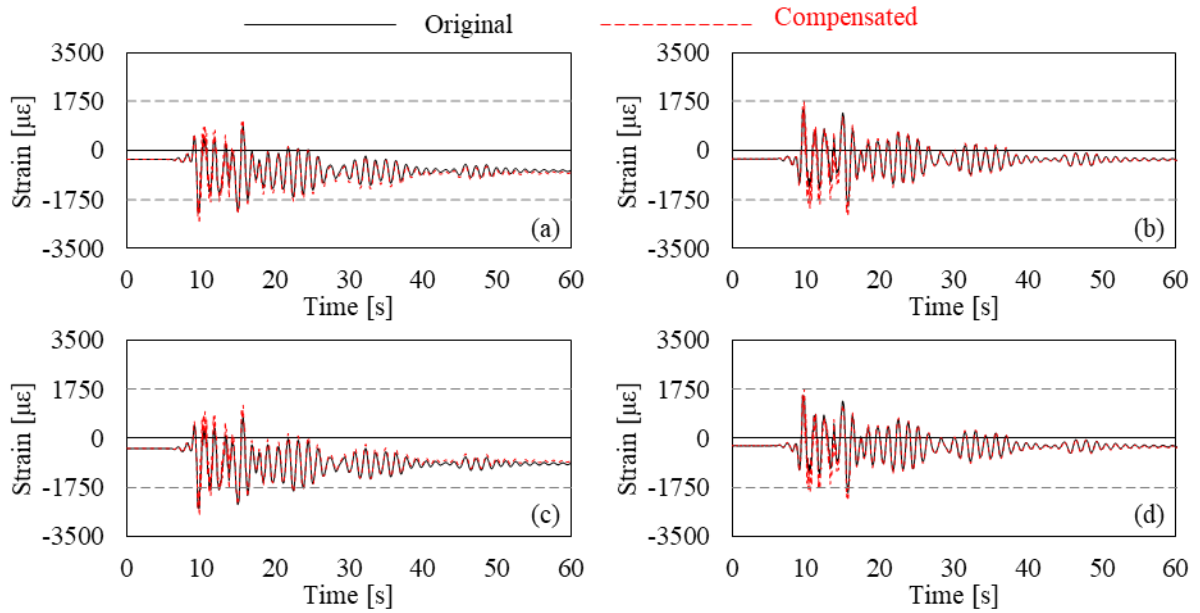


Figure C.8 Example results of Category-1 in Case 04

(a) S-B1FC1-2U; (b) S-B1FC2-1U; (c) S-B1FC3-2U; (d) S-B1FC4-1U

C.5.2.2 Category-2

Basic information:

- Strain gauges: S-B1FC3-2L.
- Excitations: Case 01~14.

Assumption:

- Axial force at the lower section of all columns has a stable linear relationship.
- Symmetricity of the specimen about the East-West direction.
- Uniform shear force in the column.

Data availability:

- Axial force of columns C1, C2 and C4.
- Reading of strain gauge S-B1FC3-1L.

Procedure:

- 1) In all cases, calculate axial force at the lower section of columns C1, C2 and C4.
- 2) In all cases, conduct linear regression between each two of them.
- 3) In all cases, apply the parameters of C1 vs C2 and C4 vs C2 (C2 as X value) to axial force of C4 and C1, respectively, getting two estimations of C3 lower section axial force estimation.
- 4) In all cases, take the average of the two estimations as the lower section axial force of C3.
- 5) In all cases, use the estimated axial force and reading of the good gauge (S-B2FC3-1L) to get the estimation of S-B1FC3-2L.

Table C.4 summarizes the compensation information of Category-2. Note that parameters of Case 01 were taken as the average of Case 02 and 03 due to the small R^2 value in this case. Figure C.9 plots the linear relationship between C1 vs C2 and C4 vs C2 in the primary cases (both with C2 as the reference). Figure C.10 plots some results of the compensation of the absolute strain in the primary cases.

Table C.4 Compensation information of Category-2

Reference	Target-1	Target-2	Case	Slope-1	Interception-1	Slope-2	Interception-2
C2	C1	C4	01	-1.02	10.57	1.00	-7.89
			02	-1.03	10.44	1.02	-7.70
			03	-1.00	10.70	0.99	-8.08
			04	-0.98	-78.60	1.01	10.68
			05	-1.03	-102.88	1.00	15.88
			06	-0.96	-105.22	0.94	10.89
			07	-0.99	-106.64	0.98	12.19
			08	-0.96	-105.57	0.95	10.77
			09	-0.99	-121.49	0.99	17.72
			10	-0.98	-124.58	0.95	14.03
			11	-1.00	-136.74	1.00	19.97
			12	-0.99	-137.69	0.97	17.41
			13	-0.98	-169.50	0.99	20.07
			14	-1.00	-177.18	0.98	18.59

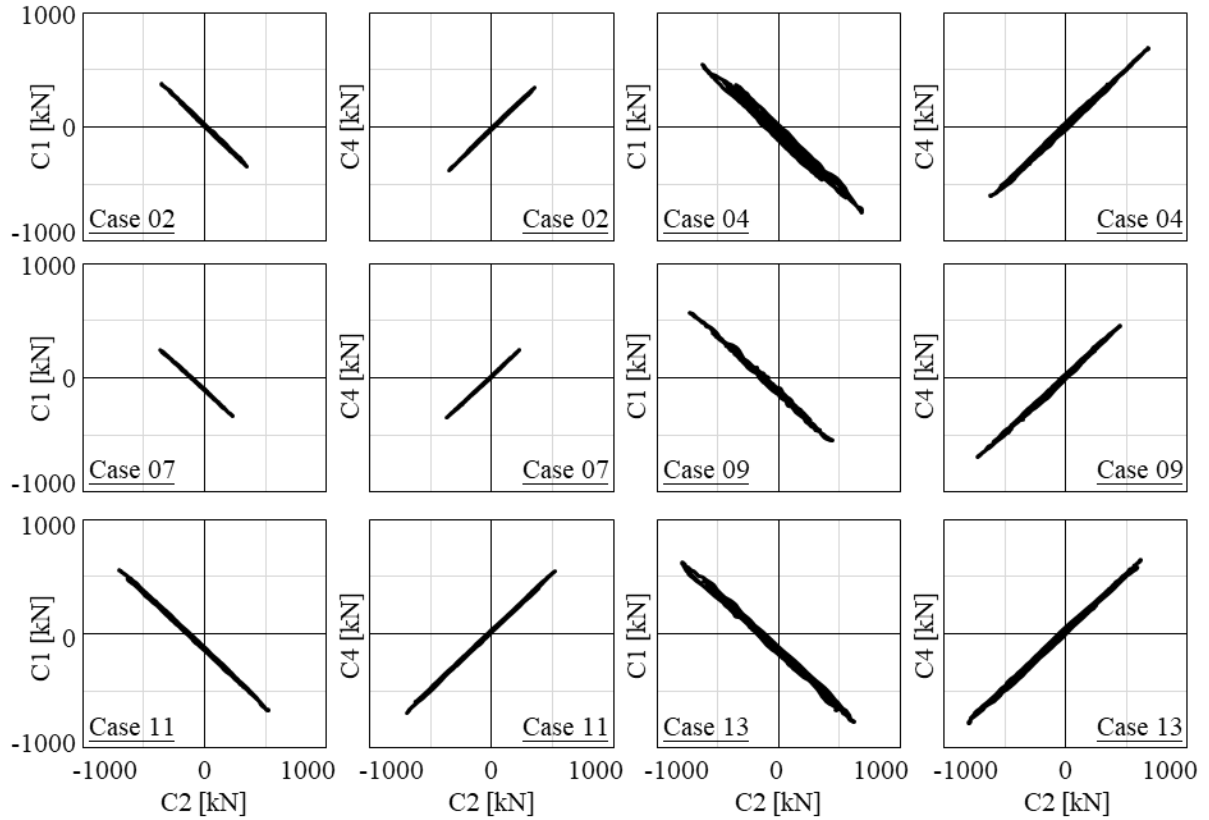


Figure C.9 Linear relationship of lower section axial force: C1 vs C2, C4 vs C2

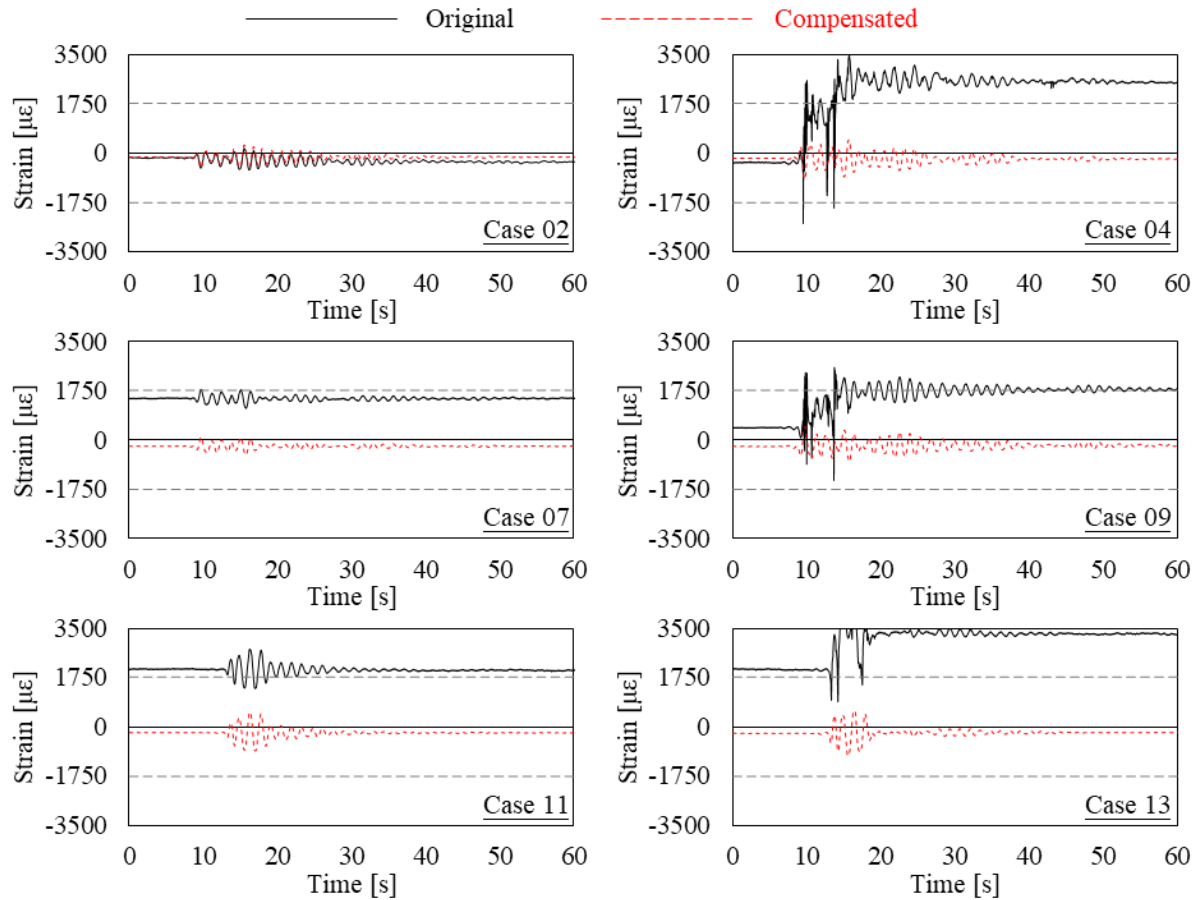


Figure C.10 Example results of Category-2 in primary cases

C.5.2.3 Category-3

Basic information:

- Strain gauges: S-B2FC2-2L, S-B2FC3-1L.
- Excitations: Case 13~14.

Assumption:

- Shear force without initial values of all columns has a stable linear relationship.
- Symmetricity of the specimen about the East-West direction.
- Uniform shear force in the column.

Data availability:

- Shear force of all columns in Case 01~12.
- Shear force of columns C1 and C4 in Case 13~14.
- Moment at upper section of C1 and C4 in Case 13~14.
- Reading of strain gauges S-B2FC2-1L and S-B2FC3-2L in Case 13~14.

Procedure:

- 1) In Case 01~12, calculate shear force of all columns.
- 2) In Case 01~12, conduct linear regression between shear force of any two of the columns and take the average of each pair as the final estimation parameters.
- 3) In Case 13~14, apply the parameters of C2 vs C1 and C2 vs C4 (with C1 and C4 as reference values) to shear force of C1 and C4, respectively, and take weighted average of the two estimations to get the final estimation of C2 shear force.
- 4) In Case 13~14, apply the parameters of C3 vs C1 and C3 vs C4 (with C1 and C4 as reference values) to shear force of C1 and C4, respectively, and take weighted average of the two estimations to get the final estimation of C3 shear force.
- 5) In Case 13~14, use the estimated shear force of C2 and C3 and the upper section moment, getting the estimated lower section moment of C2 and C3.
- 6) In Case 13~14, use the estimated lower section moment of C2 and C3, and the good gauge at the lower section (S-B2FC2-1L and S-B2FC3-2L), getting the estimation of S-B2FC2-2L and S-B2FC3-1L.

Table C.5 summarizes the compensation information of Category-3. The weight factors were determined considering the symmetricity of the specimen and excitation that ideally C1 and C3 should have the same response as well as C2 and C4, and thus estimation of C2 and C3 from C4 and C1 should be more reliable, respectively. Figure C.11 plots the linear relationship between C2/C3 vs C1/C4 in the first five primary cases. Figure C.12 plots results of the compensation of the absolute strain in Case 13.

Table C.5 Compensation information of Category-3

Reference	Target	Case	Weight	Slope	Interception
C1 shear force	C2 shear force	13 ~ 14	0.3	1.02	19.08
C4 shear force	C2 shear force		0.7	1.02	7.27
C1 shear force	C3 shear force		0.7	1.02	-9.82
C4 shear force	C3 shear force		0.3	1.02	-21.59

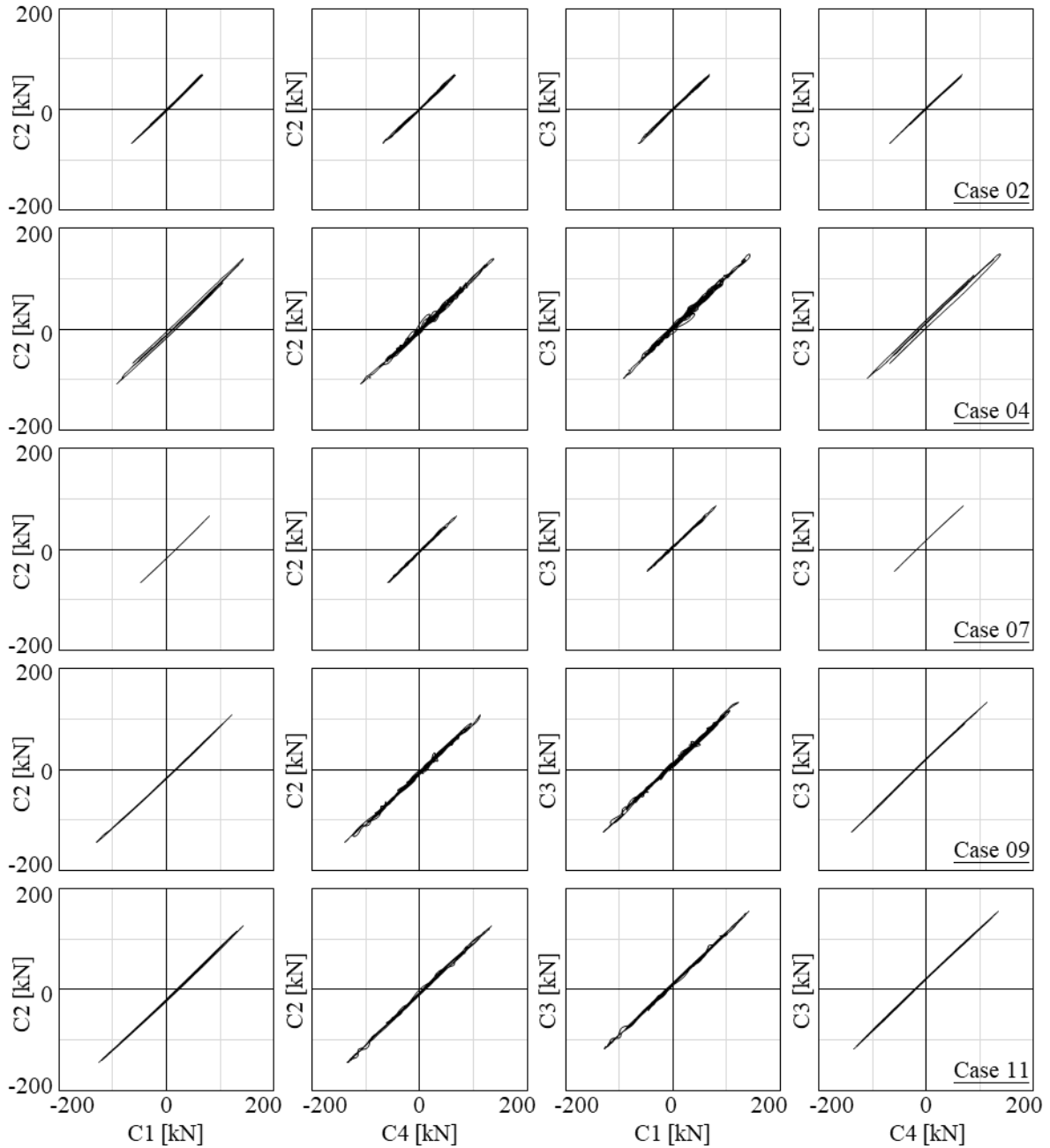


Figure C.11 Linear relationship of shear force: C2 vs C1, C2 vs C4, C3 vs C1, and C3 vs C4

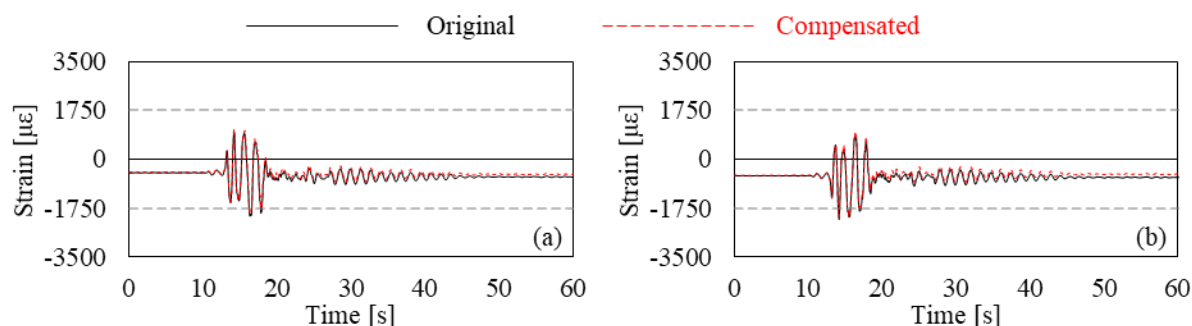


Figure C.12 Example results of Category-3 in Case 13

(a) S-B2FC2-2L; (b) S-B2FC3-1L

C.5.2.4 Category-4

Basic information:

- Strain gauges: S-B4FC1-2L.
- Excitations: Case 11~14.

Assumption:

- Shear force without initial values of all columns has a stable linear relationship.
- Symmetricity of the specimen about the East-West direction.
- Uniform shear force in the column.

Data availability:

- Shear force of all columns in Case 01~10.
- Shear force of columns C2, C3 and C4 in Case 11~14.
- Moment at upper section of C1 in Case 11~14.
- Reading of strain gauge S-B4FC1-1L in Case 11~14.

Procedure:

- 1) In Case 01~10, calculate shear force of all columns.
- 2) In Case 01~10, conduct linear regression between the shear force of C1 vs C2/C3/C4 and take the average of regression parameters as the final parameters for estimation.
- 3) In Case 11~14, apply parameters of C1 vs C2, C1 vs C3 and C1 vs C4 to shear force of C2, C3 and C4 (three estimations), then take weighted average of the three estimations as the final estimation.
- 4) In Case 11~14, use the estimated column shear force and upper section moment, getting the lower section moment of C1.
- 5) In Case 11~14, use the estimated lower section moment and reading of the good gauge (S-B4FC1-1L) to get estimation of S-B4FC1-2L.

Table C.6 summarizes the compensation information of Category-3. The weight factors were determined considering the symmetricity of the specimen and excitation that ideally C1 and C3 should have the

same response, and thus estimation of C1 from C3 should be more reliable. Figure C.13 plots the linear relationship between C1 and C2/C3/C4 in the first four primary cases. Figure C.14 plots results of the compensation of the absolute strain in Case 11 and 13.

Table C.6 Compensation information of Category-4

Reference	Target	Case	Weight	Slope	Interception
C2 shear force	C1 shear force		0.2	1.01	16.59
C3 shear force	C1 shear force	11 ~ 14	0.6	0.96	11.19
C4 shear force	C1 shear force		0.2	0.97	8.05

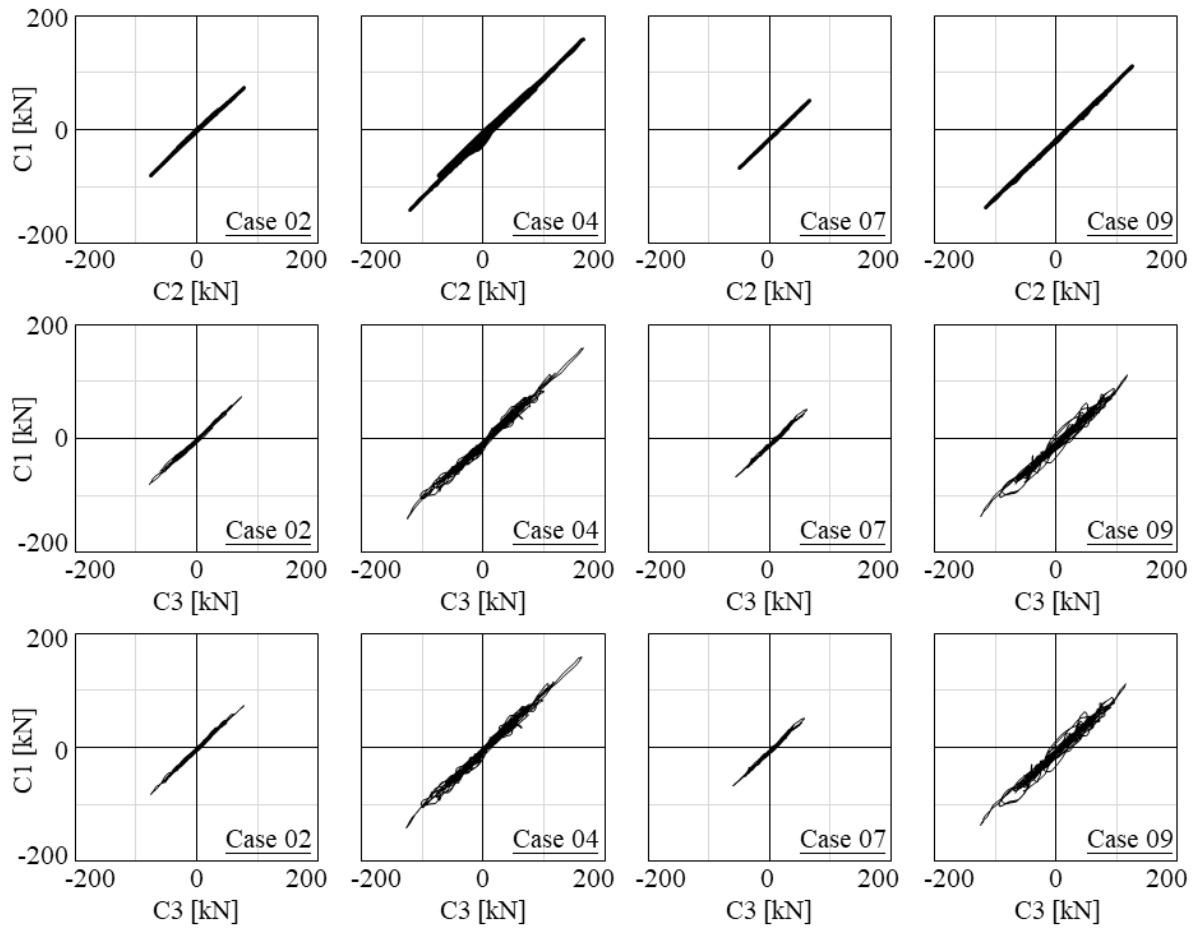


Figure C.13 Linear relationship of shear force: C1 vs C2, C1 vs C3, and C1 vs C4

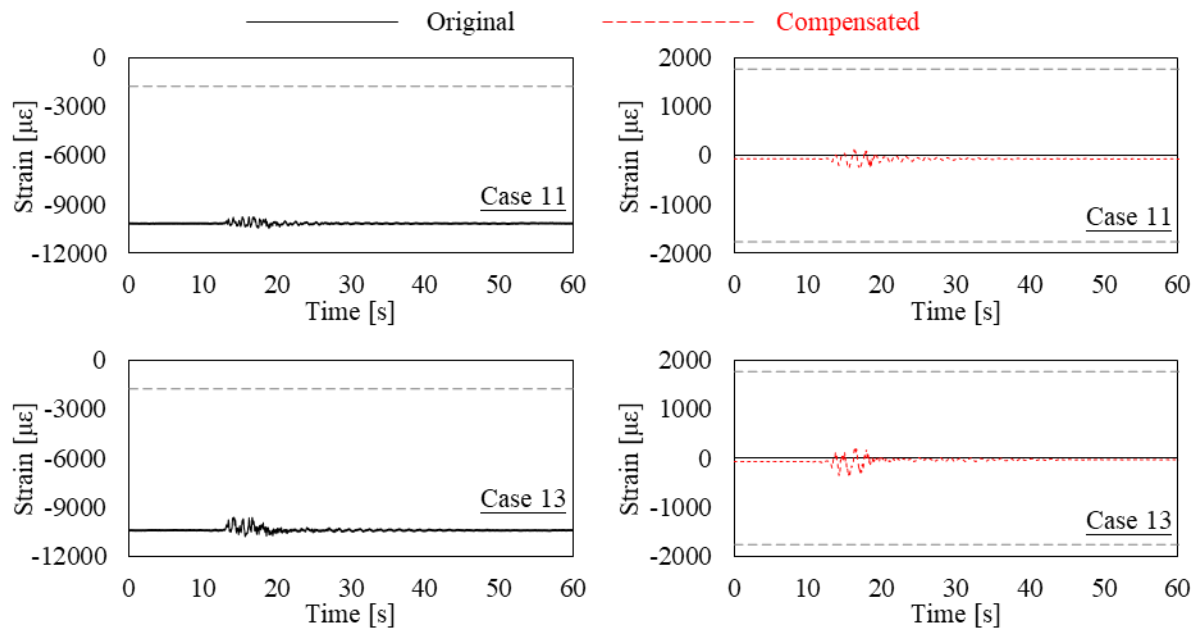


Figure C.14 Example results of Category-4

C.5.2.5 Category-5

Basic information:

- Strain gauges: S-B3FS2-2M.
- Excitations: Case 01~14.

Assumption:

- Pure-shear state of spine segment.
- Symmetricity of legs of the biaxial strain rosette.
- Uniform shear force in the spine.

Data availability:

- Reading of strain rosette at Story 1/2/4 on the north spine.
- Reading of the east leg of the strain rosette at Story 3 on the north spine (S-B3FS2-1M).

Procedure:

- 1) In Case 01~14, conduct linear regression of reading from the west and east legs of the strain rosette at Story 1/2/4 of the north spine.
- 2) In Case 01~14, take the average of the regression parameters at the three stories as the parameters for estimation at Story 3.
- 3) In Case 01~14, apply the estimated regression parameters to the reading of the west leg at Story 3, getting the estimation of S-B3FS2-2M.

Table C.7 summarizes the compensation information of Category-5. Note that parameters of white noise cases were generated by their neighboring primary cases due to the small R^2 value in the regression.

Figure C.15 plots the linear relationship between reading of two legs at Story 1/2/4 in six primary cases.

Figure C.16 plots the results in six primary cases.

Table C.7 Compensation information of Category-5

Reference	Target	Case	Slope	Interception	Note
East leg	West leg	01	-1.09	0.41	Same Case 02
		02	-1.09	0.41	
		03	-1.06	0.60	Average of Case 02 and 04
		04	-1.02	0.79	
		05	-1.02	0.79	Same Case 04
		06	-1.01	4.73	Same Case 07
		07	-1.01	4.73	
		08	-1.00	5.79	Average of Case 07 and 09
		09	-1.00	6.84	
		10	-0.99	8.11	Average of Case 09 and 11
		11	-0.97	9.38	
		12	-0.97	9.38	Average of Case 11 and 13
		13	-0.98	9.38	
		14	-0.98	9.38	Same as Case 13

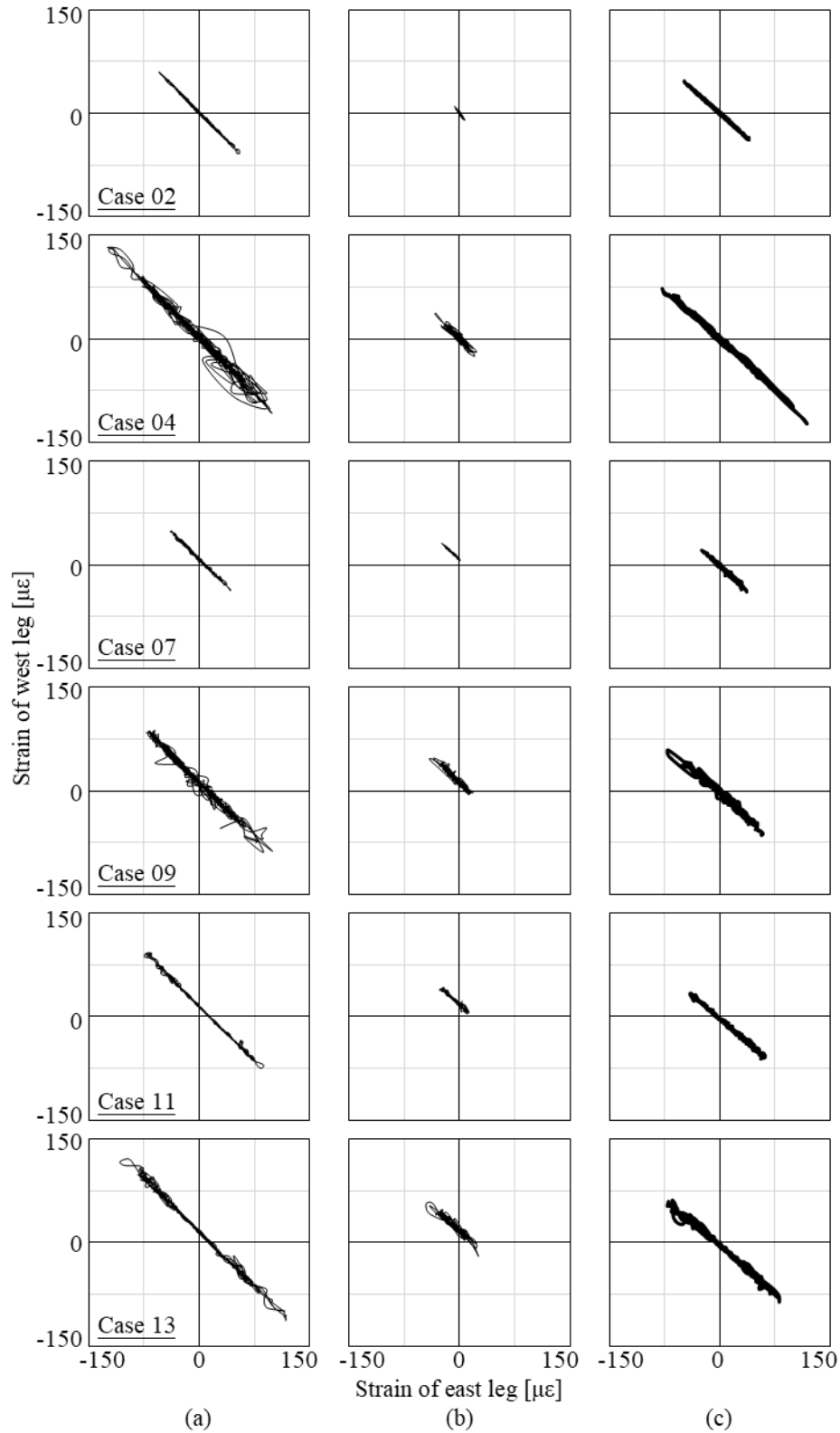


Figure C.15 Linear relationship between east and west legs at Story 1/2/4
(a) Story 1; (b) Story 2; (c) Story 3

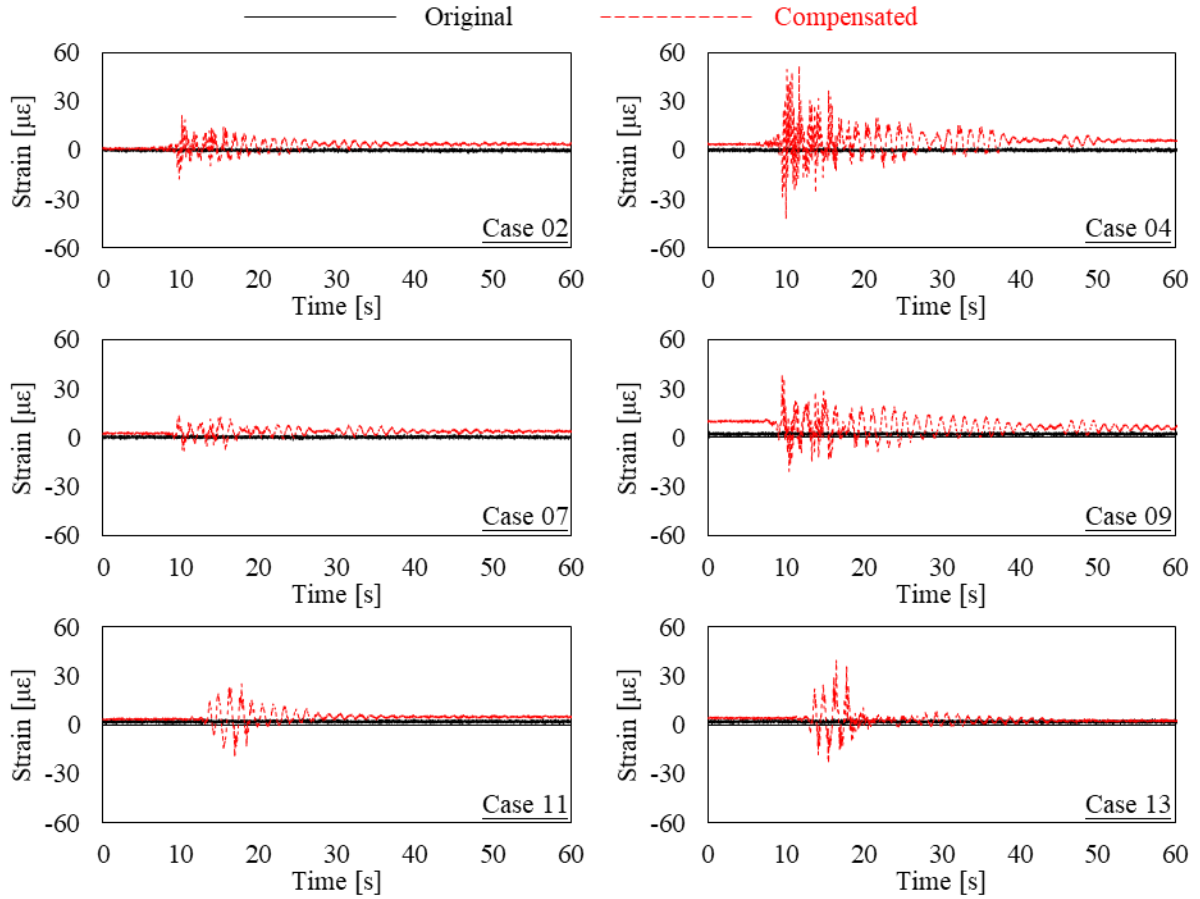


Figure C.16 Example results of Category-55 in primary cases

C.5.3 Validation of compensation results

For the compensation algorithms used for Category-1, it is possible to overestimate the story shear force at the 1st story. A comparison was made to check this condition. Figure C.17 plots the story hysteresis at the 1st story of story shear force vs story drift and drift ratio. The story shear force came from two sources: (1) compensated strain gauge (termed as “Strain gauge”) and (2) story shear force from strain gauges at the 2nd to 4th stories and inertia at the 1st floor (termed as “Mix”). It is seen that although the story shear force deduced from compensated strain gauges is larger than from the mixed source, the difference is not significant and the validity of the compensation can be confirmed.

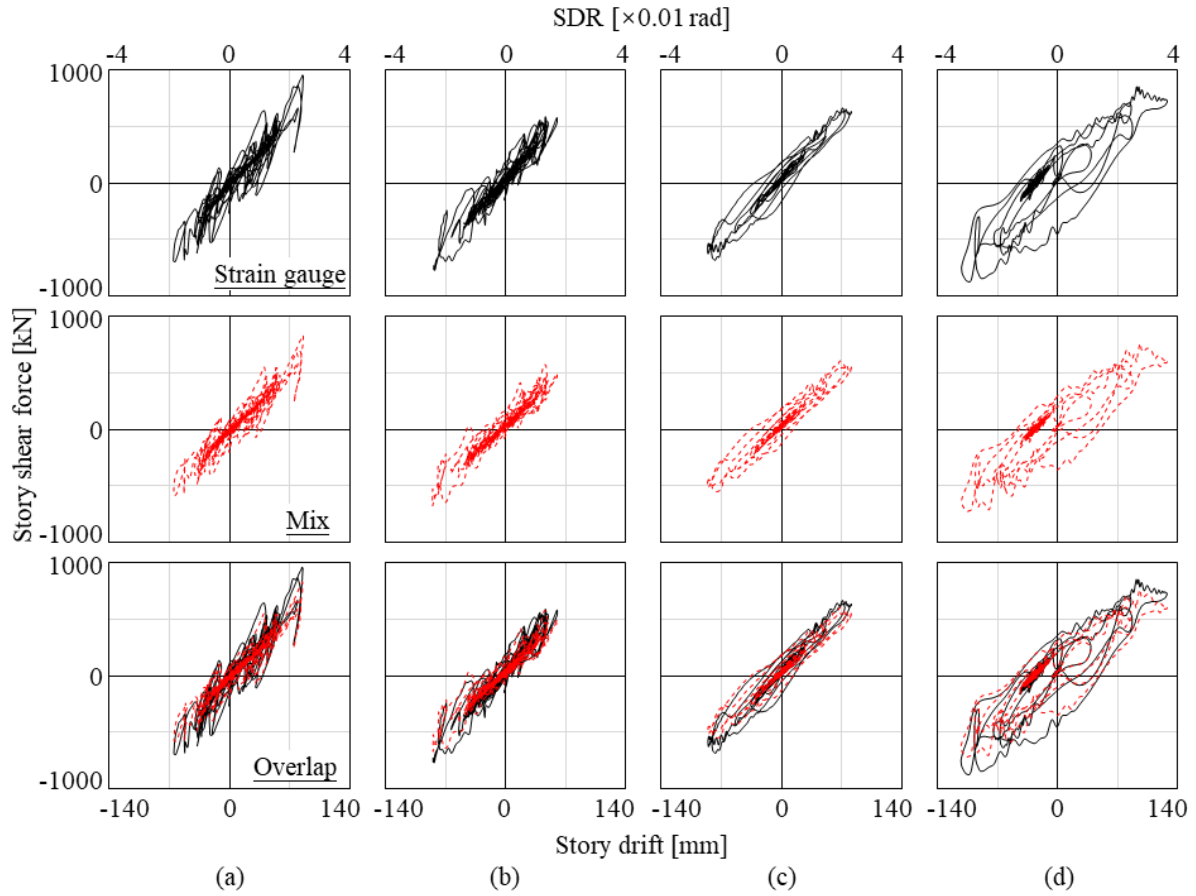


Figure C.17 Story hysteresis at 1st story of different story shear force sources
(a) Case 04; (b) Case 09; (c) Case 11; (d) Case 13

APPENDIX D. FULL RESULTS OF THE MRF-SPINE SHAKE-TABLE TEST

D.1 GENERAL

This appendix provides plots of most of the results of the 2020EDF MRF-Spine shake-table test. In addition to the results shown in Chapter 4, more results are included here. The illustrated results include: 1) transfer functions based on acceleration data; 2) envelope of global responses; 3) hysteresis of global and local responses; 4) P-M interaction curves of columns. Time history of the sensor signals are not included since it's easy to plot them. Besides, “higher-order” results such as energy dissipation is also not included here. It is expected that the full results can help fully understand the behavior of the structural system during the test.

D.2 TRANSFER FUNCTIONS

Transfer functions were calculated as the ratio between the Fourier spectra of the acceleration signals of the upper floors and the shake-table excitation. The summed transfer functions representing the “total” response of the system was plotted in Chapter 4. Here the transfer functions of each floor are plotted. Figure D.1 to Figure D.4 plot the transfer functions of the 1st to 4th floors in the white noise cases, respectively.

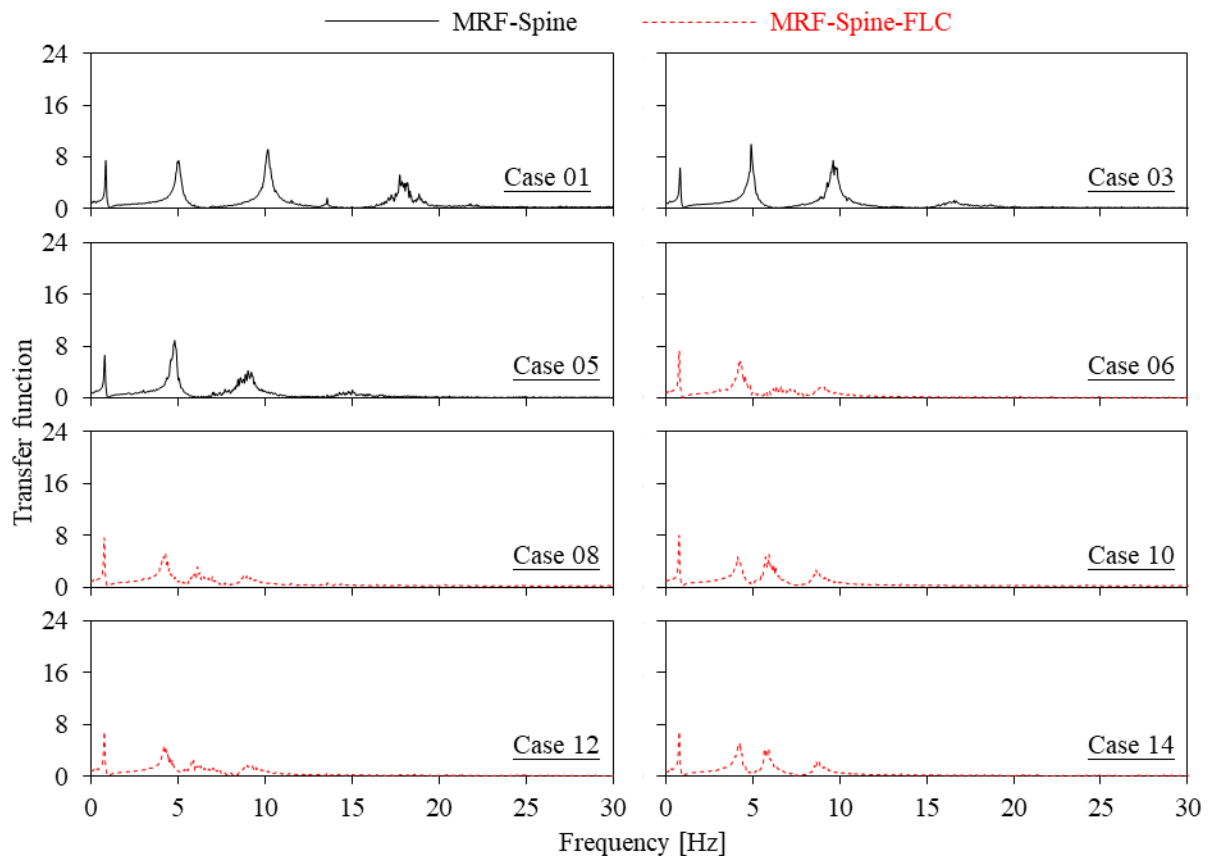


Figure D.1 Transfer functions of the 1st floor in the white noise excitations

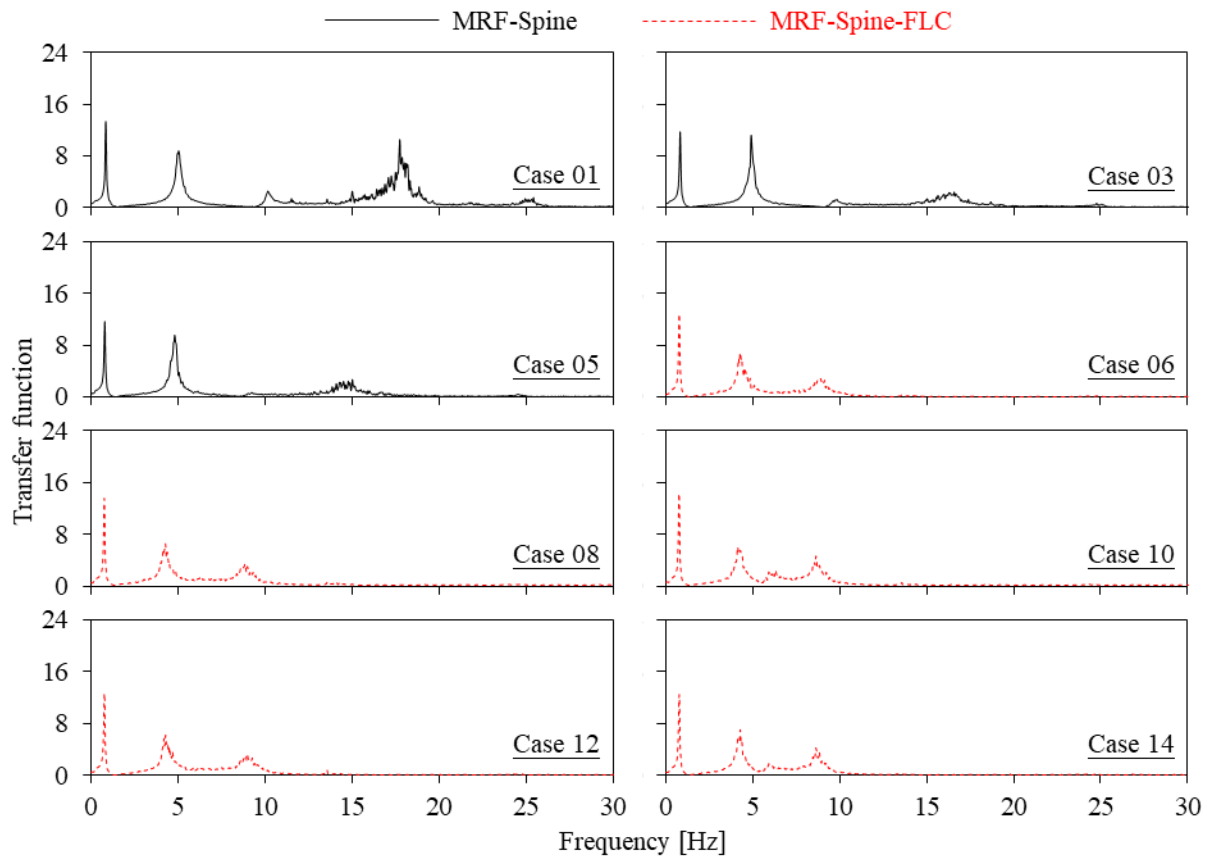


Figure D.2 Transfer functions of the 2nd floor in the white noise excitations

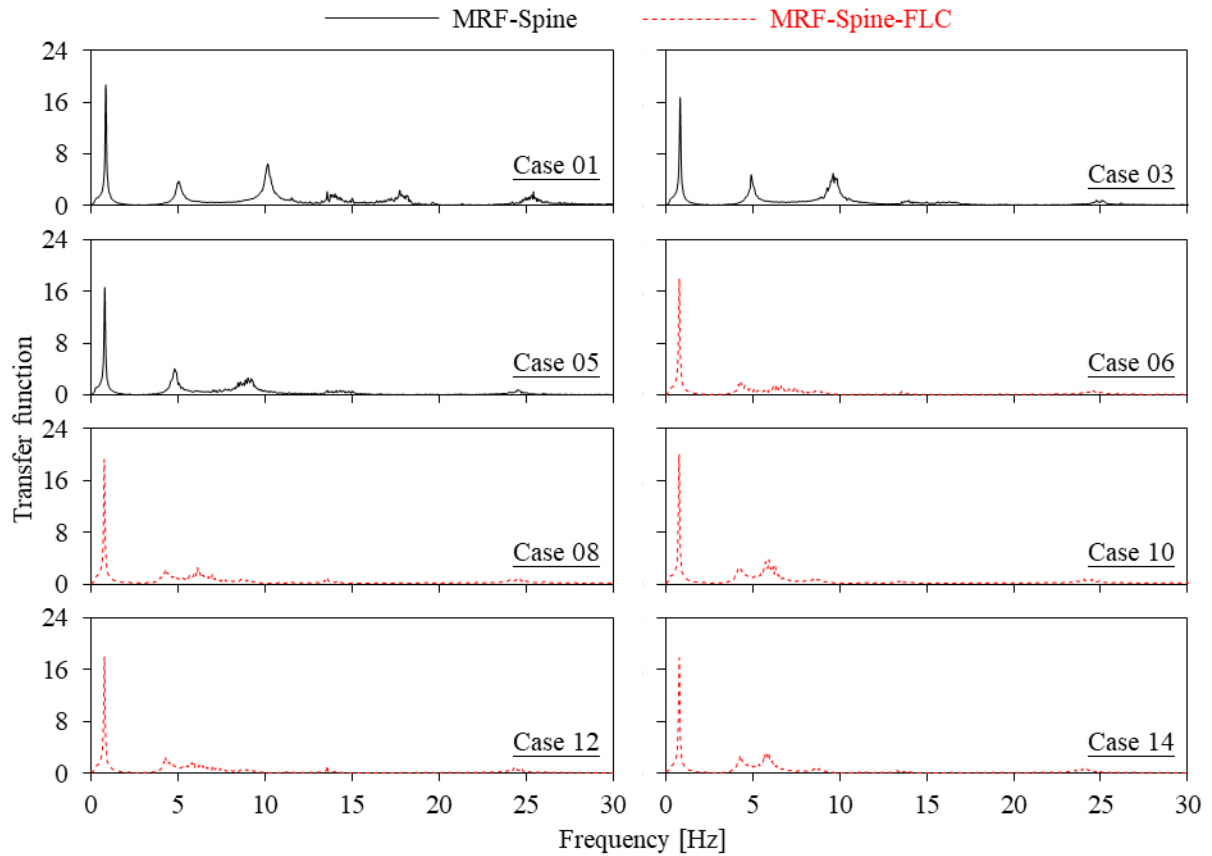


Figure D.3 Transfer functions of the 3rd floor in the white noise excitations

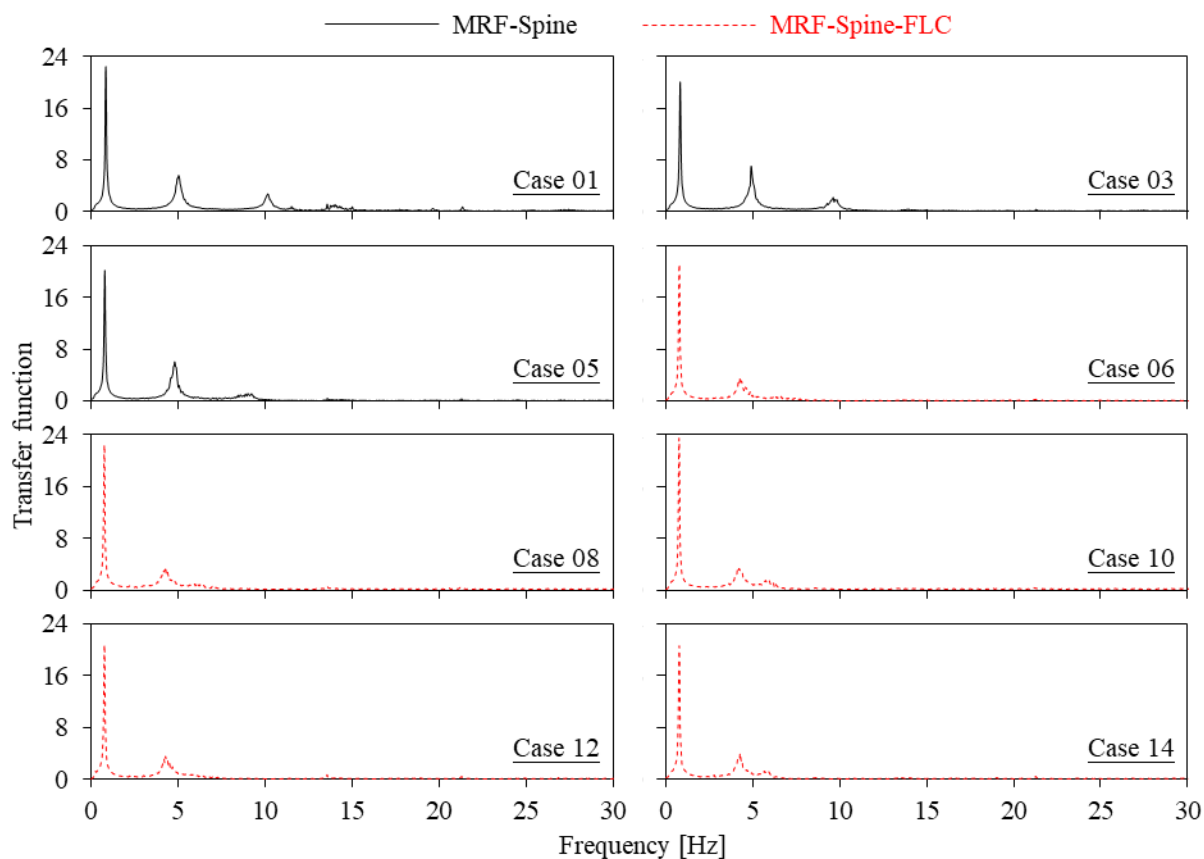


Figure D.4 Transfer functions of the 4th floor in the white noise excitations

D.3 PROFILE OF PEAK GLOBAL RESPONSES

Profile of peak floor acceleration, story drift and drift ratio, story shear force (total, MRF, spine) of the 6 major excitations are plotted in this section.

D.3.1 Floor acceleration

Figure D.5 plots profile of peak floor acceleration in the major excitations, with Table D.1 summarizing the values.

Table D.1 Peaks of floor acceleration [Unit: m/s^2]

Floor	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
4	5.0	11.8	4.1	9.3	6.3	9.3
3	3.5	10.3	3.6	9.3	5.1	6.6
2	5.5	20.2	5.4	13.1	6.9	12.4
1	5.8	16.5	4.7	11.3	6.2	12.8
Table	3.5	11.2	3.5	11.4	4.1	9.7

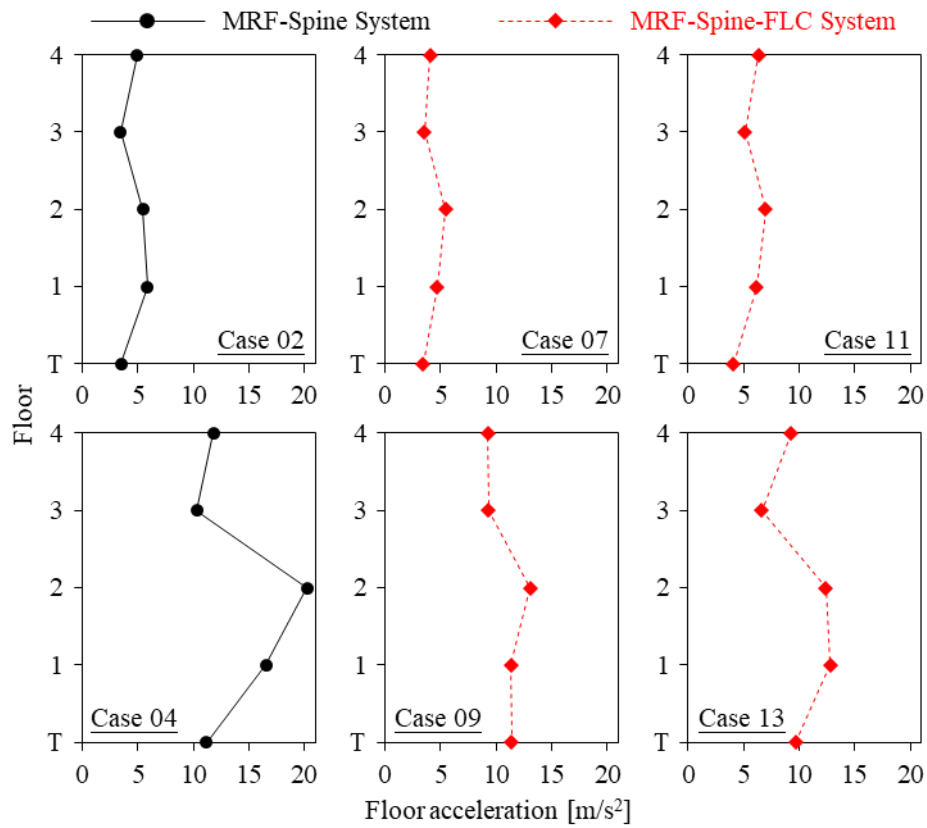


Figure D.5 Profile of peak floor acceleration

D.3.2 Story drift ratio and story drift

Figure D.6 and Figure D.7 plot the envelope of story drift and story drift ratio of the major excitations, with Table D.2 and Table D.3 summarizing the values, respectively. The synchronized story drift data obtained from the optical measurement system with residual values were used. Story heights used for calculation were 3425 mm for the 1st story and 3500 mm for the upper stories.

Table D.2 Peak story drift [Unit: mm]

Story	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
4	21.8	55.4	20.4	39.5	37.2	46.7
3	25.1	65.0	28.6	57.0	55.6	78.5
2	25.3	63.6	32.3	58.7	63.1	89.4
1	37.0	85.1	44.5	84.6	85.8	127.4

Table D.3 Peak story drift ratio [Unit: 0.01rad]

Story	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
4	0.6	1.6	0.6	1.1	1.1	1.3
3	0.7	1.9	0.8	1.6	1.6	2.2
2	0.7	1.8	0.9	1.7	1.8	2.6
1	1.1	2.5	1.3	2.5	2.5	3.7

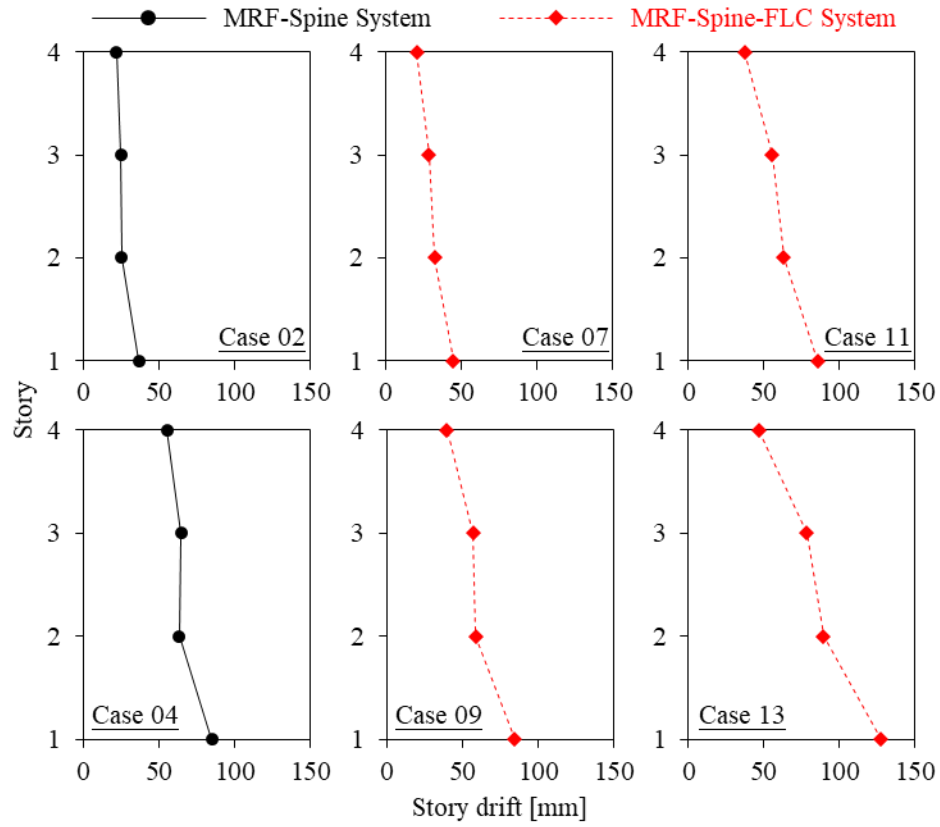


Figure D.6 Envelope of story drift (with residual values)

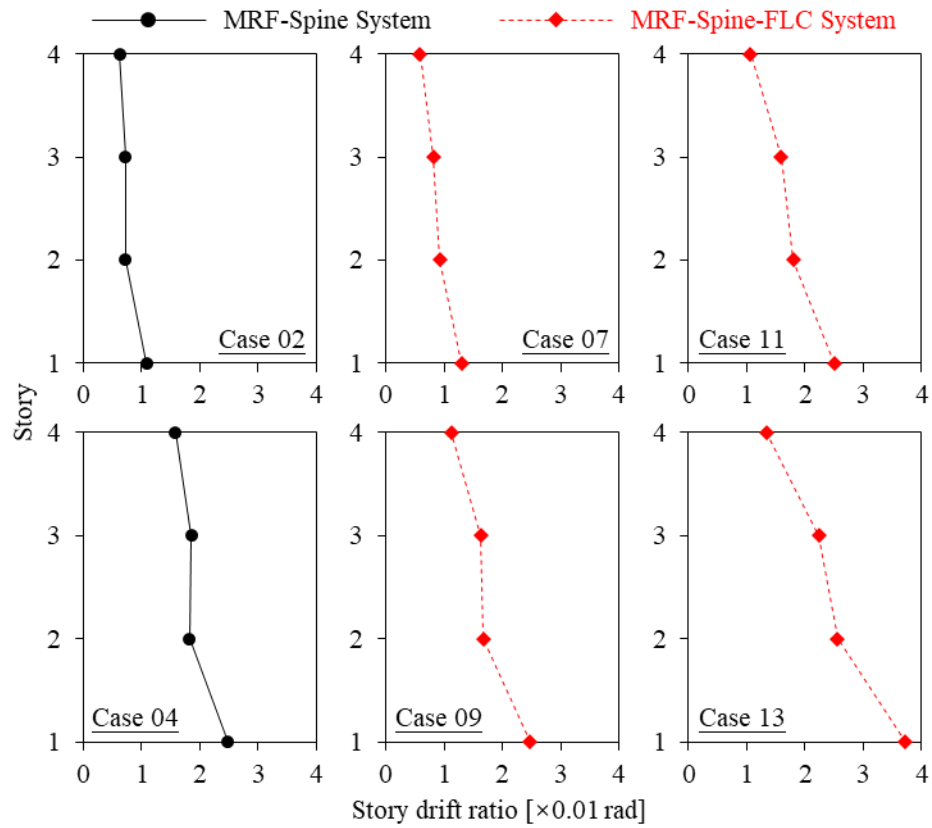


Figure D.7 Envelope of story drift ratio (with residual values)

D.3.3 Story shear force

Figure D.8 to Figure D.10 plot story shear force of the whole structure, the MRF and the spines, respectively. The shear force shown here was deduced from uniaxial strain gauges on the columns and spines and the loadcell at the spine bases. Table D.4 to Table D.6 summarize the specific values of the peak shear force of the whole structure, the MRF and the spines, respectively.

Table D.4 Peak story shear force of the whole structure [Unit: kN]

Story	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
4	252.4	558.9	236.0	468.2	347.8	504.0
3	288.8	645.2	244.2	563.2	481.3	606.6
2	287.2	623.8	303.6	513.7	544.9	632.7
1	394.1	962.8	390.9	765.8	672.6	878.4

Table D.5 Peak story shear force of the MRF [Unit: kN]

Story	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
4	321.4	639.9	246.7	513.4	415.6	538.4
3	280.3	571.8	257.2	552.4	518.4	602.3
2	270.3	572.0	303.4	538.6	556.4	635.6
1	147.2	401.7	215.3	364.6	323.0	399.3

Table D.6 Peak story shear force of the spines [Unit: kN]

Story	Case 02	Case 04	Case 07	Case 09	Case 11	Case 13
4	162.3	410.6	109.5	181.6	196.2	295.4
3	64.9	170.0	38.9	127.6	89.8	133.0
2	25.4	117.4	59.1	121.5	93.7	136.0
1	249.9	561.6	201.8	420.0	351.8	510.5

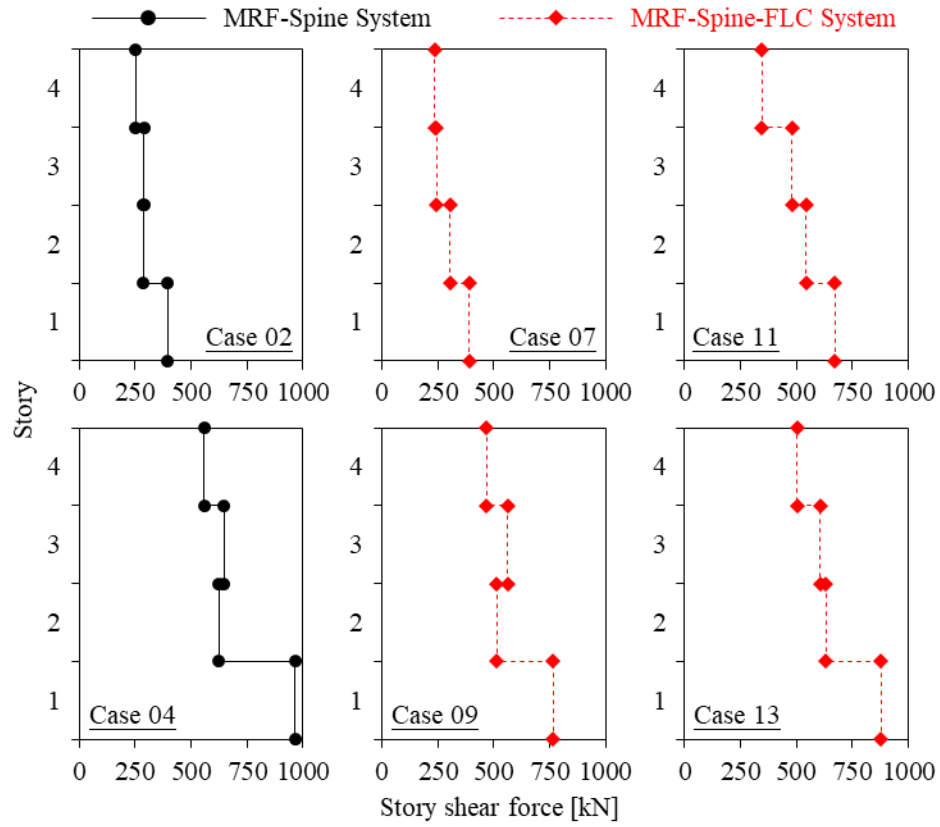


Figure D.8 Story shear force of the whole structure

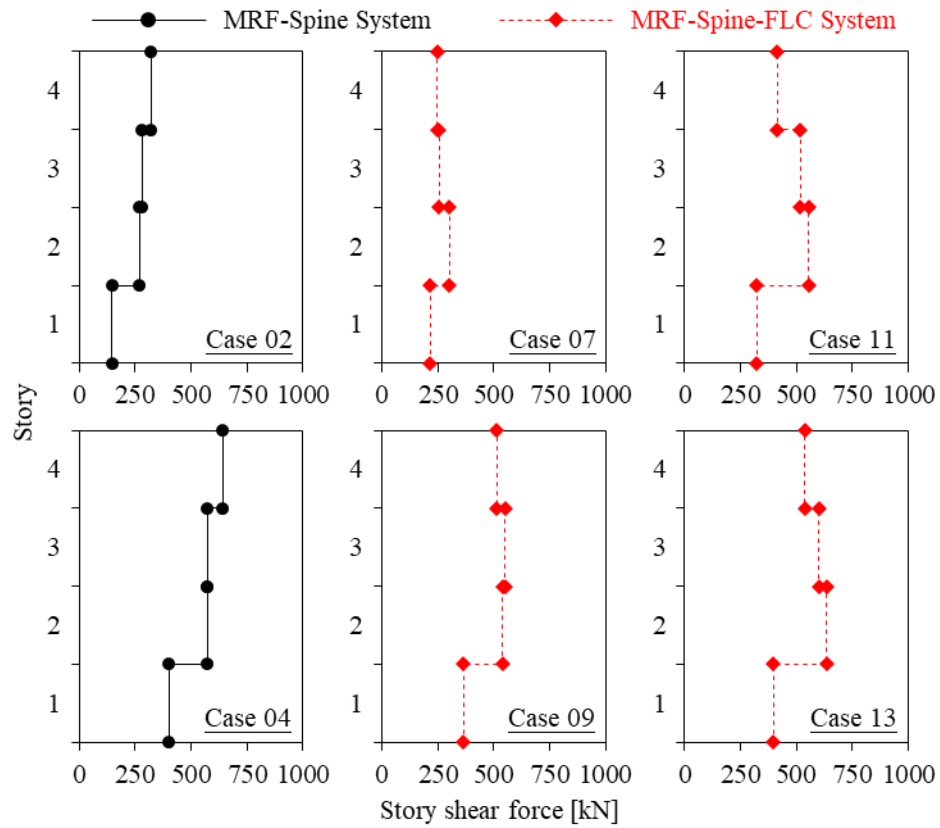


Figure D.9 Story shear force of the MRF

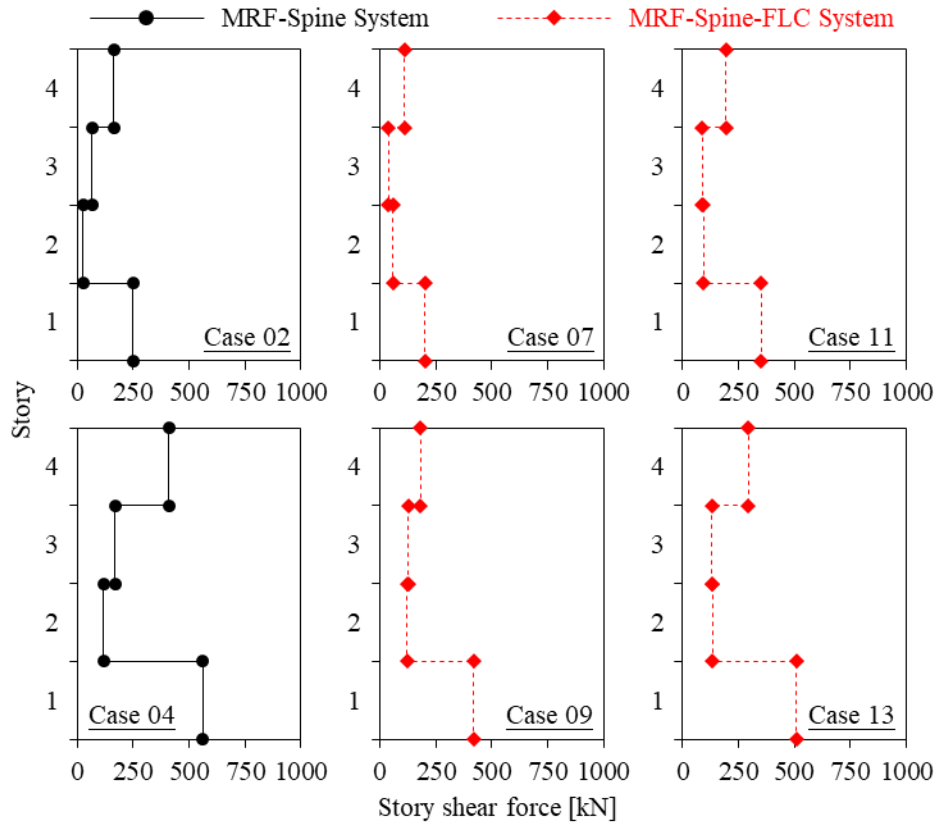


Figure D.10 Story shear force of the spines

D.4 STRUCTURE HYSTERESIS

Hysteresis of global and local responses in the 6 major excitations are plotted in this section.

D.4.1 Story hysteresis

Figure D.11 and Figure D.12 plot the hysteresis of story shear force to story drift and drift ratio of the whole structure and the MRF in the 6 major excitations, respectively.

D.4.2 MRF-spine connection hysteresis

Figure D.13 and Figure D.14 plot the MRF-spine connection force to deformation hysteresis of the SOUTH and NORTH frames in the 6 major excitations, respectively. Elastic stiffness of the FLCs obtained from static cyclic tests is also plotted.

D.4.3 Panel zone hysteresis

Figure D.15 and Figure D.16 plot the panel zone shear force to shear angle hysteresis of the EAST and WEST panels (on columns C1 and C2) at the south frame in the 6 major excitations, respectively. Shear capacity V_p of each panel is also plotted as dashed horizontal lines, being 906.3 kN for the lower three floor and 873.2 kN for the top floor.

D.4.4 Beam end hysteresis

Figure D.17 and Figure D.18 plot the hysteresis of beam end moment to rotation angle of the EAST and WEST ends (connection to columns C1 and C2) at the south frame in the 6 major excitations, respectively. Moment capacity M_p of each beam is also plotted as dashed horizontal line, being 549.2 kN·m for the lower three floors and 413.7 kN·m for the top floor.

D.4.5 Column end hysteresis

Figure D.19 to Figure D.22 plot the column end moment to story drift ratio hysteresis of the columns C1 to C4 in the 6 major excitations, respectively. The top and bottom ends of the same story are included in the same plot, with moment capacity, without influence of axial force, M_p of each column plotted as dashed horizontal line. Note that at the 4th story, the top end capacity (734.3 kN·m) is much larger than the plotted range, and the dashed lines represent values of the bottom end.

D.5 COLUMN INTERACTION CURVE

In addition to the column end hysteresis, the axial force-bending moment (P-M) interaction curve of column ends can better illustrate the comprehensive influence of large axial force and bending moment on the columns. Figure D.23 to Figure D.26 plot the P-M interaction curves of columns C1 to C4 in the 6 major excitations. In each plot, the blue solid curve indicates plastic limit while the dashed curve indicates yielding limit, with the top and bottom sections of the same story shown in the same plot.

D.6 CONSISTENCY OF STORY SHEAR FORCE

As introduced in Chapter 03 and 04, three types of story shear force can be obtained from measurement. Their consistency was confirmed by checking the linear relationship between each two of them. Table D.9 to Table D.7 summarize the linear regression factors of the three types of story shear force in the six major excitations. Figure D.27 to Figure D.29 plot the linear relationship of the data pairs in the six major excitations.

Notation of story shear types are as follows:

- 1) SSIF: Story Shear force from Inertia Force at each floor.
- 2) SSUU: Story Shear force from Uniaxial gauges on the MRF and Uniaxial gauges and loadcells on the spines.
- 3) SSUB: Story Shear force from Uniaxial gauges on the MRF and Biaxial rosettes on the spines.

Table D.7 Regression parameters of SSUB to SSUU

Story	Case 02		Case 04		Case 07		Case 09		Case 11		Case 13	
	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²
4	0.76	0.92	0.91	0.93	0.86	0.96	0.88	0.96	0.80	0.99	0.83	0.96
3	1.00	1.00	0.98	1.00	0.97	1.00	0.96	1.00	0.97	1.00	0.97	1.00
2	0.94	1.00	0.97	1.00	0.94	1.00	0.94	1.00	0.94	1.00	0.95	1.00
1	1.00	1.00	1.01	1.00	0.98	1.00	0.99	1.00	1.00	1.00	1.01	1.00

Table D.8 Regression parameters of SSIF to SSUU

Story	Case 02		Case 04		Case 07		Case 09		Case 11		Case 13	
	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²
4	1.07	1.00	1.06	0.97	1.06	1.00	1.06	0.98	1.03	0.95	1.07	0.96
3	1.07	1.00	1.04	0.95	1.08	1.00	1.08	0.97	1.05	0.98	1.05	0.98
2	1.04	1.00	0.99	0.95	1.04	1.00	1.03	0.98	1.00	0.99	1.00	0.98
1	0.92	1.00	0.85	0.96	0.87	1.00	0.85	0.97	0.85	0.99	0.84	0.97

Table D.9 Regression parameters of SSIF to SSUB

Story	Case 02		Case 04		Case 07		Case 09		Case 11		Case 13	
	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²	Slope	R ²
4	1.30	0.91	1.08	0.89	1.18	0.95	1.15	0.93	1.27	0.94	1.24	0.93
3	1.07	1.00	1.06	0.94	1.12	1.00	1.11	0.96	1.08	0.98	1.08	0.98
2	1.10	1.00	1.02	0.94	1.11	1.00	1.10	0.98	1.06	0.98	1.05	0.98
1	0.92	1.00	0.83	0.96	0.89	1.00	0.86	0.97	0.85	0.99	0.83	0.97

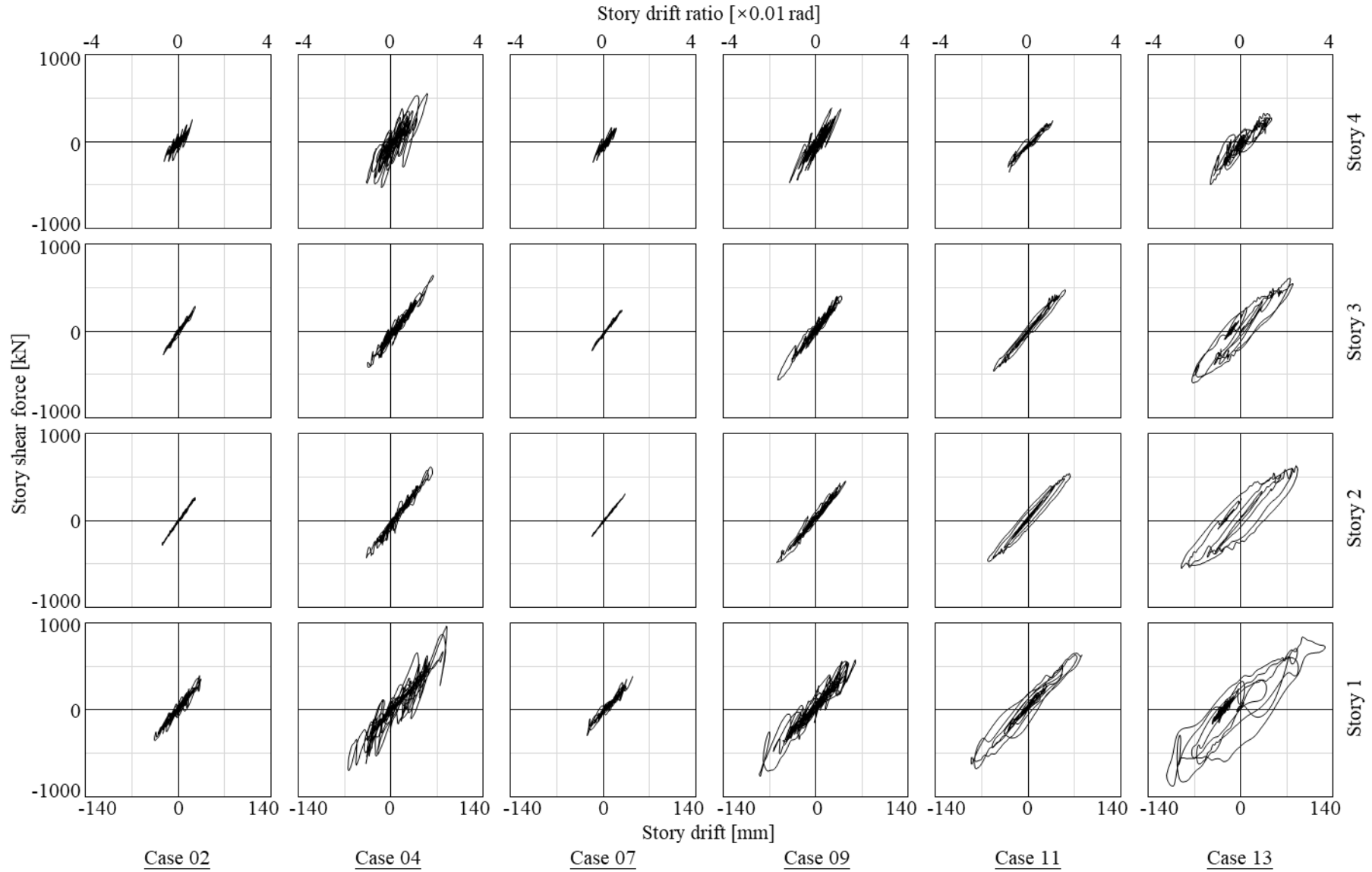


Figure D.11 Story hysteresis of the whole structure

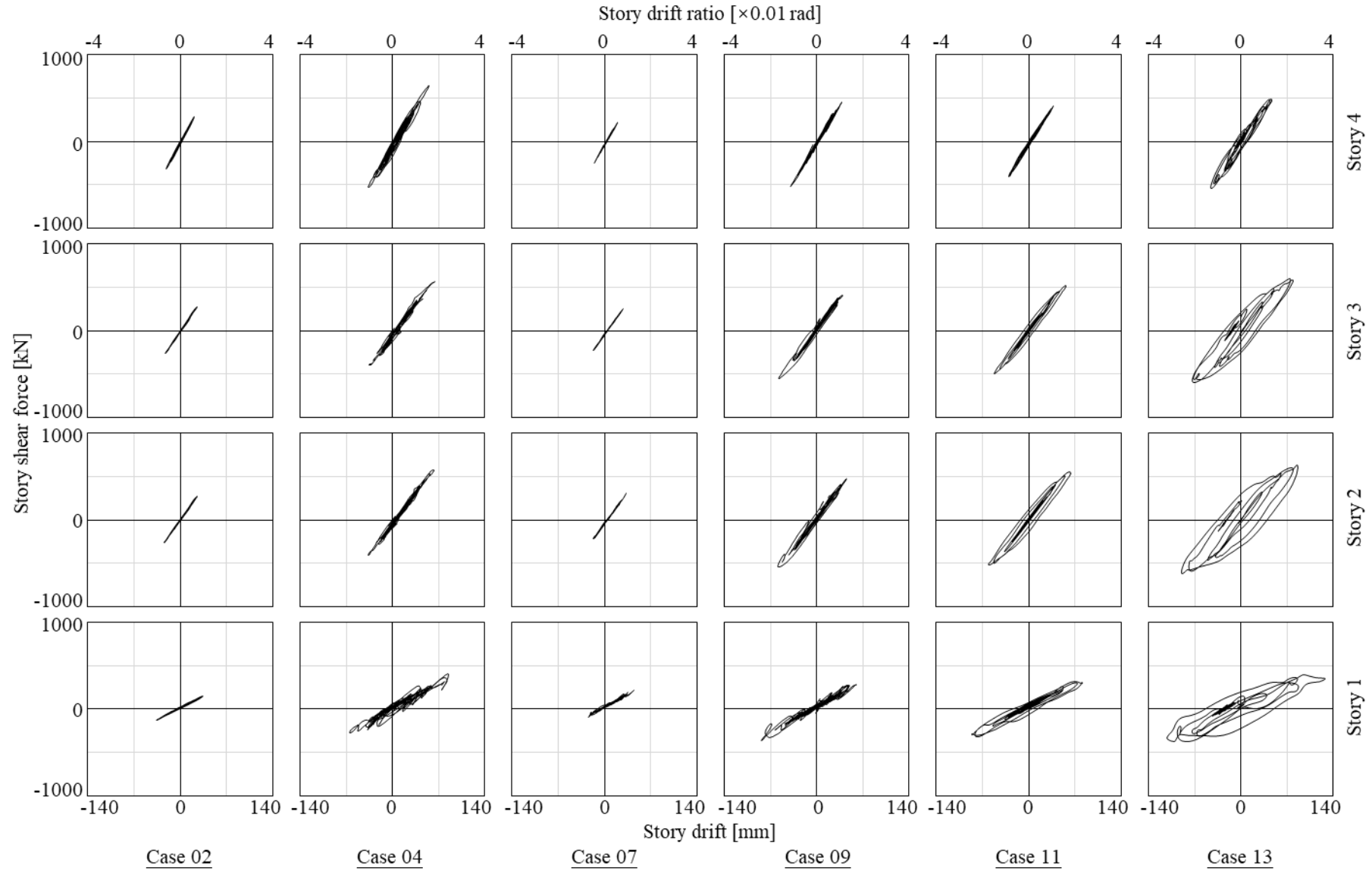


Figure D.12 Story hysteresis of the MRF

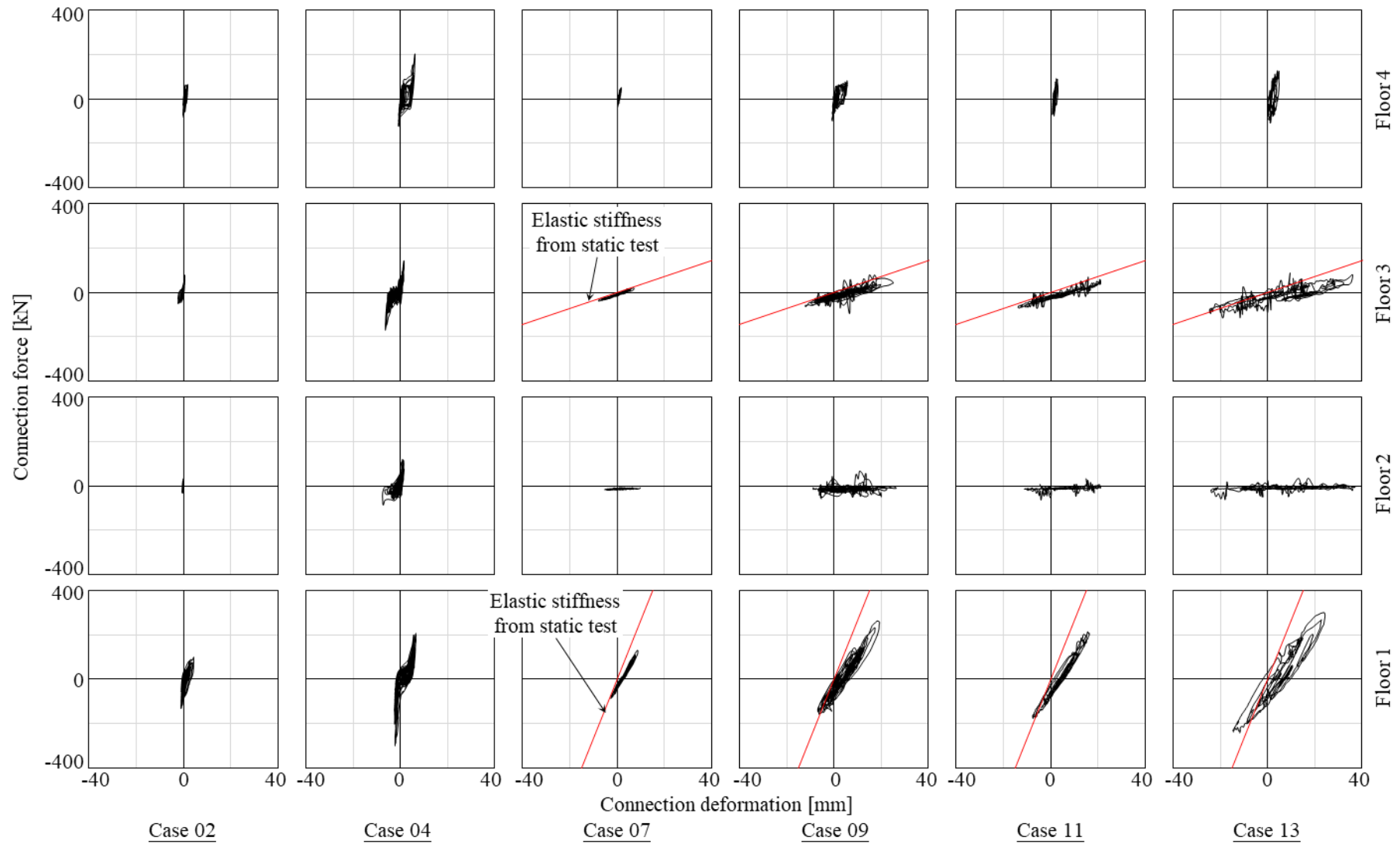


Figure D.13 MRF-spine connection hysteresis of the south frame

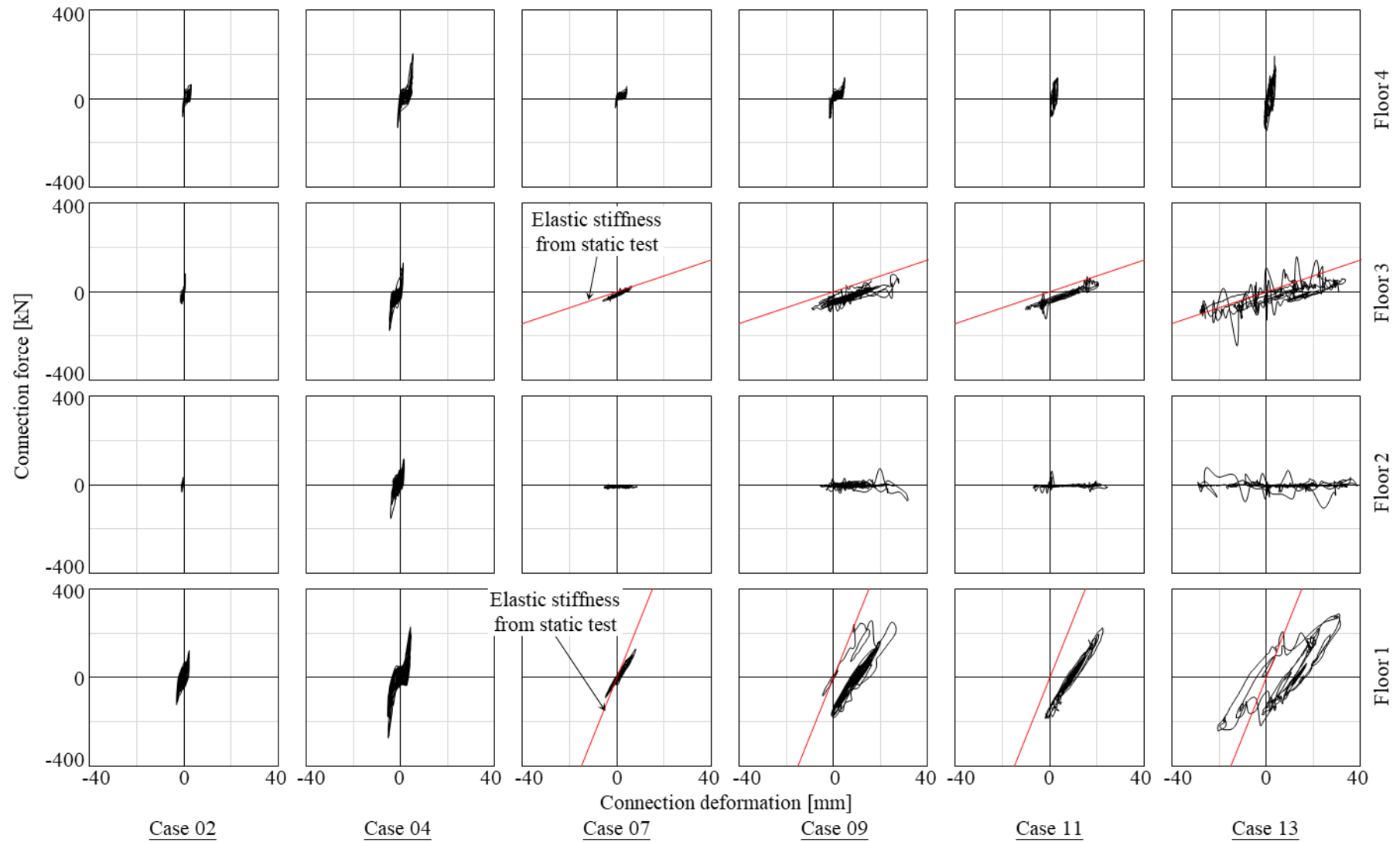


Figure D.14 MRF-spine connection hysteresis of the north frame

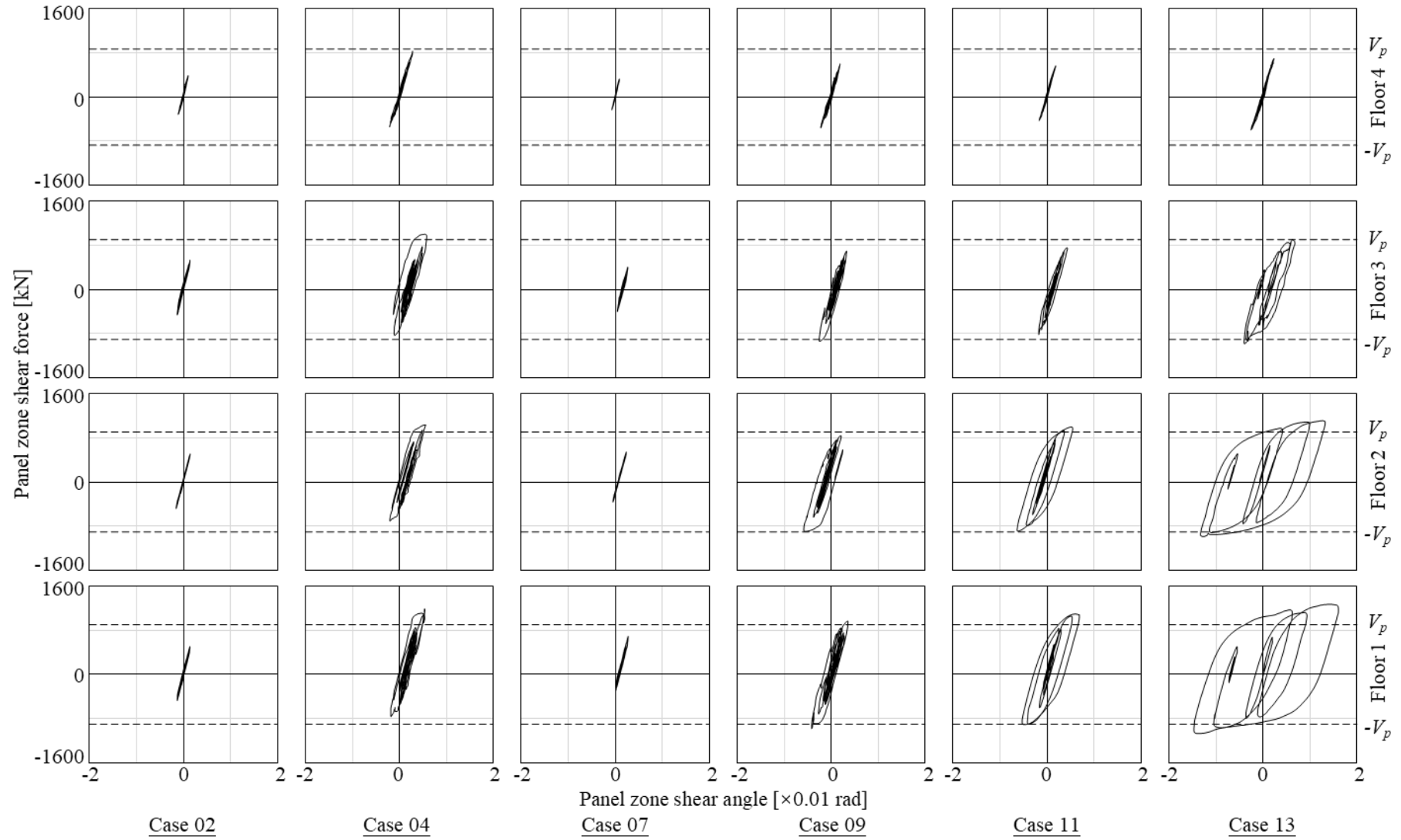


Figure D.15 Panel zone hysteresis of the east panels at the south frame (column C1)

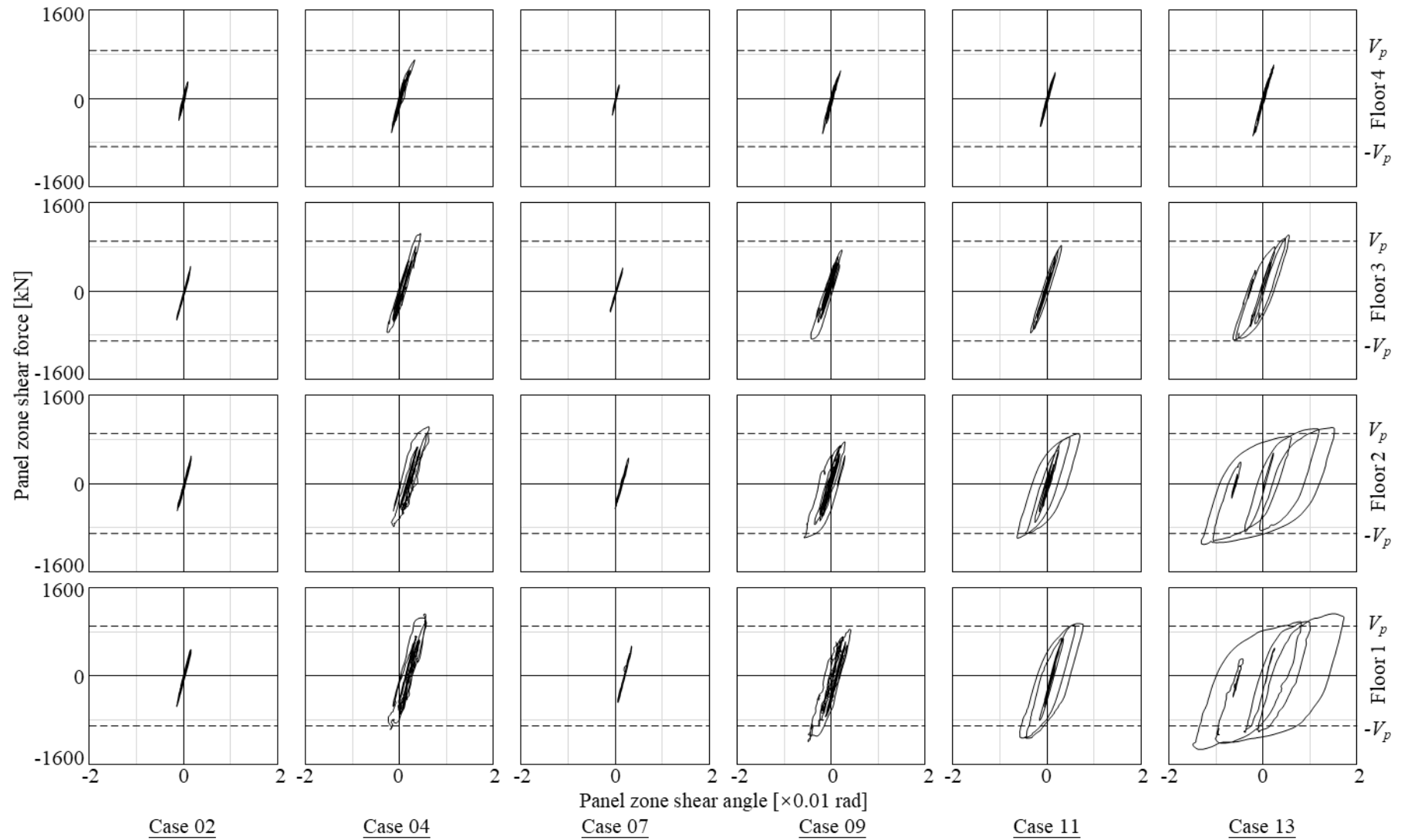


Figure D.16 Panel zone hysteresis of the west panels at the south frame (column C2)

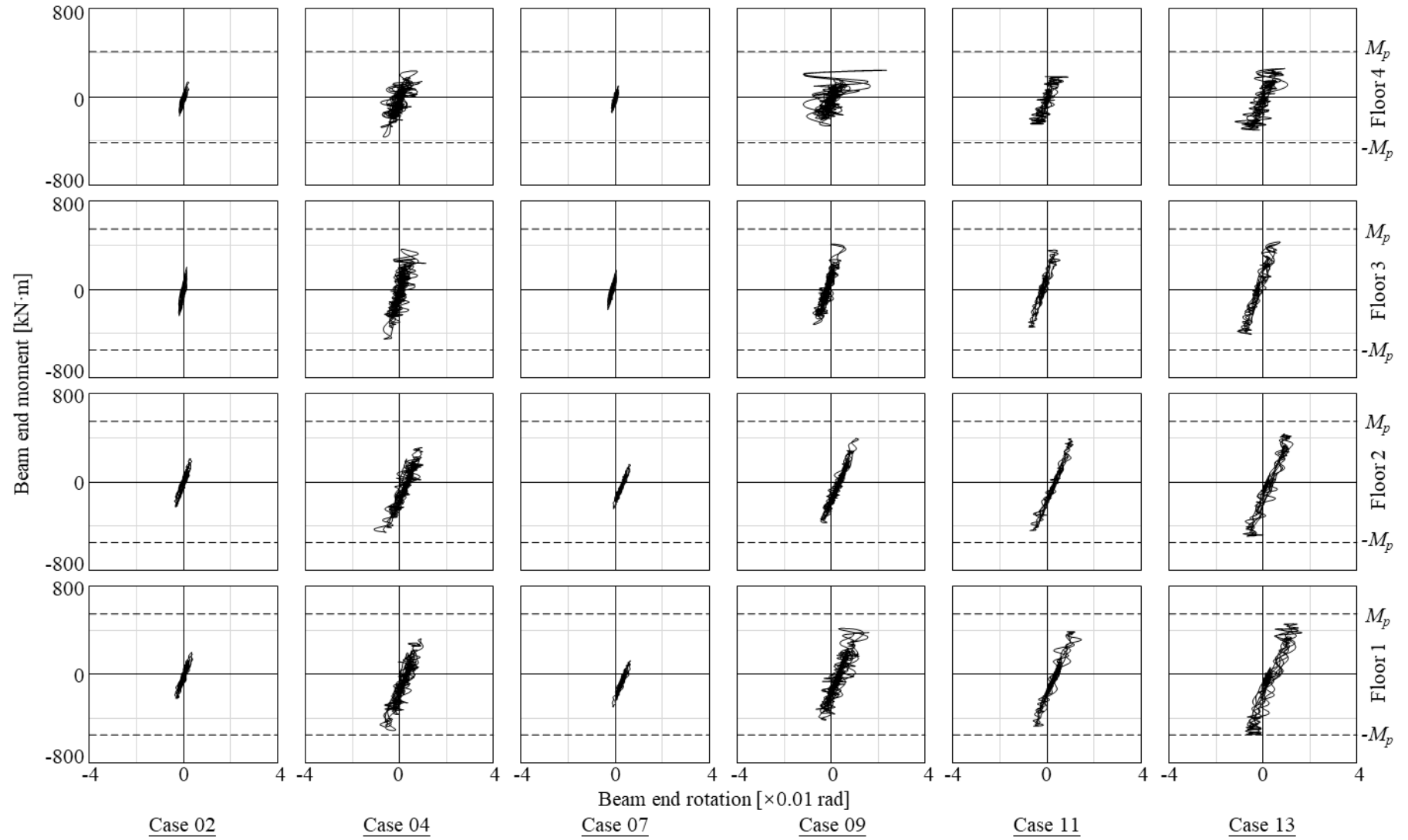


Figure D.17 Beam end hysteresis of the east ends at the south frame (intercepting to column C1)

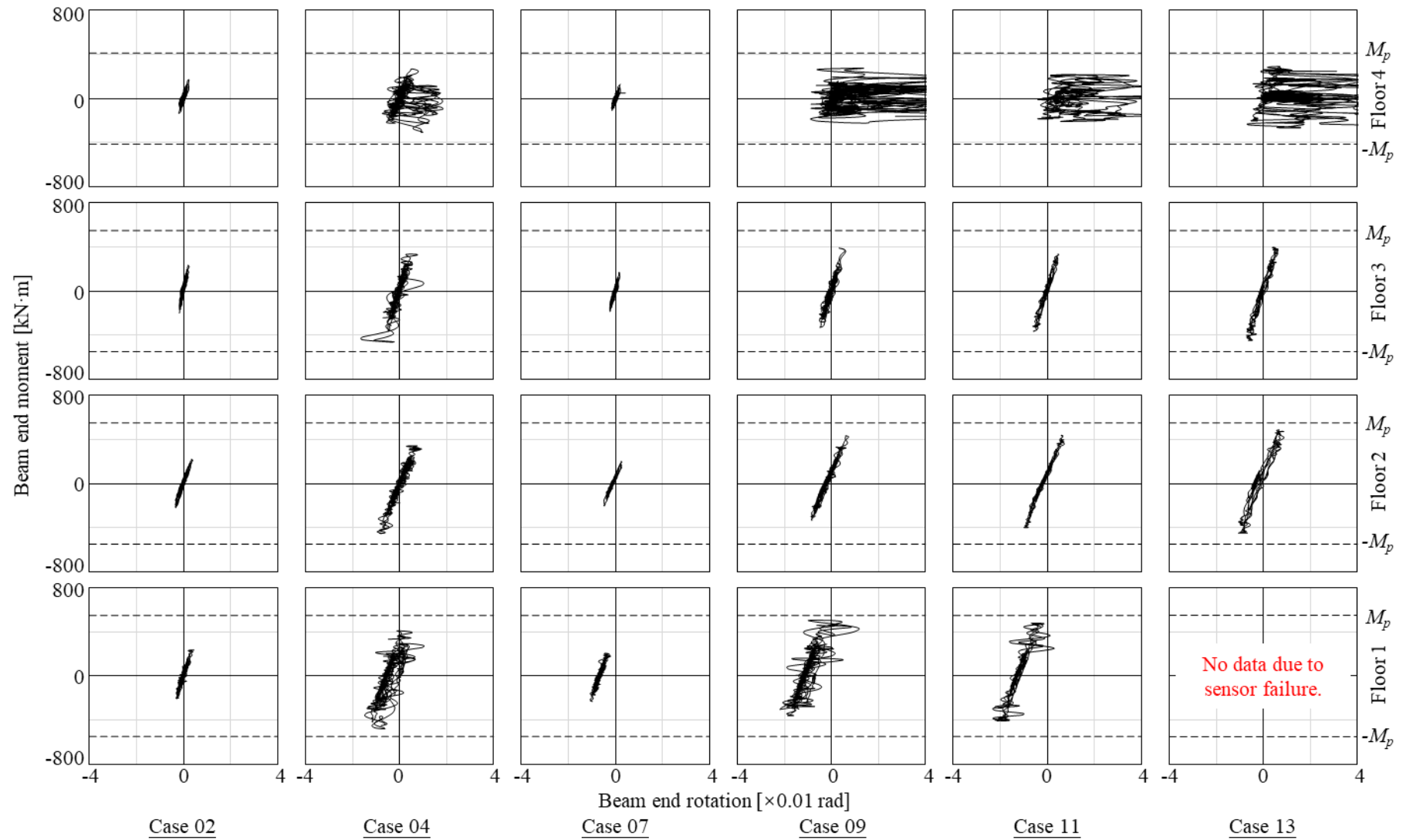


Figure D.18 Beam end hysteresis of the west ends at the south frame (intercepting to column C2)

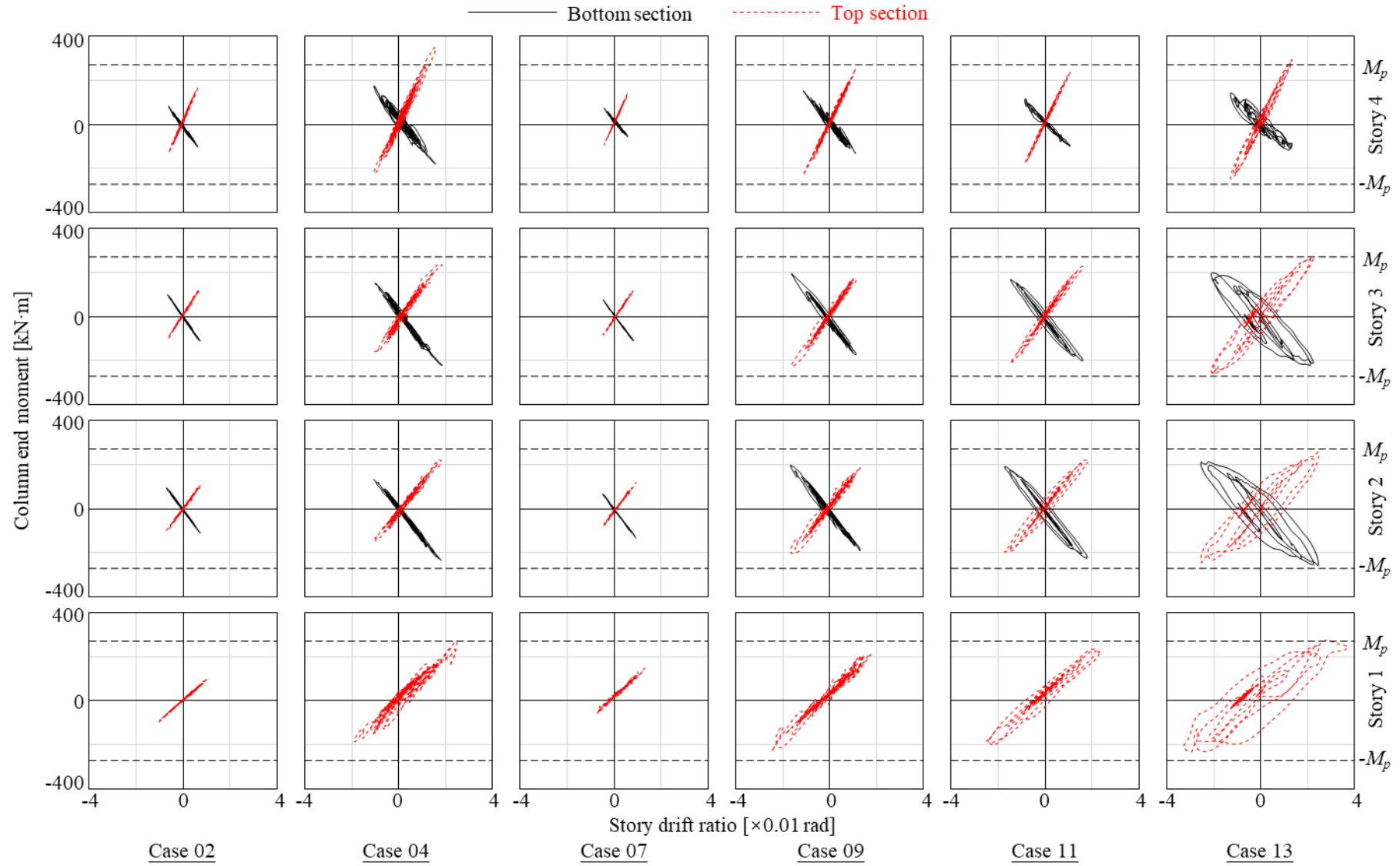


Figure D.19 Column end hysteresis of column C1 (south-east column)

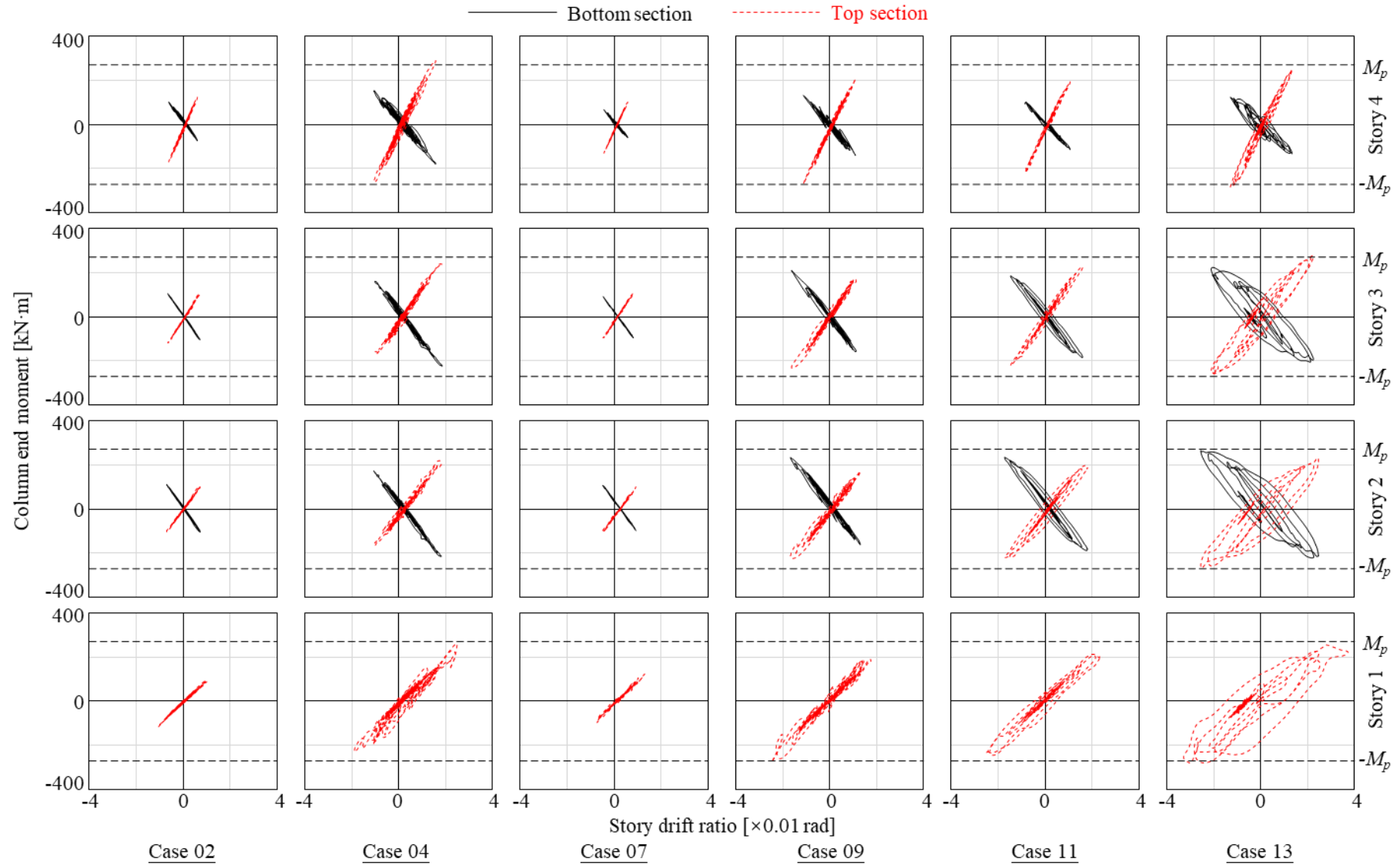


Figure D.20 Column end hysteresis of column C2 (south-west column)

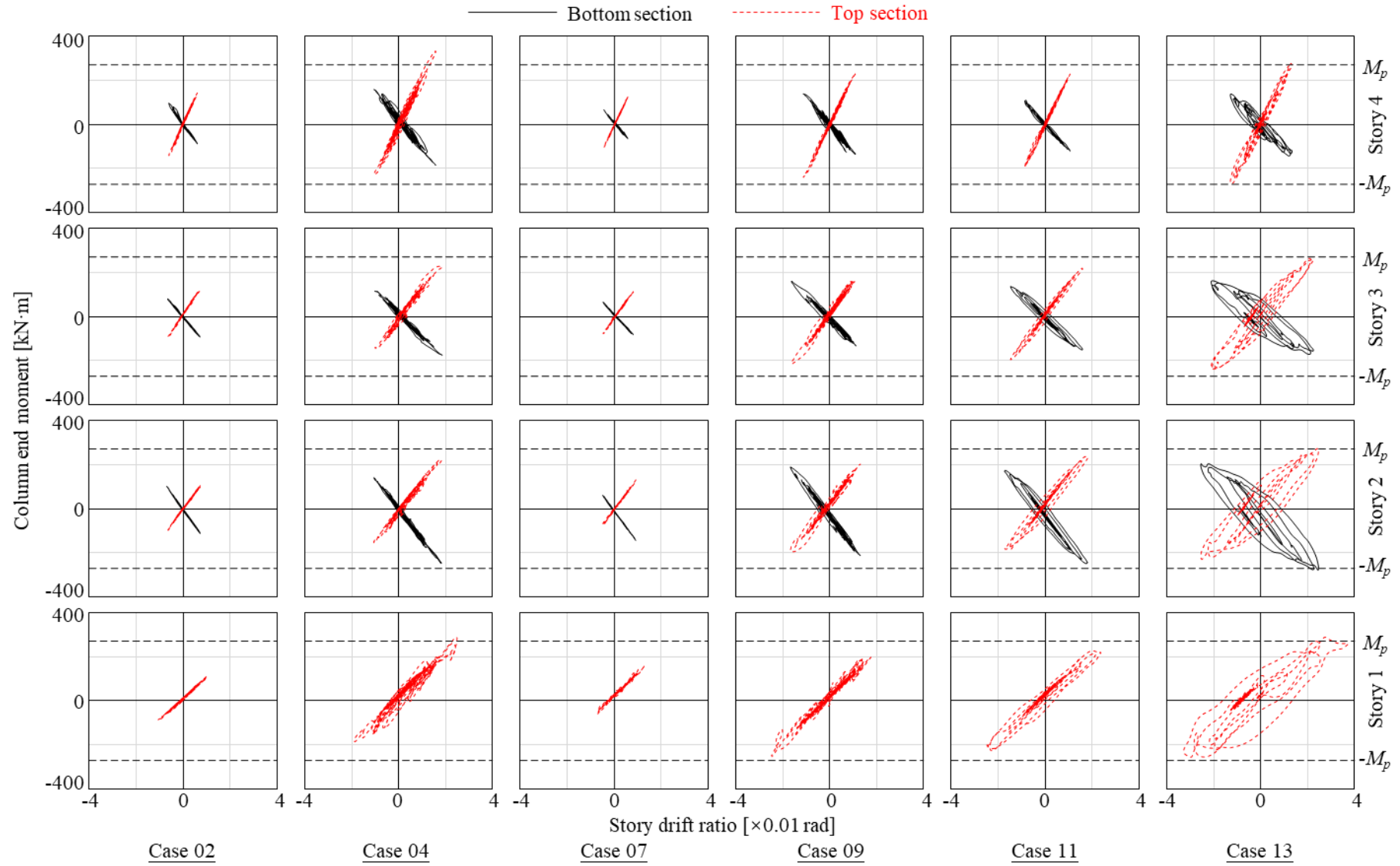


Figure D.21 Column end hysteresis of column C3 (north-east column)

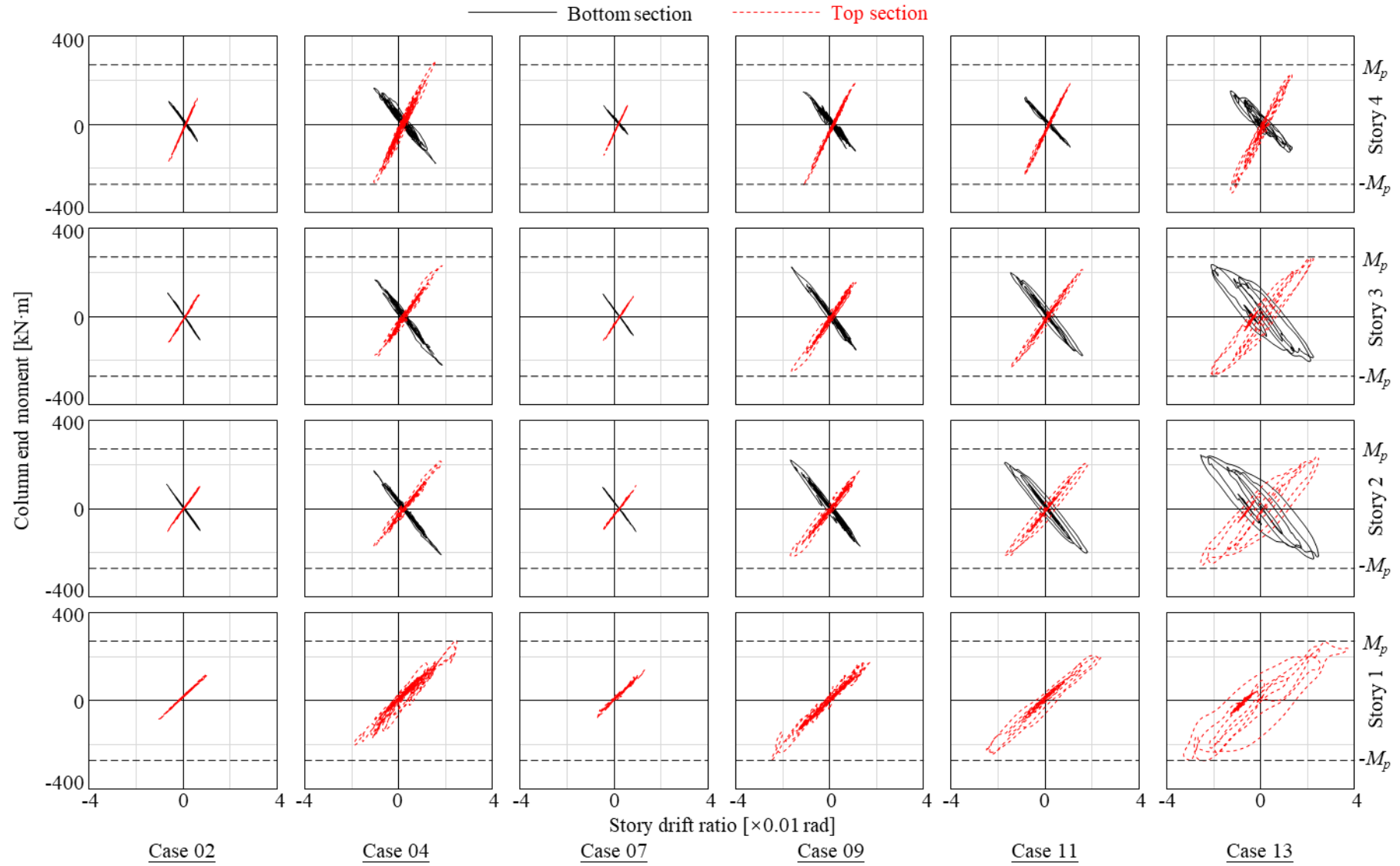


Figure D.22 Column end hysteresis of column C4 (north-west column)

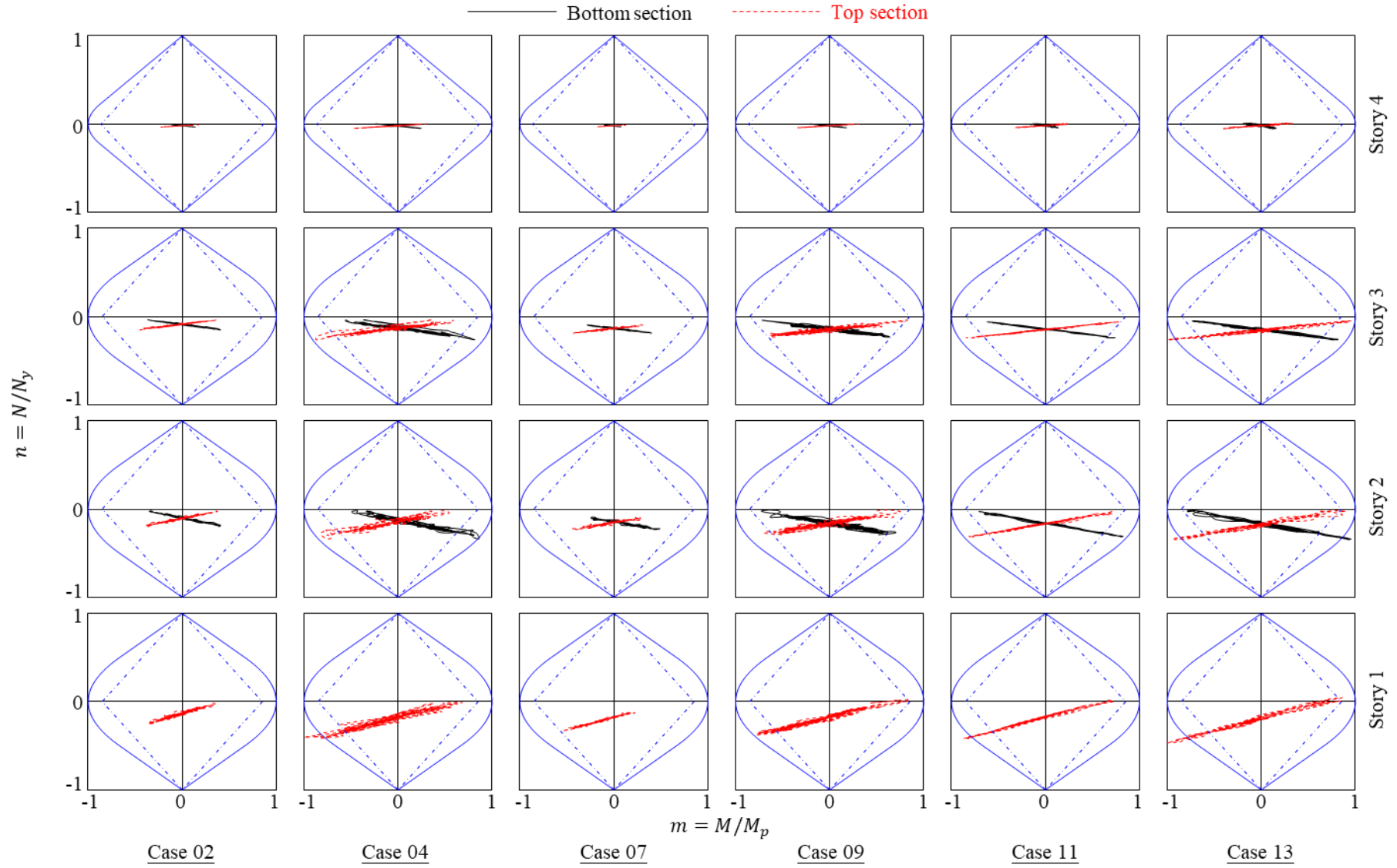


Figure D.23 P-M interaction curve of column C1 (south-east column)

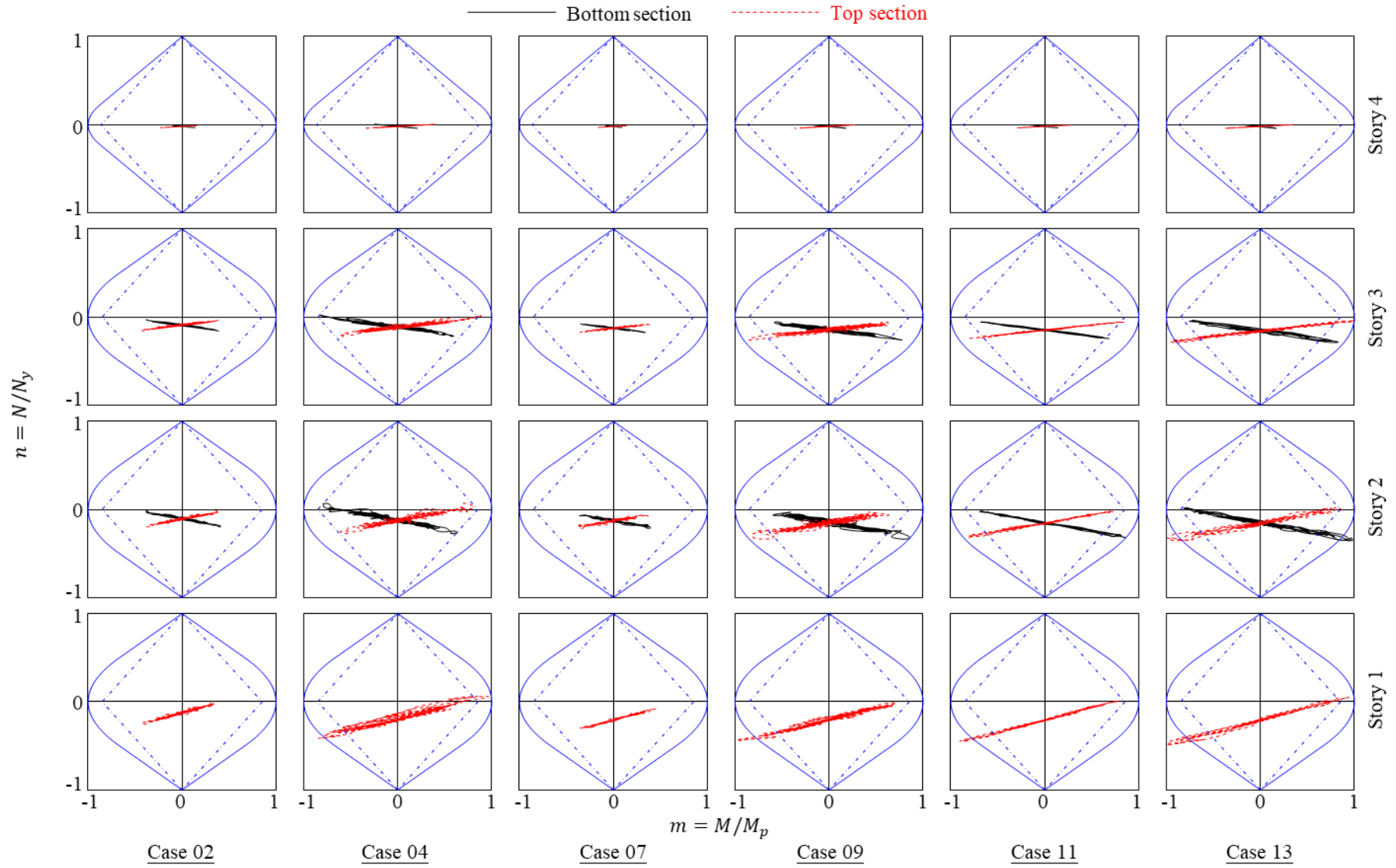


Figure D.24 P-M interaction curve of column C2 (south-west column)

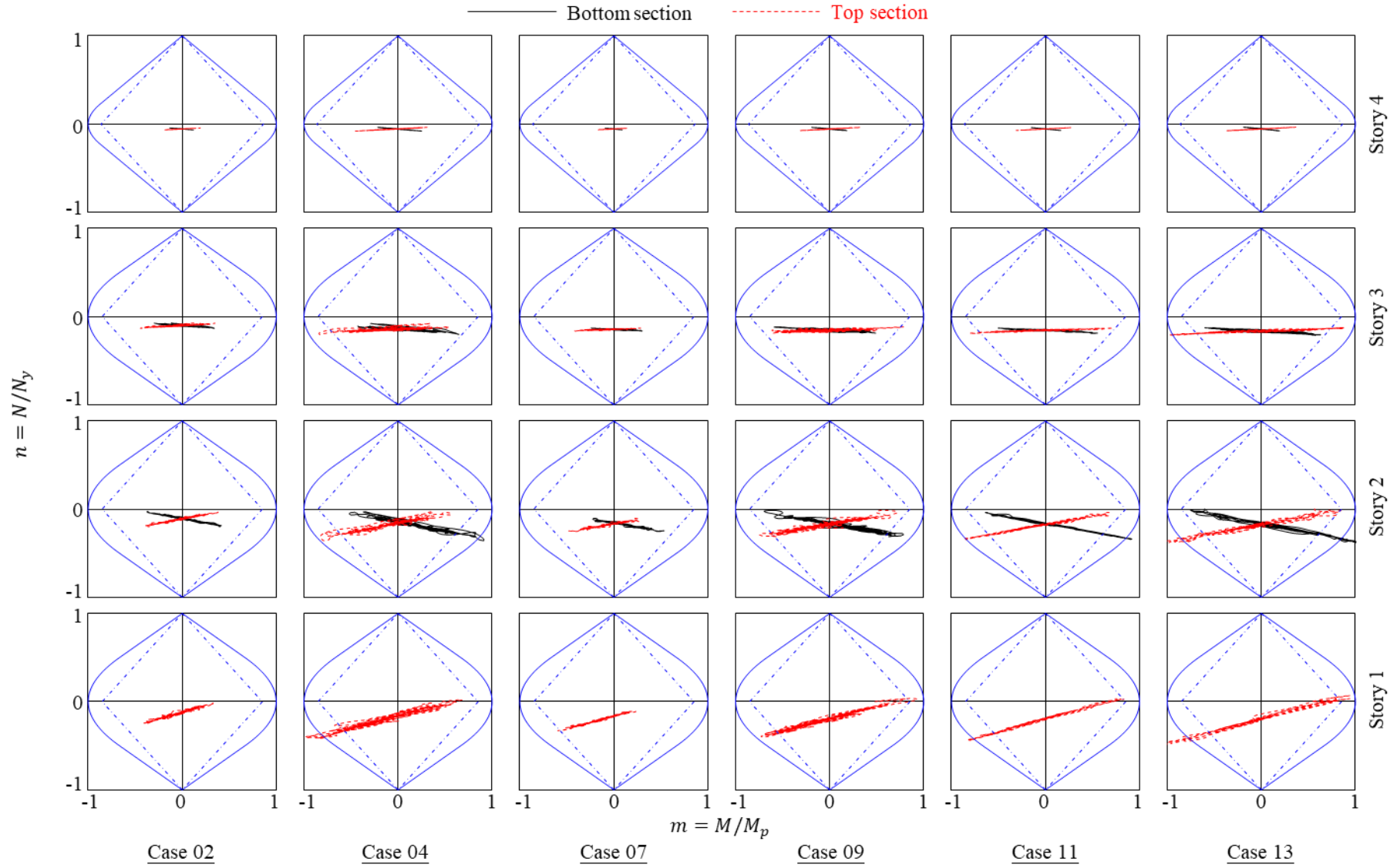


Figure D.25 P-M interaction curve of column C3 (north-east column)

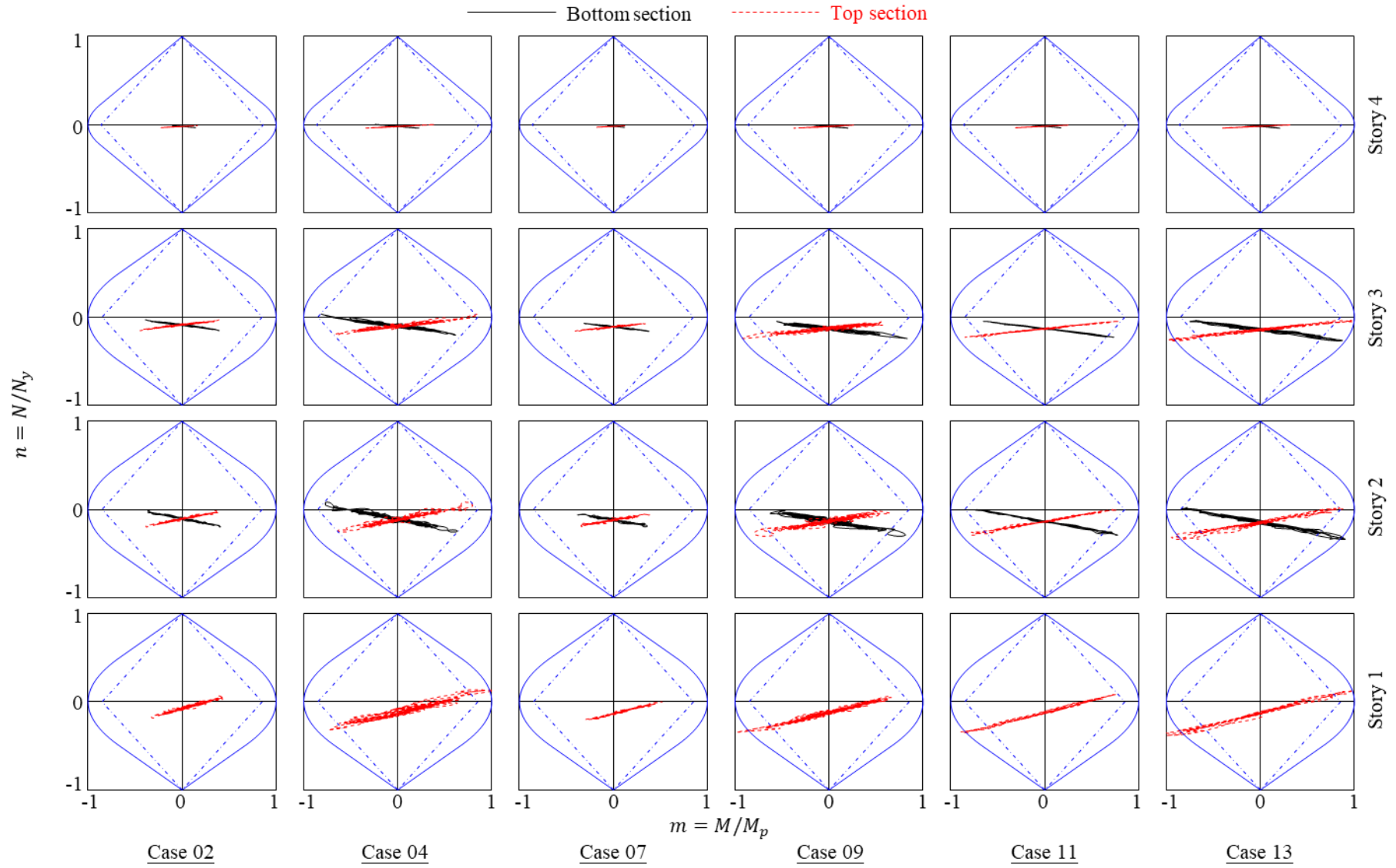


Figure D.26 P-M interaction curve of column C4 (north-west column)

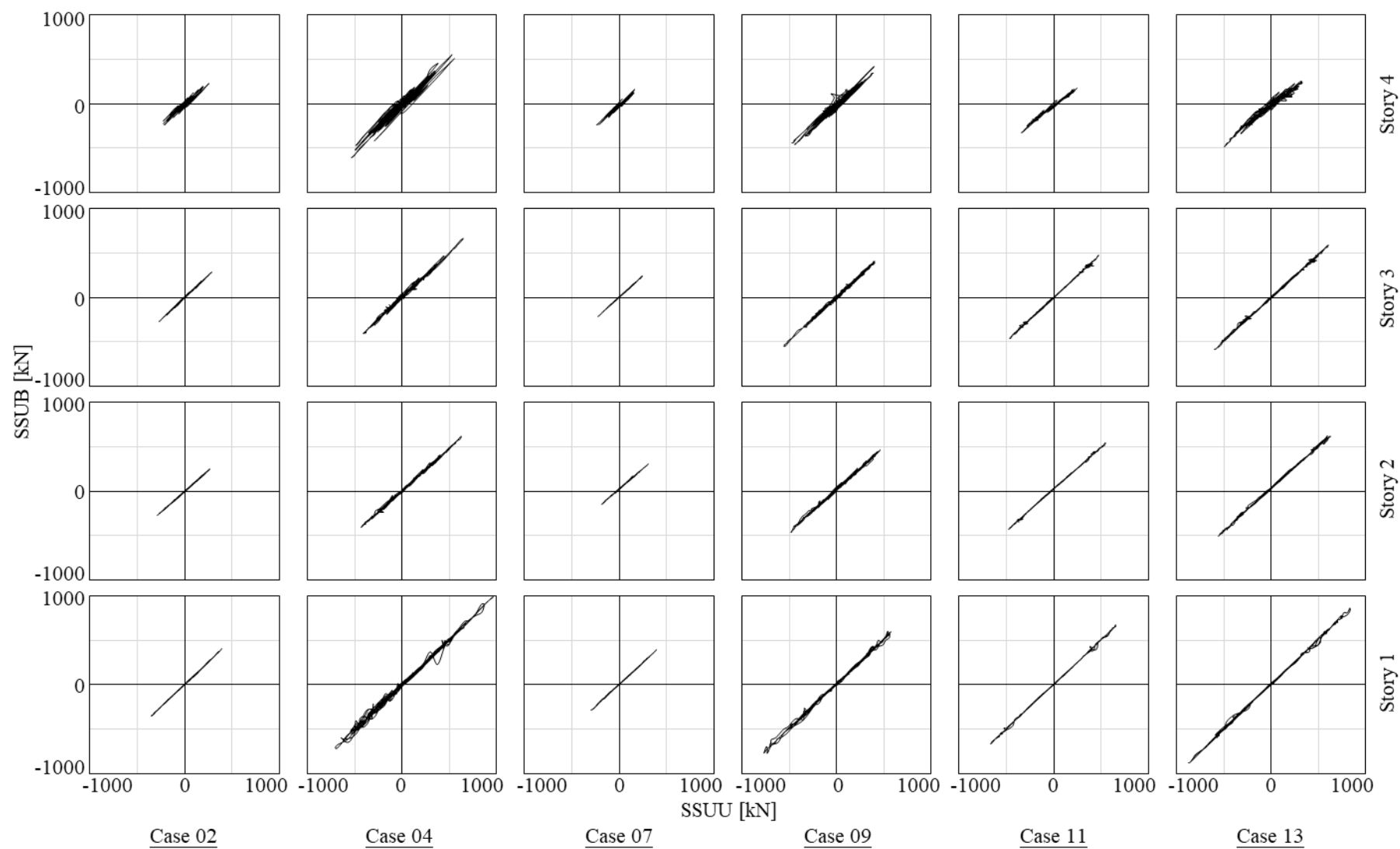


Figure D.27 Linear relationship of SSUU to SSUB

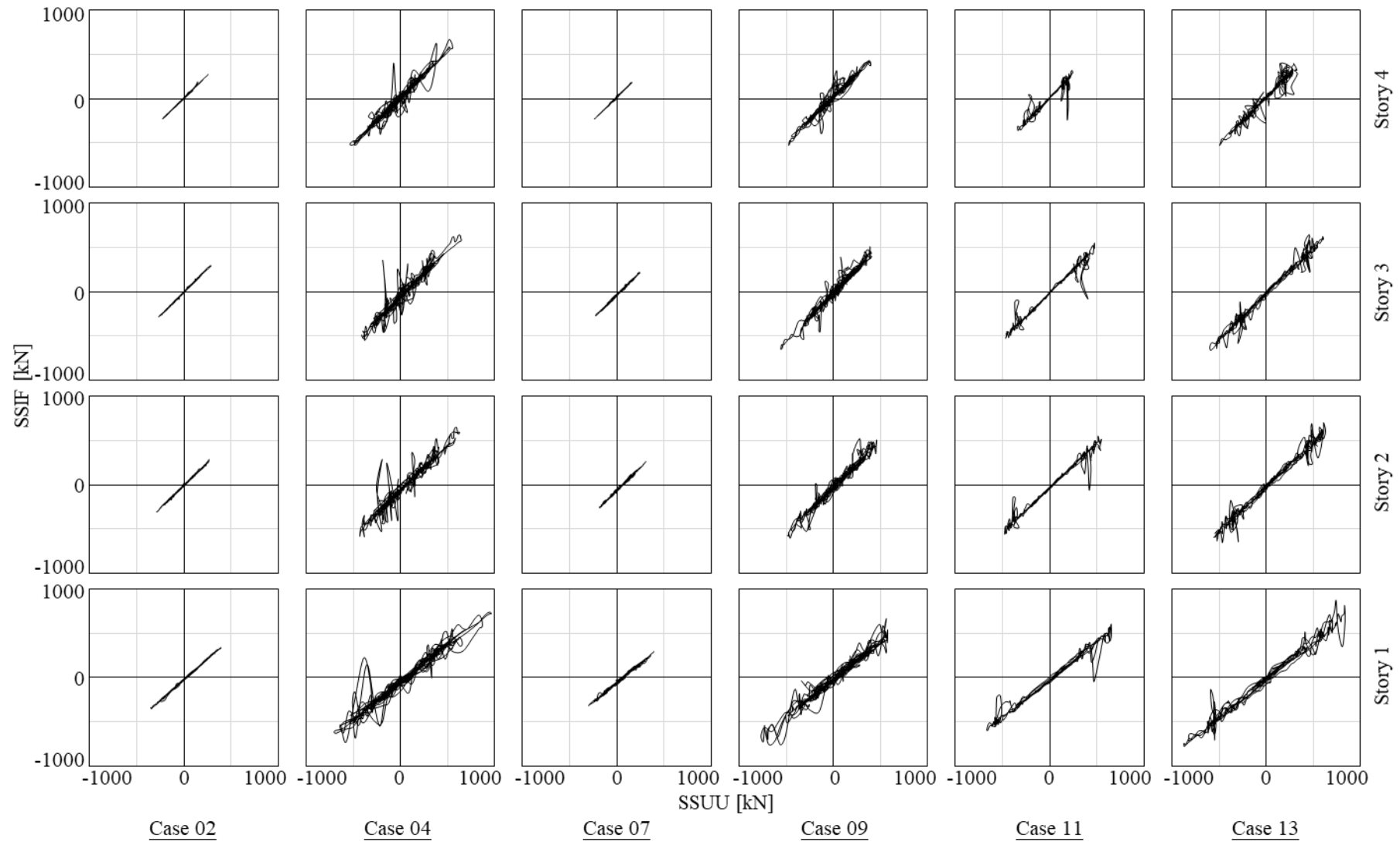


Figure D.28 Linear relationship of SSUU to SSUF

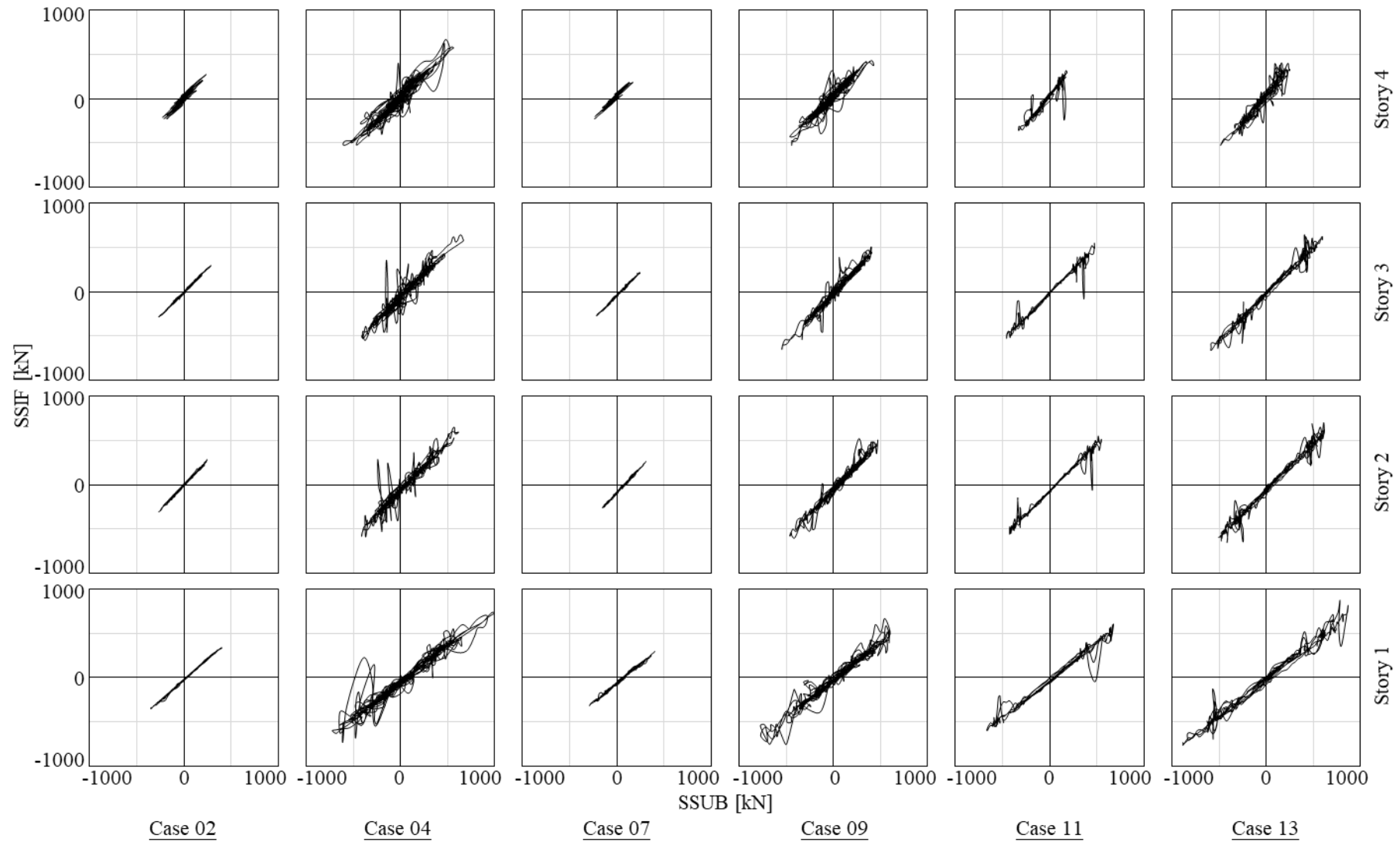


Figure D.29 Linear relationship of SSUB to SSIF

APPENDIX E. FULL RESULTS OF VIBRATION PROPERTY ANALYSIS: 20EDF

E.1 GENERAL

This appendix provides results of the vibration property analysis on the 20EDF test. The results are grouped by different tests, each including results from all analysis methods. The following results are provided in this appendix:

- 1) **Modal parameters:** natural periods, modal damping ratios, mode shapes, Modal Effective Mass (MEM) and PSD peaks from modal analysis methods.
- 2) **Spectral results:** peak values of Effective Modal Pseudo-Acceleration (EMPA).
- 3) **Time-frequency results:** magnitude of wavelet coefficients, instantaneous periods, magnitude of ridge wavelet coefficient and reconstructed modal components.

In the 20EDF test, 8 white noise and 6 ground motion cases in the East-West direction were selected for analysis. The structural system was changed from MRF-Spine to MRF-Spine-FLC between Case 05 and 06 (both being white noise cases).

E.2 MODAL PARAMETERS FROM WHITE NOISE EXCITATIONS

E.2.1 Natural period

Figure E.1 plots the natural period in the white noise cases of the 20EDF test identified using the ARX, FDD and SS methods, respectively. Table E.1 summarizes the corresponding values.

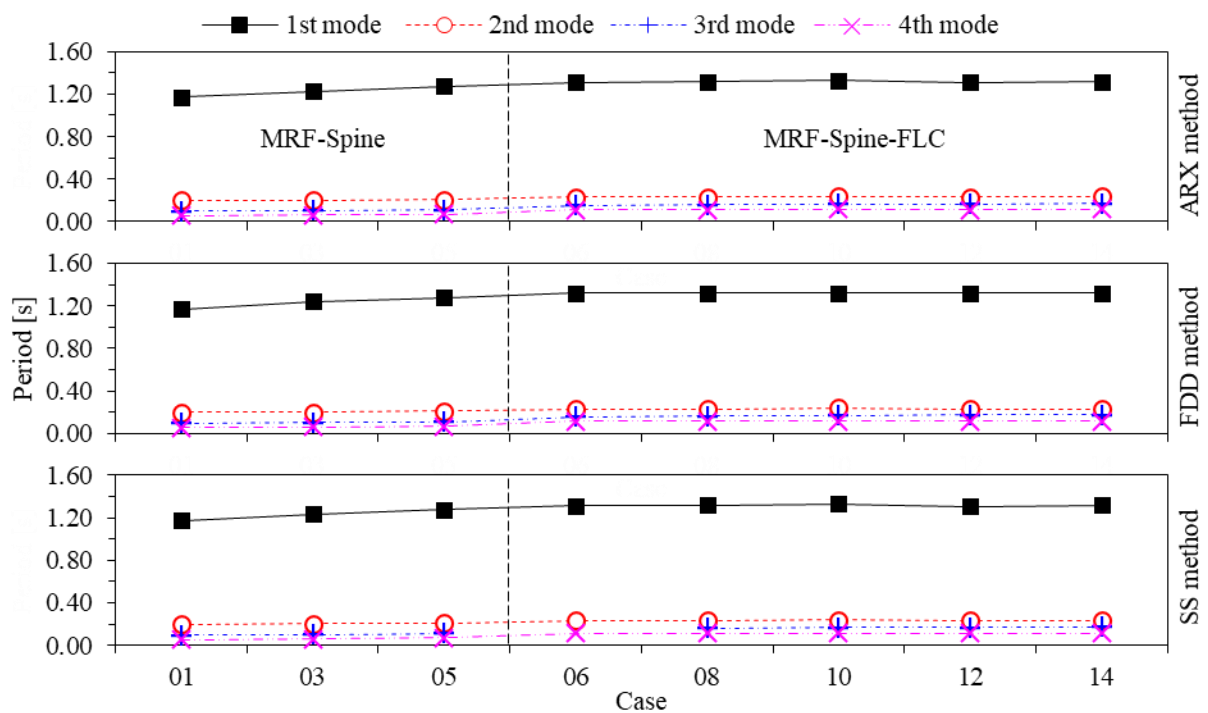


Figure E.1 Natural periods in white noise excitations of the 20EDF test

Table E.1 Natural periods in white noise excitations of the 20EDF test [Unit: s]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	1.17	1.17	1.17	0.20	0.20	0.20	0.10	0.10	0.10	0.06	0.06	0.06
03	1.23	1.24	1.24	0.20	0.20	0.20	0.10	0.10	0.10	0.06	0.06	0.06
05	1.27	1.28	1.27	0.21	0.21	0.21	0.11	0.11	0.11	0.07	0.07	0.07
06	1.31	1.32	1.31	0.23	0.23	0.23	0.16	0.16	N/A	0.11	0.11	0.11
08	1.32	1.32	1.32	0.23	0.23	0.23	0.16	0.16	0.16	0.11	0.11	0.11
10	1.33	1.32	1.33	0.24	0.24	0.24	0.17	0.17	0.17	0.11	0.12	0.12
12	1.31	1.32	1.31	0.23	0.23	0.23	0.17	0.17	0.17	0.11	0.11	0.11

E.2.2 Modal damping ratio

Figure E.2 plots the modal damping ratios in the white noise cases of the 20EDF test identified using the ARX, FDD and SS methods, respectively. Table E.2 summarizes the corresponding values.

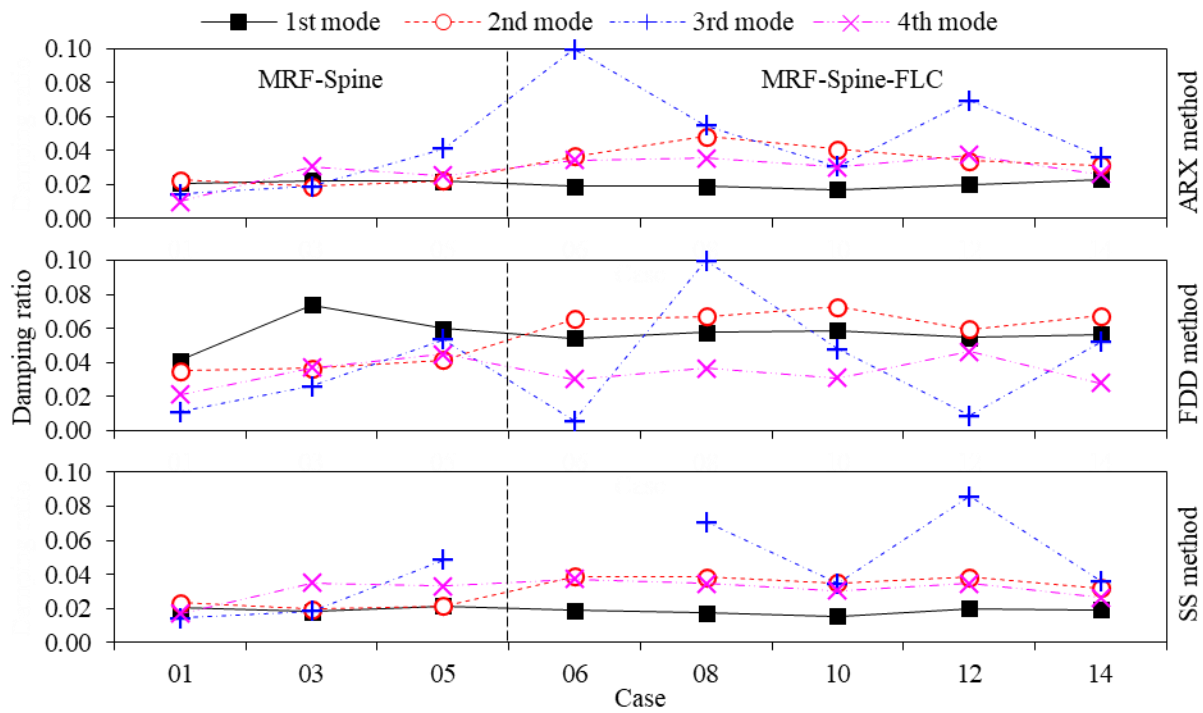


Figure E.2 Damping ratios in white noise excitations of the 20EDF test

Table E.2 Damping ratios in white noise excitations of the 20EDF test

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.02	0.04	0.02	0.02	0.04	0.02	0.01	0.01	0.01	0.01	0.02	0.02
03	0.02	0.07	0.02	0.02	0.04	0.02	0.02	0.03	0.02	0.03	0.04	0.04
05	0.02	0.06	0.02	0.02	0.04	0.02	0.04	0.05	0.05	0.03	0.04	0.03
06	0.02	0.05	0.02	0.04	0.07	0.04	0.10	0.01		0.03	0.03	0.04
08	0.02	0.06	0.02	0.05	0.07	0.04	0.05	0.10	0.07	0.04	0.04	0.03
10	0.02	0.06	0.02	0.04	0.07	0.04	0.03	0.05	0.03	0.03	0.03	0.03
12	0.02	0.05	0.02	0.03	0.06	0.04	0.07	0.01	0.09	0.04	0.05	0.03

E.2.3 Mode shapes

Figure E.3 plots the mode shapes in the white noise cases of the 20EDF test identified using the ARX, FDD and SS methods, respectively. The plotted mode shapes are normalized by the modal participation factor Γ_n .

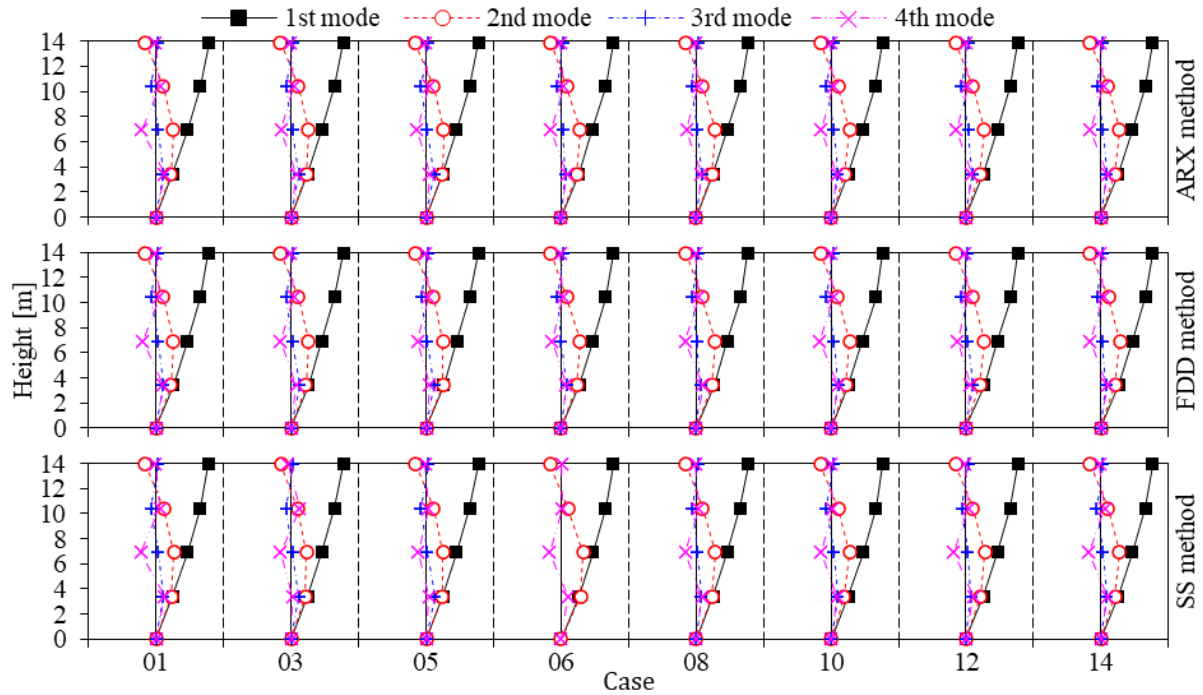


Figure E.3 Mode shapes in white noise excitations of the 20EDF test

E.2.4 Modal Effective Mass (MEM)

Figure E.4 plots the Modal Effective Mass (MEM) in the white noise cases of the 20EDF test identified using the ARX, FDD and SS methods, respectively. Table E.3 summarizes the corresponding values of the MEMs. The total mass of the structure is 153.2 tons, with Floor 1 to 4 being 26.1 tons, 16.4 tons, 53.9 tons and 56.8 tons, respectively.

Table E.3 MEMs in white noise excitations of the 20EDF test [Unit: ton]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	139.6	140.1	139.6	9.9	10.1	10.7	1.5	1.6	1.5	2.7	2.1	2.8
03	139.5	139.9	139.5	10.4	10.3	8.6	1.7	1.8	1.6	1.2	1.2	3.1
05	139.7	140.4	139.6	11.0	11.0	10.7	1.8	1.7	1.9	1.0	0.8	0.7
06	140.6	141.1	140.9	10.3	10.2	14.3	0.8	1.0		1.2	0.9	1.5
08	140.5	140.9	140.1	10.6	10.7	10.8	1.1	0.8	0.9	1.0	1.5	1.4
10	140.4	140.6	140.8	10.1	10.2	10.0	1.5	1.7	1.4	1.2	1.7	1.3
12	140.6	141.2	140.6	9.6	9.8	10.2	1.3	1.4	1.0	1.1	0.8	1.9

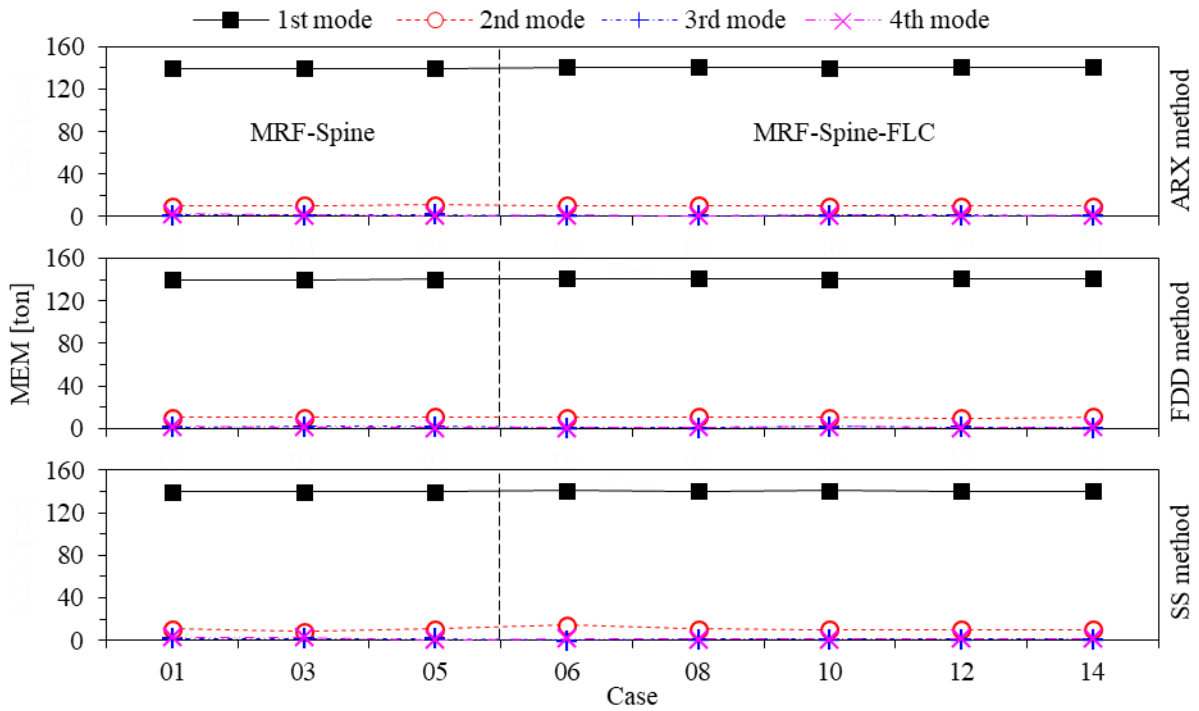


Figure E.4 MEMs in white noise excitations of the 20EDF test

E.2.5 PSD peaks

Figure E.5 plots the peak values of the Power Spectra Density (PSD) in the white noise cases identified using the FDD method. Table E.4 summarizes the corresponding values.

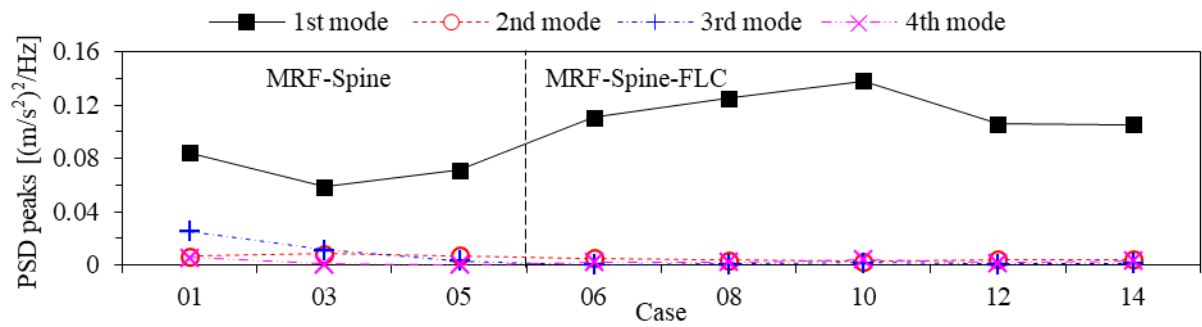


Figure E.5 PSD peaks in white noise excitations of the 20EDF test

 Table E.4 PSD peaks in white noise excitations of the 20EDF test [Unit: (m/s²)²/Hz]

Case	Mode			
	1st	2nd	3rd	4th
01	0.084	0.006	0.025	0.005
03	0.058	0.008	0.011	0.001
05	0.071	0.007	0.003	0.000
06	0.111	0.005	0.000	0.002
08	0.125	0.004	0.001	0.002
10	0.138	0.003	0.002	0.004
12	0.106	0.004	0.001	0.001
14	0.105	0.004	0.001	0.003

E.3 MODAL PARAMETERS FROM GROUND MOTION EXCITATIONS

E.3.1 Natural period

Figure E.6 plots the natural period in the ground motion cases of the 20EDF test estimated using the ARX, FDD and SS methods, respectively. Table E.5 summarizes the corresponding values in each segment.

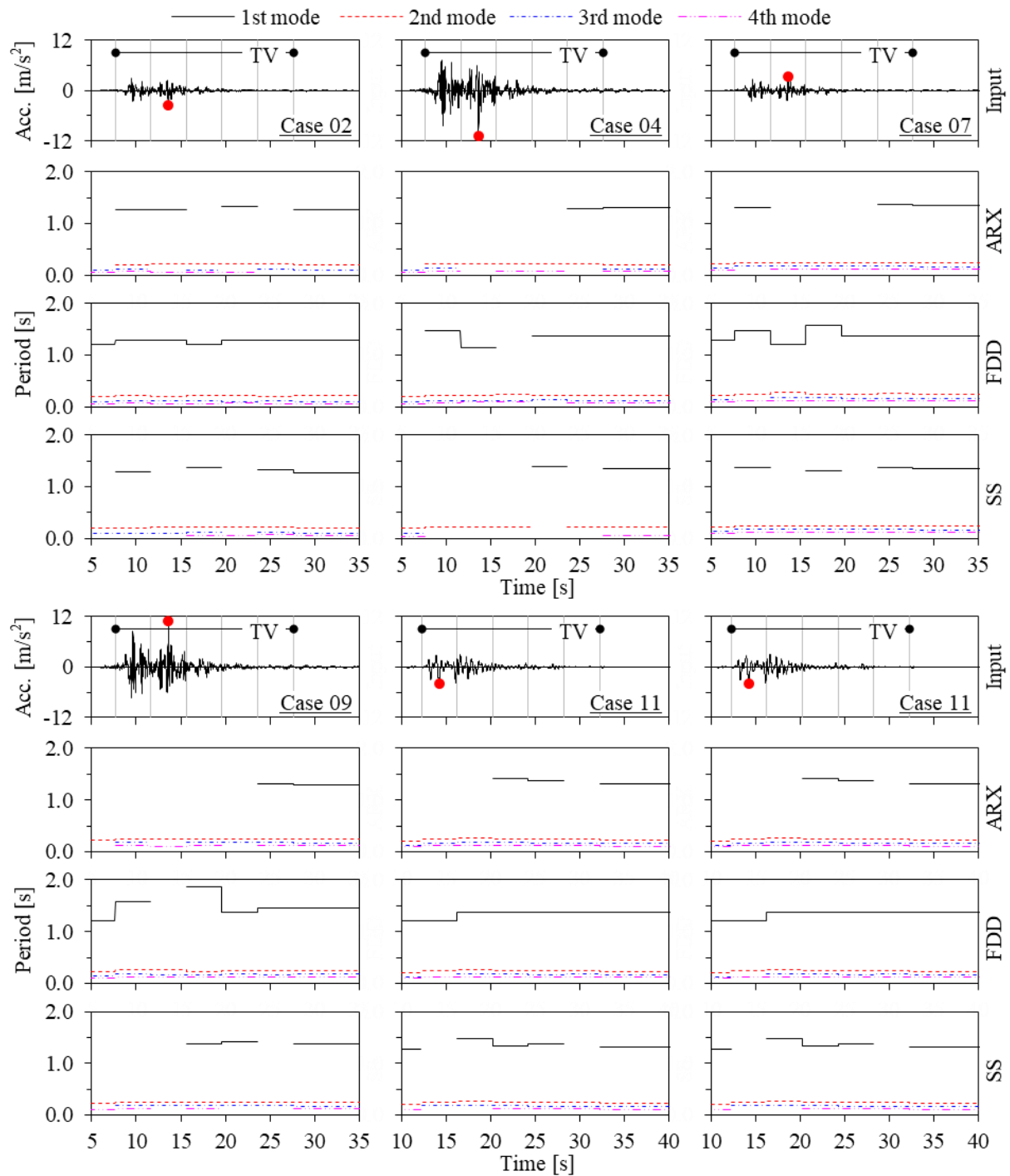


Figure E.6 Natural periods in ground motion excitations of the 20EDF test

Table E.5 Natural periods in ground motion excitations of the 20EDF test [Unit: s]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01		1.20			0.20	0.20	0.10	0.10	0.10	0.06	0.06	
02-02	1.27	1.28	1.28	0.21	0.21	0.22	0.12	0.12	0.10	0.07	0.07	
02-03	1.27	1.28		0.21	0.21	0.20		0.12	0.10	0.06	0.06	
02-04		1.20	1.37	0.21	0.21	0.22	0.10	0.12	0.11	0.06	0.06	0.06
02-05	1.33	1.28		0.21	0.21	0.21		0.10		0.06	0.07	0.06
02-06		1.28	1.32	0.21	0.21	0.20	0.12	0.11	0.12		0.06	0.07
02-07	1.27	1.28	1.27	0.21	0.21	0.21	0.11	0.11	0.11		0.06	0.06
04-01					0.21	0.20	0.10	0.10	0.10	0.06	0.06	0.06
04-02		1.46		0.22	0.22	0.22	0.13	0.12		0.07	0.07	
04-03		1.14		0.23	0.23	0.23		0.11			0.10	
04-04				0.22	0.22	0.24		0.11		0.07	0.09	
04-05		1.37	1.39	0.22		0.22		0.13		0.07		
04-06	1.29	1.37		0.22	0.22	0.22		0.12			0.07	
04-07	1.32	1.37	1.35	0.22	0.22	0.23	0.12	0.12		0.07	0.07	0.06
07-01		1.28		0.22	0.23	0.22	0.13	0.13	0.13	0.11	0.11	0.11
07-02	1.30	1.46	1.37	0.24	0.24	0.24	0.18		0.17		0.12	0.12
07-03		1.20		0.25	0.24	0.27	0.17	0.17	0.17	0.12	0.12	0.12
07-04		1.58	1.31	0.24	0.24	0.24	0.17	0.18	0.17	0.12	0.12	0.12
07-05		1.37		0.24	0.24	0.25	0.17	0.17	0.17	0.11	0.12	0.12
07-06	1.36	1.37	1.36	0.24	0.24	0.26	0.17	0.17	0.17	0.11	0.12	0.11
07-07	1.34	1.37	1.34	0.24	0.24	0.25	0.17	0.17	0.17	0.11	0.12	0.11
09-01		1.20		0.24	0.23	0.23		0.14			0.11	0.11
09-02		1.58		0.25	0.25	0.27	0.18	0.18	0.18	0.11	0.12	0.12
09-03				0.25	0.25	0.28		0.17	0.18	0.11	0.12	
09-04		1.86	1.37	0.25	0.25	0.24	0.18	0.17	0.18	0.12	0.12	0.12
09-05		1.37	1.41	0.24	0.25	0.25	0.18	0.18	0.18		0.12	
09-06	1.32	1.46		0.24	0.24	0.26	0.17	0.17	0.18	0.12	0.12	
09-07	1.29	1.46	1.37	0.24	0.24	0.25	0.17	0.18	0.17	0.12	0.12	0.12
11-01		1.20	1.27	0.21	0.21	0.22	0.13	0.13		0.10	0.10	0.10
11-02		1.20		0.25	0.25	0.24	0.17		0.18	0.12	0.12	
11-03		1.37	1.49	0.26	0.26	0.27	0.18	0.19	0.18	0.12	0.12	
11-04	1.41	1.37	1.33	0.24	0.24	0.25	0.18	0.19	0.17	0.12	0.12	0.12
11-05	1.37	1.37	1.37	0.25	0.25	0.25	0.17	0.17	0.17	0.12	0.12	0.11
11-06		1.37		0.24	0.24	0.25	0.17	0.18	0.17	0.11	0.11	0.12
11-07	1.31	1.37	1.31	0.23	0.23	0.23	0.17	0.17	0.17	0.11	0.11	0.11
13-01				0.24	0.23	0.23	0.17	0.18		0.12	0.11	0.11
13-02		1.20		0.26	0.26	0.29	0.18	0.19	0.19		0.12	
13-03		1.58		0.25	0.25	0.27	0.18	0.18	0.18	0.13	0.12	0.12
13-04		1.58	1.33	0.24	0.24	0.25	0.18	0.18	0.18	0.12	0.12	0.12
13-05	1.36	1.46	1.36	0.24	0.25	0.25	0.17	0.17		0.12	0.12	0.12
13-06	1.37	1.37		0.25	0.24	0.25	0.18	0.18	0.17	0.12	0.13	0.12
13-07	1.32	1.37	1.35	0.24	0.24	0.24	0.17	0.17	0.17	0.12	0.12	0.11

E.3.2 Modal damping ratio

Figure E.7 plots the modal damping ratios in the ground motion cases of the 20EDF test estimated using the ARX, FDD and SS methods, respectively. Table E.6 summarizes the corresponding values in each segment. Note that damping ratio values larger than 0,1 are not plotted.

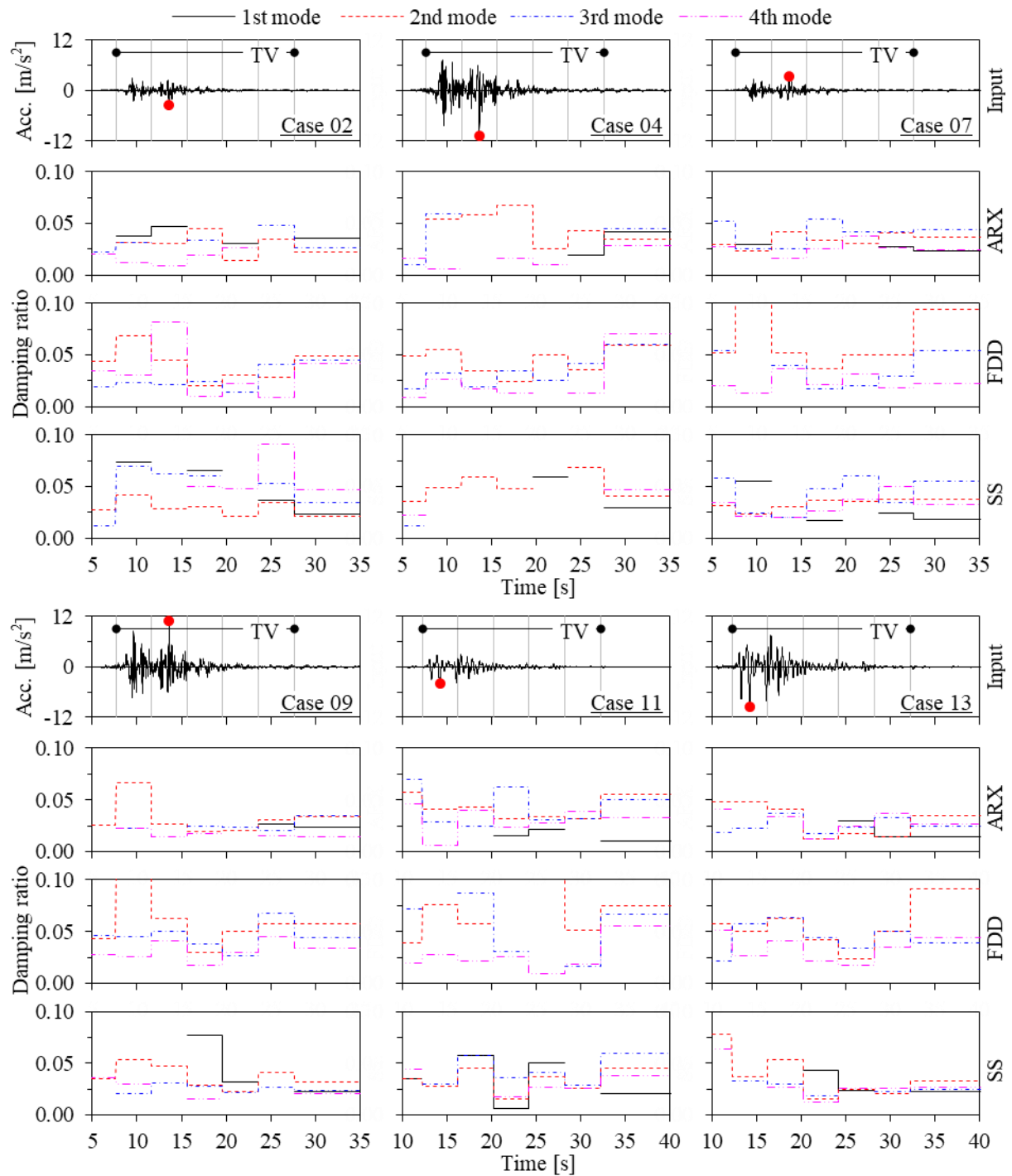


Figure E.7 Damping ratio in ground motion excitations of the 20EDF test

Table E.6 Damping ratio in ground motion excitations of the 20EDF test

Case-Segment	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01					0.03	0.04	0.02	0.02	0.01	0.02	0.03	
02-02	0.04		0.07	0.03	0.04	0.07	0.03	0.02	0.07	0.01	0.03	
02-03	0.05			0.03	0.03	0.04		0.02	0.06	0.01	0.08	
02-04			0.07	0.04	0.03	0.02	0.03	0.02	0.06	0.02	0.01	0.05
02-05	0.03			0.01	0.02	0.03		0.01		0.03	0.02	0.05
02-06			0.04	0.03	0.03	0.03	0.05	0.04	0.05		0.01	0.09
02-07	0.04		0.02	0.02	0.02	0.05	0.03	0.04	0.03		0.04	0.05
04-01					0.04	0.05	0.01	0.02	0.01	0.02	0.01	0.02
04-02				0.05	0.05	0.05	0.06	0.03		0.01	0.03	
04-03				0.06	0.06	0.03		0.02			0.02	
04-04				0.07	0.05	0.02		0.03		0.02	0.01	
04-05			0.06	0.03		0.05		0.03		0.01		
04-06	0.02			0.04	0.07	0.04		0.04			0.01	
04-07	0.04		0.03	0.03	0.04	0.06	0.04	0.06		0.03	0.07	0.05
07-01				0.03	0.03	0.05	0.05	0.05	0.06	0.03	0.02	0.03
07-02	0.03		0.06	0.02	0.02	0.12	0.03		0.02		0.01	0.02
07-03				0.04	0.03	0.05	0.03	0.04	0.02	0.02	0.04	0.02
07-04			0.02	0.03	0.04	0.04	0.05	0.02	0.05	0.03	0.02	0.03
07-05				0.03	0.04	0.05	0.04	0.02	0.06	0.04	0.03	0.04
07-06	0.03		0.02	0.04	0.04	0.05	0.04	0.03	0.03	0.03	0.02	0.05
07-07	0.02		0.02	0.04	0.04	0.09	0.04	0.05	0.06	0.02	0.02	0.03
09-01				0.03	0.03	0.04		0.05			0.03	0.04
09-02				0.07	0.05	0.12	0.02	0.05	0.02	0.02	0.03	0.03
09-03				0.03	0.05	0.06		0.05	0.03	0.01	0.04	
09-04			0.08	0.02	0.03	0.03	0.02	0.04	0.03	0.02	0.02	0.02
09-05			0.03	0.02	0.02	0.05	0.02	0.03	0.02		0.03	
09-06	0.03			0.03	0.04	0.06	0.02	0.07	0.03	0.02	0.04	
09-07	0.02		0.02	0.03	0.03	0.06	0.04	0.04	0.02	0.01	0.03	0.02
11-01			0.04	0.06	0.03	0.04	0.07	0.07		0.05	0.02	0.04
11-02				0.04	0.03	0.08	0.03		0.03	0.01	0.03	
11-03			0.06	0.04	0.05	0.06	0.02	0.09	0.06	0.04	0.02	
11-04	0.02		0.01	0.03	0.02		0.06	0.03	0.04	0.02	0.03	0.02
11-05	0.02		0.05	0.03	0.04	0.19	0.03		0.04	0.03	0.01	0.03
11-06				0.03	0.03	0.05	0.03	0.02	0.03	0.04	0.02	0.03
11-07	0.01		0.02	0.06	0.05	0.08	0.05	0.07	0.06	0.03	0.06	0.04
13-01				0.05	0.08	0.06	0.02	0.02		0.04	0.05	0.06
13-02				0.05	0.04	0.05	0.02	0.06	0.03		0.03	
13-03				0.04	0.05	0.06	0.04	0.06	0.03	0.03	0.04	0.03
13-04			0.04	0.01	0.02	0.04	0.02	0.04	0.02	0.01	0.02	0.01
13-05	0.03		0.02	0.02	0.02	0.02	0.02	0.03		0.02	0.02	0.03
13-06	0.01			0.01	0.02	0.05	0.03	0.05	0.02	0.04	0.04	0.03
13-07	0.01		0.02	0.03	0.03	0.09	0.02	0.04	0.02	0.03	0.04	0.03

E.3.3 Mode shapes

Figure E.8 to Figure E.10 plot the mode shapes in the ground motion cases of the 20EDF test estimated using the ARX, FDD and SS methods, respectively.

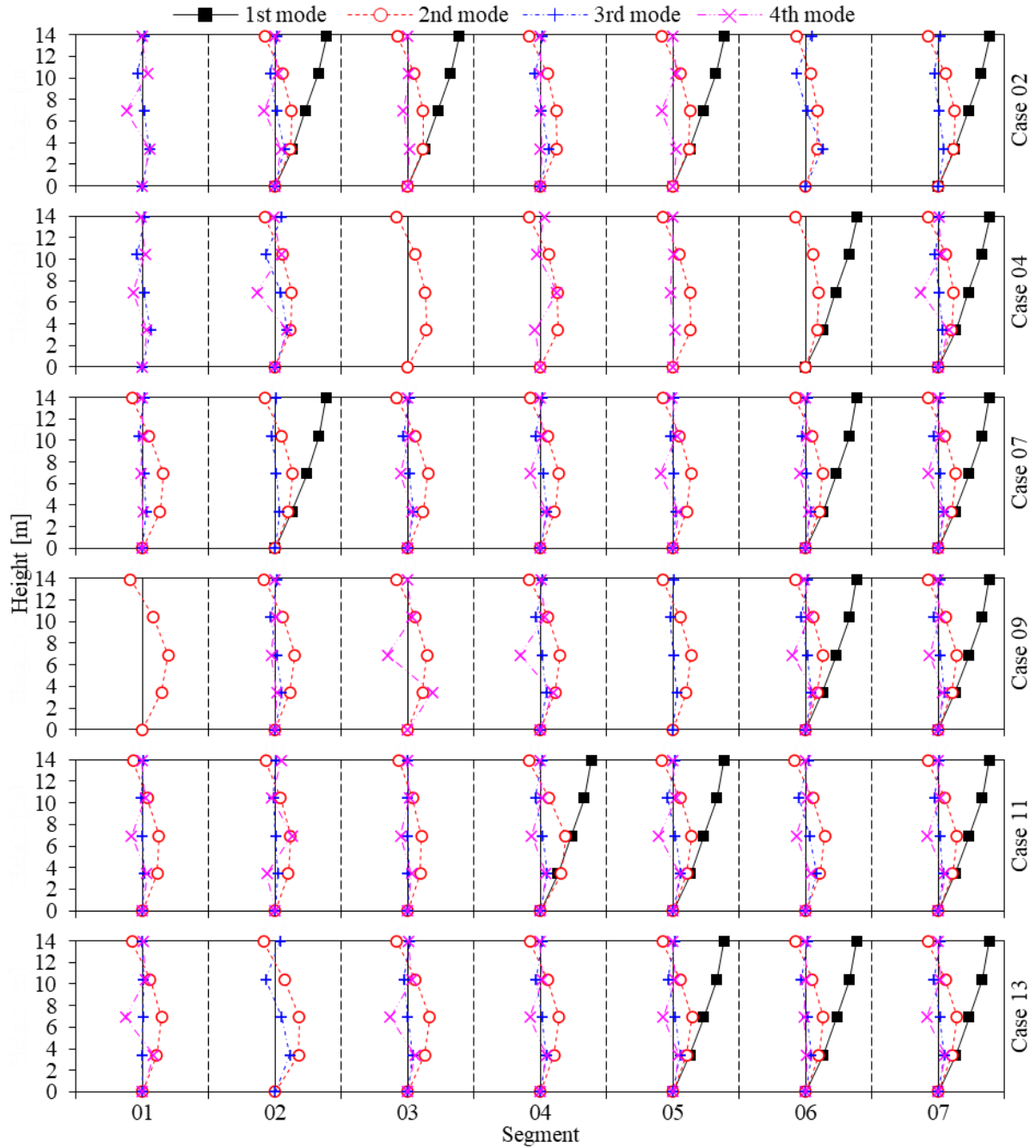


Figure E.8 Mode shapes in ground motion excitations of the 20EDF test (ARX)

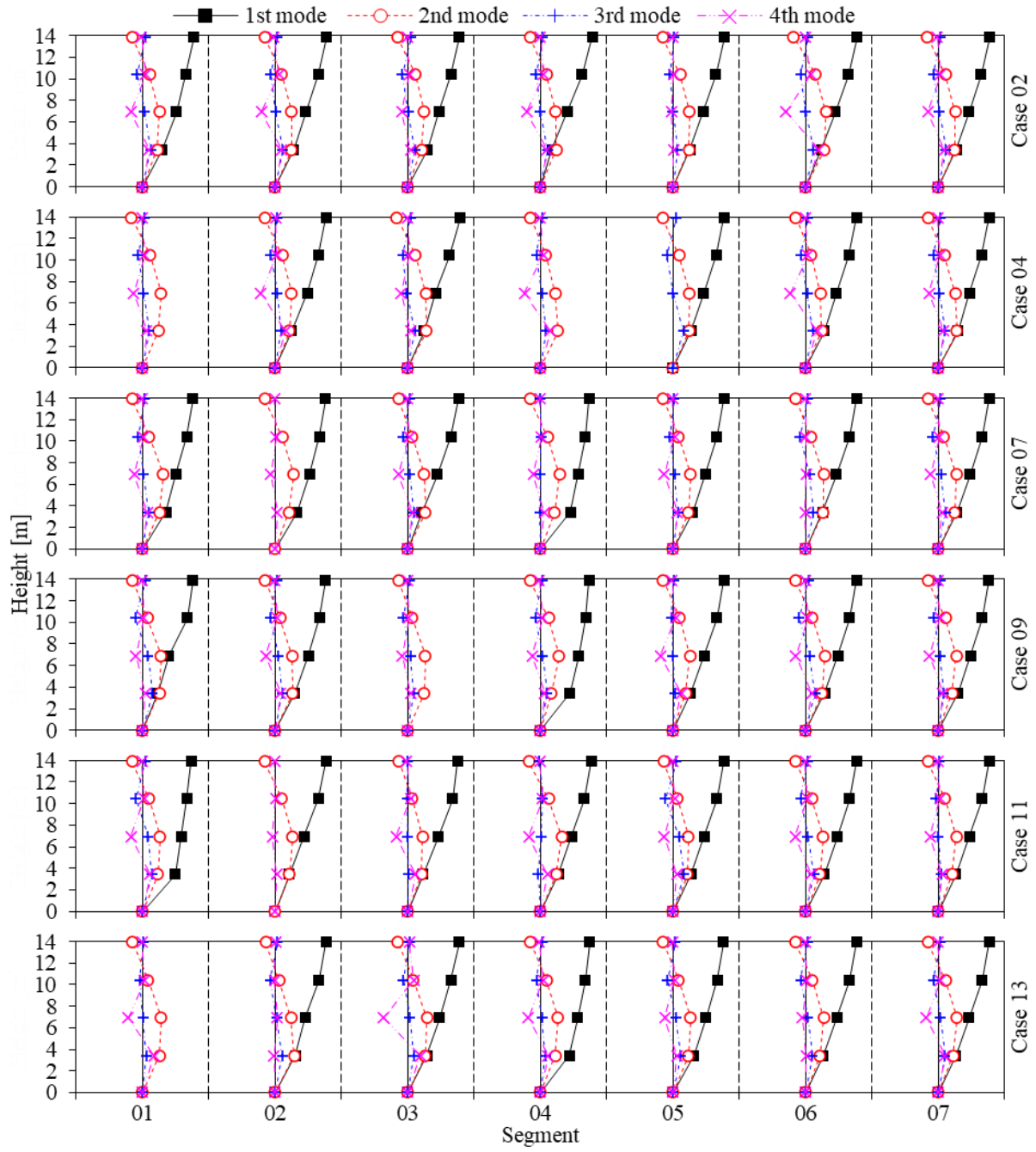


Figure E.9 Mode shapes in ground motion excitations of the 20EDF test (FDD)

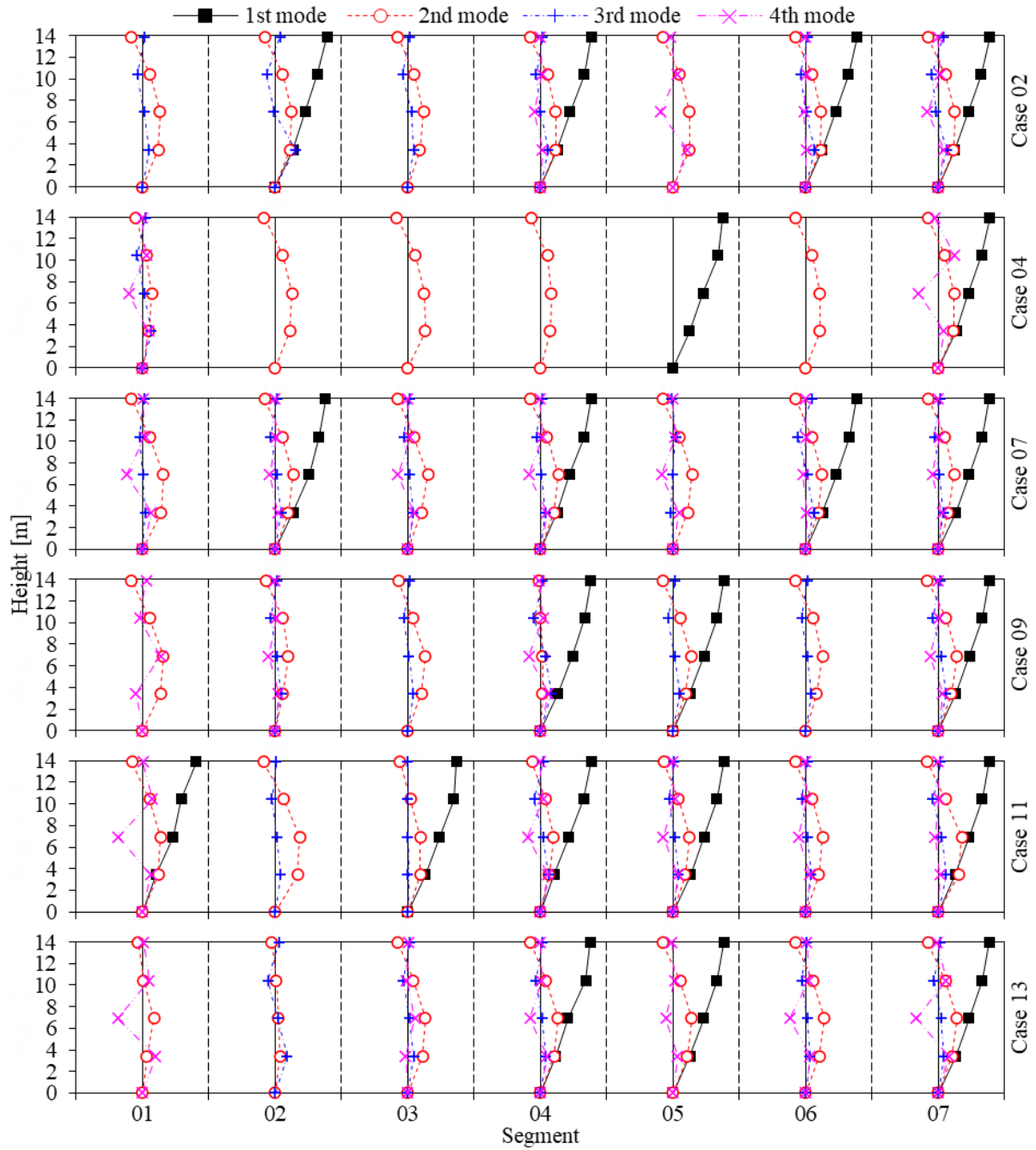


Figure E.10 Mode shapes in ground motion excitations of the 20EDF test (SS)

E.3.4 Modal Effective Mass (MEM)

Figure E.11 plots the natural period in the ground motion cases of the 20EDF test estimated using the ARX, FDD and SS methods, respectively. Table E.7 summarizes the corresponding values in each segment.

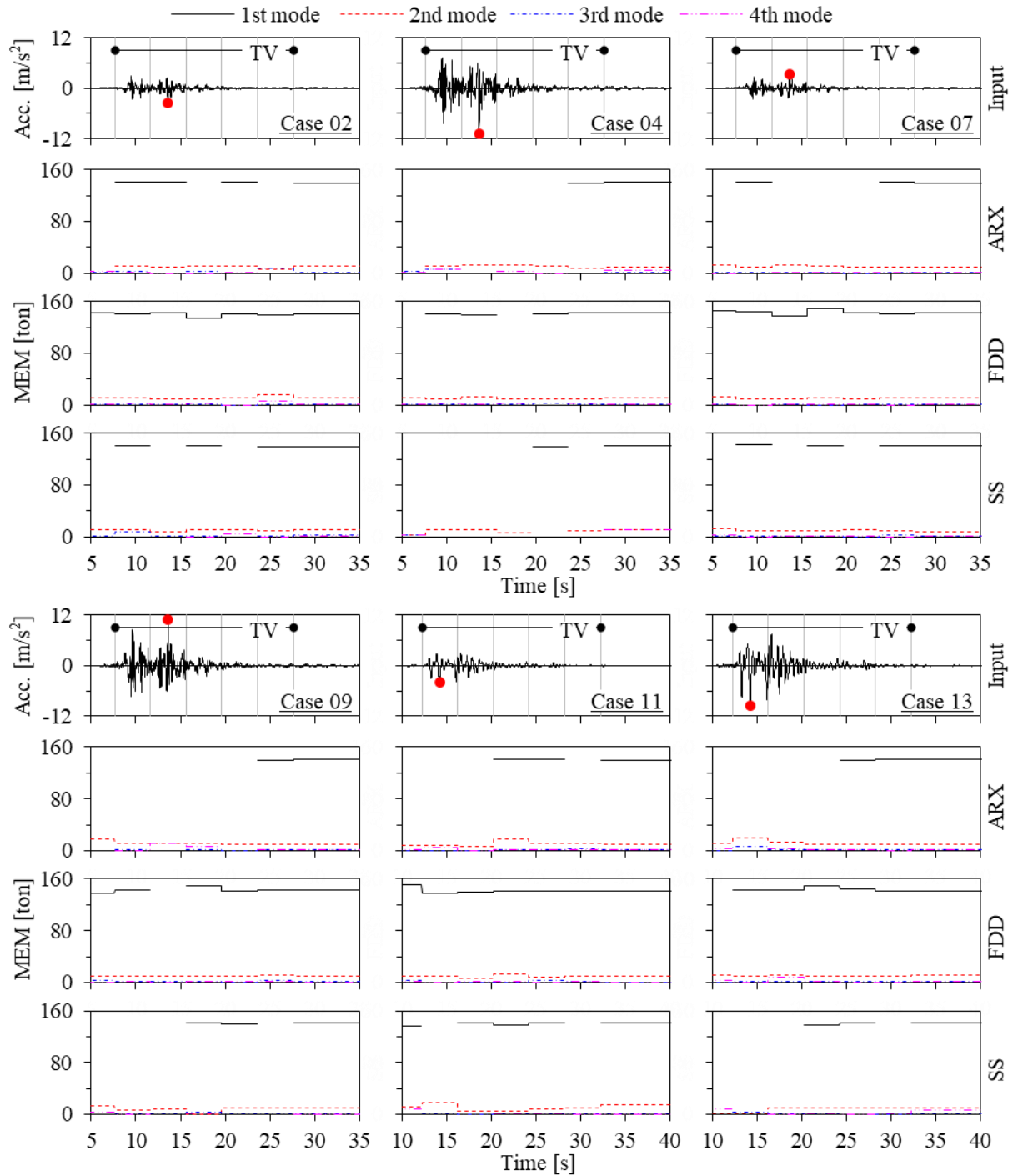


Figure E.11 MEMs in ground motion excitations of the 20EDF test

Table E.7 MEMs in ground motion excitations of the 20EDF test [Unit: ton]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01		142.9			10.5	11.2	1.6	1.9	1.4	3.1	1.8	
02-02	140.2	141.2	140.5	10.4	10.5	10.4	2.1	1.5	8.2	1.5	2.3	
02-03	140.1	142.1		9.7	10.1	8.3		1.8	1.4	0.2	0.4	
02-04		134.3	140.0	10.8	9.9	10.3	2.0	1.4	1.5	0.0	2.1	0.4
02-05	140.2	139.8		10.8	10.7	10.7		0.6		1.4	0.0	4.6
02-06		139.0	139.8	6.0	15.2	9.5	7.5	1.7	1.9		5.9	0.0
02-07	139.7	140.9	139.7	10.6	11.0	10.4	1.0	1.7	2.8		1.4	1.4
04-01					11.6	3.2	2.0	1.3	2.1	1.0	0.9	2.2
04-02		140.2		10.3	9.9	10.6	5.8	1.3		5.5	3.0	
04-03		138.4		13.0	12.8	11.8		1.6			0.4	
04-04				12.7	10.1	6.8		1.0		3.5	3.4	
04-05		141.1	140.4	10.9	10.0			2.9		0.1		
04-06	140.6	141.7		7.8	9.4	9.2		1.6			3.6	
04-07	140.6	142.6	140.9	9.5	11.7	10.2	0.6	1.3		4.0	1.0	11.0
07-01		145.2		12.7	12.5	13.2	0.6	1.3	0.4	0.0	0.7	3.1
07-02	140.7	144.5	141.9	9.5	10.0	9.9	0.7		1.2		0.2	0.3
07-03		137.7		12.2	9.8	10.0	1.1	1.2	1.0	0.7	1.0	1.3
07-04		149.6	140.1	10.6	10.9	10.0	1.5	0.0	0.9	1.2	0.6	1.7
07-05		142.8		10.0	9.8	10.7	0.3	0.8	0.3	1.9	0.9	1.5
07-06	140.8	140.4	140.6	9.6	10.7	8.8	0.9	1.9	3.6	0.4	0.0	0.1
07-07	140.7	141.8	140.7	9.6	10.7	7.6	1.0	1.7	0.8	1.3	0.6	0.4
09-01		138.3		18.6	10.5	13.2		3.4			0.4	4.0
09-02		143.1		11.5	10.3	6.8	1.4	1.6	1.2	0.1	1.2	0.5
09-03				11.2	9.5	8.8		1.2	1.0	11.6	0.3	
09-04		149.5	141.4	11.6	10.1	0.3	1.3	1.4	3.3	5.8	0.9	2.0
09-05		141.1	140.6	10.0	9.0	9.8	0.5	0.1	1.4		2.2	
09-06	140.6	142.5		9.3	11.5	9.2	1.1	2.8	1.0	2.4	1.3	
09-07	140.7	142.9	140.9	10.3	10.4	10.4	1.3	1.0	1.9	0.9	0.8	0.8
11-01		150.5	135.8	8.7	10.2	11.2	0.1	2.8		1.3	1.8	7.9
11-02		137.5		7.9	9.7	18.0	0.2		0.9	4.7	0.1	
11-03		139.4	141.7	6.9	7.4	5.6	0.0	0.0	0.0	0.6	2.0	
11-04	140.8	141.4	137.2	17.7	13.2	5.5	1.4	0.3	2.3	1.0	1.7	2.1
11-05	140.7	141.1	140.8	11.1	8.1	8.0	1.9	4.0	1.0	2.4	0.8	1.1
11-06		141.7		11.0	9.6	9.2	3.2	1.7	1.0	1.0	1.2	0.6
11-07	140.7	141.2	140.6	10.2	10.0	15.6	0.9	0.3	2.0	1.5	0.8	0.1
13-01				10.9	10.9	2.2	0.1	0.5		4.1	3.3	8.0
13-02		142.5		19.0	10.2	0.9	6.4	1.7	4.0		0.1	
13-03		142.2		13.4	11.5	9.2	0.9	1.3	1.2	3.5	7.4	0.6
13-04		148.8	138.6	10.4	9.8	9.1	1.2	0.9	1.1	1.3	2.0	1.4
13-05	140.3	143.7	140.4	10.7	9.7	10.4	1.6	2.0		1.2	0.7	0.7
13-06	140.7	141.0		9.2	10.1	10.5	1.1	1.5	0.7	0.0	0.1	2.4
13-07	140.8	140.8	140.8	10.5	10.8	10.4	1.3	1.5	1.1	1.7	1.8	6.7

E.3.5 PSD peaks

Figure E.12 plots the peak values of the Power Spectra Density (PSD) in the white noise cases of the 20EDF test identified using the FDD method. Table E.8 summarizes the corresponding values.

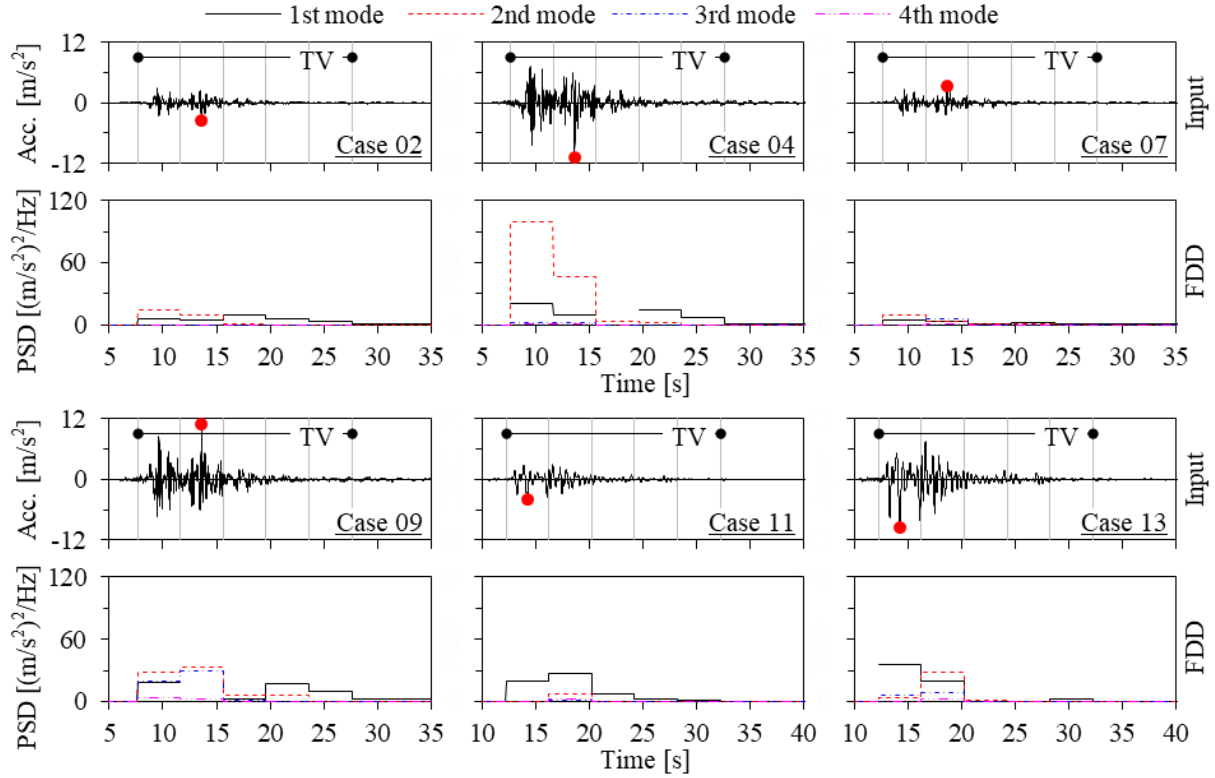


Figure E.12 PSD peaks in ground motion excitations of the 20EDF test

Table E.8 PSD peaks in ground motion excitations of the 20EDF test [Unit: $(\text{m/s}^2)^2/\text{Hz}$]

Case-Segment	Mode			
	1st	2nd	3rd	4th
C02-S01	0.0	0.0	0.0	0.0
C02-S02	5.5	14.5	0.1	0.1
C02-S03	4.8	9.6	0.1	0.1
C02-S04	9.7	1.0	0.1	0.0
C02-S05	5.9	0.2	0.0	0.0
C02-S06	3.4	0.0	0.0	0.0
C02-S07	0.6	0.0	0.0	0.0
C04-S01	0.0	0.0	0.0	0.0
C04-S02	20.8	99.8	2.5	1.3
C04-S03	9.2	46.2	1.7	1.4
C04-S04	0.0	3.4	0.2	0.1
C04-S05	14.1	1.8	0.2	0.0
C04-S06	7.4	0.2	0.1	0.0
C04-S07	1.3	0.0	0.0	0.0
C07-S01	0.0	0.0	0.0	0.0
C07-S02	4.5	9.3	0.0	0.1
C07-S03	3.2	3.8	5.6	0.3

C07-S04	1.0	1.3	0.0	0.0
C07-S05	1.6	0.4	0.0	0.0
C07-S06	1.4	0.0	0.0	0.0
C07-S07	0.5	0.0	0.0	0.0
C09-S01	0.0	0.0	0.0	0.0
C09-S02	19.0	28.1	19.3	3.2
C09-S03	0.0	32.9	30.1	2.3
C09-S04	2.3	5.6	1.2	0.2
C09-S05	17.8	6.2	0.4	0.1
C09-S06	9.4	0.5	0.1	0.0
C09-S07	2.0	0.0	0.0	0.0
C11-S01	0.0	0.0	0.0	0.0
C11-S02	19.4	0.2	0.0	0.0
C11-S03	26.4	7.9	1.6	2.9
C11-S04	7.1	0.0	0.0	0.0
C11-S05	2.5	0.0	0.0	0.0
C11-S06	0.8	0.0	0.0	0.0
C11-S07	0.1	0.0	0.0	0.0
C13-S01	0.0	0.0	0.0	0.0
C13-S02	36.1	3.4	5.8	0.5
C13-S03	19.9	28.5	8.8	2.2
C13-S04	0.3	0.8	0.4	0.0
C13-S05	0.5	0.5	0.0	0.0
C13-S06	2.7	0.1	0.0	0.0
C13-S07	0.2	0.0	0.0	0.0

E.4 EFFECTIVE MODAL PSEUDO-ACCELERATION (EMPA)

Figure E.13 plots the peak EMPA values identified in the ground motion cases of the 20EDF test, including the elastic pseudo-acceleration spectra with 2% damping of the measured excitations.

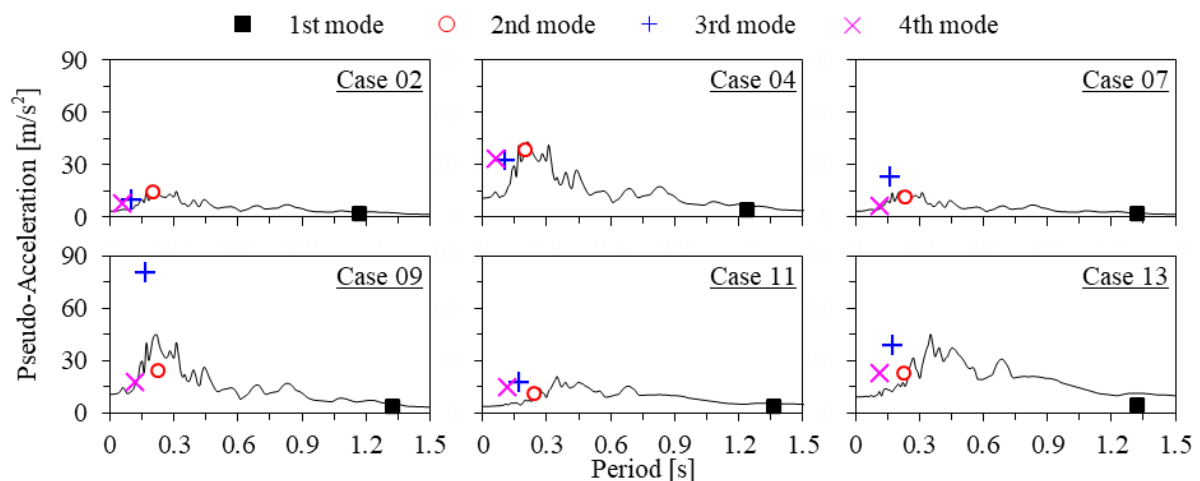


Figure E.13 Peak EMPA values in ground motion excitations of the 20EDF test

E.5 TIME-FREQUENCY RESULTS FROM GROUND MOTION EXCITATIONS

E.5.1 Magnitude of wavelet coefficients and instantaneous periods

Figure E.14 plots the magnitude of wavelet coefficients on the time-frequency plane and the identified instantaneous periods in the ground motion cases of the 20EDF test.

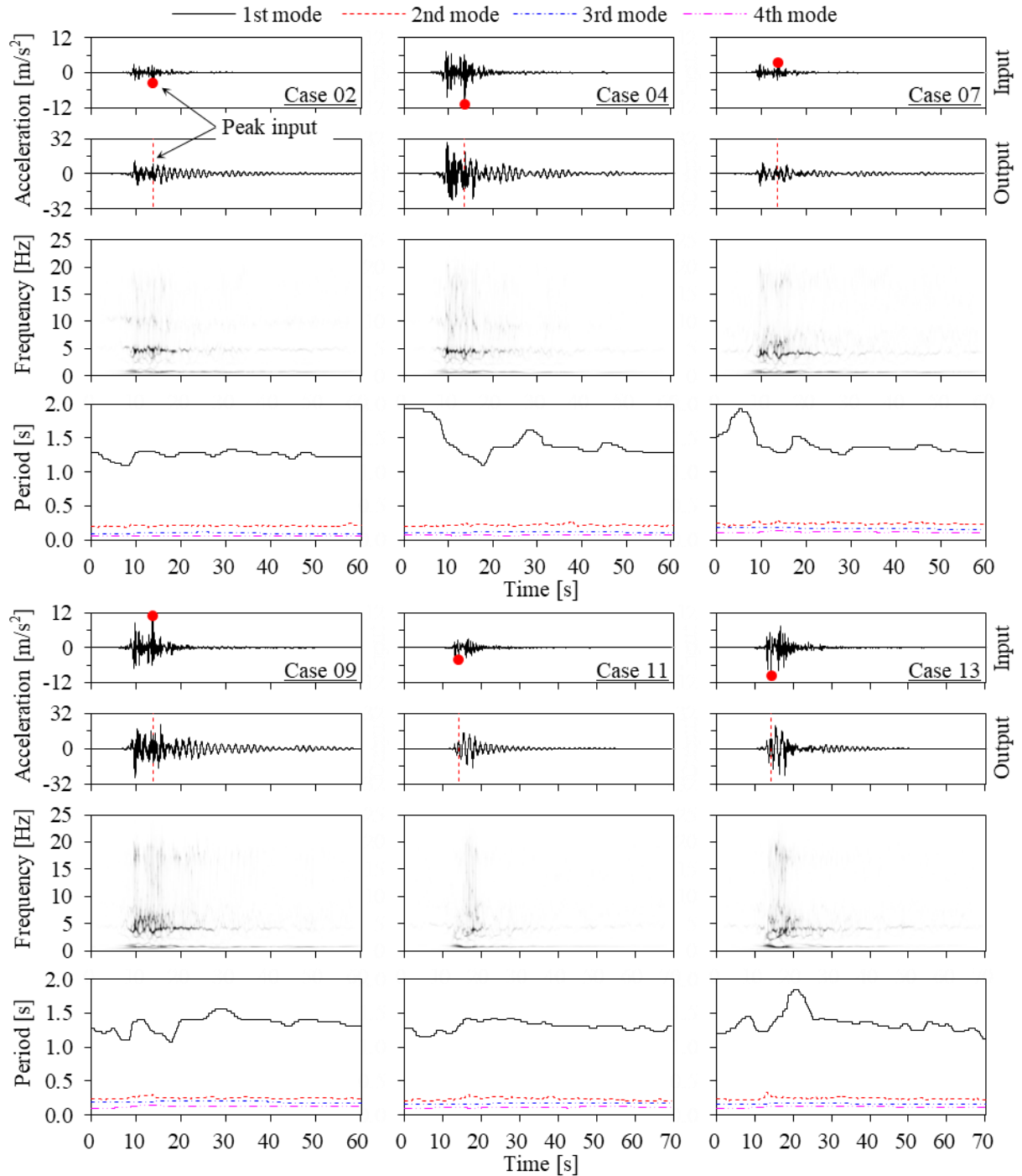


Figure E.14 Magnitude of wavelet coefficients in ground motion excitations of the 20EDF test

E.5.2 Magnitude of ridge wavelet coefficient

Figure E.15 plots the magnitude of wavelet coefficient on the wavelet ridges in the ground motion cases of the 20EDF test.

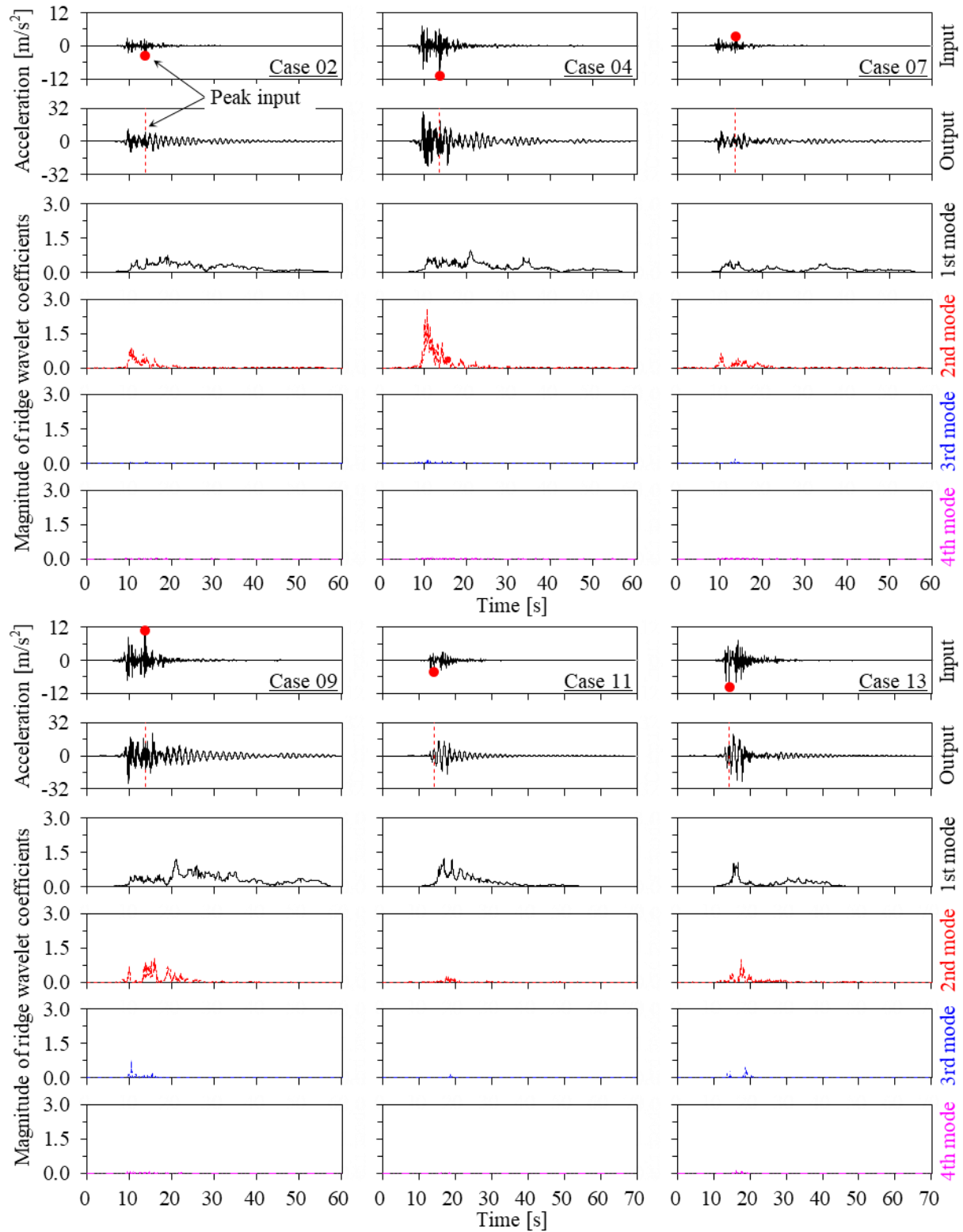


Figure E.15 Magnitude on wavelet ridges in ground motion excitations of the 20EDF test

E.5.3 Reconstructed modal components

Figure E.16 plots the time-domain modal components reconstructed from the SSWT results and ridge identification in the ground motion cases of the 20EDF test.

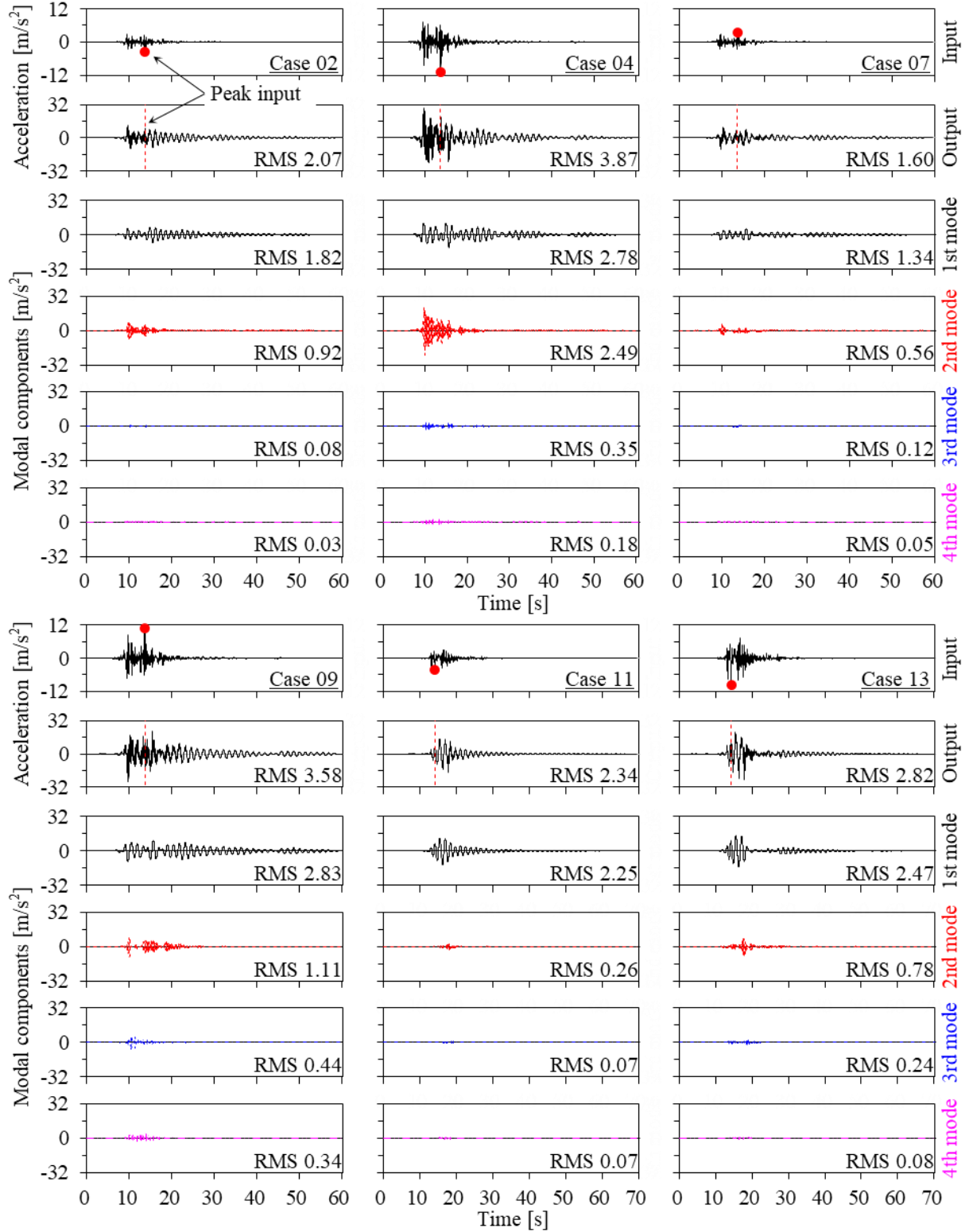


Figure E.16 Modal components in ground motion excitations of the 20EDF test

APPENDIX F. FULL RESULTS OF VIBRATION PROPERTY ANALYSIS: 21TKB

F.1 GENERAL

This appendix provides results of the vibration property analysis on the 21TKB test. The results are grouped by different tests, each including results from all analysis methods. The following results are provided in this appendix:

- 1) **Modal parameters:** natural periods, modal damping ratios, mode shapes, Modal Effective Mass (MEM) and PSD peaks from modal analysis methods.
- 2) **Spectral results:** peak values of Effective Modal Pseudo-Acceleration (EMPA).
- 3) **Time-frequency results:** magnitude of wavelet coefficients, instantaneous periods, magnitude of ridge wavelet coefficient and reconstructed modal components.

In the 21TKB test, 5 white noise and 7 ground motion cases in the East-West direction were selected for analysis. The structural system was not changed during the test.

F.2 MODAL PARAMETERS FROM WHITE NOISE EXCITATIONS

F.2.1 Natural period

Figure F.1 plots the natural period in the white noise cases of the 21TKB test identified using the ARX, FDD and SS methods, respectively. Table F.1 summarizes the corresponding values.

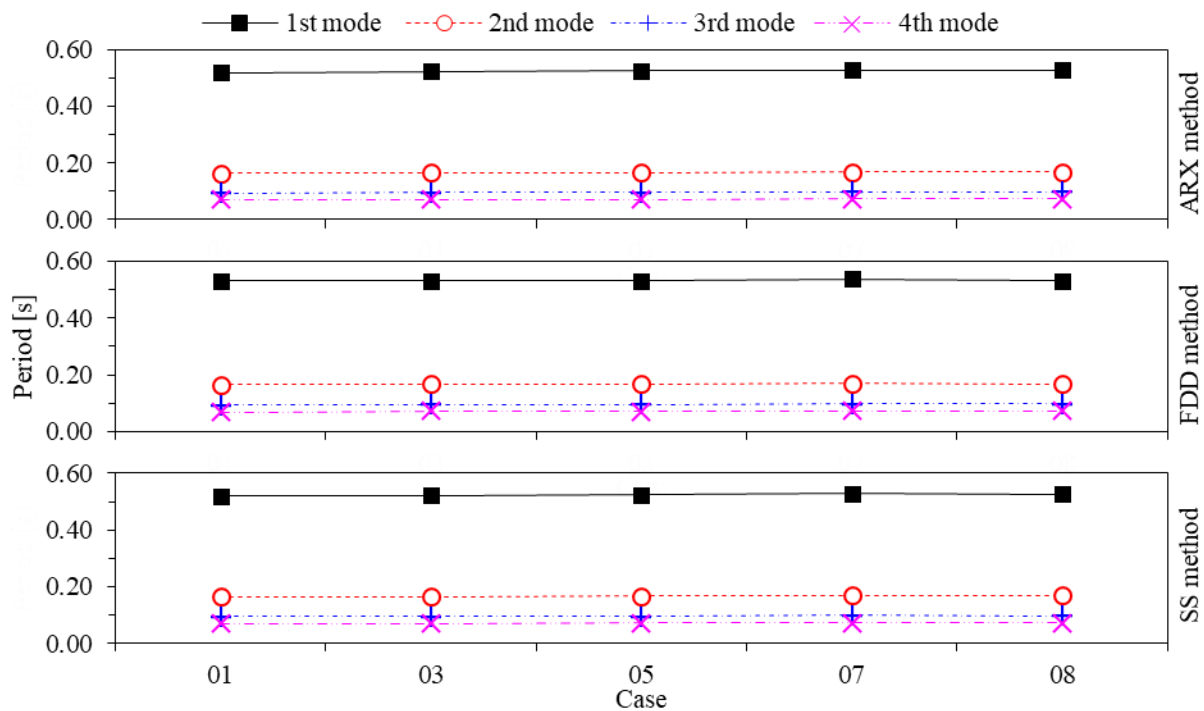


Figure F.1 Natural periods in white noise excitations of the 21TKB test

Table F.1 Natural periods in white noise excitations of the 21TKB test [Unit: s]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.52	0.53	0.52	0.16	0.17	0.16	0.09	0.09	0.09	0.07	0.07	0.07
03	0.52	0.53	0.52	0.17	0.17	0.16	0.10	0.10	0.09	0.07	0.07	0.07
05	0.52	0.53	0.52	0.17	0.17	0.17	0.10	0.10	0.10	0.07	0.07	0.07
07	0.53	0.54	0.53	0.17	0.17	0.17	0.10	0.10	0.10	0.07	0.07	0.07
08	0.53	0.53	0.53	0.17	0.17	0.17	0.10	0.10	0.10	0.07	0.07	0.07

F.2.2 Modal damping ratio

Figure F.2 plots the modal damping ratios in the white noise cases of the 21TKB test identified using the ARX, FDD and SS methods, respectively. Table F.2 summarizes the corresponding values.

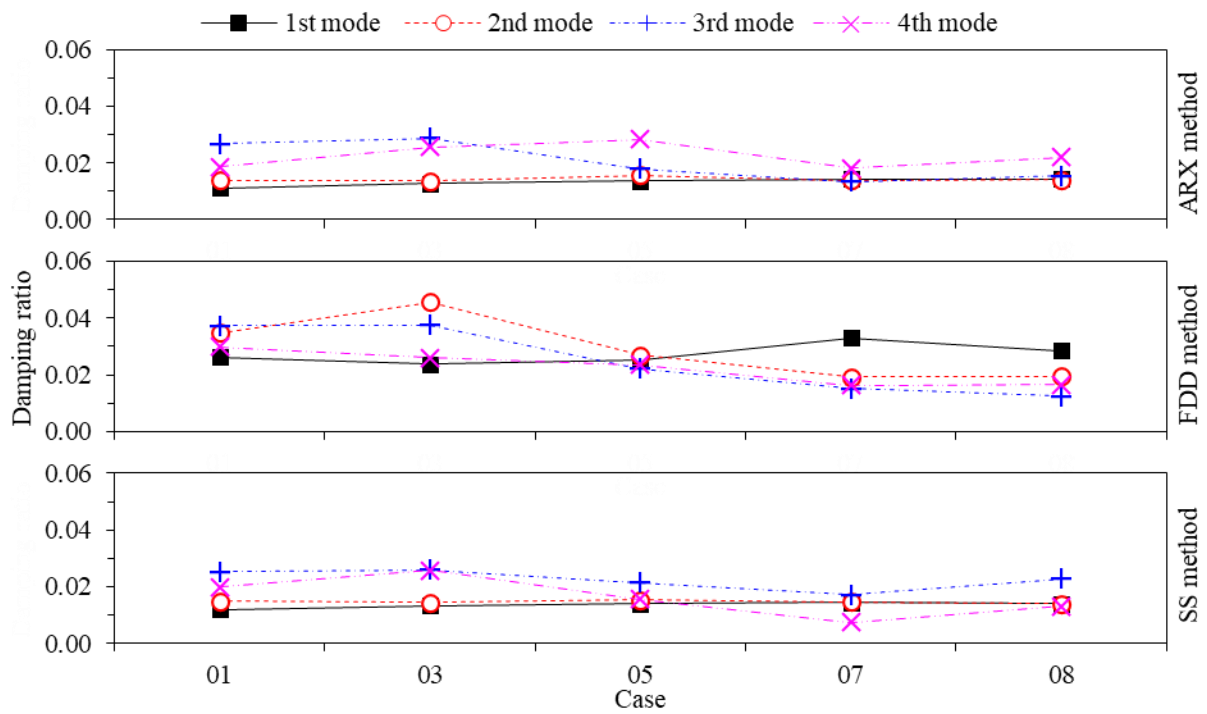


Figure F.2 Damping ratios in white noise excitations of the 21TKB test

Table F.2 Damping ratios in white noise excitations of the 21TKB test

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.01	0.03	0.01	0.01	0.03	0.01	0.03	0.04	0.03	0.02	0.03	0.02
03	0.01	0.02	0.01	0.01	0.05	0.01	0.03	0.04	0.03	0.03	0.03	0.03
05	0.01	0.03	0.01	0.02	0.03	0.02	0.02	0.02	0.02	0.03	0.02	0.02
07	0.01	0.03	0.01	0.01	0.02	0.01	0.01	0.02	0.02	0.02	0.02	0.01
08	0.01	0.03	0.01	0.01	0.02	0.01	0.02	0.01	0.02	0.02	0.02	0.01

F.2.3 Mode shapes

Figure F.3 plots the mode shapes in the white noise cases of the 21TKB test identified using the ARX, FDD and SS methods, respectively. The plotted mode shapes are normalized by the modal participation factor Γ_n .

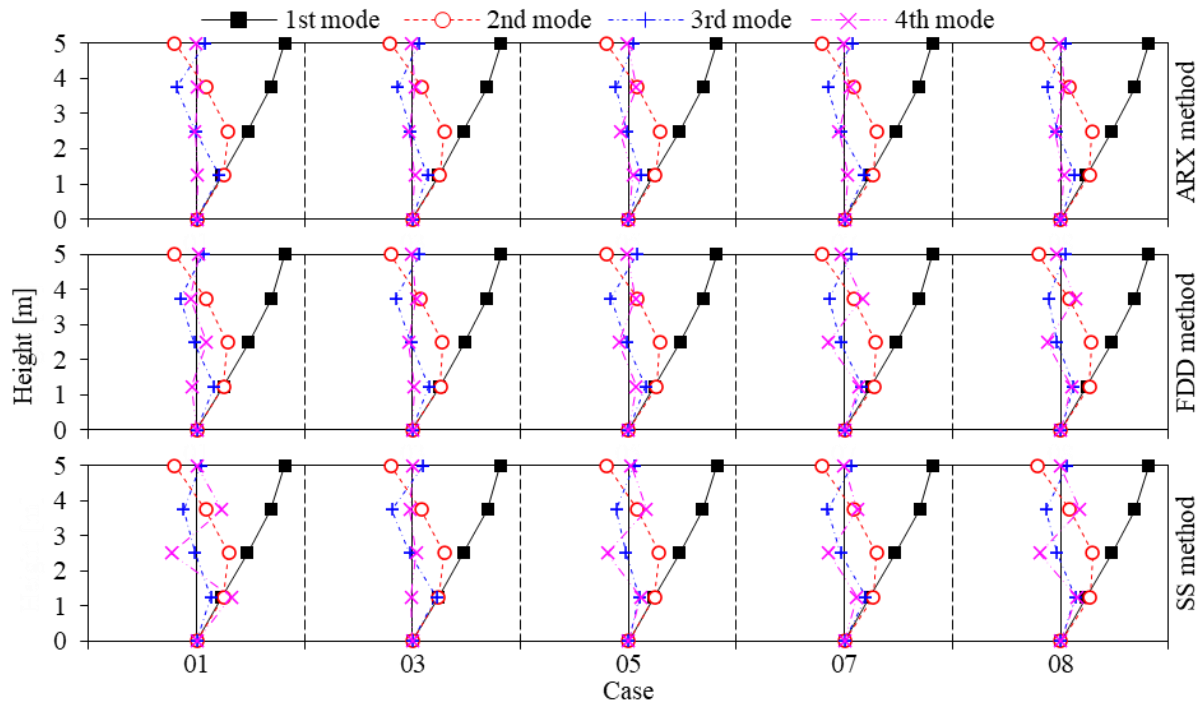


Figure F.3 Mode shapes in white noise excitations of the 21TKB test

F.2.4 Modal Effective Mass (MEM)

Figure F.4 plots the Modal Effective Mass (MEM) in the white noise cases of the 21TKB test identified using the ARX, FDD and SS methods, respectively. Table F.3 summarizes the corresponding values of the MEMs. The total mass of the structure is 50.8 tons, with Floor 1 to 4 all being 12.7 tons.

Table F.3 MEMs in white noise excitations of the 21TKB test [Unit: ton]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	43.7	44.3	43.7	11.4	11.2	11.4	1.2	0.6	0.4	0.1	0.1	6.1
03	43.8	44.2	43.9	11.4	10.9	11.1	0.4	0.6	1.4	0.2	0.3	0.0
05	43.7	44.1	43.8	11.4	11.4	11.0	0.2	0.6	0.1	0.6	0.7	1.9
07	43.8	44.3	43.8	11.7	11.7	11.7	0.8	0.6	1.0	0.5	2.6	1.5
08	43.9	44.1	43.9	11.6	11.4	11.8	0.3	0.2	0.4	0.4	2.0	2.7

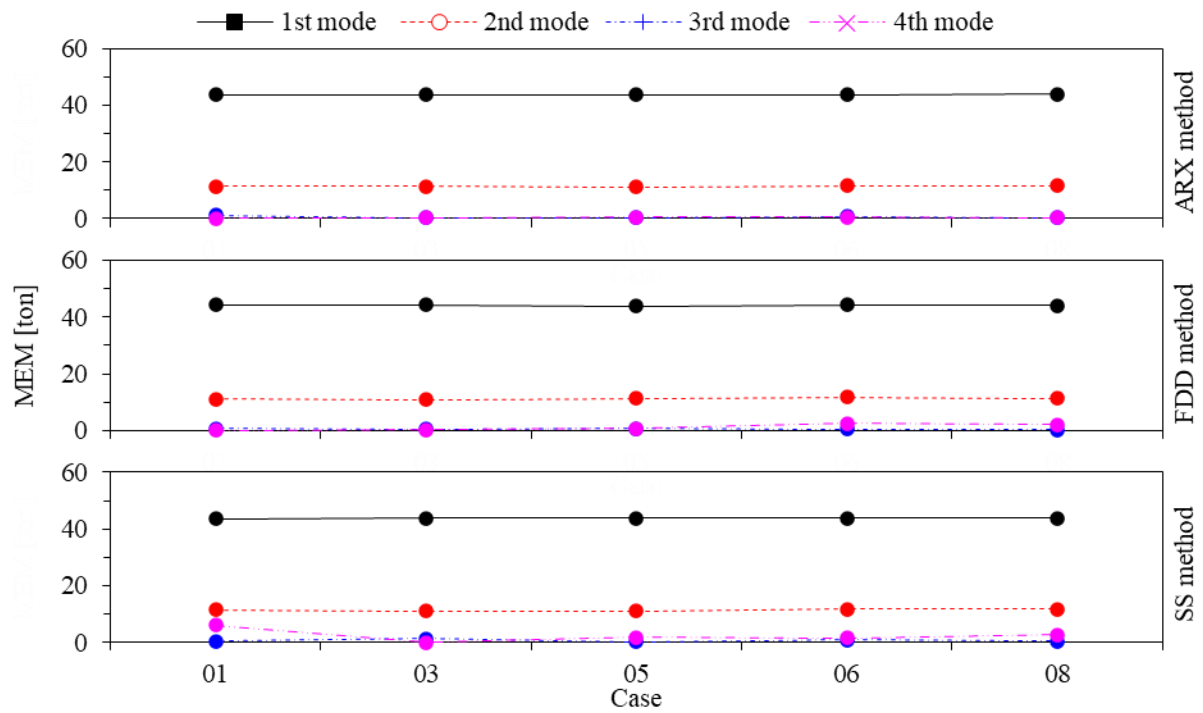


Figure F.4 MEMs in white noise excitations of the 21TKB test

F.2.5 PSD peaks

Figure F.5 plots the peak values of the Power Spectra Density (PSD) in the white noise cases identified using the FDD method. Table F.4 summarizes the corresponding values.

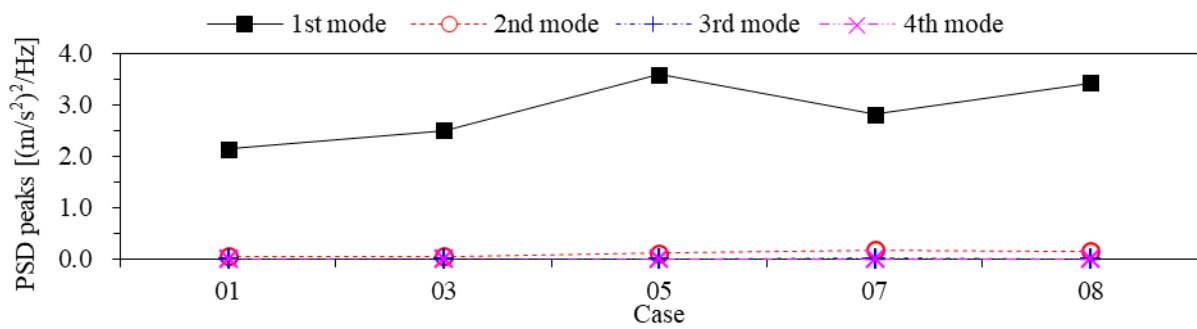


Figure F.5 PSD peaks in white noise excitations of the 21TKB test

 Table F.4 PSD peaks in white noise excitations of the 21TKB test [Unit: (m/s²)²/Hz]

Case	Mode			
	1st	2nd	3rd	4th
01	2.139	0.056	0.003	0.001
03	2.509	0.053	0.003	0.001
05	3.591	0.112	0.005	0.002
07	2.820	0.174	0.007	0.002
08	3.433	0.155	0.006	0.001

F.3 MODAL PARAMETERS FROM GROUND MOTION EXCITATIONS

F.3.1 Natural period

Figure F.6 plots the natural period in the ground motion cases of the 21TKB test estimated using the ARX, FDD and SS methods, respectively. Table F.5 summarizes the corresponding values in each segment.

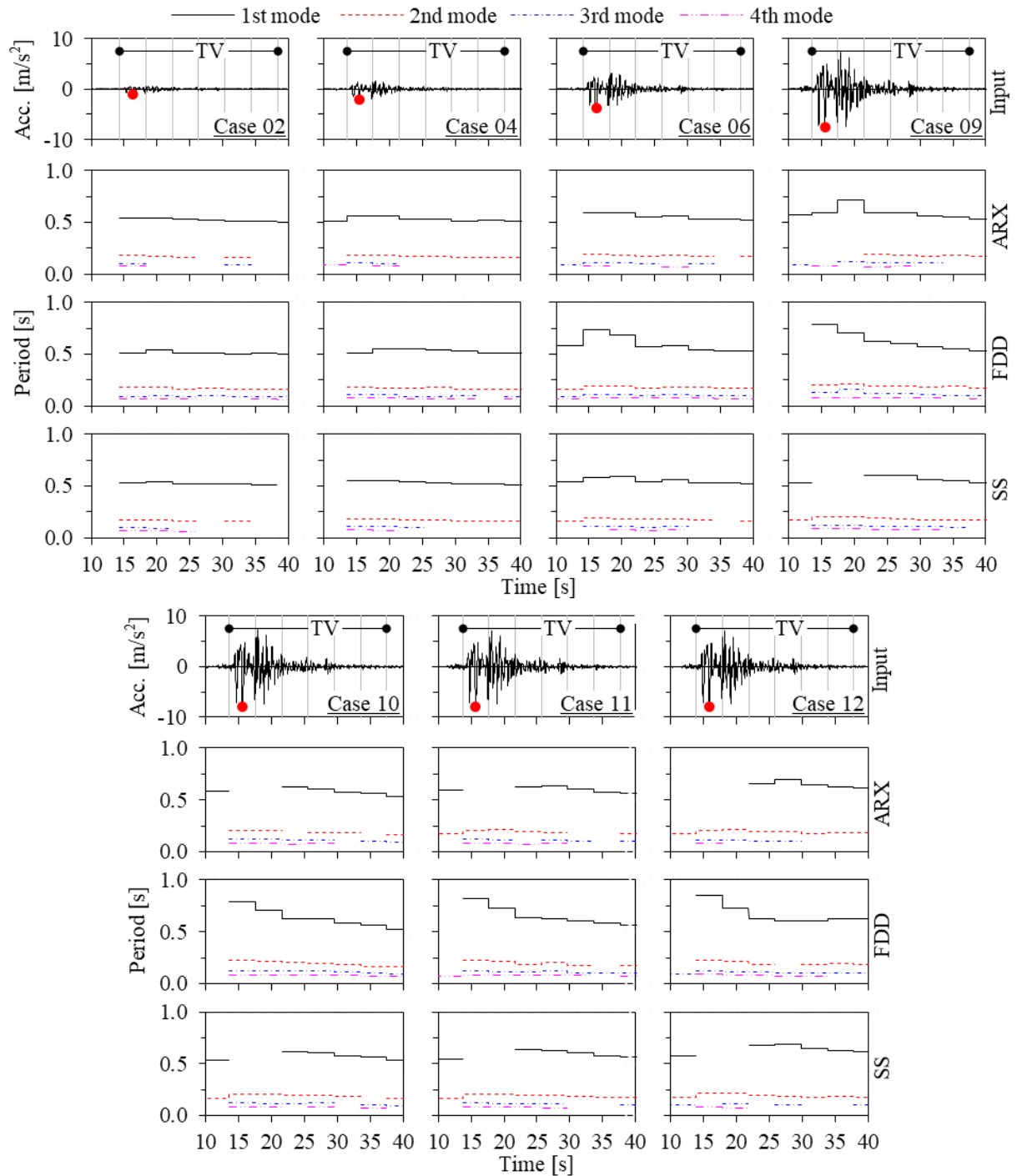


Figure F.6 Natural periods in ground motion excitations of the 21TKB test

Table F.5 Natural periods in ground motion excitations of the 21TKB test [Unit: s]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01												
02-02	0.54	0.51	0.53	0.18	0.18	0.17	0.10	0.09	0.10	0.07	0.07	0.07
02-03	0.54	0.54	0.54	0.17	0.18	0.17		0.10	0.10		0.07	0.07
02-04	0.53	0.51	0.52	0.16	0.17	0.16		0.09			0.07	0.07
02-05	0.52	0.51	0.52		0.17			0.10				
02-06	0.51	0.50	0.52	0.16	0.16	0.16	0.09	0.09				
02-07	0.51	0.51	0.51		0.16			0.09			0.07	
02-08	0.50	0.50			0.16			0.09			0.07	
04-01	0.51									0.09		
04-02	0.56	0.51	0.55	0.18	0.18	0.18	0.11	0.11	0.11		0.08	0.08
04-03	0.56	0.55	0.56	0.18	0.18	0.18	0.11	0.11	0.11	0.08	0.08	0.08
04-04	0.53	0.55	0.54	0.17	0.17	0.17		0.09	0.10		0.07	
04-05	0.53	0.54	0.53	0.17	0.18	0.17		0.09			0.07	
04-06	0.51	0.53	0.52	0.16	0.16	0.16		0.10			0.07	
04-07	0.52	0.51	0.52	0.16	0.16	0.16						
04-08	0.51	0.51	0.51	0.16	0.16	0.16		0.09			0.07	
06-01		0.59	0.55		0.16	0.17	0.09	0.09			0.07	
06-02	0.59	0.73	0.58	0.20	0.20	0.19	0.11	0.11	0.11	0.08	0.08	
06-03	0.59	0.68	0.59	0.19	0.19	0.19	0.11	0.11	0.11		0.08	0.08
06-04	0.55	0.57	0.54	0.17	0.18	0.18	0.10	0.10	0.10		0.08	0.07
06-05	0.56	0.59	0.56	0.18	0.18	0.18		0.11	0.11	0.08	0.08	0.08
06-06	0.53	0.54	0.53	0.17	0.18	0.17	0.10	0.10			0.07	
06-07	0.53	0.53	0.53		0.17			0.10			0.07	
06-08	0.52	0.53	0.52	0.17	0.17	0.16		0.10			0.07	
09-01	0.58		0.53			0.17	0.09					
09-02	0.59	0.79			0.20	0.20		0.13	0.12	0.08	0.08	0.09
09-03	0.72	0.71			0.21	0.21	0.12	0.17	0.12		0.08	0.09
09-04	0.59	0.62	0.61	0.19	0.20	0.19	0.11	0.12	0.11	0.08	0.08	0.08
09-05	0.59	0.60	0.60	0.19	0.20	0.19	0.11	0.12	0.11	0.08	0.08	0.08
09-06	0.56	0.57	0.56	0.18	0.18	0.18	0.11	0.11	0.11		0.08	0.08
09-07	0.55	0.55	0.55	0.18	0.19	0.18		0.10	0.10			
09-08	0.53	0.53	0.53	0.17	0.17	0.17		0.10			0.07	
10-01	0.58		0.53			0.17						
10-02		0.79		0.20	0.23	0.21	0.12	0.12	0.12	0.09	0.08	0.09
10-03		0.71		0.21	0.21	0.21	0.12	0.12	0.12	0.08	0.08	0.08
10-04	0.63	0.62	0.61		0.20	0.20	0.11	0.12	0.11	0.08	0.08	
10-05	0.60	0.62	0.61	0.19	0.20	0.20	0.12	0.12	0.12	0.08	0.08	0.08
10-06	0.58	0.59	0.58	0.18	0.18	0.18		0.11			0.08	
10-07	0.56	0.57	0.56		0.16		0.10	0.10	0.10		0.08	0.08
10-08	0.53	0.53	0.53	0.17	0.17	0.17	0.10	0.10	0.10		0.07	
11-01	0.59		0.54	0.18		0.17					0.07	
11-02		0.82		0.21	0.23	0.21	0.12	0.12	0.12	0.09	0.08	0.09
11-03		0.73		0.21	0.21	0.21	0.12	0.12	0.12	0.08	0.09	0.08
11-04	0.63	0.64	0.63	0.19	0.19	0.19		0.11	0.11	0.08	0.08	0.08
11-05	0.63	0.62	0.63	0.19	0.20	0.19	0.12	0.12	0.12	0.08	0.08	0.08
11-06	0.60	0.60	0.60		0.17	0.18	0.10	0.10			0.08	

11-07	0.57	0.59	0.58			0.18		0.10			
11-08	0.57	0.57	0.56	0.17	0.17	0.17	0.10	0.10	0.10		0.07
12-01			0.57	0.17		0.17		0.10	0.10		
12-02		0.85		0.21	0.23	0.21	0.12	0.12		0.09	0.09 0.08
12-03		0.73		0.22	0.22	0.21	0.12	0.11	0.11		0.09 0.08
12-04	0.65	0.62	0.68	0.19	0.19	0.19	0.11	0.11			0.08
12-05	0.69	0.60	0.68	0.19		0.19	0.11	0.11	0.11		0.08
12-06	0.65	0.60	0.65	0.18	0.18	0.18		0.10			0.07
12-07	0.62	0.62	0.63	0.18	0.20	0.18		0.10			
12-08	0.62	0.62	0.62	0.18	0.18	0.18		0.10	0.10		

F.3.2 Modal damping ratio

Figure F.7 plots the modal damping ratios in the ground motion cases of the 21TKB test estimated using the ARX, FDD and SS methods, respectively. Table F.6 summarizes the corresponding values in each segment.

Table F.6 Damping ratio in ground motion excitations of the 21TKB test

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01												
02-02	0.03	0.09	0.03	0.02	0.03	0.01	0.01	0.02	0.05	0.02	0.04	0.04
02-03	0.03	0.11	0.03	0.02	0.02	0.01		0.01	0.07		0.01	0.04
02-04	0.02	0.11	0.01	0.03	0.02	0.06		0.01			0.04	0.03
02-05	0.01	0.13	0.01		0.03			0.08				
02-06	0.01	0.12	0.01	0.01	0.03	0.07	0.01	0.01				
02-07	0.01	0.10	0.01		0.02			0.01			0.05	
02-08	0.04	0.10			0.04			0.03			0.01	
04-01	0.10									0.02		
04-02	0.04	0.07	0.03	0.06	0.04	0.02	0.04	0.05	0.03		0.04	0.02
04-03	0.05	0.12	0.04	0.01	0.07	0.02	0.02	0.03	0.02	0.02	0.01	0.04
04-04	0.02	0.09	0.01	0.01	0.02	0.02		0.03	0.04		0.04	
04-05	0.02	0.10	0.02	0.01	0.05	0.03		0.01			0.03	
04-06	0.02	0.13	0.01	0.03	0.03	0.03		0.02			0.01	
04-07	0.02	0.11	0.02	0.01	0.03	0.01						
04-08	0.02	0.11	0.01	0.03	0.03	0.04		0.02			0.01	
06-01		0.20	0.08		0.02	0.10	0.02	0.03			0.02	
06-02	0.05	0.10	0.08	0.01	0.04	0.03	0.01	0.01	0.04	0.01	0.02	
06-03	0.06	0.22	0.07	0.03	0.05	0.03	0.00	0.02	0.03		0.03	0.05
06-04	0.03	0.14	0.03	0.03	0.02	0.03	0.02	0.02	0.03		0.00	0.04
06-05	0.03	0.10	0.04	0.02	0.05	0.00		0.03	0.04	0.01	0.02	0.03
06-06	0.02	0.09	0.01	0.03	0.03	0.06	0.01	0.01			0.04	
06-07	0.03	0.11	0.02		0.06			0.02			0.01	
06-08	0.01	0.11	0.01	0.03	0.03	0.04		0.04			0.02	
09-01	0.05		0.10			0.07	0.03					
09-02	0.02	0.20			0.01	0.06		0.02	0.08	0.01	0.01	0.07
09-03	0.07				0.05	0.07	0.02	0.03	0.05		0.04	0.07
09-04	0.06	0.11	0.05	0.03	0.03	0.04	0.01	0.05	0.04	0.02	0.03	0.05

Appendix F Full Results of Vibration Property Analysis: 21TKB

09-05	0.05	0.07	0.04	0.02	0.07	0.03	0.02	0.02	0.03	0.01	0.02	0.02
09-06	0.03	0.10	0.03	0.02	0.04	0.02	0.02	0.03	0.04		0.02	0.04
09-07	0.02	0.13	0.03	0.01	0.03	0.08		0.02	0.08			
09-08	0.01	0.12	0.02	0.01	0.03	0.02		0.04			0.03	
10-01	0.09		0.09			0.08						
10-02		0.21		0.06	0.03	0.06	0.03	0.02	0.04	0.02	0.01	0.04
10-03				0.07	0.05	0.05	0.02	0.04	0.04	0.02	0.02	0.07
10-04	0.05	0.11	0.05		0.04	0.03	0.04		0.03	0.02	0.02	
10-05	0.06	0.08	0.05	0.03	0.07	0.03	0.03	0.03	0.03	0.02	0.03	0.04
10-06	0.07	0.09	0.04	0.02	0.04	0.02		0.03			0.02	
10-07	0.03	0.10	0.03		0.02		0.02	0.05	0.03		0.03	0.03
10-08	0.02	0.13	0.02	0.02	0.02	0.02	0.04	0.03	0.04		0.04	
11-01	0.10		0.09	0.08		0.06					0.01	
11-02		0.23		0.05	0.04	0.06	0.02	0.02	0.03	0.01	0.02	0.03
11-03				0.06	0.06	0.06	0.01	0.04	0.04	0.02	0.01	0.06
11-04	0.05	0.09	0.04	0.07	0.02	0.05		0.01	0.04	0.02	0.01	0.05
11-05	0.05	0.10	0.07	0.07	0.05	0.04	0.03	0.05	0.07	0.03	0.03	0.06
11-06	0.05	0.10	0.04		0.03	0.04	0.03	0.01			0.01	
11-07	0.03	0.09	0.02			0.06		0.01				
11-08	0.02	0.12	0.02	0.02	0.04	0.02	0.03	0.02	0.03		0.02	
12-01			0.08	0.08		0.06		0.01	0.05			
12-02		0.24		0.05	0.04	0.07	0.02	0.02		0.01	0.04	0.04
12-03				0.06	0.08	0.07	0.02	0.02	0.06		0.01	0.06
12-04	0.06	0.09	0.03	0.03	0.03	0.03	0.02	0.02			0.04	
12-05	0.06	0.07	0.04	0.04		0.02	0.01	0.02	0.01		0.01	
12-06	0.02	0.09	0.03	0.03	0.04	0.01		0.02			0.01	
12-07	0.06	0.07	0.04	0.02	0.03	0.02		0.02				
12-08	0.02	0.09	0.02	0.02	0.04	0.02		0.02	0.03			

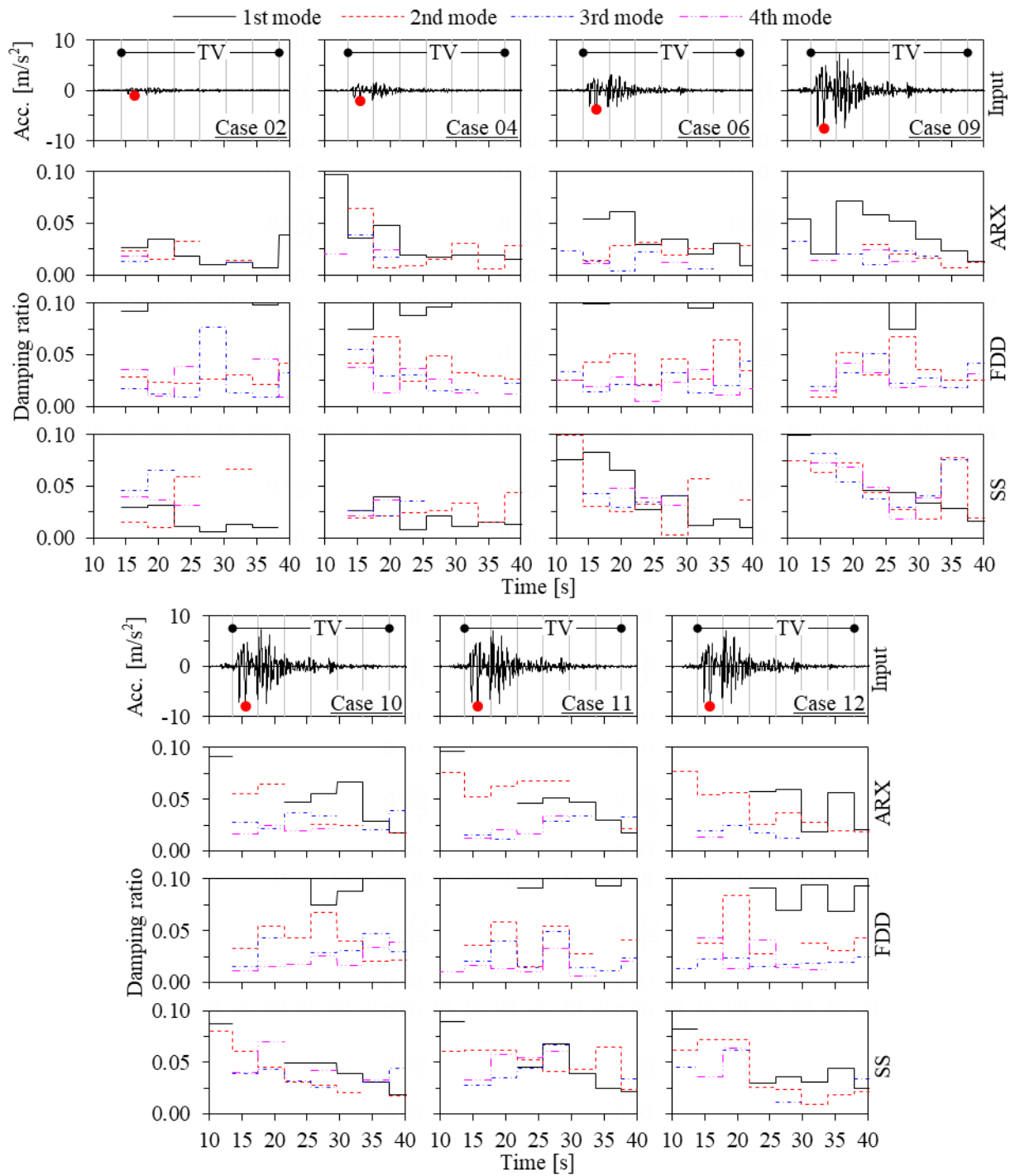


Figure F.7 Damping ratio in ground motion excitations of the 21TKB test

F.3.3 Mode shapes

Figure F.8 to Figure F.10 plot the mode shapes in the ground motion cases of the 21TKB test estimated using the ARX, FDD and SS methods, respectively.

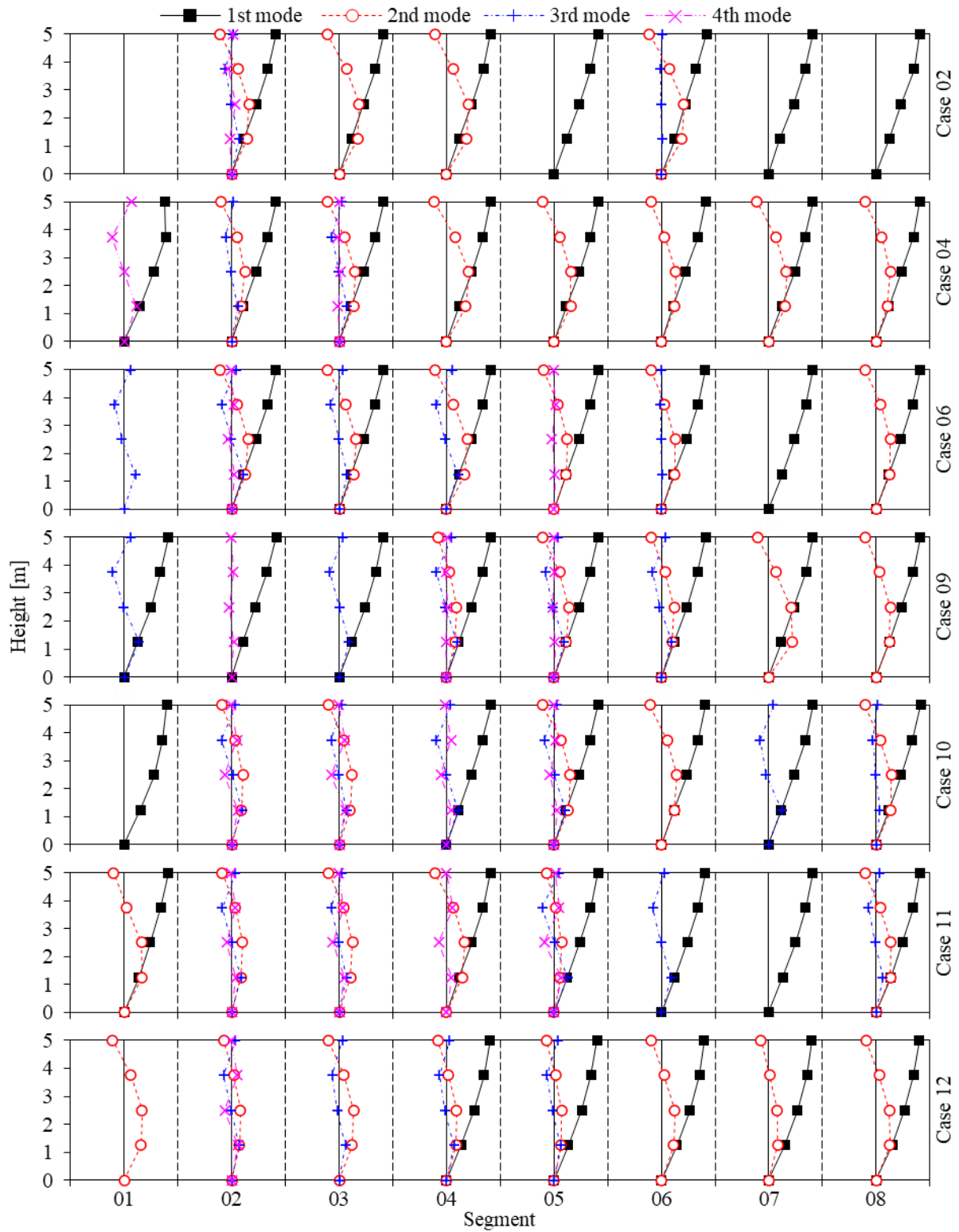


Figure F.8 Mode shapes in ground motion excitations of the 21TKB test (ARX)

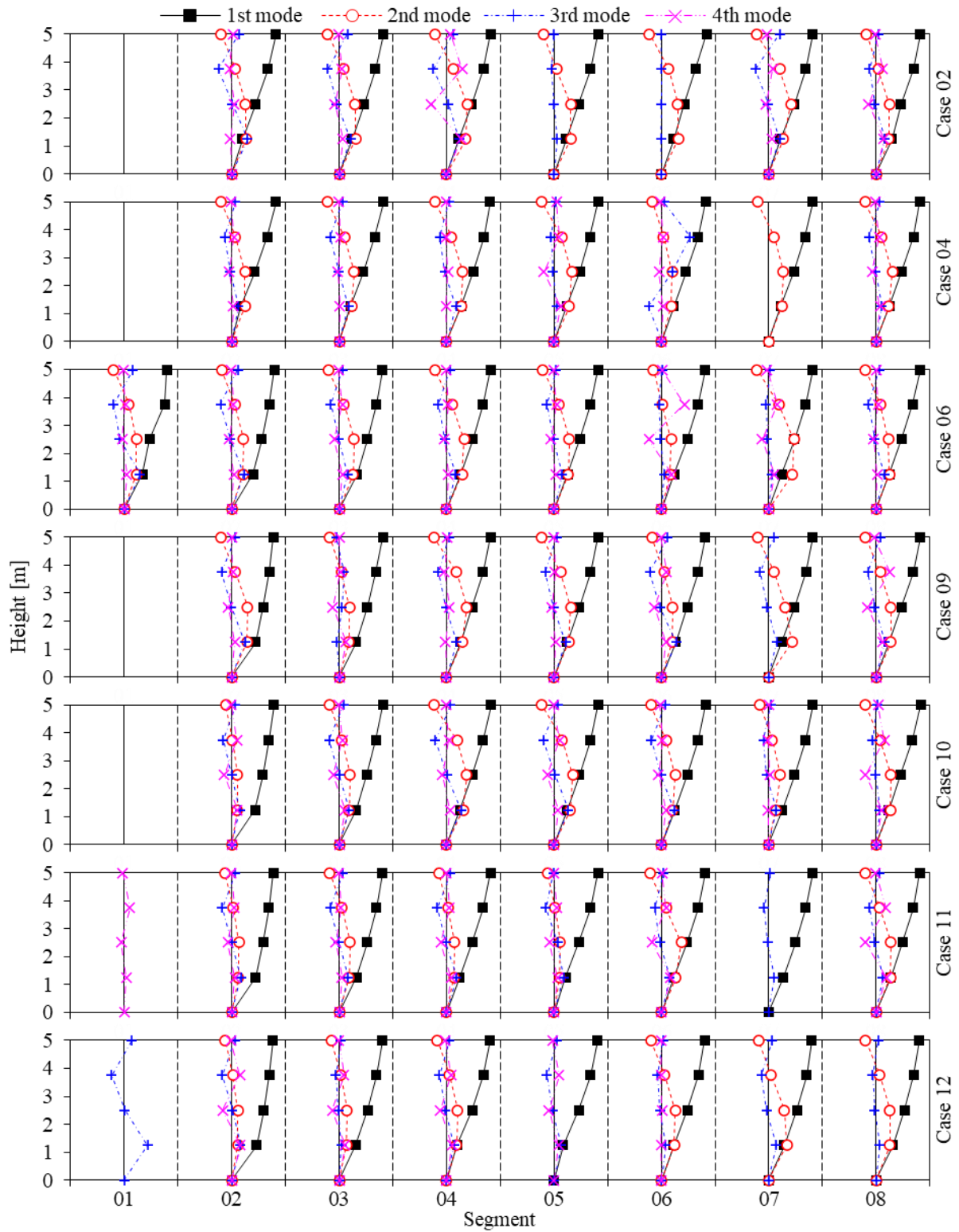


Figure F.9 Mode shapes in ground motion excitations of the 21TKB test (FDD)

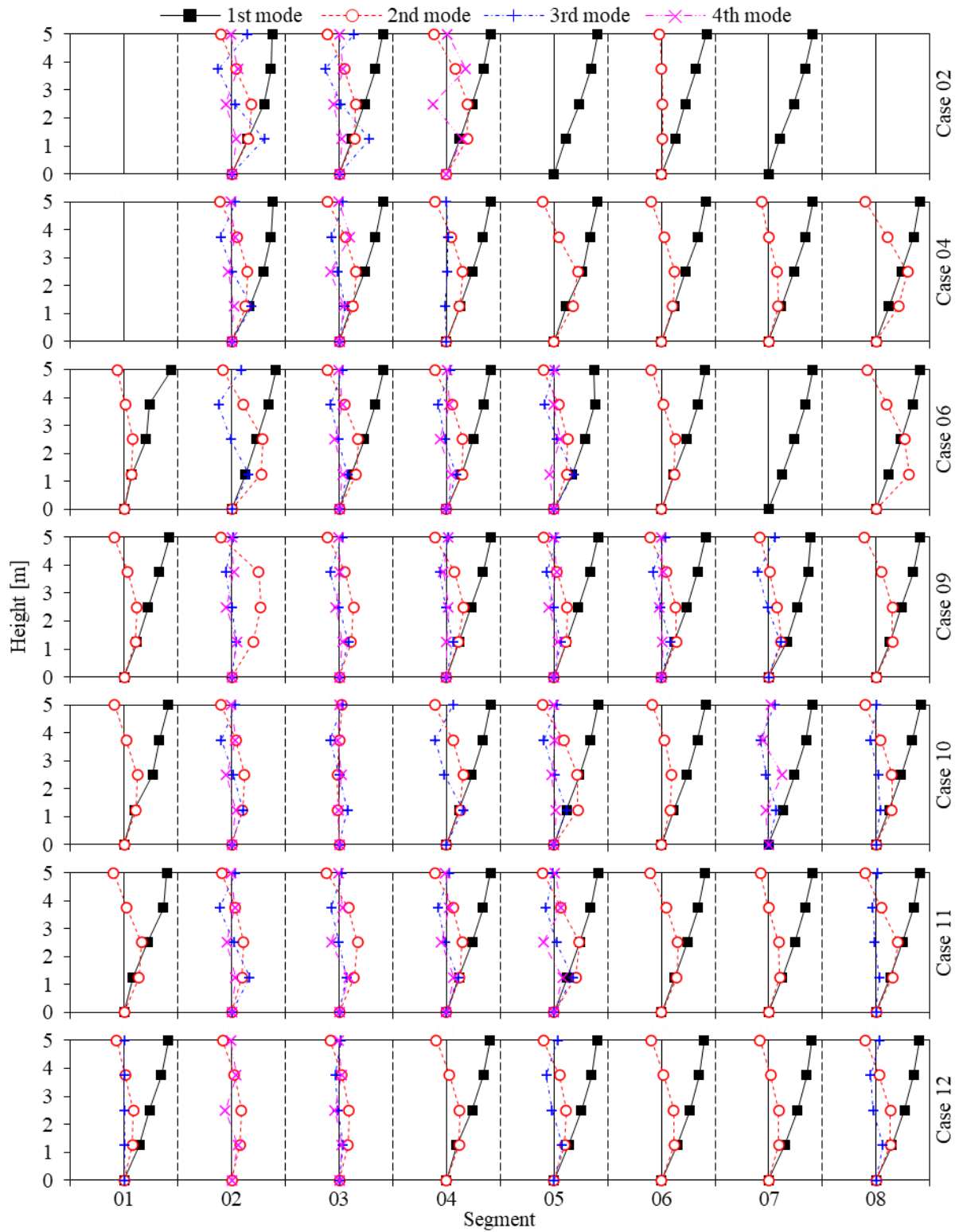


Figure F.10 Mode shapes in ground motion excitations of the 21TKB test (SS)

F.3.4 Modal Effective Mass (MEM)

Figure F.11 plots the natural period in the ground motion cases of the 21TKB test estimated using the ARX, FDD and SS methods, respectively. Table F.7 summarizes the corresponding values in each segment.

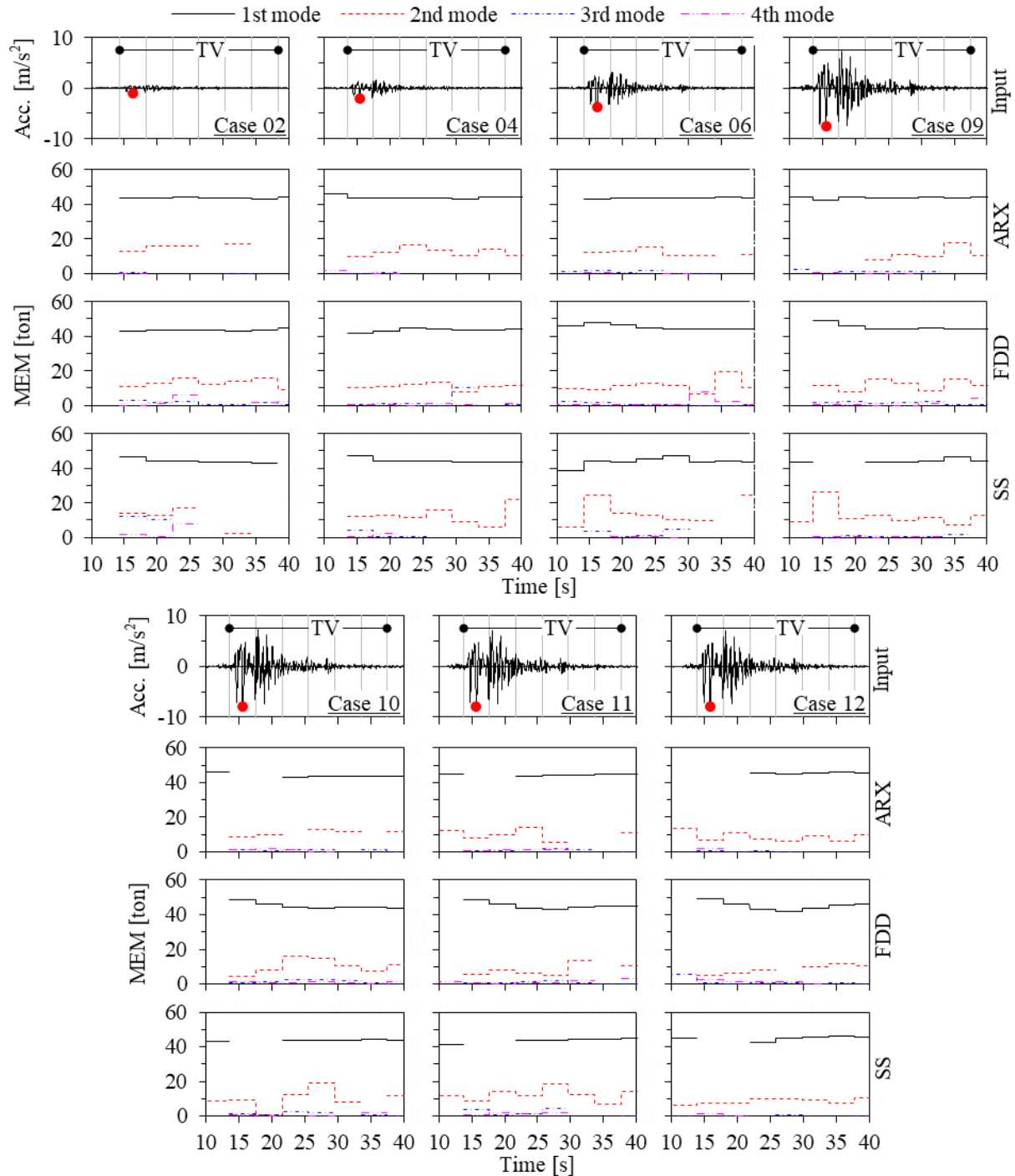


Figure F.11 MEMs in ground motion excitations of the 21TKB test

Table F.7 MEMs in ground motion excitations of the 21TKB test [Unit: ton]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01												
02-02	43.3	42.7	46.5	13.0	10.8	13.7	0.5	2.9	12.0	0.0	0.0	1.4
02-03	43.5	43.8	44.1	15.5	12.4	12.5		1.4	10.3		0.8	0.4
02-04	43.8	43.6	44.1	16.0	15.7	16.9		2.1			6.0	7.7
02-05	43.7	43.5	43.4		12.0			0.2				
02-06	43.6	43.0	43.7	16.9	13.8	2.1	0.2	0.2				
02-07	43.0	43.4	42.9		16.1			1.9			1.6	
02-08	44.0	45.0			9.2			0.7			2.1	
04-01	45.8									1.7		
04-02	43.2	41.5	47.0	9.7	10.2	11.9	0.3	0.3	4.1		0.3	0.7
04-03	43.2	42.8	43.8	11.9	11.1	12.6	0.5	0.9	0.2	0.0	0.2	2.4
04-04	43.7	44.9	44.3	16.7	12.0	11.5		0.8	0.3		0.0	
04-05	43.6	43.9	44.3	13.5	13.6	15.9		0.0			1.0	
04-06	43.1	43.6	43.5	10.5	7.9	9.3		10.2			0.5	
04-07	44.1	43.5	43.7	13.8	11.2	6.1						
04-08	43.9	44.0	43.7	10.5	11.6	21.8		0.2			0.8	
06-01		46.0	38.6		9.8	5.7	1.1	2.4			0.5	
06-02	43.2	47.9	44.2	12.0	8.9	24.4	1.4	1.6	3.4	0.4	0.2	
06-03	43.3	46.5	43.5	12.5	11.3	13.9	0.6	0.6	0.6		0.7	0.5
06-04	43.6	44.6	45.1	15.3	12.9	12.5	1.3	0.6	0.8		0.6	0.6
06-05	43.5	44.4	47.1	10.1	11.7	10.5		0.6	4.7	0.3	0.5	0.5
06-06	43.8	44.1	43.8	10.2	6.6	9.5	0.1	0.0			7.7	
06-07	44.4	44.0	44.1		19.6			0.0			2.2	
06-08	43.8	43.9	43.8	11.0	10.0	24.3		0.5			0.2	
09-01	44.4		43.4			9.3	2.3					
09-02	42.5	48.9			11.6	26.1		1.8	0.1	0.2	0.2	0.6
09-03	44.2	46.1			8.0	11.0	0.9	2.0	0.9		0.7	0.2
09-04	43.4	44.4	43.6	7.5	15.3	12.5	1.0	1.0	0.3	0.0	0.0	0.0
09-05	43.4	44.2	43.3	11.0	13.0	9.8	0.9	1.4	0.3	0.1	0.1	0.5
09-06	43.9	44.7	43.8	9.4	8.6	11.5	0.9	2.2	0.6		1.1	0.3
09-07	43.7	44.1	46.8	17.6	14.9	7.4		0.5	1.4			
09-08	44.1	43.8	44.3	10.5	11.2	12.9		0.6			3.9	
10-01	46.1		43.0			8.9						
10-02		48.8		8.7	4.6	9.5	1.2	0.6	1.4	1.3	1.2	0.7
10-03		46.1		9.7	8.0	0.6	0.5	1.0	0.7	1.5	0.9	0.0
10-04	43.3	44.1	43.6		15.9	12.4	1.5	2.3	2.7	1.2	0.6	
10-05	43.4	43.9	43.9	12.7	14.6	19.0	1.4	2.3	1.7	0.3	1.1	0.2
10-06	43.9	44.3	43.7	11.4	10.2	7.7		1.7			0.8	
10-07	43.7	44.2	44.4		7.6		1.5	0.4	0.4		0.0	1.8
10-08	43.7	43.6	44.0	11.5	11.1	11.9	0.1	0.0	0.3		1.5	
11-01	44.9		41.3	12.5		11.4					1.3	
11-02		48.9		8.2	5.5	8.8	0.9	0.8	3.6	0.8	0.5	0.5
11-03		46.3		10.1	8.1	14.3	0.4	0.7	0.5	0.9	0.3	1.6
11-04	43.8	44.0	43.8	13.9	6.1	11.8		0.9	1.2	1.2	0.7	1.3
11-05	44.0	43.2	44.0	5.8	5.0	18.2	2.0	1.8	4.5	1.3	0.7	2.1
11-06	44.4	44.1	44.2		13.3	12.3	0.9	0.6			1.8	

11-07	44.7	44.9	44.5			7.1		0.2		
11-08	44.9	44.9	44.8	11.1	10.8	14.2	0.3	0.2	0.0	3.1
12-01			44.9	13.7		6.3		5.6	0.1	
12-02		49.2		6.7	5.0	7.3	0.4	0.9		1.5 2.6 1.3
12-03		46.1		10.9	6.2	7.7	0.2	0.0	0.0	1.2 0.3
12-04	45.2	42.9	42.7	7.4	8.1	9.6	0.4	0.4		1.3
12-05	45.1	42.0	45.2	6.0		9.8	0.3	0.3	0.5	1.3
12-06	45.7	43.8	45.8	9.5	10.1	9.4		0.0		0.0
12-07	45.9	45.6	45.9	6.2	11.8	7.5		0.3		
12-08	45.8	46.1	45.7	10.0	10.3	10.5		0.0	0.2	

F.3.5 PSD peaks

Figure F.12 plots the peak values of the Power Spectra Density (PSD) in the white noise cases of the 21TKB test identified using the FDD method. Table F.8 summarizes the corresponding values.

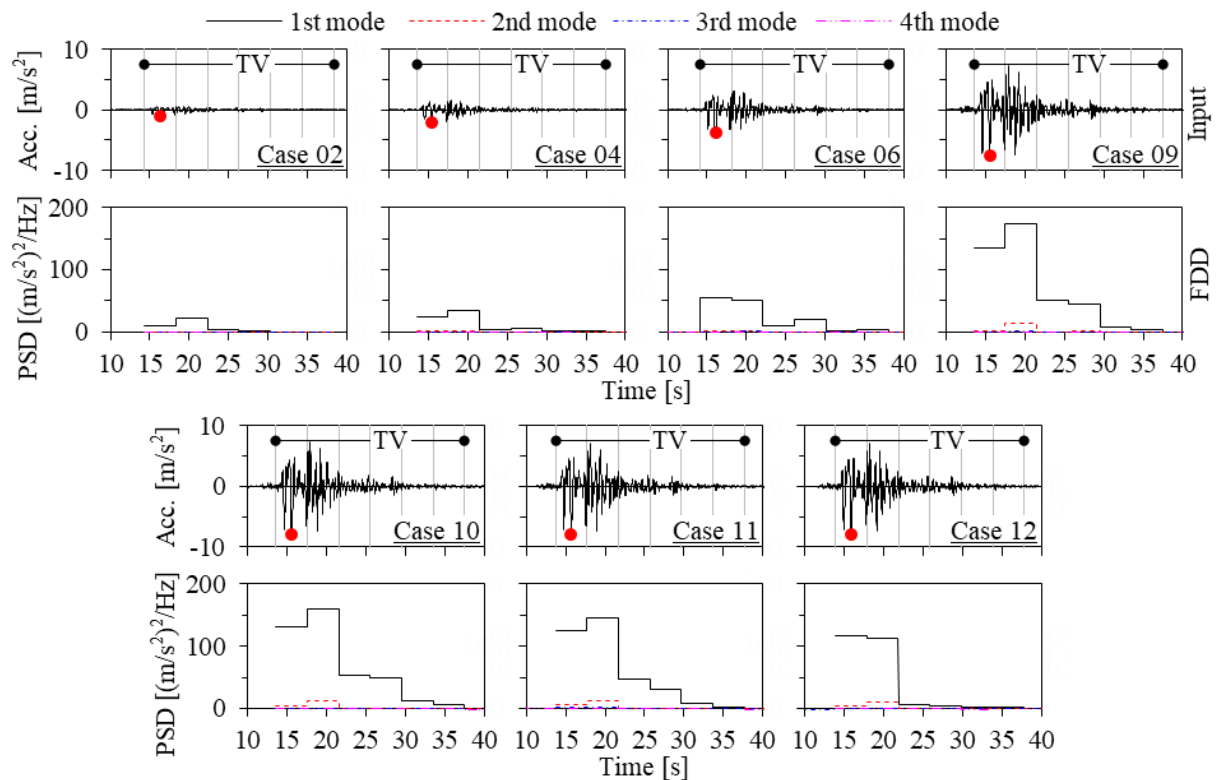


Figure F.12 PSD peaks in ground motion excitations of the 21TKB test

Table F.8 PSD peaks in ground motion excitations of the 21TKB test [Unit: $(\text{m/s}^2)^2/\text{Hz}$]

Case-Segment	Mode			
	1st	2nd	3rd	4th
02-01				
02-02	9.0	0.2	0.0	0.0
02-03	21.0	0.1	0.0	0.0
02-04	2.8	0.0	0.0	0.0
02-05	1.4	0.0	0.0	
02-06	0.2	0.0	0.0	
02-07	0.2	0.0	0.0	0.0

Appendix F Full Results of Vibration Property Analysis: 21TKB

02-08	0.0	0.0	0.0	0.0
04-01				
04-02	24.5	0.6	0.0	0.0
04-03	35.1	0.5	0.1	0.0
04-04	3.2	0.1	0.0	0.0
04-05	6.4	0.0	0.0	0.0
04-06	1.2	0.0	0.0	0.0
04-07	0.4	0.0		
04-08	0.0	0.0	0.0	0.0
06-01	0.0	0.0	0.0	0.0
06-02	54.1	1.5	0.1	0.1
06-03	50.3	2.1	0.5	0.1
06-04	9.8	0.1	0.0	0.0
06-05	19.1	0.3	0.0	0.0
06-06	1.9	0.0	0.0	0.0
06-07	2.5	0.0	0.0	0.0
06-08	0.2	0.0	0.0	0.0
09-01				
09-02	133.7	1.9	0.2	0.1
09-03	173.9	12.9	0.7	0.0
09-04	50.9	0.4	0.1	0.0
09-05	45.3	0.7	0.1	0.0
09-06	7.4	0.1	0.0	0.0
09-07	4.4	0.0	0.0	
09-08	0.1	0.0	0.0	0.0
10-01				
10-02	131.8	4.7	0.5	0.1
10-03	160.8	13.2	0.9	0.2
10-04	52.3	0.3	0.1	0.0
10-05	48.4	0.5	0.1	0.0
10-06	11.8	0.2	0.0	0.0
10-07	5.6	0.0	0.0	0.0
10-08	0.1	0.0	0.0	0.0
11-01				0.0
11-02	125.5	5.6	1.2	0.2
11-03	146.0	12.9	1.7	0.1
11-04	46.4	0.1	0.0	0.0
11-05	30.6	0.3	0.0	0.0
11-06	8.0	0.0	0.0	0.0
11-07	1.6		0.0	
11-08	0.1	0.0	0.0	0.0
12-01			0.0	
12-02	116.4	5.0	0.7	0.1
12-03	112.3	9.5	0.5	0.1
12-04	7.0	0.5	0.1	0.0
12-05	4.6		0.0	0.0
12-06	1.3	0.1	0.0	0.0
12-07	1.1	0.0	0.0	
12-08	0.2	0.0	0.0	

F.4 EFFECTIVE MODAL PSEUDO-ACCELERATION (EMPA)

Figure F.13 plots the peak EMPA values identified in the ground motion cases of the 21TKB test, including the elastic pseudo-acceleration spectra with 2% damping of the measured excitations.

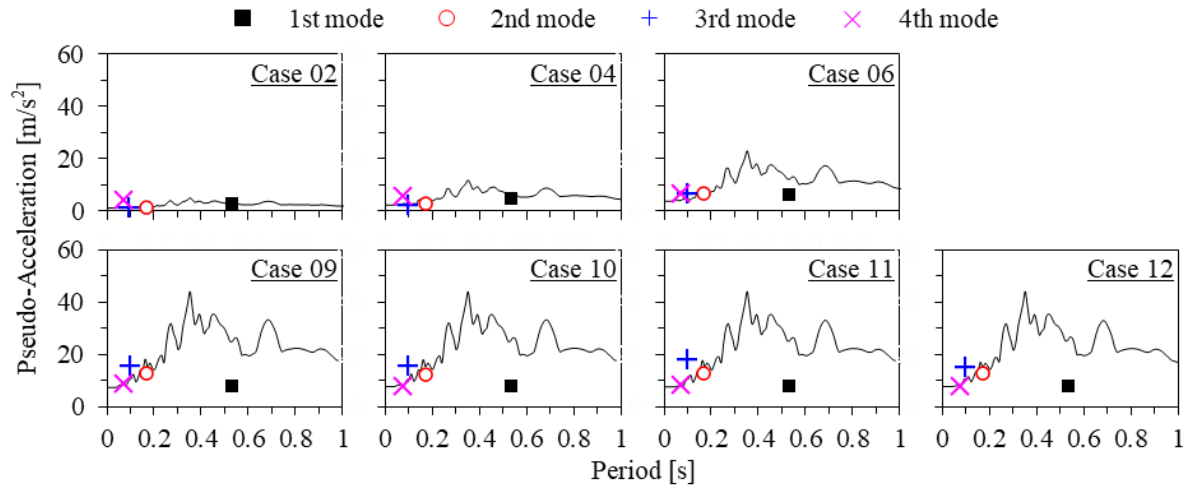


Figure F.13 Peak EMPA values in ground motion excitations of the 21TKB test

F.5 TIME-FREQUENCY RESULTS FROM GROUND MOTION EXCITATIONS

F.5.1 Magnitude of wavelet coefficients and instantaneous periods

Figure F.14 plots the magnitude of wavelet coefficients on the time-frequency plane and the identified instantaneous periods in the ground motion cases of the 21TKB test.

F.5.2 Magnitude of ridge wavelet coefficient

Figure F.15 plots the magnitude of wavelet coefficient on the wavelet ridges in the ground motion cases of the 21TKB test.

F.5.3 Reconstructed modal components

Figure F.16 plots the time-domain modal components reconstructed from the SSWT results and ridge identification in the ground motion cases of the 21TKB test.

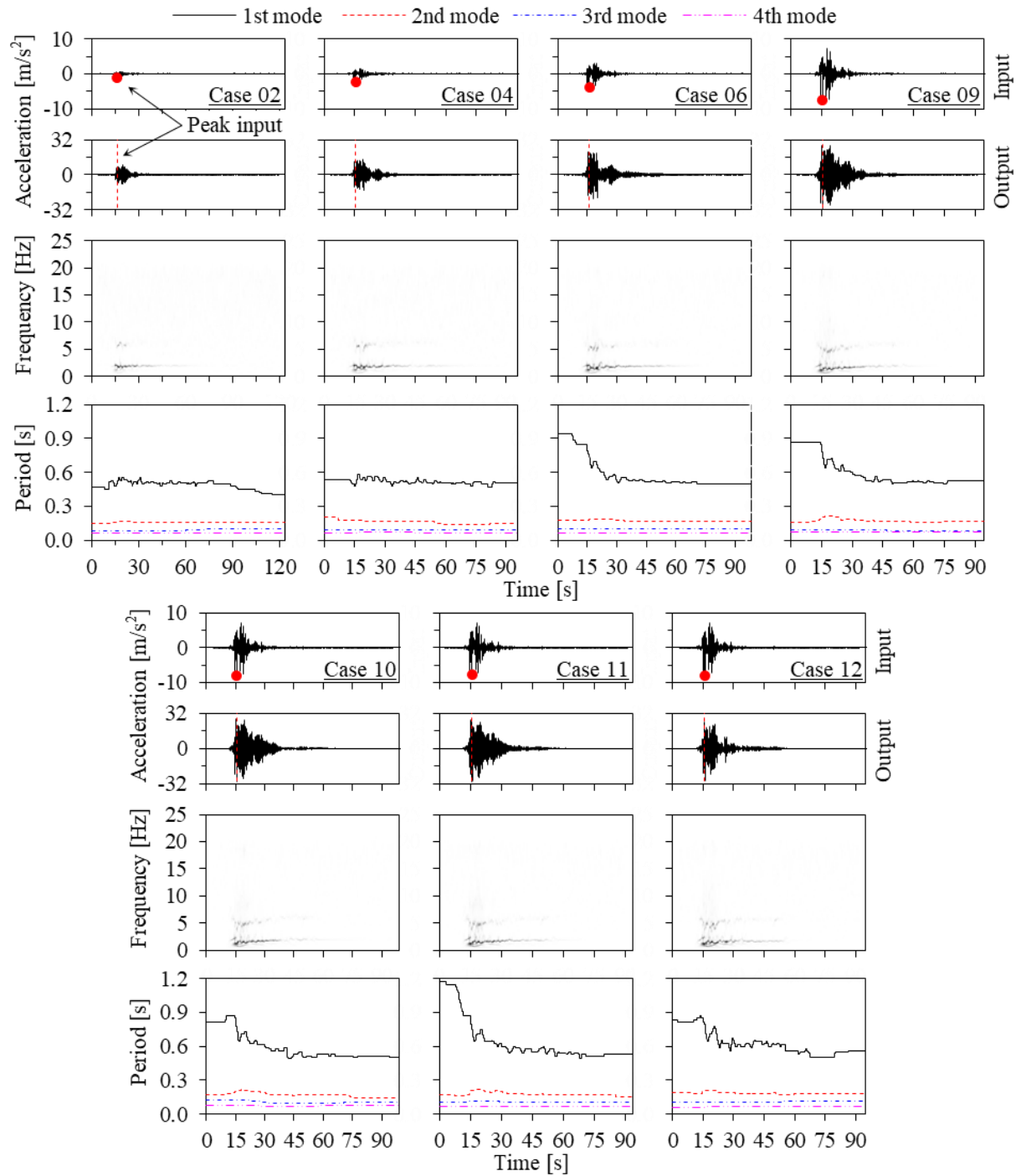


Figure F.14 Magnitude of wavelet coefficients in ground motion excitations of the 21TKB test

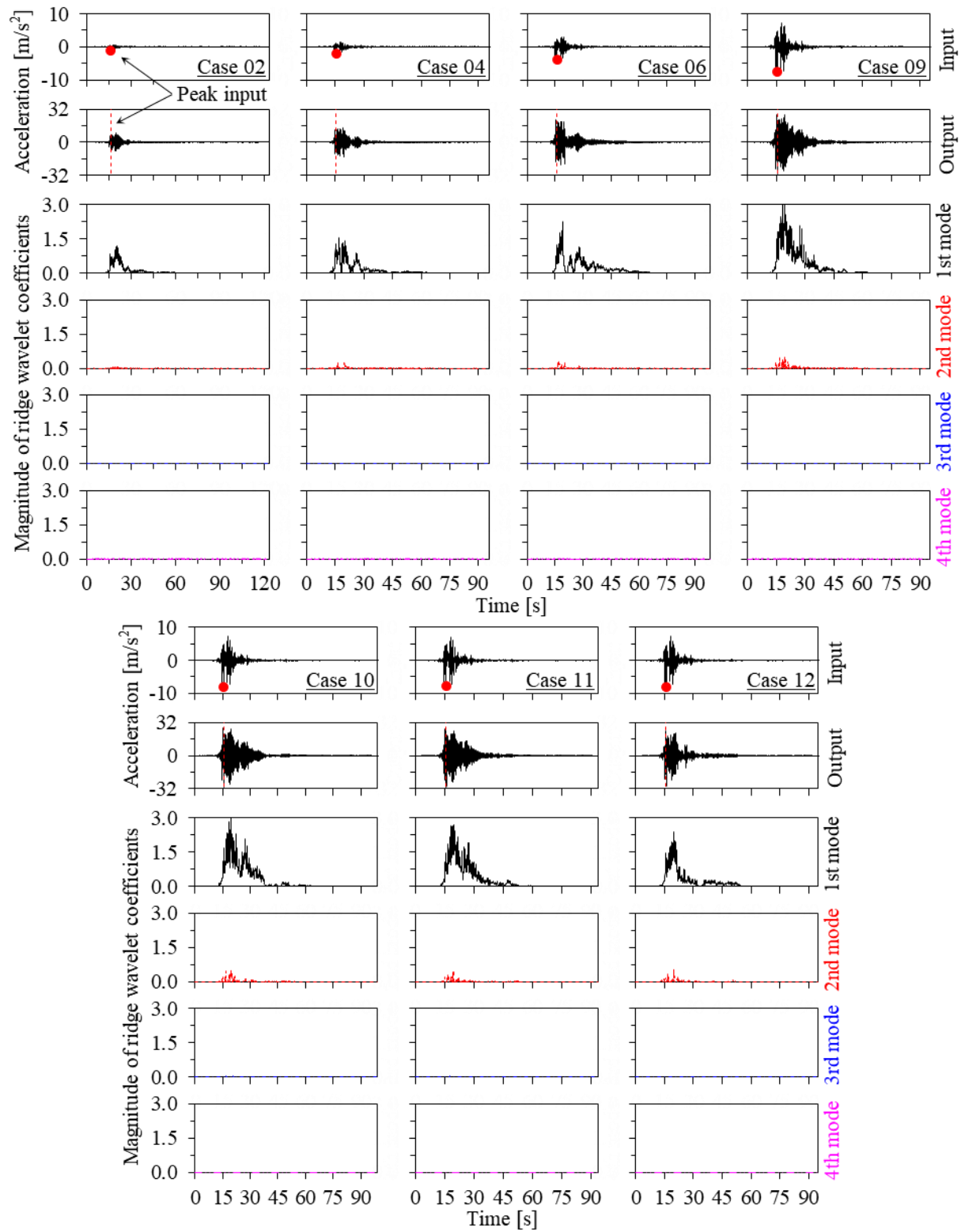


Figure F.15 Magnitude on wavelet ridges in ground motion excitations of the 21TKB test

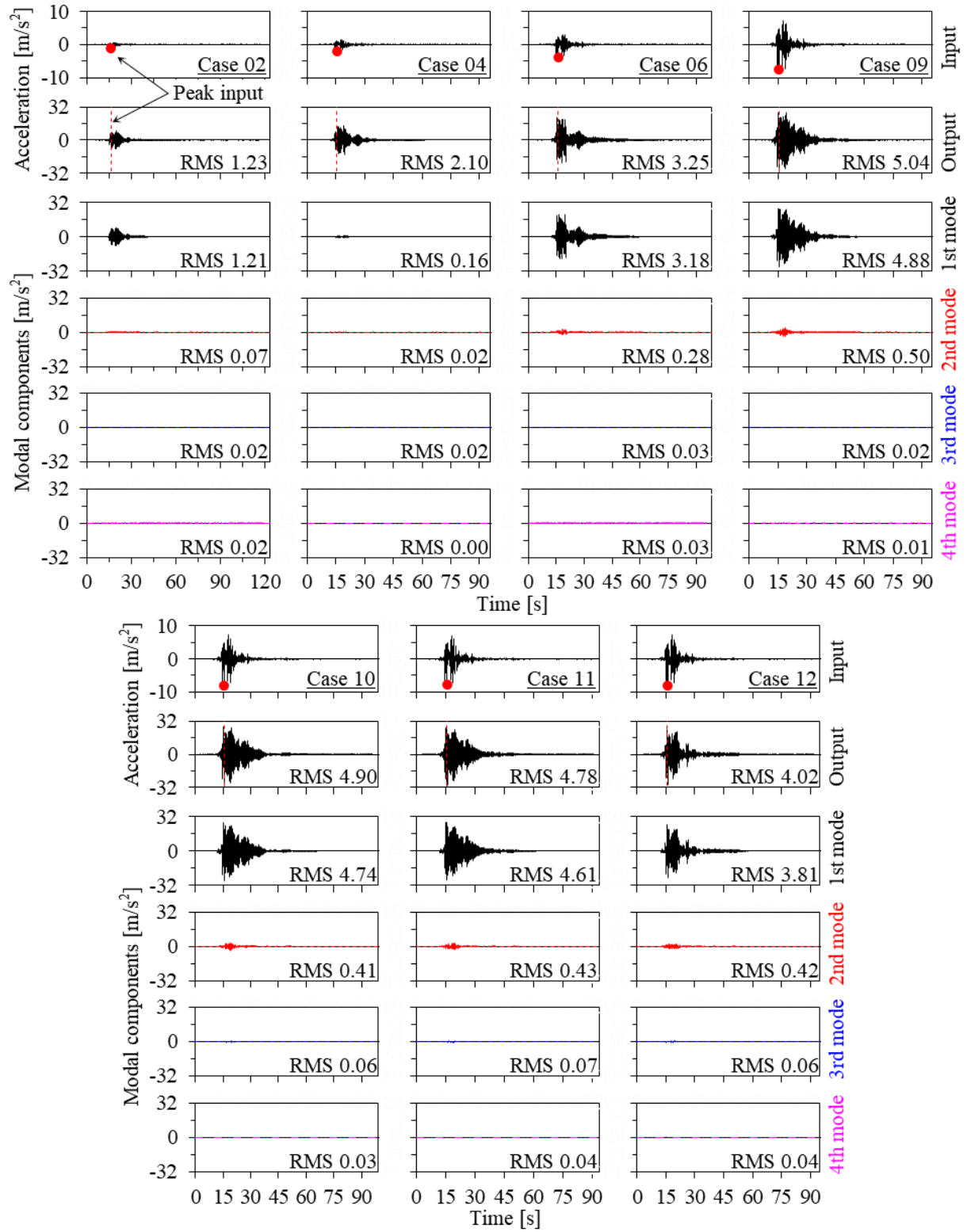


Figure F.16 Modal components in ground motion excitations of the 21TKB test

APPENDIX G. FULL RESULTS OF VIBRATION PROPERTY ANALYSIS: 07EDF

G.1 GENERAL

This appendix provides results of the vibration property analysis on the 07EDF test. The results are grouped by different tests, each including results from all analysis methods. The following results are provided in this appendix:

- 1) **Modal parameters:** natural periods, modal damping ratios, mode shapes, Modal Effective Mass (MEM) and PSD peaks from modal analysis methods.
- 2) **Spectral results:** peak values of Effective Modal Pseudo-Acceleration (EMPA).
- 3) **Time-frequency results:** magnitude of wavelet coefficients, instantaneous periods, magnitude of ridge wavelet coefficient and reconstructed modal components.

In the 07EDF test, 12 white noise and 8 ground motion cases in the East-West direction were selected for analysis. The structural system was not changed during the test.

G.2 MODAL PARAMETERS FROM WHITE NOISE EXCITATIONS

G.2.1 Natural period

Figure G.1 plots the natural period in the white noise cases of the 07EDF test identified using the ARX, FDD and SS methods, respectively. Table G.1 summarizes the corresponding values.

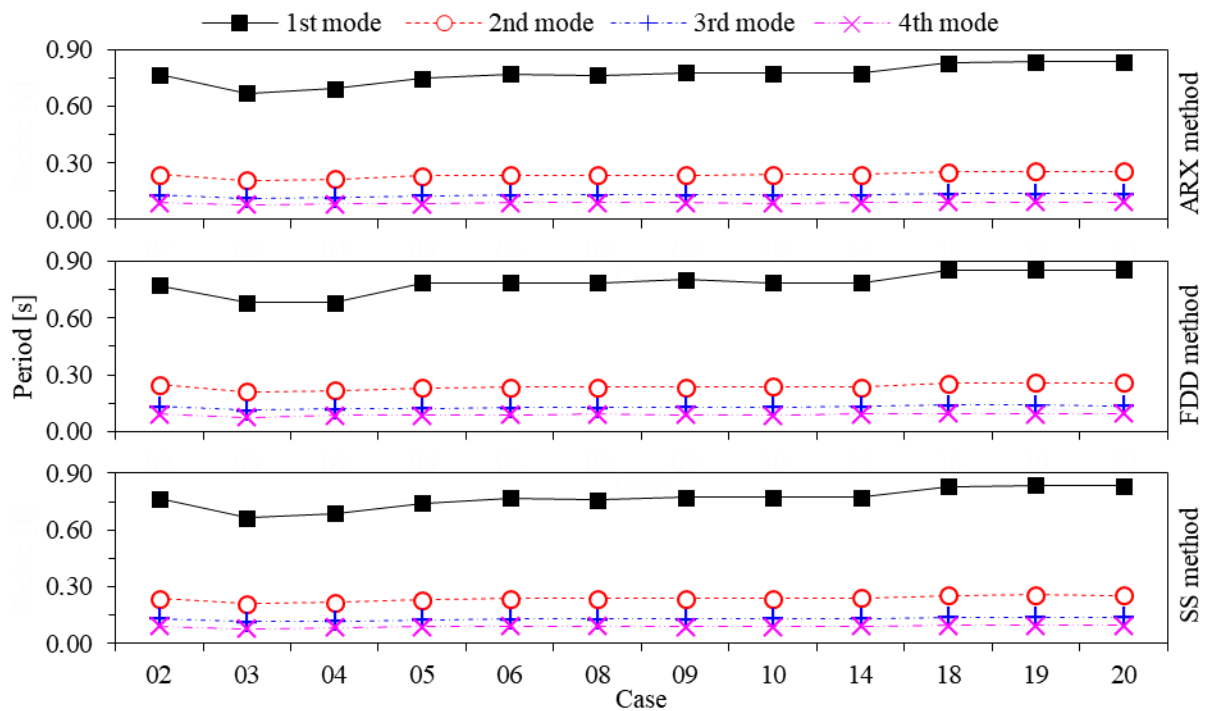


Figure G.1 Natural periods in white noise excitations of the 07EDF test

Table G.1 Natural periods in white noise excitations of the 07EDF test [Unit: s]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02	0.77	0.77	0.77	0.24	0.25	0.24	0.13	0.13	0.13	0.09	0.09	0.09
03	0.67	0.68	0.66	0.21	0.21	0.21	0.11	0.11	0.11	0.08	0.08	0.08
04	0.69	0.68	0.69	0.22	0.22	0.22	0.12	0.12	0.12	0.08	0.08	0.08
05	0.75	0.79	0.74	0.23	0.23	0.23	0.13	0.12	0.13	0.08	0.09	0.09
06	0.77	0.79	0.77	0.24	0.23	0.24	0.13	0.13	0.13	0.09	0.09	0.09
08	0.76	0.79	0.76	0.24	0.23	0.24	0.13	0.13	0.13	0.09	0.09	0.09
09	0.78	0.80	0.77	0.24	0.23	0.24	0.13	0.13	0.13	0.09	0.09	0.09
10	0.77	0.79	0.77	0.24	0.24	0.24	0.13	0.13	0.13	0.09	0.09	0.09
14	0.78	0.79	0.77	0.24	0.24	0.24	0.13	0.13	0.13	0.09	0.09	0.09
18	0.83	0.85	0.83	0.25	0.26	0.25	0.14	0.14	0.14	0.09	0.10	0.09
19	0.84	0.85	0.84	0.26	0.26	0.26	0.14	0.14	0.14	0.09	0.09	0.09
20	0.83	0.85	0.83	0.26	0.26	0.26	0.14	0.14	0.14	0.09	0.10	0.09

G.2.2 Modal damping ratio

Figure G.2 plots the modal damping ratios in the white noise cases of the 07EDF test identified using the ARX, FDD and SS methods, respectively. Table G.2 summarizes the corresponding values.

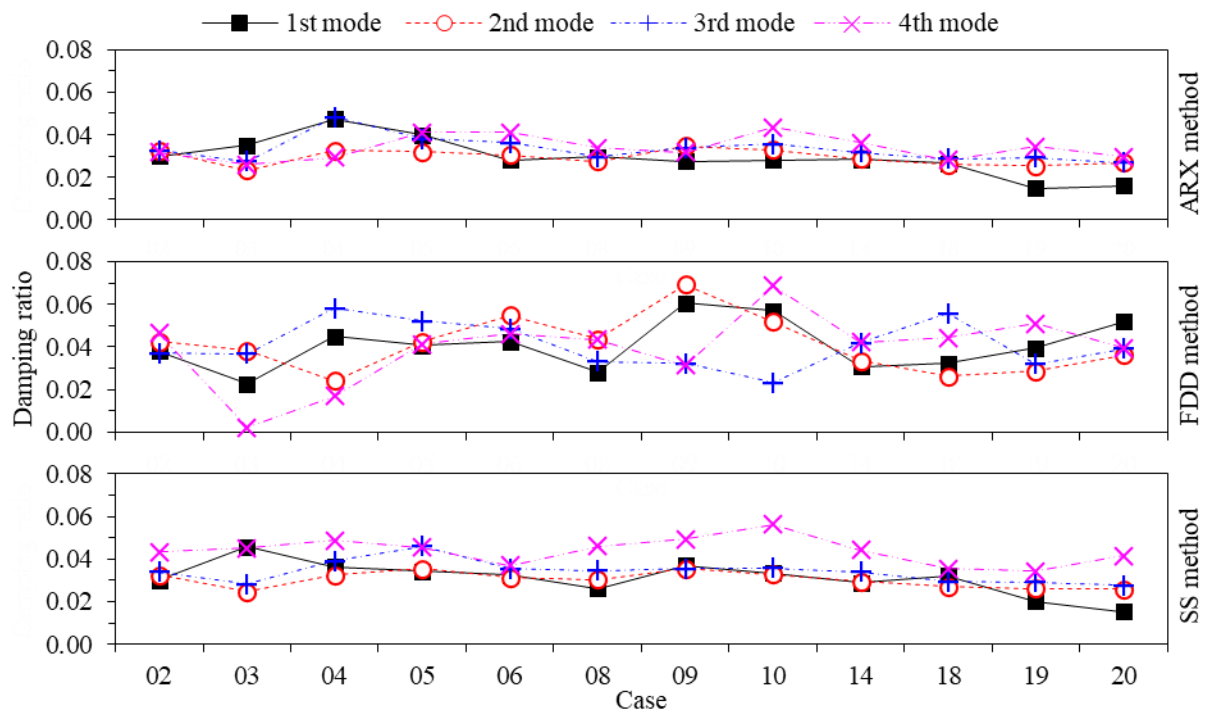


Figure G.2 Damping ratios in white noise excitations of the 07EDF test

Table G.2 Damping ratios in white noise excitations of the 07EDF test

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02	0.03	0.05	0.03	0.03	0.03	0.03	0.03	0.05	0.03	0.03	0.03	0.04
03	0.04	0.04	0.05	0.02	0.03	0.02	0.03	0.04	0.03	0.03	0.04	0.05
04	0.05	0.03	0.04	0.03	0.03	0.03	0.05	0.04	0.04	0.03	0.04	0.05
05	0.04	0.03	0.03	0.03	0.03	0.04	0.04	0.03	0.05	0.04	0.03	0.05
06	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.03	0.04	0.04	0.03	0.04
08	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.03	0.03	0.04	0.05
09	0.03	0.03	0.04	0.03	0.03	0.04	0.03	0.03	0.04	0.03	0.04	0.05
10	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.03	0.04	0.04	0.03	0.06
14	0.03	0.01	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.03	0.04
18	0.03	0.02	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.04
19	0.01	0.04	0.02	0.03	0.03	0.03	0.03	0.02	0.03	0.03	0.04	0.03
20	0.02	0.05	0.02	0.03	0.04	0.03	0.03	0.03	0.03	0.03	0.03	0.04

G.2.3 Mode shapes

Figure G.3 plots the mode shapes in the white noise cases of the 07EDF test identified using the ARX, FDD and SS methods, respectively. The plotted mode shapes are normalized by the modal participation factor Γ_n .

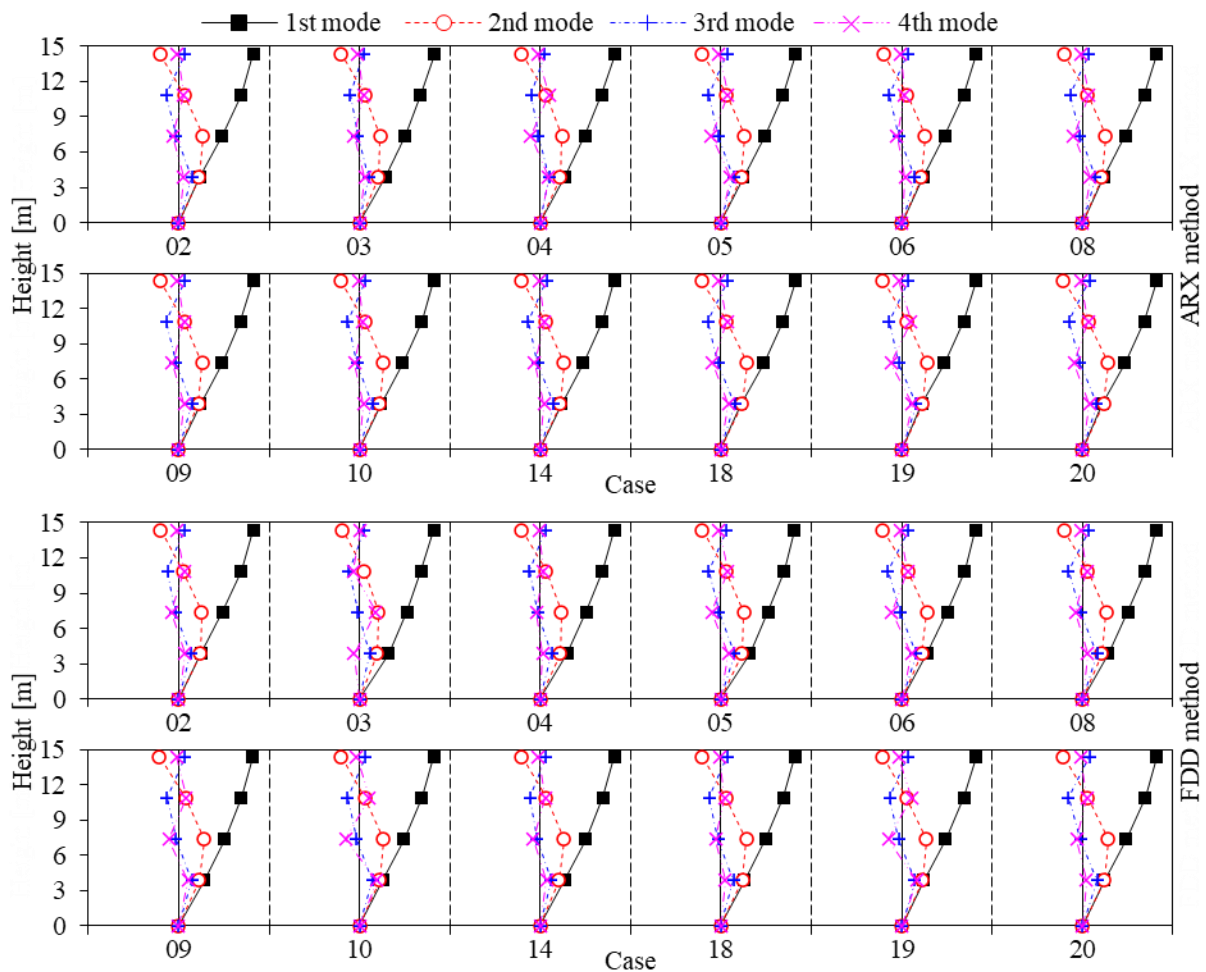


Figure G.3 Mode shapes in white noise excitations of the 07EDF test

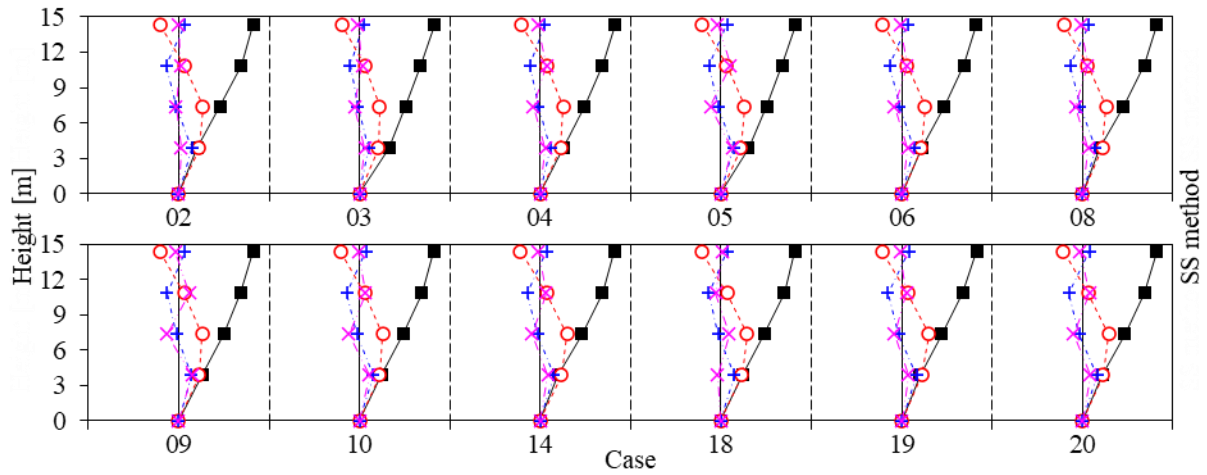


Figure G.3 (continued) Mode shapes in white noise excitations of the 07EDF test

G.2.4 Modal Effective Mass (MEM)

Figure G.4 plots the Modal Effective Mass (MEM) in the white noise cases of the 07EDF test identified using the ARX, FDD and SS methods, respectively. Table G.3 summarizes the corresponding values of the MEMs. The total mass of the structure is 209 tons, with Floor 1 to 3 being 48 tons and Floor 4 being 64 tons.

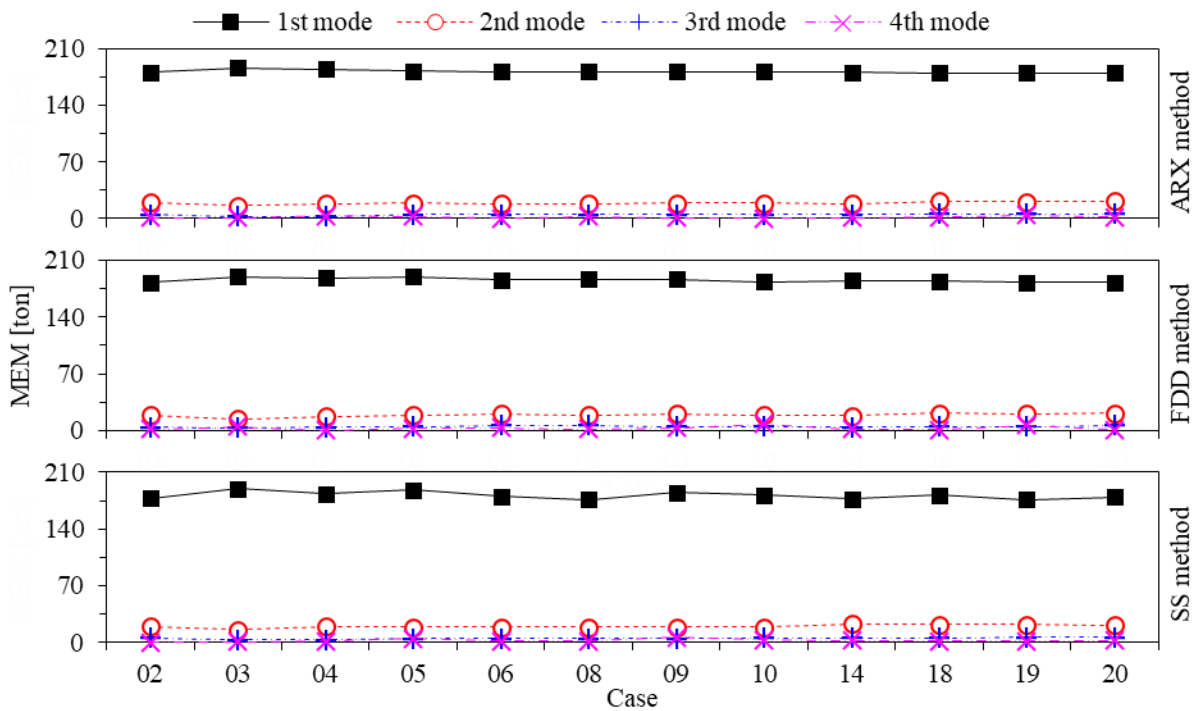


Figure G.4 MEMs in white noise excitations of the 07EDF test

Table G.3 MEMs in white noise excitations of the 07EDF test [Unit: ton]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02	180.9	182.6	177.6	19.7	19.2	19.8	5.0	4.3	5.2	1.2	1.7	0.2
03	186.2	190.1	190.0	16.5	14.2	15.8	2.7	3.6	2.8	1.0	4.6	0.8
04	184.7	188.1	183.6	18.2	17.6	19.5	2.6	3.8	3.2	3.2	0.4	1.6
05	182.6	190.3	188.3	19.2	18.8	19.0	4.8	5.1	4.7	3.0	2.6	4.6
06	181.3	186.2	180.0	18.4	20.8	19.2	5.1	6.2	4.8	0.6	3.3	1.7
08	181.3	186.9	176.3	18.9	19.1	19.2	5.2	6.3	5.0	2.6	1.3	1.6
09	181.3	186.6	184.6	19.1	21.0	19.2	5.1	4.9	4.8	1.5	3.4	6.0
10	181.3	183.8	182.0	19.5	19.2	19.1	4.8	4.9	5.5	0.7	7.8	3.2
14	181.0	185.5	177.4	18.8	18.6	22.5	5.2	3.7	5.4	0.9	1.6	2.2
18	179.7	184.2	181.6	21.7	21.2	22.0	5.6	4.9	5.1	2.2	0.8	1.5
19	179.9	182.6	176.1	21.1	20.5	22.4	5.8	5.0	6.3	4.2	6.6	1.7
20	179.9	182.6	179.5	21.5	21.9	21.5	6.1	6.4	6.1	2.2	1.0	2.5

G.2.5 PSD peaks

Figure G.5 plots the peak values of the Power Spectra Density (PSD) in the white noise cases identified using the FDD method. Table G.4 summarizes the corresponding values.

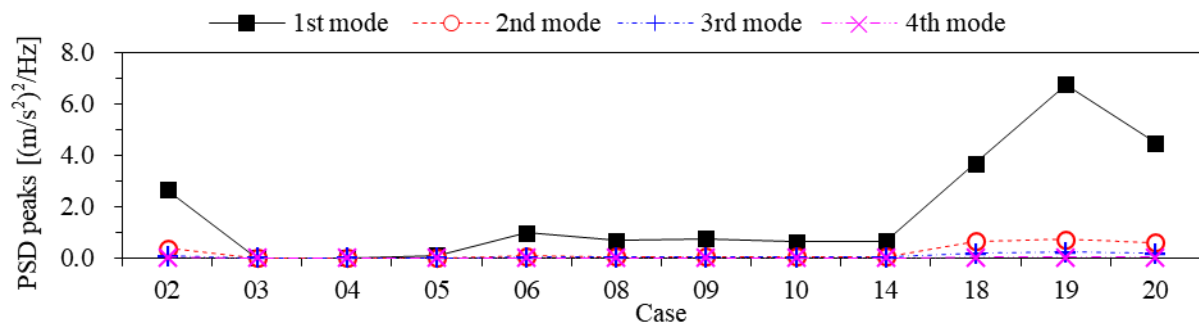


Figure G.5 PSD peaks in white noise excitations of the 07EDF test

Table G.4 PSD peaks in white noise excitations of the 07EDF test [Unit: (m/s²)²/Hz]

Case	Mode			
	1st	2nd	3rd	4th
02	2.655	0.387	0.085	0.016
03	0.002	0.000	0.001	0.000
04	0.007	0.001	0.001	0.000
05	0.100	0.012	0.006	0.001
06	0.969	0.069	0.028	0.004
08	0.665	0.062	0.024	0.003
09	0.748	0.039	0.036	0.003
10	0.628	0.062	0.019	0.003
14	0.656	0.052	0.024	0.004
18	3.687	0.659	0.173	0.027
19	6.765	0.725	0.237	0.021
20	4.472	0.597	0.185	0.020

G.3 MODAL PARAMETERS FROM GROUND MOTION EXCITATIONS

G.3.1 Natural period

Figure G.6 plots the natural period in the ground motion cases of the 07EDF test estimated using the ARX, FDD and SS methods, respectively. Table G.5 summarizes the corresponding values in each segment.

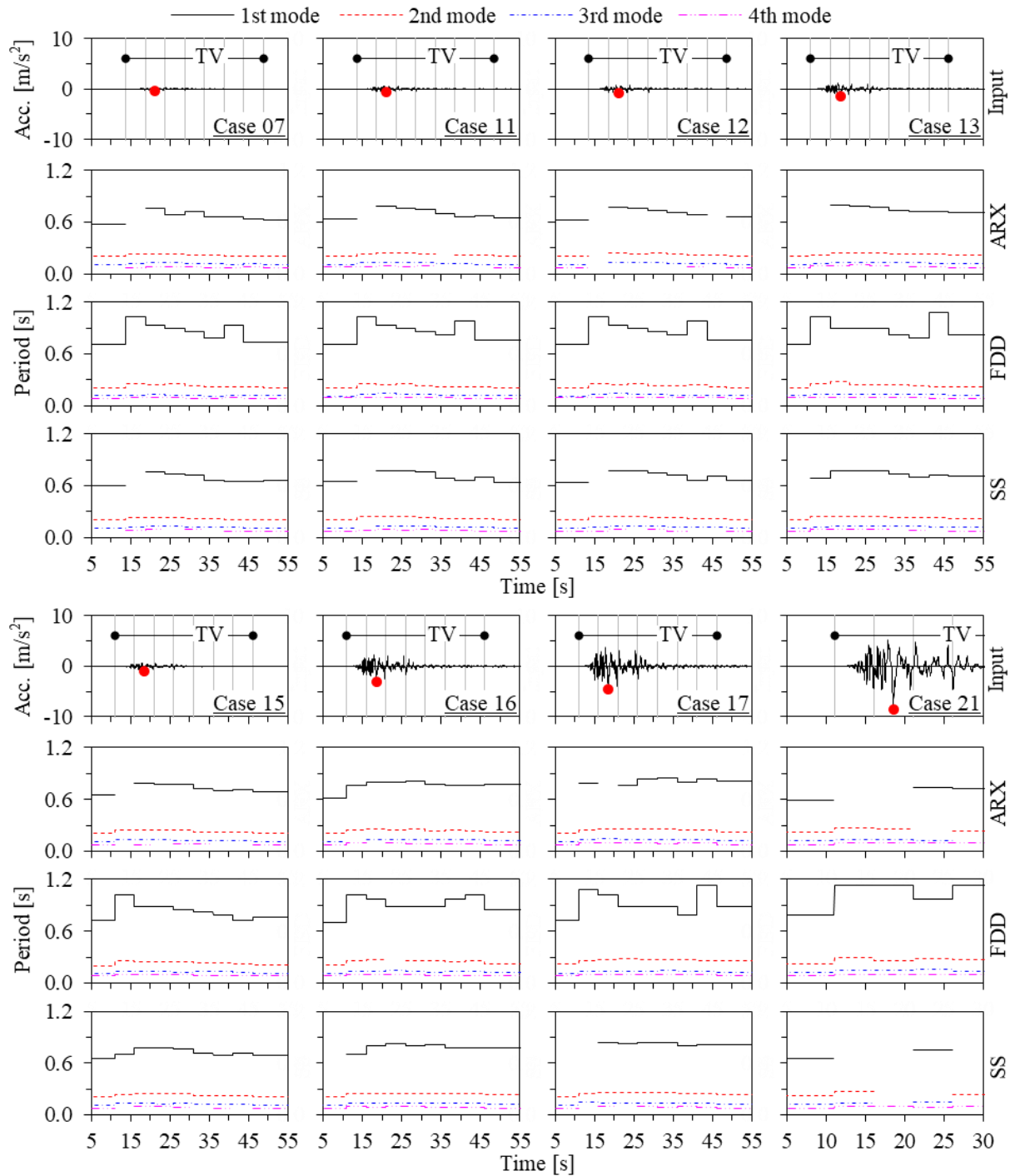


Figure G.6 Natural periods in ground motion excitations of the 07EDF test

Table G.5 Natural periods in ground motion excitations of the 07EDF test [Unit: s]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
07-01	0.57	0.71	0.60	0.21	0.21	0.21	0.11	0.11	0.11		0.08	
07-02		1.02		0.23	0.26	0.23	0.11	0.12	0.12	0.07	0.09	0.08
07-03	0.76	0.93	0.76	0.23	0.24	0.23	0.13	0.13	0.13	0.09	0.09	0.09
07-04	0.68	0.89	0.73	0.23	0.25	0.23	0.13	0.12	0.13	0.09	0.09	
07-05	0.72	0.85	0.72	0.22	0.23	0.22	0.12	0.12	0.12	0.08	0.09	0.09
07-06	0.66	0.79	0.66	0.22	0.21	0.21	0.12	0.11	0.12	0.08	0.09	
07-07	0.66	0.93	0.65	0.21	0.22	0.21	0.11	0.12	0.11	0.08	0.09	0.08
07-08	0.63	0.73	0.65	0.21	0.22	0.21	0.11	0.12	0.11	0.08	0.08	0.07
07-09	0.63	0.73	0.66	0.21	0.21	0.21	0.11	0.12	0.11	0.08	0.09	0.07
11-01	0.64	0.71	0.64	0.20	0.21	0.20	0.11	0.11	0.11	0.08	0.09	0.07
11-02		1.02		0.23	0.26	0.24	0.13	0.13		0.09	0.10	0.08
11-03	0.78	0.93	0.77	0.24	0.25	0.24	0.13	0.14	0.13	0.10	0.09	0.09
11-04	0.76	0.89	0.77	0.24	0.25	0.24	0.13	0.13	0.13	0.09	0.09	0.09
11-05	0.74	0.85	0.75	0.24	0.23	0.23	0.14	0.13	0.13	0.09	0.10	0.09
11-06	0.70	0.82	0.68		0.22	0.22	0.12	0.12	0.12		0.09	0.08
11-07	0.66	0.98	0.66	0.22	0.22	0.22	0.12	0.12	0.12		0.09	0.08
11-08	0.67	0.76	0.70	0.22	0.22	0.22	0.11	0.12	0.12		0.09	0.07
11-09	0.65	0.76	0.64	0.21	0.21	0.21	0.11	0.12	0.11	0.07	0.08	0.07
12-01	0.62	0.71	0.64	0.21	0.21	0.21	0.11	0.11	0.11	0.07	0.09	0.07
12-02		1.02			0.25	0.24		0.13	0.12		0.10	0.07
12-03	0.78	0.93	0.77	0.24	0.25	0.24	0.13	0.14	0.13		0.09	0.09
12-04	0.76	0.89	0.77	0.23	0.26	0.24	0.13	0.13	0.13		0.09	
12-05	0.74	0.85	0.75	0.24	0.23	0.23	0.13	0.13	0.13		0.09	0.09
12-06	0.71	0.82	0.72	0.23	0.23	0.22	0.12	0.12	0.12		0.09	0.07
12-07	0.69	0.98	0.66	0.22	0.24	0.22	0.12	0.12	0.12	0.08	0.09	
12-08		0.76	0.71	0.22	0.22	0.22	0.12	0.12	0.12		0.09	
12-09	0.66	0.76	0.66	0.21	0.21	0.21	0.11	0.12	0.11	0.07	0.09	0.07
13-01		0.71		0.21	0.21	0.21	0.11	0.11	0.11	0.07	0.09	0.07
13-02		1.02	0.69	0.22	0.25	0.24	0.11	0.13	0.13	0.10	0.10	0.09
13-03	0.79	0.89	0.77	0.24	0.28	0.24	0.13	0.14	0.13	0.09	0.09	0.09
13-04	0.78	0.89	0.78	0.24	0.24	0.24	0.14	0.13	0.13	0.10	0.09	0.09
13-05	0.78	0.89	0.77	0.24	0.24	0.24	0.13	0.13	0.13	0.09	0.10	0.09
13-06	0.74	0.82	0.74	0.23	0.24	0.23	0.13	0.13	0.13		0.09	
13-07	0.72	0.79	0.70	0.23	0.23	0.23	0.13	0.13	0.13	0.09	0.09	0.09
13-08	0.72	1.08	0.72	0.23	0.22	0.23	0.12	0.13	0.12	0.08	0.09	
13-09	0.71	0.82	0.71	0.21	0.22	0.21	0.11	0.12	0.11	0.07	0.09	0.07
15-01	0.65	0.73	0.66	0.21	0.20	0.21	0.11	0.11	0.11	0.07	0.09	0.07
15-02		1.02	0.71	0.24	0.26	0.24	0.13	0.13	0.13		0.10	
15-03	0.79	0.89	0.78	0.24	0.24	0.24	0.13	0.13	0.13	0.08	0.09	0.09
15-04	0.77	0.89	0.78	0.24	0.24	0.24	0.13	0.13	0.13		0.09	0.09
15-05	0.78	0.85	0.77	0.24	0.24	0.24	0.13	0.13	0.13	0.09	0.10	0.09
15-06	0.72	0.82	0.71	0.23	0.24	0.23	0.12	0.13	0.12	0.08	0.09	
15-07	0.70	0.79	0.69	0.22	0.23	0.22	0.13	0.13	0.12		0.09	0.08
15-08	0.72	0.73	0.72	0.23	0.22	0.23	0.12	0.12	0.12		0.09	
15-09	0.69	0.76	0.69	0.21	0.21	0.21	0.11	0.12	0.11	0.07	0.09	0.07
16-01	0.61	0.71		0.21	0.21	0.21	0.11	0.11	0.11	0.07	0.09	0.07

16-02	0.76	1.02	0.71	0.24	0.26	0.24	0.13	0.13	0.10	0.10	0.10
16-03	0.80	0.98	0.80	0.26	0.27	0.25	0.14	0.14	0.13	0.10	0.09
16-04	0.80	0.89	0.82	0.25		0.25	0.14	0.15	0.14	0.10	0.09
16-05	0.81	0.89	0.81	0.25	0.26	0.25	0.14	0.14	0.14	0.09	0.10
16-06	0.78	0.89	0.82	0.24	0.26	0.24	0.13	0.13	0.13	0.09	0.09
16-07	0.77	0.98	0.77	0.24	0.25	0.24	0.13	0.14	0.13	0.09	0.09
16-08	0.76	1.02	0.78	0.24	0.25	0.24	0.13	0.13	0.13	0.09	0.09
16-09	0.78	0.85	0.78	0.22	0.22	0.23	0.12	0.12	0.12	0.08	0.09
17-01		0.73		0.21	0.22	0.21	0.12	0.12	0.12	0.08	0.09
17-02	0.79	1.08		0.25	0.26	0.25	0.13	0.14	0.15	0.10	0.10
17-03		1.02	0.84	0.26	0.27	0.26	0.14	0.14	0.14	0.10	0.10
17-04	0.76	0.89	0.82	0.25	0.29	0.26	0.14	0.13	0.14	0.10	0.10
17-05	0.83	0.89	0.83	0.25	0.27	0.26	0.14	0.14	0.14	0.10	0.10
17-06	0.85	0.89	0.84	0.26	0.27	0.25	0.14	0.15	0.14	0.09	0.10
17-07	0.80	0.79	0.80	0.25	0.27	0.25	0.13	0.14	0.14	0.09	0.09
17-08	0.83	1.14	0.81	0.25	0.26	0.25	0.13	0.13	0.13	0.09	0.09
17-09	0.82	0.89	0.82	0.23	0.26	0.24	0.12	0.12	0.13	0.08	0.09
21-01	0.59	0.79	0.65	0.22	0.22	0.22	0.12	0.12	0.12	0.08	0.09
21-02		1.14		0.27	0.29	0.27	0.14	0.14	0.14	0.10	0.10
21-03		1.14		0.26	0.26		0.13	0.15		0.10	0.10
21-04	0.73	0.98	0.75		0.28		0.12	0.15	0.14	0.10	0.10
21-05	0.73	1.14		0.23	0.27	0.23		0.14		0.10	0.10

G.3.2 Modal damping ratio

Figure G.7 plots the modal damping ratios in the ground motion cases of the 07EDF test estimated using the ARX, FDD and SS methods, respectively. Table G.6 summarizes the corresponding values in each segment.

Table G.6 Damping ratio in ground motion excitations of the 07EDF test

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
07-01	0.03	0.09	0.06	0.01	0.06	0.01	0.02	0.04	0.02		0.03	
07-02		0.23		0.07	0.05	0.04	0.03	0.02	0.05	0.03	0.01	0.07
07-03	0.03	0.14	0.05	0.02	0.02	0.03	0.03	0.01	0.04	0.03	0.01	0.06
07-04	0.04	0.07	0.04	0.01	0.03	0.02	0.03	0.02	0.02	0.04	0.04	
07-05	0.05	0.07	0.05	0.03	0.02	0.03	0.02	0.04	0.03	0.01	0.02	0.05
07-06	0.04	0.32	0.05	0.03	0.05	0.03	0.03	0.02	0.03	0.02	0.02	
07-07	0.03	0.09	0.05	0.03	0.04	0.03	0.03	0.01	0.03	0.01	0.03	0.02
07-08	0.09	0.10	0.04	0.04	0.01	0.03	0.02	0.02	0.02	0.02	0.02	0.02
07-09	0.08	0.09	0.05	0.02	0.04	0.02	0.02	0.03	0.02	0.03	0.03	0.02
11-01	0.02	0.12	0.04	0.02	0.03	0.02	0.02	0.03	0.02	0.03	0.04	0.02
11-02		0.14		0.07	0.14	0.04	0.05	0.02		0.02	0.04	0.07
11-03	0.04	0.12	0.05	0.01	0.03	0.03	0.01	0.01	0.04	0.02	0.02	0.05
11-04	0.05	0.07	0.04	0.02	0.03	0.02	0.03	0.06	0.04	0.02	0.02	0.02
11-05	0.04	0.07	0.05	0.02	0.03	0.03	0.02	0.02	0.04	0.03	0.03	0.06
11-06	0.07	0.14	0.05		0.02	0.05	0.02	0.02	0.04		0.02	0.07
11-07	0.09		0.05	0.03	0.03	0.03	0.05	0.01	0.02		0.01	0.05

11-08	0.08		0.07	0.01	0.03	0.03	0.04	0.05	0.05		0.01	0.03
11-09	0.06	0.15	0.08	0.02	0.04	0.02	0.02	0.02	0.02	0.02	0.04	0.02
12-01	0.03	0.10	0.03	0.02	0.03	0.01	0.02	0.04	0.02	0.01	0.03	0.02
12-02		0.16			0.15	0.05		0.02	0.02		0.01	0.01
12-03	0.04	0.12	0.04	0.06	0.02	0.03	0.01	0.01	0.02		0.01	0.07
12-04	0.05	0.07	0.03	0.03	0.03	0.04	0.02	0.04	0.03		0.01	
12-05	0.04	0.07	0.04	0.02	0.04	0.03	0.05	0.01	0.05		0.01	0.04
12-06	0.06	0.14	0.02	0.03	0.03	0.02	0.03	0.02	0.05		0.01	0.01
12-07	0.06		0.08	0.03	0.03	0.03	0.03	0.01	0.04	0.01	0.01	
12-08			0.09	0.03	0.03	0.02	0.03	0.02	0.02		0.01	
12-09	0.07	0.22	0.07	0.02	0.05	0.02	0.02	0.04	0.03	0.02	0.03	0.02
13-01		0.09		0.01	0.04	0.01	0.02	0.03	0.02	0.01	0.03	0.02
13-02		0.17	0.08	0.02	0.03	0.04	0.03	0.02	0.08	0.01	0.01	0.07
13-03	0.05	0.12	0.04	0.03	0.04	0.04	0.01	0.03	0.03	0.03	0.02	0.05
13-04	0.03	0.10	0.03	0.01	0.01	0.03	0.03	0.09	0.05	0.02	0.01	0.04
13-05	0.04	0.11	0.03	0.04	0.02	0.03	0.02	0.02	0.04	0.02	0.01	0.04
13-06	0.05		0.04	0.05	0.03	0.04	0.03	0.04	0.03		0.05	
13-07	0.06	0.11	0.04	0.03	0.03	0.04	0.06	0.02	0.05	0.02	0.01	0.06
13-08	0.05	0.30	0.05	0.03	0.02	0.04	0.02	0.03	0.04	0.01	0.02	
13-09	0.05	0.18	0.06	0.03	0.06	0.03	0.03	0.08	0.02	0.01	0.04	0.02
15-01	0.03	0.13	0.03	0.01	0.04	0.01	0.02	0.04	0.02	0.01	0.01	0.01
15-02			0.08	0.04	0.03	0.03	0.05	0.03	0.03		0.04	
15-03	0.03	0.13	0.03	0.03	0.03	0.03	0.01	0.04	0.03	0.03	0.01	0.04
15-04	0.03	0.09	0.03	0.04	0.04	0.03	0.04	0.02	0.03		0.01	0.07
15-05	0.04	0.10	0.03	0.05	0.02	0.04	0.03	0.02	0.03	0.02	0.01	0.04
15-06	0.06		0.10	0.03	0.03	0.03	0.05	0.01	0.04	0.06	0.02	
15-07	0.06	0.08	0.05	0.02	0.04	0.03	0.08	0.02	0.01		0.02	0.01
15-08	0.06	0.25	0.06	0.01	0.03	0.03	0.04	0.01	0.04		0.01	
15-09	0.05	0.18	0.05	0.02	0.04	0.02	0.03	0.04	0.02	0.02	0.04	0.02
16-01	0.01	0.08		0.01	0.03	0.01	0.02	0.02	0.02	0.02	0.02	0.03
16-02	0.06	0.18	0.09	0.03	0.03	0.03		0.05	0.07	0.01		0.03
16-03	0.05	0.17	0.06	0.06	0.03	0.06	0.01	0.01	0.04		0.01	0.05
16-04	0.04		0.03	0.02	0.05	0.03	0.02	0.02	0.03	0.01	0.01	0.02
16-05	0.03		0.03	0.04	0.02	0.03	0.02	0.03	0.04	0.02	0.01	0.04
16-06	0.02		0.04	0.05	0.05	0.04	0.04	0.01	0.04	0.02	0.02	0.03
16-07	0.07		0.03	0.03	0.03	0.03	0.02	0.04	0.03	0.01	0.01	0.07
16-08	0.06		0.04	0.03	0.03	0.04	0.05	0.01	0.04	0.02	0.01	0.04
16-09	0.04	0.13	0.03	0.04	0.06	0.04	0.03	0.03	0.04	0.02	0.02	0.01
17-01		0.09		0.02	0.09	0.02	0.02	0.01	0.02	0.01	0.05	0.01
17-02	0.10	0.18		0.03	0.04	0.02	0.03	0.04	0.04	0.02	0.02	
17-03			0.09	0.06	0.05	0.06	0.04	0.02	0.08	0.02	0.02	0.05
17-04	0.05		0.03	0.05	0.03	0.04	0.04	0.02	0.06	0.01	0.03	
17-05	0.04		0.04	0.03	0.03	0.03	0.01	0.01	0.04	0.02	0.03	0.03
17-06	0.08		0.04	0.04	0.04	0.04	0.02	0.02	0.03	0.01	0.02	0.05
17-07	0.06		0.04	0.02	0.08	0.03	0.04	0.02	0.03	0.02	0.01	0.03
17-08	0.02		0.04	0.05	0.03	0.03	0.03	0.04	0.03	0.03	0.02	0.06
17-09	0.03	0.11	0.03	0.05	0.07	0.04	0.03	0.03	0.04	0.01	0.02	0.03
21-01	0.02	0.15	0.03	0.02	0.03	0.02	0.02	0.03	0.01	0.02	0.03	0.01
21-02				0.02	0.10	0.02	0.03	0.03	0.06	0.02	0.01	0.03

21-03	0.02	0.07	0.02	0.06	0.04	0.01	0.02	0.03
21-04	0.06	0.06	0.07	0.02	0.02	0.05	0.07	0.00
21-05	0.04	0.04	0.06	0.05	0.02	0.01	0.01	0.03

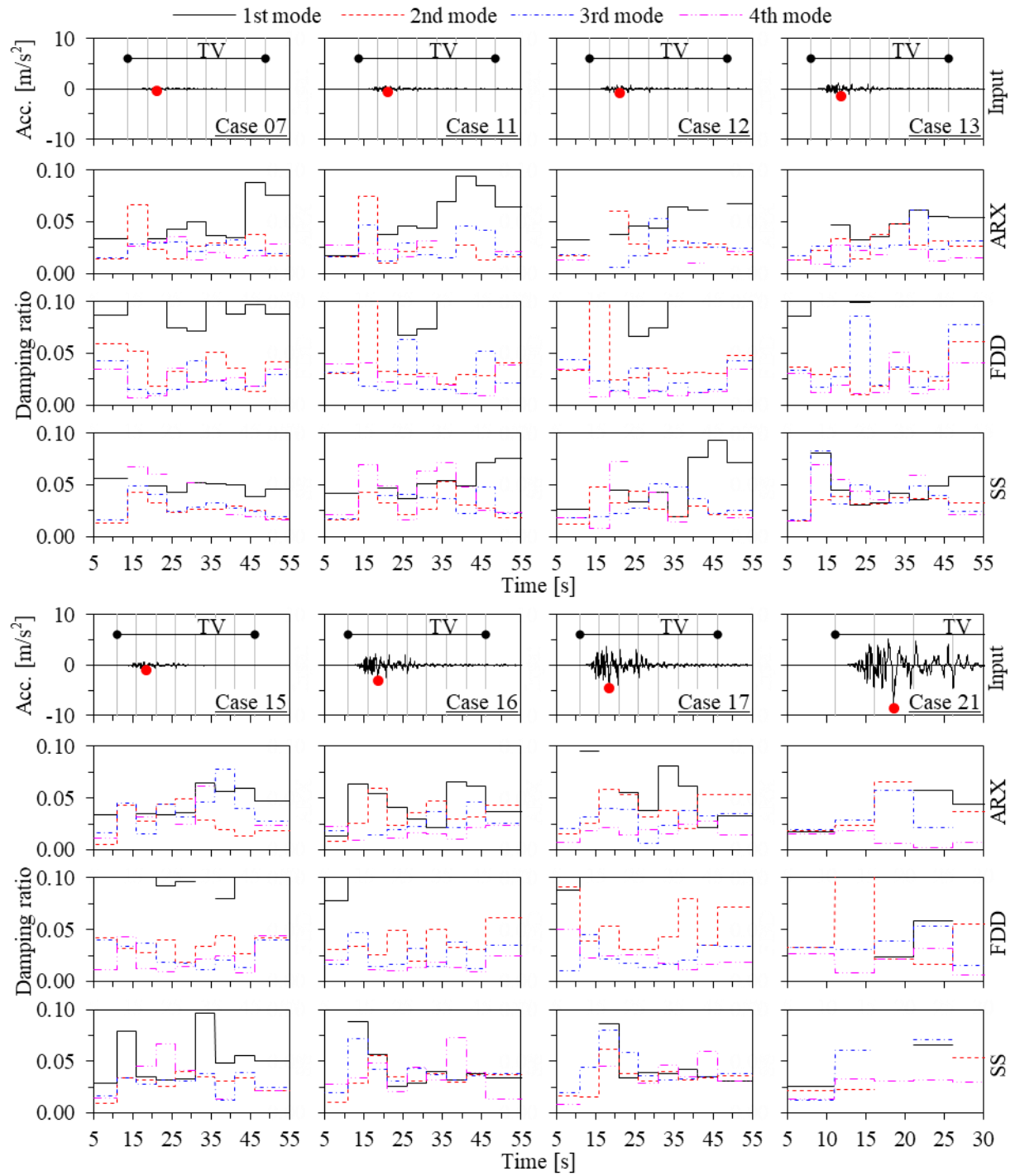


Figure G.7 Damping ratio in ground motion excitations of the 07EDF test

G.3.3 Mode shapes

Figure G.8 to Figure G.10 plot the mode shapes in the ground motion cases of the 07EDF test estimated using the ARX, FDD and SS methods, respectively.

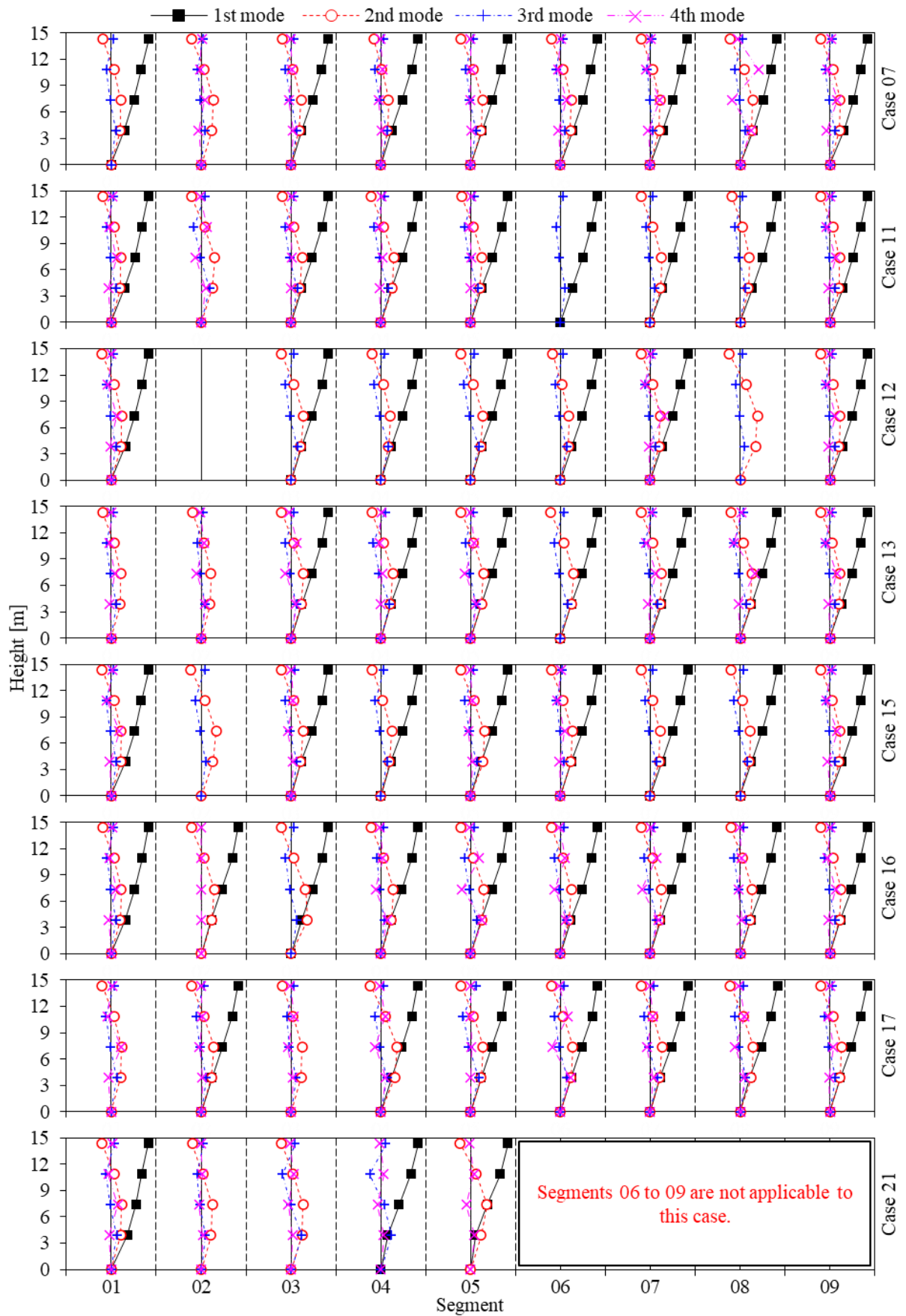


Figure G.8 Mode shapes in ground motion excitations of the 07EDF test (ARX)

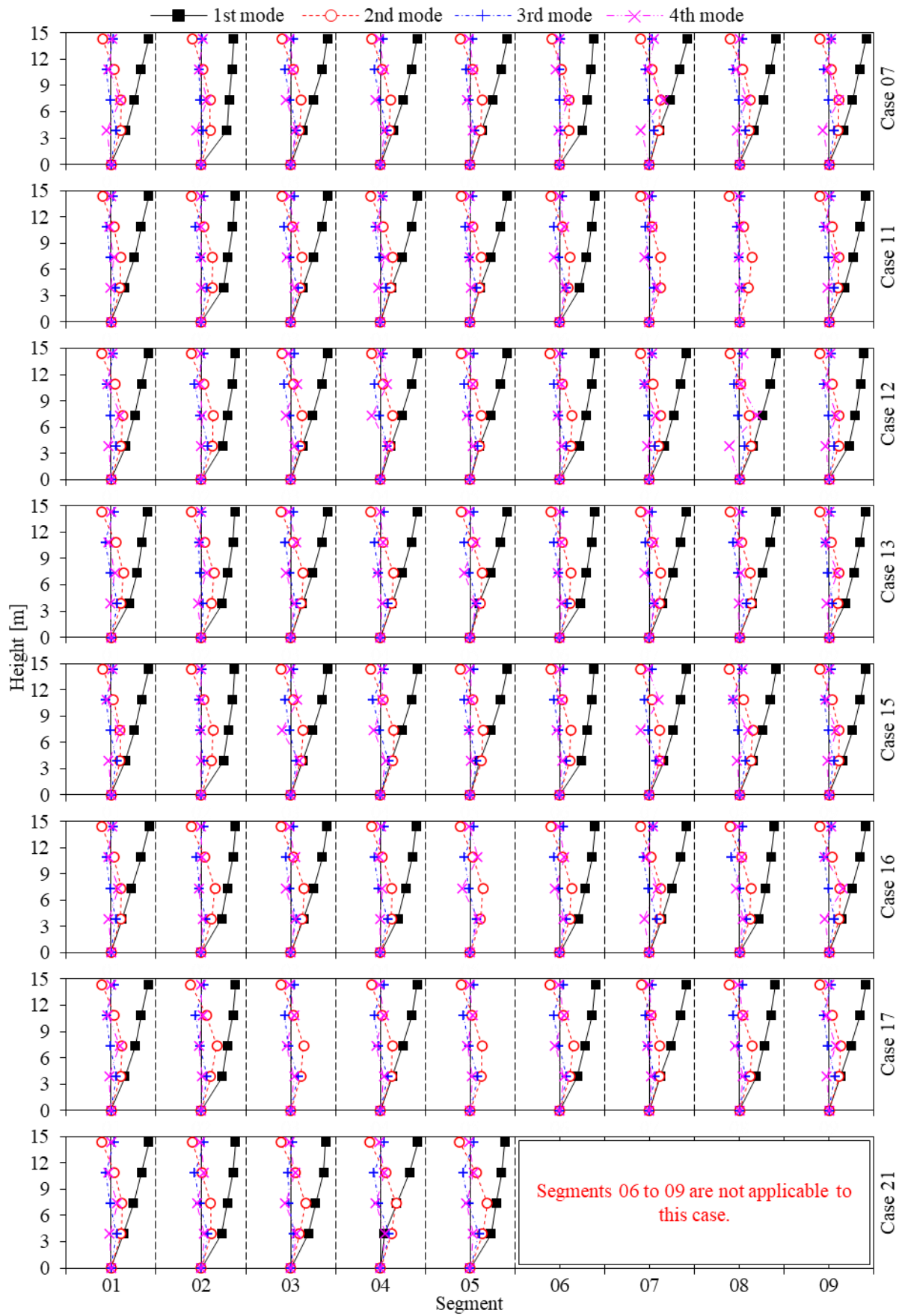


Figure G.9 Mode shapes in ground motion excitations of the 07EDF test (FDD)

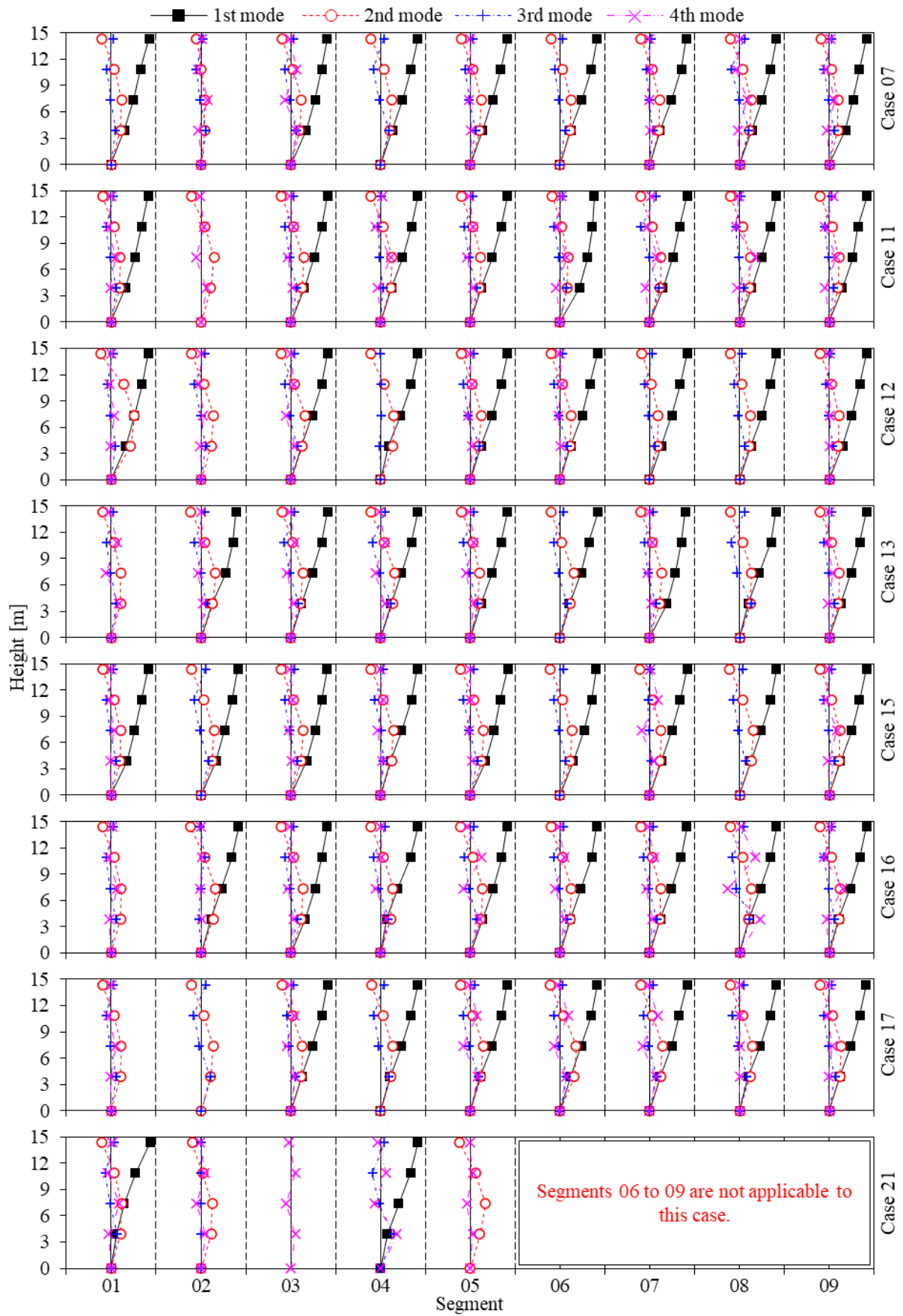


Figure G.10 Mode shapes in ground motion excitations of the 07EDF test (SS)

G.3.4 Modal Effective Mass (MEM)

Figure G.11 plots the natural period in the ground motion cases of the 07EDF test estimated using the ARX, FDD and SS methods, respectively. Table G.7 summarizes the corresponding values in each segment.

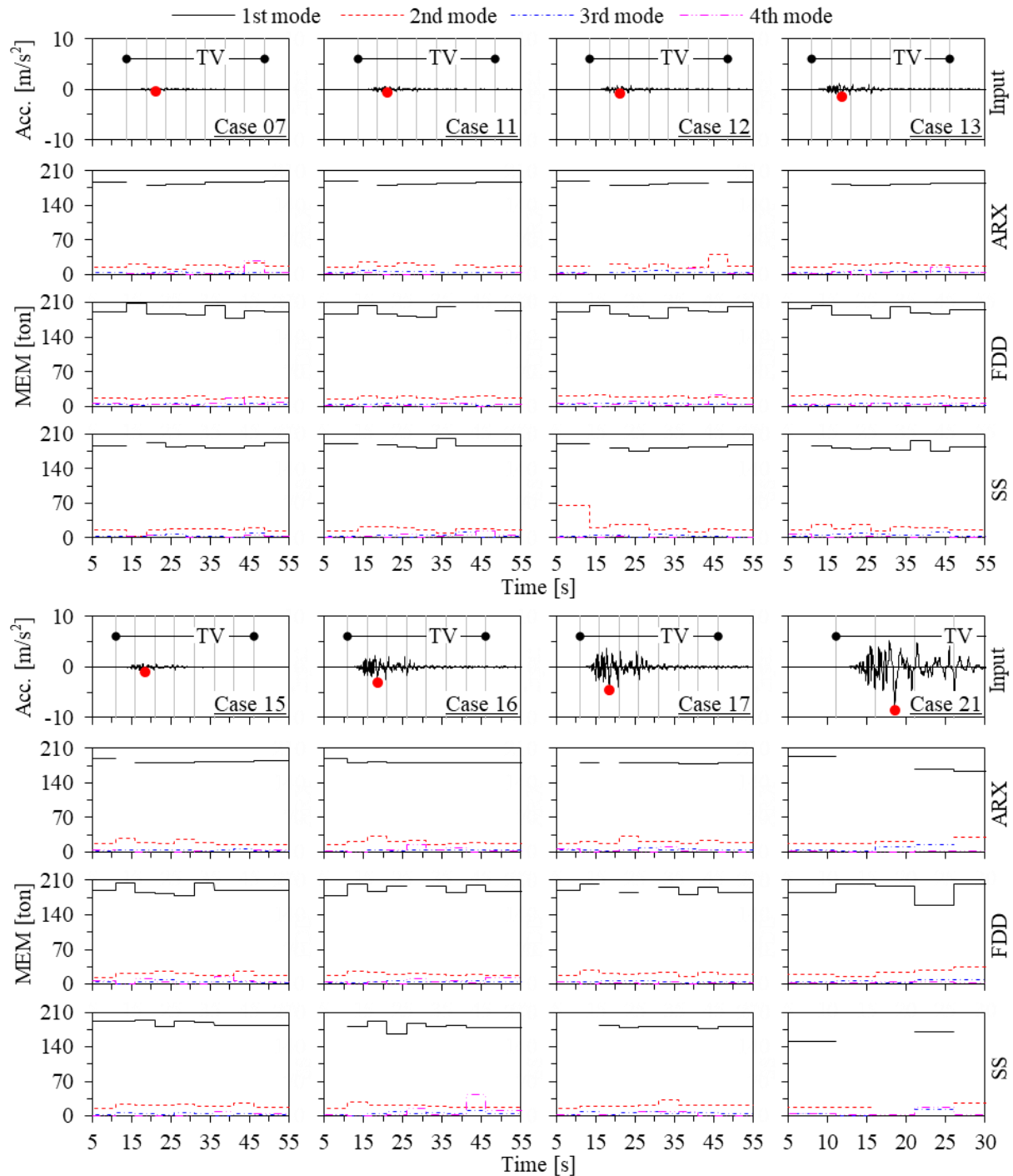


Figure G.11 MEMs in ground motion excitations of the 07EDF test

Table G.7 MEMs in ground motion excitations of the 07EDF test [Unit: ton]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
07-01	187.1	189.8	186.7	15.0	16.2	17.0	2.9	2.9	2.3		5.7	
07-02		207.4		21.3	14.0	2.7	1.8	0.6	2.8	1.1	3.5	3.2
07-03	181.3	186.8	192.8	14.8	15.8	16.4	4.1	4.4	4.5	0.5	2.7	5.8
07-04	182.1	187.3	184.3	9.2	15.6	18.0	4.8	5.4	7.5	0.6	2.9	
07-05	183.0	185.2	186.0	19.7	20.1	17.9	3.1	2.7	3.6	0.1	1.3	0.2
07-06	185.9	204.7	181.9	18.0	14.2	18.2	2.4	0.0	3.5	2.4	4.7	
07-07	186.4	178.7	181.0	15.2	17.0	16.0	1.1	2.7	1.4	5.6	16.3	0.0
07-08	187.4	191.9	186.4	24.0	19.2	19.9	3.1	5.0	10.4	28.3	3.7	4.3
07-09	188.9	190.5	193.1	16.6	15.5	13.3	2.8	3.2	3.0	4.0	8.1	2.5
11-01	187.8	187.2	189.6	14.4	14.3	13.9	2.9	2.4	3.0	2.5	0.7	1.8
11-02		204.1		24.1	20.7	22.5	7.8	3.6		4.6	0.0	3.6
11-03	181.1	185.7	188.5	16.6	17.2	22.9	4.8	5.9	4.5	0.1	2.1	0.9
11-04	181.4	183.2	183.1	22.8	20.6	19.6	5.9	3.9	0.8	0.3	2.1	7.4
11-05	181.9	179.4	181.8	17.7	17.1	17.0	5.7	4.4	4.8	0.2	0.0	1.4
11-06	184.5	201.0	201.1		14.4	10.5	2.8	4.9	4.3		4.6	3.6
11-07	185.3		187.0	18.9	18.2	19.0	2.6	2.4	11.3		4.9	4.5
11-08	185.7		185.4	14.0	21.1	17.6	3.4	0.6	2.0		0.0	14.8
11-09	187.4	192.0	186.9	15.8	16.0	15.1	2.9	3.1	2.4	2.8	3.2	5.9
12-01	189.6	190.1	190.0	17.2	19.6	65.1	2.6	2.4	2.2	2.4	5.3	0.9
12-02		202.8			22.2	21.3		5.3	4.7		0.1	0.5
12-03	180.8	185.6	181.6	19.9	18.4	26.2	4.7	6.0	5.0		4.3	2.6
12-04	180.7	182.0	175.0	13.1	18.5	25.8	6.4	5.1	0.1		9.2	
12-05	182.7	177.9	182.3	20.2	18.6	15.5	7.0	6.1	8.2		1.0	0.6
12-06	183.6	200.0	182.4	12.0	21.3	18.4	4.3	4.5	5.8		0.5	0.2
12-07	183.8	192.0	184.0	15.2	17.3	11.8	3.1	3.3	3.5	11.4	4.0	
12-08		189.9	184.1	39.9	18.3	17.4	2.6	3.8	3.8		23.3	
12-09	186.5	200.9	187.5	16.0	16.9	15.4	2.8	3.2	1.5	2.2	3.9	0.0
13-01		198.0		14.9	21.1	15.6	3.0	4.3	3.1	1.1	1.1	7.0
13-02		203.0	185.9	14.9	22.8	27.4	1.8	0.7	4.7	2.3	2.4	1.0
13-03	181.5	183.9	181.8	20.6	21.5	19.1	3.9	3.7	6.5	5.4	3.7	1.8
13-04	180.7	183.3	180.6	18.8	22.8	26.4	8.7	5.8	10.4	0.2	1.1	3.6
13-05	181.0	177.7	182.6	21.8	20.4	13.3	3.8	5.1	6.9	4.1	5.5	2.5
13-06	182.5	202.5	178.2	23.6	17.4	21.8	5.8	6.0	5.4		0.6	
13-07	183.3	187.9	196.7	18.9	18.2	19.5	5.1	2.7	4.3	2.9	3.7	0.6
13-08	183.5	187.4	175.6	19.8	20.6	20.8	5.0	5.4	12.8	14.1	0.2	
13-09	183.9	194.5	183.5	16.5	15.9	16.0	3.0	1.8	3.3	3.6	4.7	0.9
15-01	189.5	189.1	191.9	16.2	14.0	14.8	3.1	3.7	3.0	4.6	5.9	0.7
15-02		204.8	191.1	27.6	21.5	23.3	3.9	0.8	7.1		0.0	
15-03	180.9	184.9	193.3	19.3	20.9	20.5	4.7	4.4	5.3	1.1	10.5	0.5
15-04	180.8	182.1	180.0	16.3	25.0	22.2	4.4	7.7	2.8		4.1	0.8
15-05	181.0	179.2	191.0	26.0	22.4	23.7	4.5	5.1	5.9	0.8	0.1	0.2
15-06	182.5	203.5	189.2	19.7	17.9	20.4	1.7	4.9	4.5	1.6	0.9	
15-07	183.8	189.6	183.5	15.4	16.6	19.9	4.0	3.8	0.3		14.5	8.5
15-08	183.3	188.5	182.8	15.8	25.2	25.2	6.2	4.6	4.5		7.4	
15-09	184.9	189.1	183.4	16.0	16.7	17.6	3.0	3.0	3.0	3.8	3.8	3.9
16-01	189.4	179.8		15.4	16.3	16.0	2.7	2.9	2.8	2.2	3.7	2.6

16-02	180.4	201.9	180.2	22.2	25.1	27.4	2.8	1.0	0.0	0.5	0.1
16-03	183.0	187.9	191.4	32.6	24.6	20.8	4.2	3.7	4.5	3.6	1.0
16-04	180.4	197.1	166.3	21.3	19.7	20.6	2.0	5.8	7.8	4.0	0.4
16-05	180.9		186.5	23.4	21.7	20.9	5.1	5.0	5.1	14.7	10.2
16-06	181.0	198.6	180.3	15.5	19.3	18.5	5.1	4.7	6.0	4.9	3.5
16-07	180.3	185.3	182.1	17.1	18.2	17.4	5.1	5.4	5.4	9.0	6.2
16-08	181.4	201.0	179.2	19.5	19.8	17.9	5.1	7.5	10.1	0.5	1.9
16-09	181.0	187.3	179.2	17.8	18.1	18.1	3.5	3.7	3.6	2.4	12.6
17-01		189.3		16.9	17.9	16.1	3.7	3.4	3.0	6.5	4.9
17-02	181.7	202.5		21.0	27.6	19.7	3.2	4.2	9.2	0.3	0.5
17-03			182.9	18.3	21.5	19.4	2.9	6.2	2.4	1.2	2.2
17-04	180.7	185.5	179.4	33.1	20.2	19.4	4.7	5.9	7.6	4.1	2.2
17-05	181.3		181.0	20.6	20.7	22.2	8.7	6.8	8.2	0.0	1.0
17-06	181.6	196.7	181.4	20.7	23.0	32.5	5.2	5.3	4.7	11.8	3.3
17-07	179.7	181.7	181.3	18.0	15.4	21.4	5.4	3.0	5.5	1.6	0.6
17-08	179.5	196.1	176.8	22.7	22.7	21.5	4.1	4.6	6.5	3.7	2.7
17-09	180.1	184.2	181.3	19.6	19.4	20.8	3.7	3.9	4.1	0.9	3.0
21-01	193.6	185.3	151.7	17.3	18.4	16.8	4.1	3.8	4.5	2.9	2.6
21-02		202.1		17.4	15.7	17.9	2.3	5.6	0.0	0.4	1.9
21-03		197.3		20.5	24.4		11.0	1.2		0.9	4.8
21-04	168.4	159.0	169.6		29.0		14.1	7.6	13.9	1.6	3.5
21-05	163.8	202.1		29.2	33.8	26.8		8.3		2.3	1.8

G.3.5 PSD peaks

Figure G.12 plots the peak values of the Power Spectra Density (PSD) in the white noise cases of the 07EDF test identified using the FDD method. Table G.8 summarizes the corresponding values.

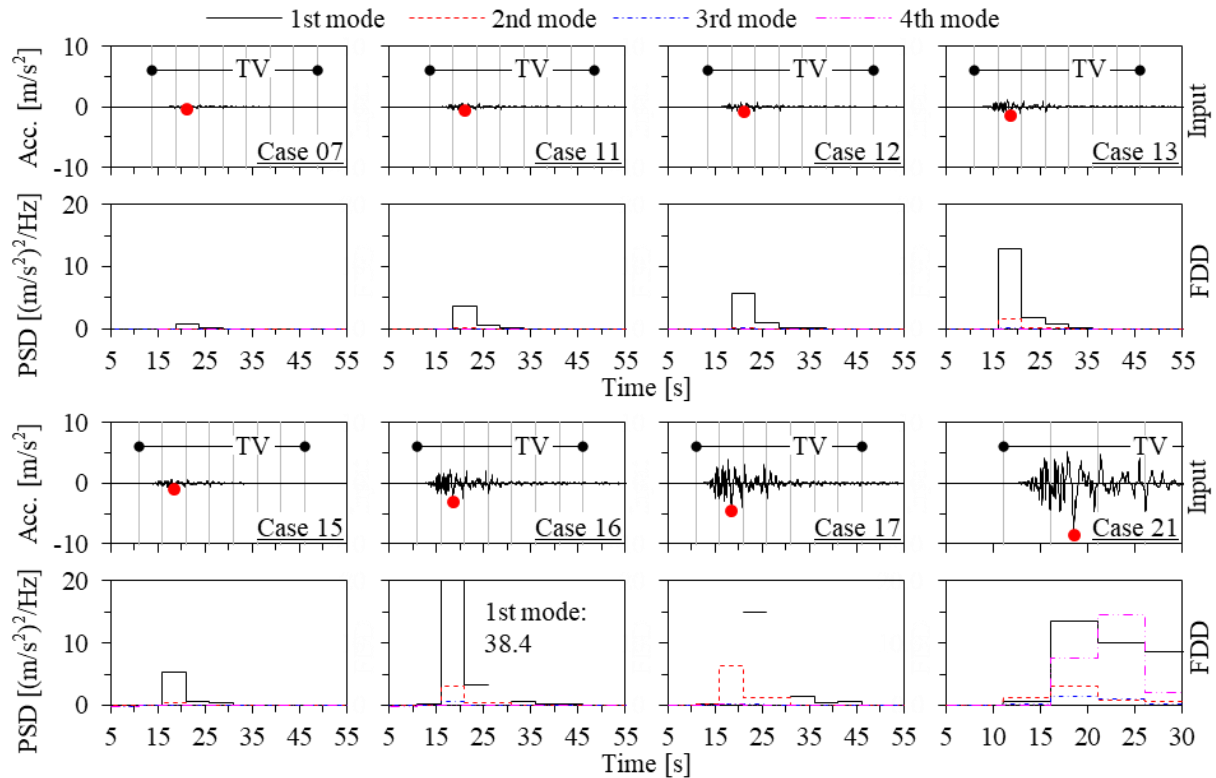


Figure G.12 PSD peaks in ground motion excitations of the 07EDF test

Table G.8 PSD peaks in ground motion excitations of the 07EDF test [Unit: (m/s²)²/Hz]

Case-Segment	Mode			
	1st	2nd	3rd	4th
07-01	0.0	0.0	0.0	0.0
07-02	0.0	0.0	0.0	0.0
07-03	0.7	0.0	0.0	0.0
07-04	0.1	0.0	0.0	0.0
07-05	0.0	0.0	0.0	0.0
07-06	0.0	0.0	0.0	0.0
07-07	0.0	0.0	0.0	0.0
07-08	0.0	0.0	0.0	0.0
07-09	0.0	0.0	0.0	0.0
11-01	0.0	0.0	0.0	0.0
11-02	0.0	0.0	0.0	0.0
11-03	3.6	0.0	0.0	0.0
11-04	0.5	0.0	0.0	0.0
11-05	0.1	0.0	0.0	0.0
11-06	0.0	0.0	0.0	0.0
11-07		0.0	0.0	0.0
11-08		0.0	0.0	0.0
11-09	0.0	0.0	0.0	0.0
12-01	0.0	0.0	0.0	0.0
12-02	0.0	0.0	0.0	0.0
12-03	5.7	0.1	0.1	0.0
12-04	0.9	0.0	0.0	0.0
12-05	0.1	0.0	0.0	0.0
12-06	0.0	0.0	0.0	0.0
12-07	0.0	0.0	0.0	0.0
12-08	0.0	0.0	0.0	0.0
12-09	0.0	0.0	0.0	0.0
13-01	0.0	0.0	0.0	0.0
13-02	0.0	0.0	0.0	0.0
13-03	12.9	1.5	0.2	0.0
13-04	1.7	0.1	0.0	0.0
13-05	0.8	0.1	0.0	0.0
13-06	0.1	0.0	0.0	0.0
13-07	0.0	0.0	0.0	0.0
13-08	0.0	0.0	0.0	0.0
13-09	0.0	0.0	0.0	0.0
15-01	0.0	0.0	0.0	0.0
15-02	0.0	0.0	0.0	0.0
15-03	5.3	0.5	0.0	0.0
15-04	0.7	0.0	0.0	0.0
15-05	0.4	0.0	0.0	0.0
15-06	0.0	0.0	0.0	0.0
15-07	0.0	0.0	0.0	0.0
15-08	0.0	0.0	0.0	0.0
15-09	0.0	0.0	0.0	0.0

16-01	0.0	0.0	0.0	0.0
16-02	0.1	0.1	0.0	0.0
16-03	38.4	3.1	0.6	0.0
16-04	3.3	0.4	0.0	0.0
16-05		0.3	0.0	0.0
16-06	0.5	0.0	0.0	0.0
16-07	0.2	0.0	0.0	0.0
16-08	0.1	0.0	0.0	0.0
16-09	0.0	0.0	0.0	0.0
17-01	0.0	0.0	0.0	0.0
17-02	0.3	0.2	0.1	0.0
17-03		6.4	0.2	0.1
17-04	14.9	1.3	0.1	0.1
17-05		1.2	0.1	0.0
17-06	1.5	0.1	0.0	0.0
17-07	0.3	0.1	0.0	0.0
17-08	0.5	0.1	0.0	0.0
17-09	0.1	0.0	0.0	0.0
21-01	0.0	0.0	0.0	0.0
21-02	0.6	1.3	0.2	0.1
21-03	13.6	3.1	1.4	7.6
21-04	10.1	0.9	1.0	14.6
21-05	8.6	0.6	0.2	2.0

G.4 EFFECTIVE MODAL PSEUDO-ACCELERATION (EMPA)

Figure G.13 plots the peak EMPA values identified in the ground motion cases of the 07EDF test, including the elastic pseudo-acceleration spectra with 2% damping of the measured excitations.

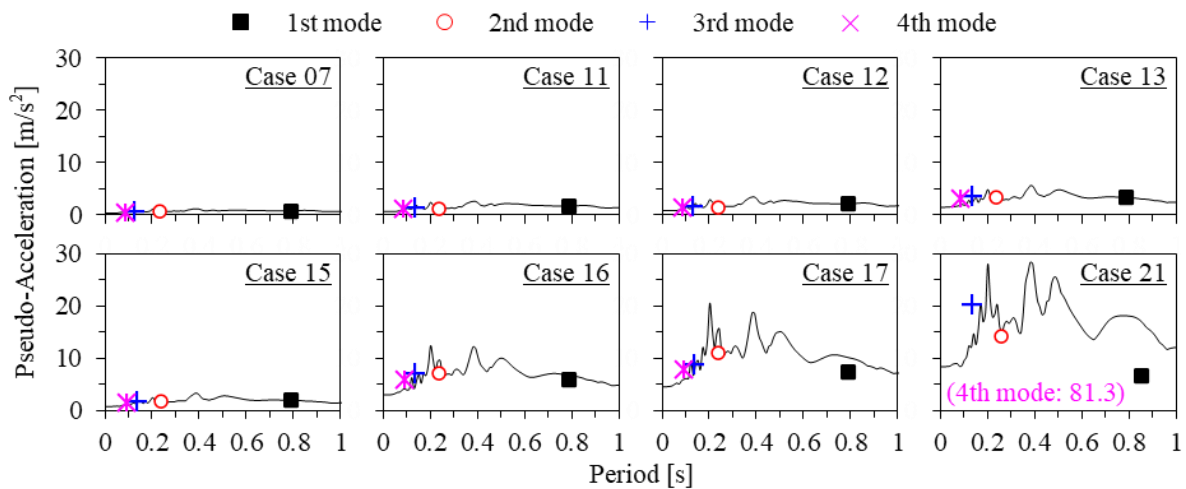


Figure G.13 Peak EMPA values in ground motion excitations of the 07EDF test

G.5 TIME-FREQUENCY RESULTS FROM GROUND MOTION EXCITATIONS

G.5.1 Magnitude of wavelet coefficients and instantaneous periods

Figure G.14 plots the magnitude of wavelet coefficients on the time-frequency plane and the identified instantaneous periods in the ground motion cases of the 07EDF test.

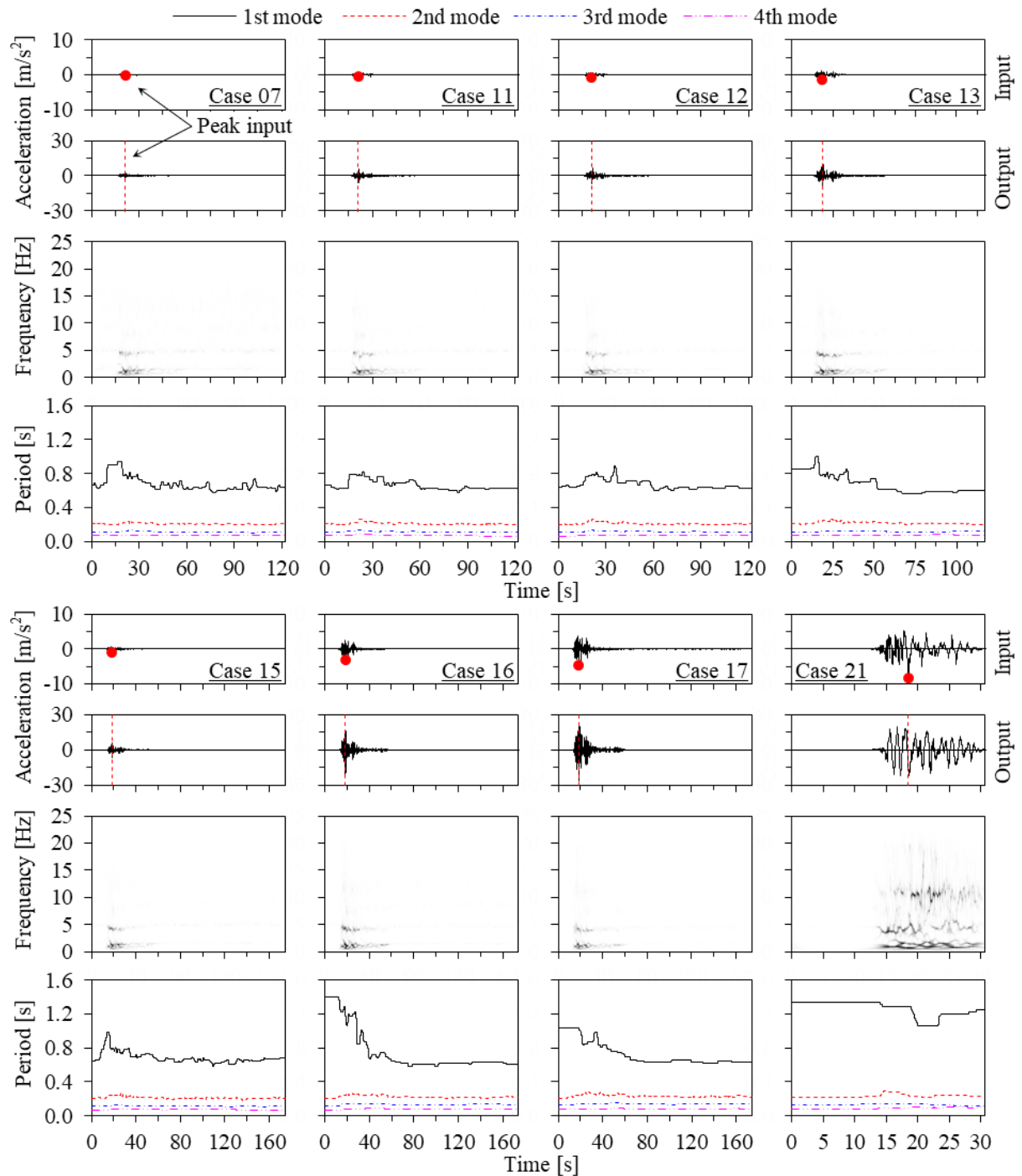


Figure G.14 Magnitude of wavelet coefficients in ground motion excitations of the 07EDF test

G.5.2 Magnitude of ridge wavelet coefficient

Figure G.15 plots the magnitude of wavelet coefficient on the wavelet ridges in the ground motion cases of the 07EDF test.

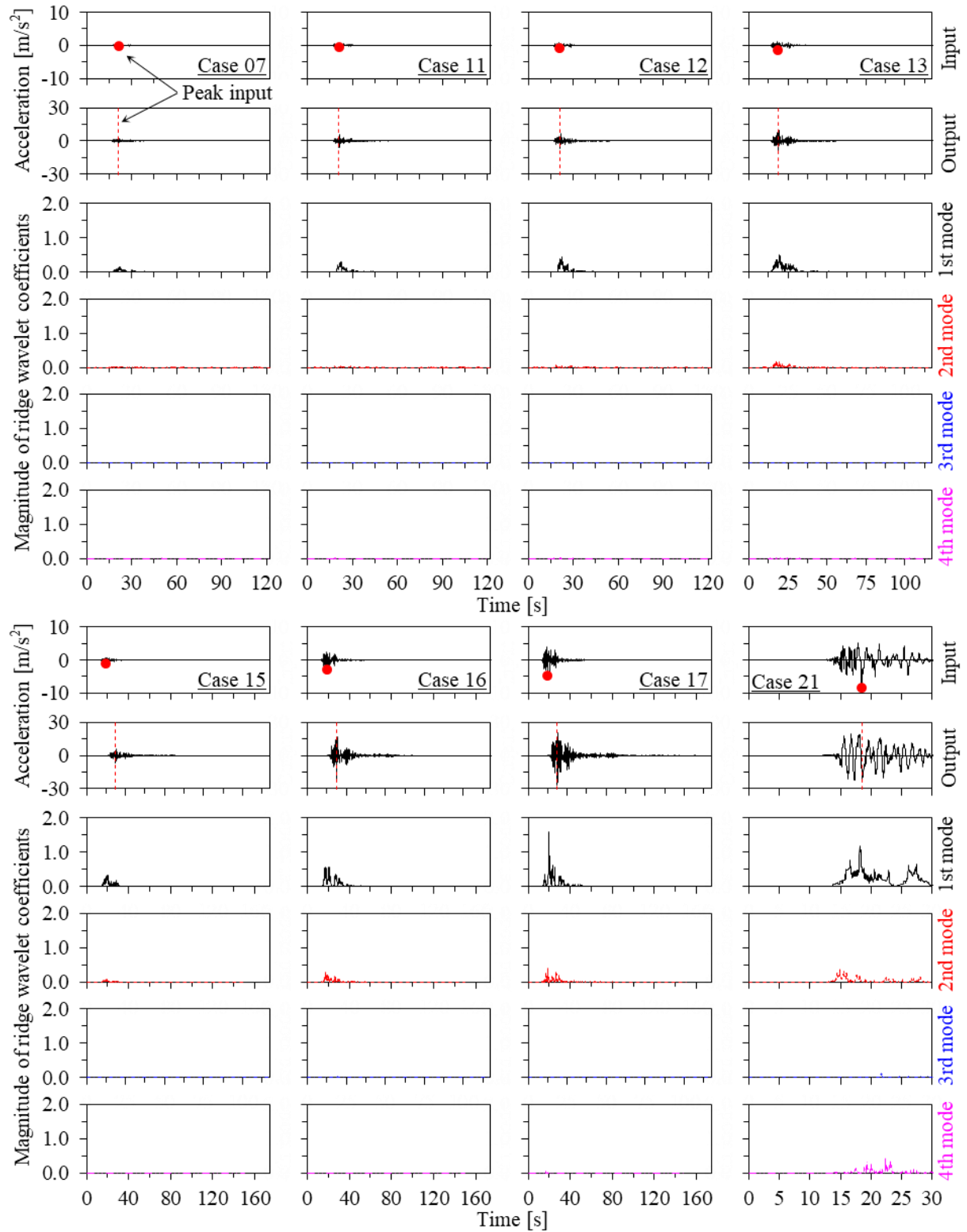


Figure G.15 Magnitude on wavelet ridges in ground motion excitations of the 07EDF test

G.5.3 Reconstructed modal components

Figure G.16 plots the time-domain modal components reconstructed from the SSWT results and ridge identification in the ground motion cases of the 07EDF test.

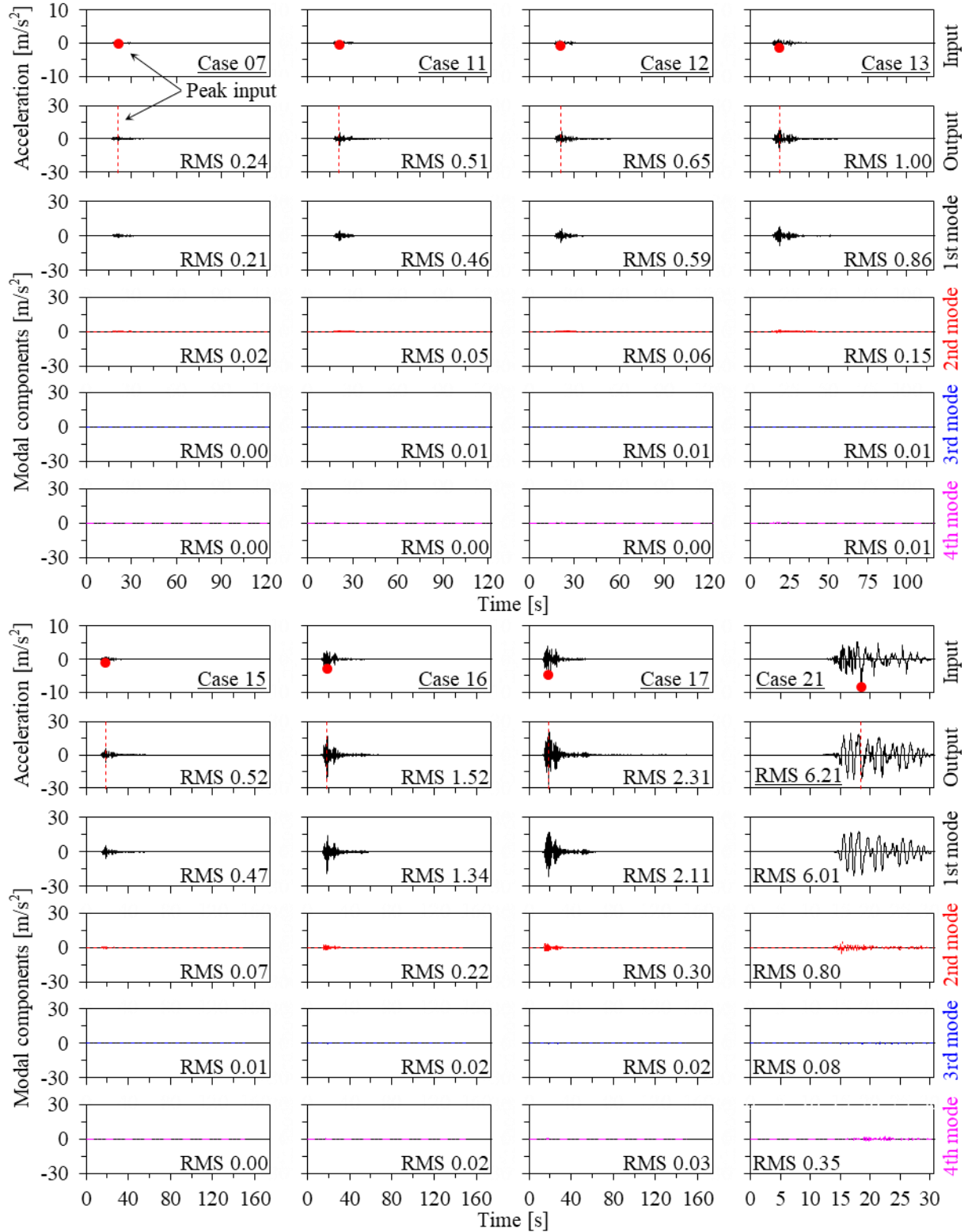


Figure G.16 Modal components in ground motion excitations of the 07EDF test

APPENDIX H. FULL RESULTS OF VIBRATION PROPERTY ANALYSIS: 08EDF

H.1 GENERAL

This appendix provides results of the vibration property analysis on the 08EDF test. The results are grouped by different tests, each including results from all analysis methods. The following results are provided in this appendix:

- 1) **Modal parameters:** natural periods, modal damping ratios, mode shapes, Modal Effective Mass (MEM) and PSD peaks from modal analysis methods.
- 2) **Spectral results:** peak values of Effective Modal Pseudo-Acceleration (EMPA).
- 3) **Time-frequency results:** magnitude of wavelet coefficients, instantaneous periods, magnitude of ridge wavelet coefficient and reconstructed modal components.

In the 08EDF test, 15 white noise and 13 ground motion cases in the East-West direction were selected for analysis. The structural system was not changed during the test.

H.2 MODAL PARAMETERS FROM WHITE NOISE EXCITATIONS

H.2.1 Natural period

Figure H.1 plots the natural period in the white noise cases of the 08EDF test identified using the ARX, FDD and SS methods, respectively. Table H.1 summarizes the corresponding values.

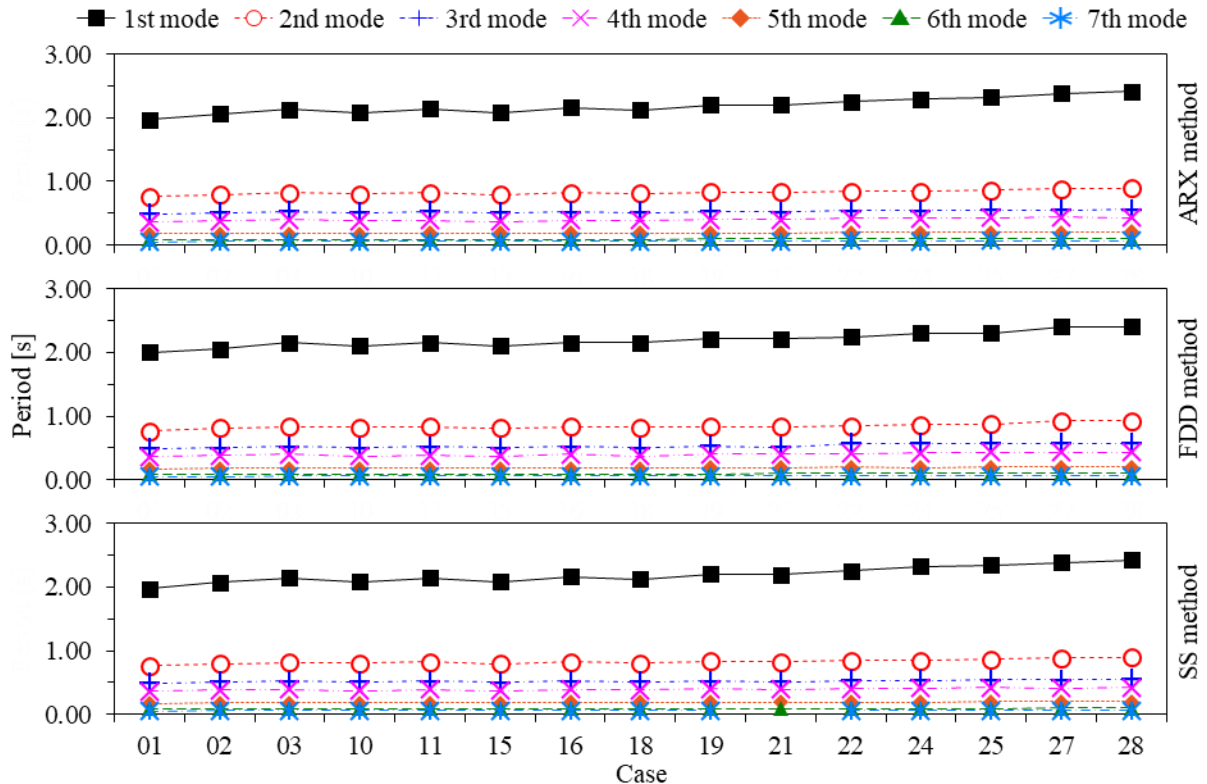


Figure H.1 Natural periods in white noise excitations of the 08EDF test

Table H.1 Natural periods in white noise excitations of the 08EDF test [Unit: s]

(a) 1st to 4th modes

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	1.97	2.00	1.97	0.76	0.77	0.76	0.49	0.49	0.49	0.36	0.36	0.36
02	2.06	2.05	2.06	0.79	0.82	0.79	0.51	0.50	0.51	0.38	0.38	0.38
03	2.13	2.16	2.13	0.82	0.83	0.82	0.53	0.53	0.53	0.40	0.40	0.40
10	2.08	2.10	2.08	0.80	0.83	0.80	0.51	0.50	0.51	0.38	0.37	0.38
11	2.14	2.16	2.14	0.82	0.83	0.82	0.53	0.52	0.53	0.39	0.38	0.39
15	2.08	2.10	2.08	0.80	0.82	0.80	0.51	0.50	0.51	0.37	0.38	0.37
16	2.16	2.16	2.16	0.82	0.83	0.82	0.52	0.52	0.52	0.39	0.40	0.39
18	2.12	2.16	2.12	0.81	0.83	0.81	0.51	0.50	0.51	0.39	0.38	0.39
19	2.21	2.21	2.21	0.83	0.84	0.83	0.53	0.54	0.53	0.40	0.40	0.40
21	2.20	2.21	2.20	0.83	0.83	0.83	0.52	0.51	0.52	0.40	0.40	0.40
22	2.26	2.24	2.26	0.85	0.85	0.85	0.54	0.57	0.54	0.42	0.41	0.42
24	2.29	2.31	2.29	0.85	0.87	0.85	0.54	0.57	0.54	0.42	0.42	0.42
25	2.32	2.31	2.32	0.86	0.88	0.86	0.55	0.57	0.55	0.43	0.43	0.43
27	2.38	2.41	2.38	0.88	0.92	0.88	0.55	0.57	0.55	0.44	0.43	0.44
28	2.42	2.41	2.42	0.89	0.92	0.89	0.56	0.57	0.56	0.43	0.43	0.43

(b) 5th to 7th modes

Case	5th mode			6th mode			7th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.17	0.17	0.17	0.09	0.09	0.09	0.06	0.06	0.06
02	0.18	0.18	0.18	0.09	0.09	0.09	0.06	0.06	0.06
03	0.18	0.18	0.18	0.09	0.09	0.09	0.06	0.06	0.06
10	0.18	0.18	0.18	0.09	0.09	0.09	0.06	0.06	0.06
11	0.19	0.18	0.19	0.09	0.09	0.09	0.06	0.06	0.06
15	0.19	0.18	0.19	0.09	0.09	0.09	0.06	0.06	0.06
16	0.19	0.18	0.19	0.09	0.09	0.09	0.06	0.06	0.06
18	0.19	0.18	0.19	0.09	0.09	0.09	0.06	0.06	0.06
19	0.19	0.19	0.19	0.09	0.09	0.09	0.06	0.06	0.06
21	0.19	0.19	0.19	0.10	0.09	0.09	0.06	0.06	
22	0.20	0.20	0.20	0.10	0.09	0.10	0.06	0.06	0.06
24	0.19	0.19	0.19	0.10	0.09	0.10	0.06	0.06	0.06
25	0.20	0.20	0.20	0.10	0.09	0.10	0.06	0.06	0.06
27	0.20	0.20	0.20	0.10	0.10	0.10	0.06	0.06	0.06
28	0.21	0.20	0.20	0.10	0.10	0.10	0.06	0.06	0.06

H.2.2 Modal damping ratio

Figure H.2 plots the modal damping ratios in the white noise cases of the 08EDF test identified using the ARX, FDD and SS methods, respectively. Table H.2 summarizes the corresponding values.

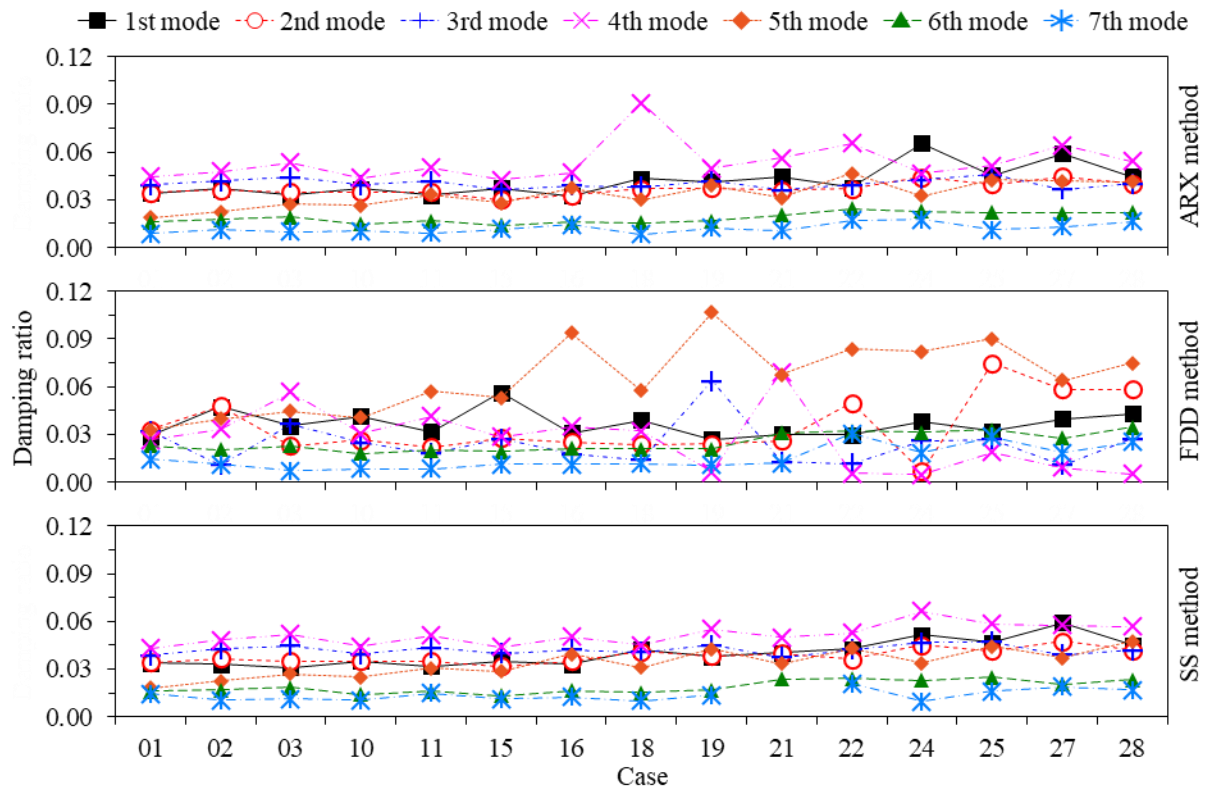


Figure H.2 Damping ratios in white noise excitations of the 08EDF test

Table H.2 Damping ratios in white noise excitations of the 08EDF test
(a) 1st to 4th modes

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.03	0.04	0.04	0.03	0.04
02	0.04	0.05	0.03	0.04	0.05	0.04	0.04	0.01	0.04	0.05	0.03	0.05
03	0.03	0.04	0.03	0.03	0.02	0.04	0.04	0.04	0.04	0.05	0.06	0.05
10	0.04	0.04	0.04	0.03	0.03	0.04	0.04	0.02	0.04	0.04	0.03	0.04
11	0.03	0.03	0.03	0.04	0.02	0.04	0.04	0.02	0.04	0.05	0.04	0.05
15	0.04	0.06	0.03	0.03	0.03	0.03	0.04	0.03	0.04	0.04	0.03	0.04
16	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.02	0.04	0.05	0.03	0.05
18	0.04	0.04	0.04	0.04	0.02	0.04	0.04	0.01	0.04	0.09	0.03	0.05
19	0.04	0.03	0.04	0.04	0.02	0.04	0.04	0.06	0.04	0.05	0.01	0.06
21	0.04	0.03	0.04	0.04	0.03	0.04	0.04	0.01	0.04	0.06	0.07	0.05
22	0.04	0.03	0.04	0.04	0.05	0.04	0.04	0.01	0.04	0.07	0.01	0.05
24	0.07	0.04	0.05	0.04	0.01	0.05	0.04	0.03	0.05	0.05	0.00	0.07
25	0.05	0.03	0.05	0.04	0.07	0.04	0.05	0.03	0.05	0.05	0.02	0.06
27	0.06	0.04	0.06	0.04	0.06	0.05	0.04	0.01	0.04	0.06	0.01	0.06
28	0.04	0.04	0.05	0.04	0.06	0.04	0.04	0.03	0.04	0.05	0.01	0.06

(b) 5th to 7th modes

Case	5th mode			6th mode			7th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.02	0.03	0.02	0.02	0.02	0.02	0.01	0.01	0.01
02	0.02	0.04	0.02	0.02	0.02	0.02	0.01	0.01	0.01
03	0.03	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01
10	0.03	0.04	0.02	0.01	0.02	0.01	0.01	0.01	0.01
11	0.03	0.06	0.03	0.02	0.02	0.02	0.01	0.01	0.02
15	0.03	0.05	0.03	0.01	0.02	0.01	0.01	0.01	0.01
16	0.04	0.09	0.04	0.02	0.02	0.02	0.01	0.01	0.01
18	0.03	0.06	0.03	0.02	0.02	0.02	0.01	0.01	0.01
19	0.04	0.11	0.04	0.02	0.02	0.02	0.01	0.01	0.01
21	0.03	0.07	0.03	0.02	0.03	0.02	0.01	0.01	
22	0.05	0.08	0.04	0.02	0.03	0.02	0.02	0.03	0.02
24	0.03	0.08	0.03	0.02	0.03	0.02	0.02	0.02	0.01
25	0.04	0.09	0.04	0.02	0.03	0.03	0.01	0.03	0.02
27	0.04	0.06	0.04	0.02	0.03	0.02	0.01	0.02	0.02
28	0.04	0.07	0.05	0.02	0.03	0.02	0.02	0.03	0.02

H.2.3 Mode shapes

Figure H.3 plots the mode shapes in the white noise cases of the 08EDF test identified using the ARX, FDD and SS methods, respectively. The plotted mode shapes are normalized by the modal participation factor Γ_n .

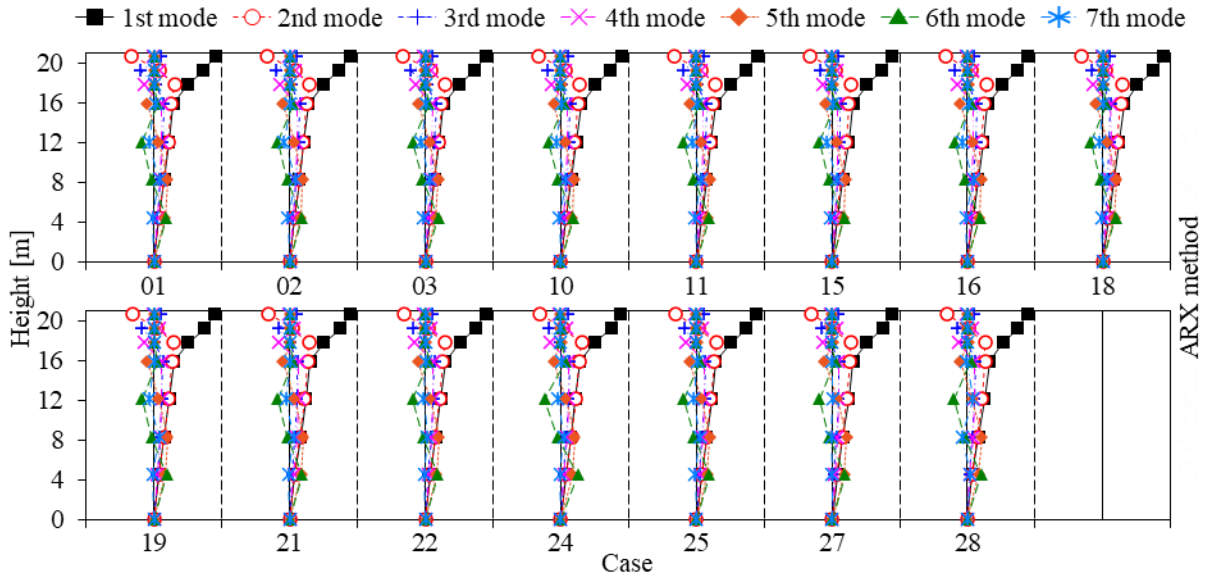


Figure H.3 Mode shapes in white noise excitations of the 08EDF test

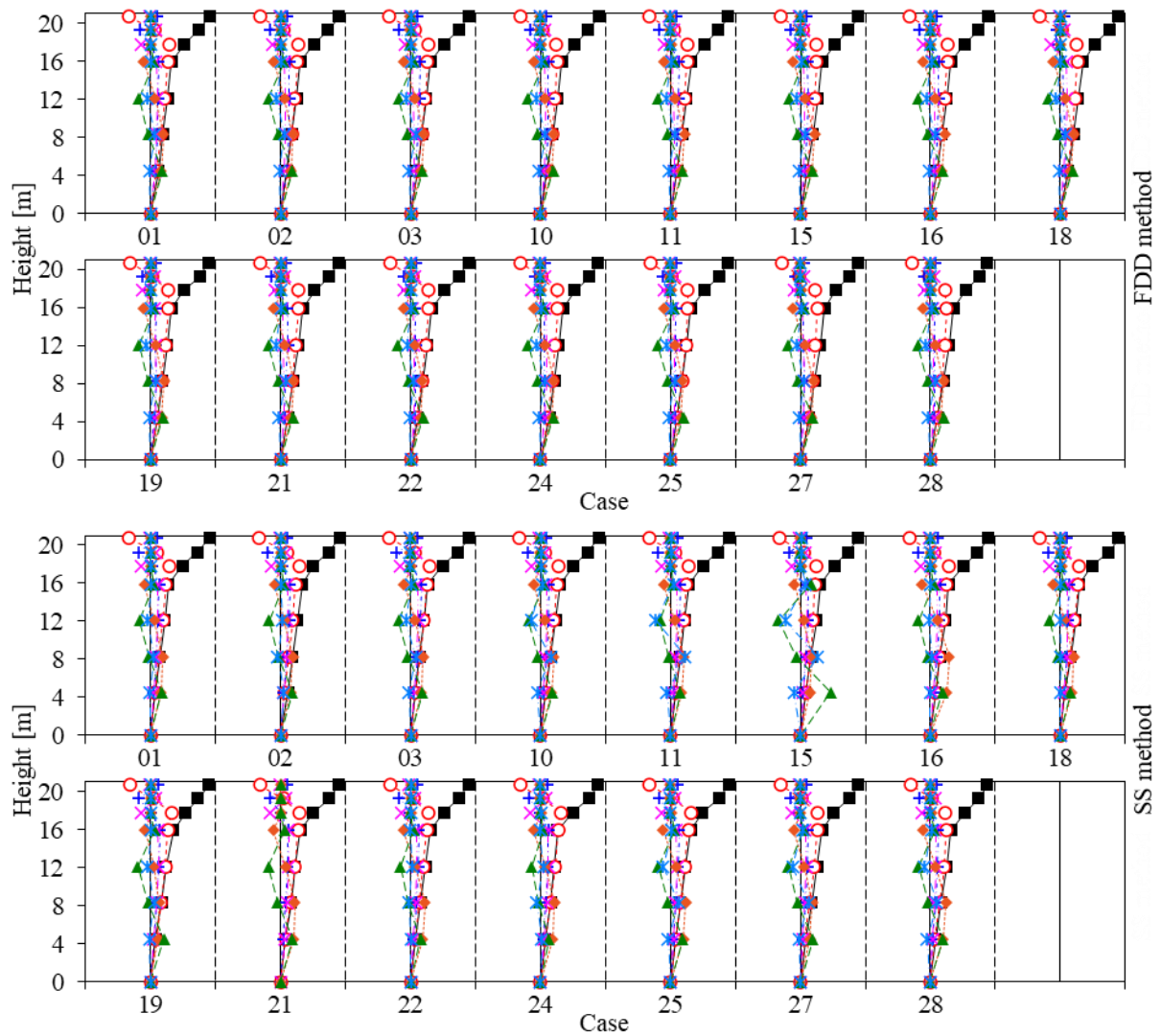


Figure H.3 (continued) Mode shapes in white noise excitations of the 08EDF test

H.2.4 Modal Effective Mass (MEM)

Figure H.4 plots the Modal Effective Mass (MEM) in the white noise cases of the 08EDF test identified using the ARX, FDD and SS methods, respectively. Table H.3 summarizes the corresponding values of the MEMs. The total mass of the structure is 209 tons, with Floor 1 to 7 being 57 tons, 54 tons, 54 tons, 166 tons, 195 tons, 195 tons and 180 tons, respectively.

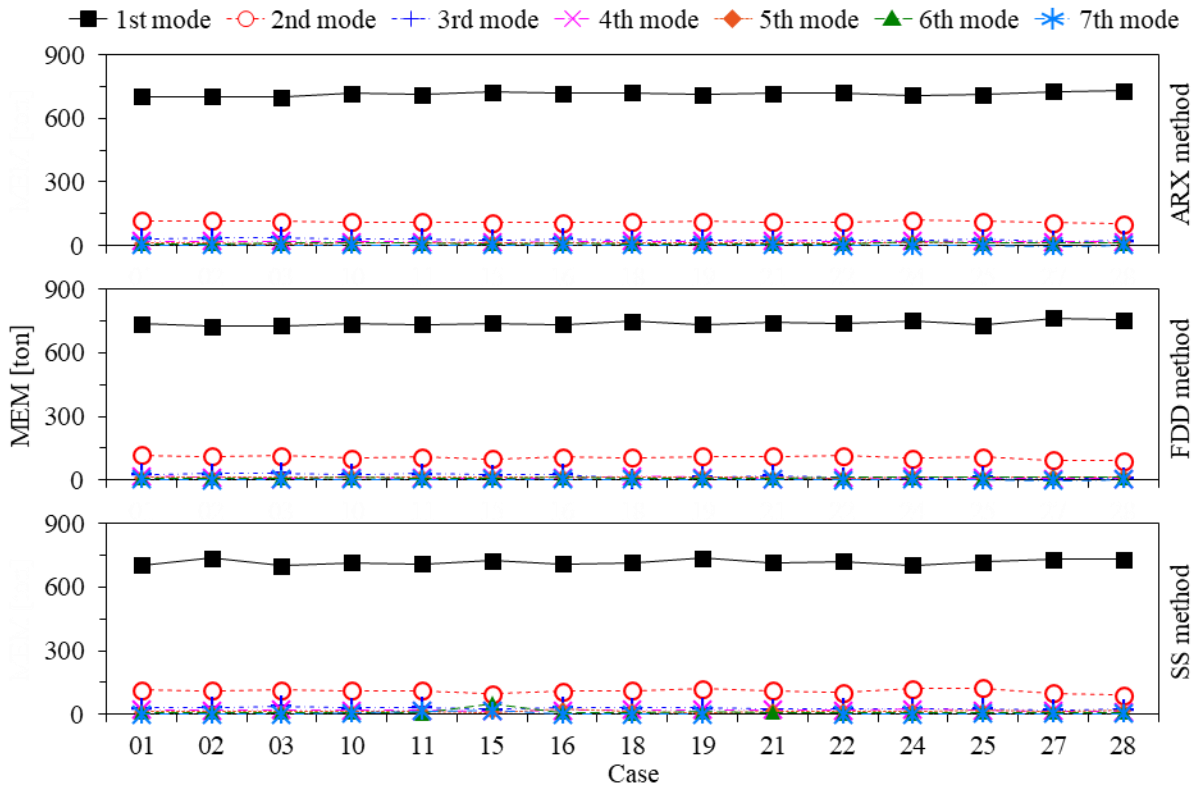


Figure H.4 MEMs in white noise excitations of the 08EDF test

Table H.3 MEMs in white noise excitations of the 08EDF test [Unit: ton]

(a) 1st to 4th modes

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	704.3	735.9	703.7	118.0	118.1	114.3	32.5	26.4	29.6	21.3	17.8	20.9
02	702.9	724.7	736.2	116.2	111.2	112.4	34.8	29.4	33.6	20.2	11.6	19.4
03	699.4	726.9	702.0	115.2	114.8	114.7	37.4	32.9	39.7	19.3	5.2	22.2
10	718.3	736.2	716.9	110.4	102.9	110.4	29.7	25.4	28.7	17.0	10.8	16.7
11	710.8	733.2	708.7	110.9	109.1	111.3	33.6	28.4	32.3	17.7	7.2	17.1
15	725.5	738.7	724.9	108.0	100.5	95.7	26.9	22.3	24.6	15.6	8.8	14.9
16	719.3	734.0	710.2	108.9	107.9	108.4	31.0	28.1	33.1	16.7	9.6	18.1
18	719.7	748.6	714.8	109.8	105.1	113.0	23.2	6.6	29.9	20.3	16.7	21.8
19	713.6	733.3	736.3	113.1	111.8	122.3	27.3	15.3	30.8	20.2	12.4	21.3
21	718.7	743.6	717.3	111.9	109.8	111.5	23.9	20.6	23.6	23.5	6.3	22.6
22	719.8	738.8	722.1	110.8	112.9	102.8	27.7	12.0	27.5	20.8	7.8	17.7
24	707.9	752.9	705.2	119.1	102.4	119.7	27.1	9.1	26.9	20.9	15.2	25.2
25	712.6	730.6	718.8	113.1	108.2	122.6	29.4	16.2	27.2	21.1	9.2	17.2
27	727.5	763.5	731.5	107.4	93.4	99.5	21.1	6.5	21.7	16.7	6.7	13.9
28	731.0	755.7	731.5	102.9	91.4	90.4	21.9	9.9	23.6	16.3	6.6	12.9

(b) 5th to 7th modes

Case	5th mode			6th mode			7th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	12.3	12.4	11.8	9.5	9.3	8.2	1.5	1.1	0.5
02	12.3	11.6	11.3	9.3	9.6	8.9	1.8	1.0	0.7

03	11.8	12.6	12.4	10.1	9.9	9.6	1.1	1.7	1.4
10	13.0	13.4	13.2	9.9	10.2	9.5	1.3	1.2	6.3
11	12.8	14.1	13.5	10.1	10.0	6.6	1.0	1.4	14.1
15	13.6	14.7	10.2	9.6	9.2	52.8	1.4	1.2	18.6
16	13.9	15.0	25.5	9.8	10.2	9.8	1.9	1.2	0.1
18	12.9	13.6	13.9	9.4	9.5	7.4	0.9	1.4	0.0
19	12.5	14.1	6.8	9.6	9.8	11.6	1.4	1.3	0.7
21	13.2	12.6	16.5	9.4	9.8	10.1	0.9	1.6	
22	13.5	11.4	15.4	9.5	10.3	7.0	0.2	0.7	0.1
24	13.7	13.8	15.5	17.5	10.3	5.0	0.5	1.1	0.7
25	13.0	11.9	17.9	10.3	11.0	9.9	0.5	1.1	3.6
27	15.5	13.8	10.9	10.6	10.7	10.4	0.0	0.7	4.1
28	14.2	13.7	18.0	11.7	10.9	10.5	1.9	1.3	2.6

H.2.5 PSD peaks

Figure H.5 plots the peak values of the Power Spectra Density (PSD) in the white noise cases identified using the FDD method. Table H.4 summarizes the corresponding values.

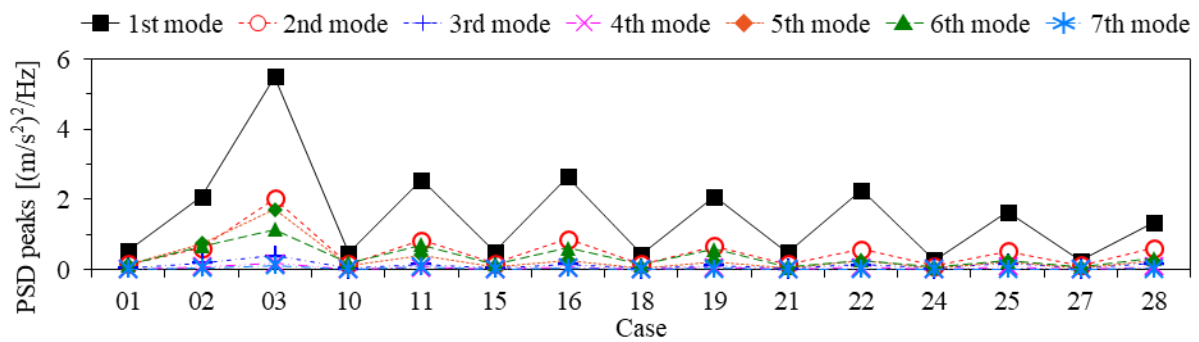


Figure H.5 PSD peaks in white noise excitations of the 08EDF test

Table H.4 PSD peaks in white noise excitations of the 08EDF test [Unit: $(\text{m/s}^2)^2/\text{Hz}$]

Case	Mode						
	1st	2nd	3rd	4th	5th	6th	7th
01	0.550	0.168	0.052	0.021	0.133	0.135	0.009
02	2.073	0.610	0.204	0.090	0.742	0.674	0.051
03	5.506	2.022	0.393	0.173	1.712	1.144	0.128
10	0.472	0.163	0.038	0.014	0.108	0.190	0.013
11	2.550	0.845	0.160	0.063	0.416	0.713	0.079
15	0.514	0.172	0.044	0.014	0.084	0.162	0.011
16	2.642	0.867	0.159	0.063	0.270	0.609	0.053
18	0.442	0.153	0.033	0.016	0.049	0.149	0.012
19	2.080	0.674	0.116	0.068	0.226	0.584	0.038
21	0.492	0.161	0.026	0.012	0.049	0.077	0.007
22	2.263	0.574	0.143	0.071	0.248	0.276	0.021
24	0.270	0.099	0.024	0.012	0.045	0.076	0.007
25	1.640	0.535	0.183	0.079	0.239	0.254	0.022
27	0.245	0.121	0.024	0.014	0.046	0.085	0.007
28	1.348	0.610	0.171	0.087	0.258	0.325	0.028

H.3 MODAL PARAMETERS FROM GROUND MOTION EXCITATIONS

H.3.1 Natural period

Figure H.6 plots the natural period in the ground motion cases of the 08EDF test estimated using the ARX, FDD and SS methods, respectively. Table H.5 summarizes the corresponding values in each segment.

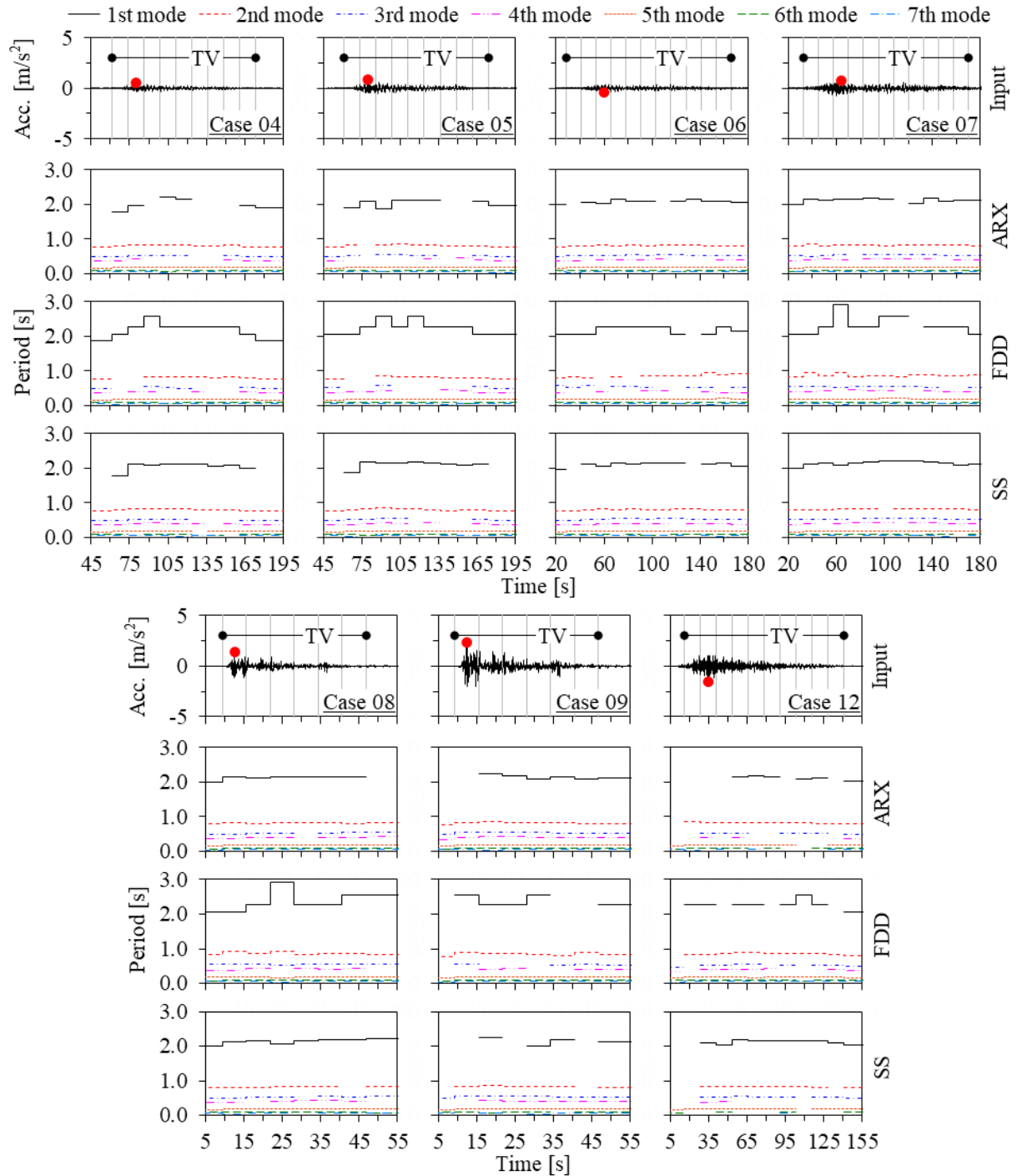


Figure H.6 Natural periods in ground motion excitations of the 08EDF test

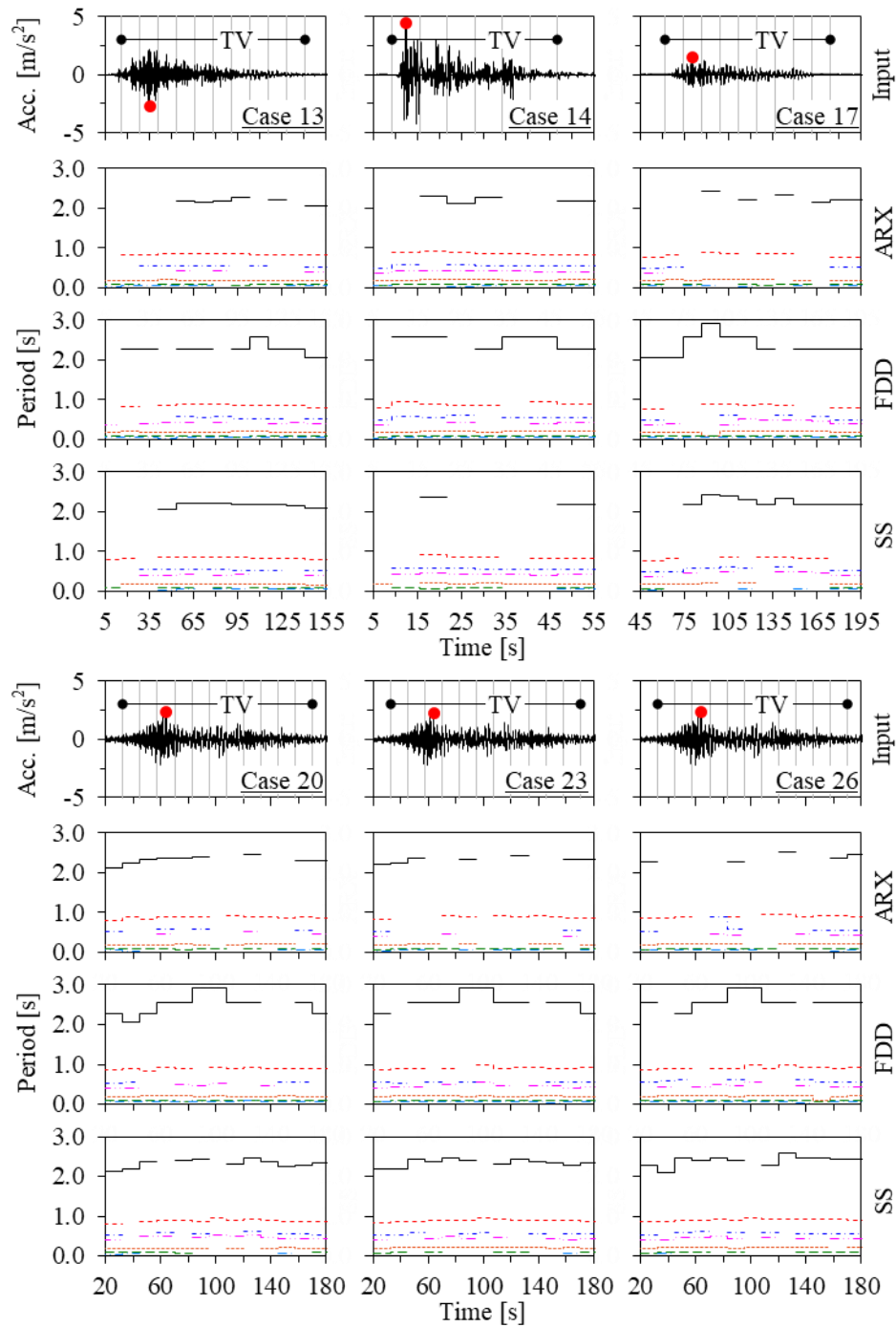


Figure H.6 (continued) Natural periods in ground motion excitations of the 08EDF test

Table H.5 Natural periods in ground motion excitations of the 08EDF test [Unit: s]

Case-Segment	1st mode			2nd mode			3rd mode			4th mode			5th mode			6th mode			7th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01		0.75	0.48	0.35	0.17	0.09	0.06	1.86	0.76	0.48	0.35	0.17	0.09	0.06		0.75	0.47	0.35	0.17	0.09	0.06
04-02	1.79	0.80	0.49	0.37	0.18	0.09	0.06	2.05				0.18	0.09	0.06	1.78	0.80	0.50	0.37	0.18	0.09	0.06
04-03	1.96	0.81	0.52	0.42	0.18	0.09	0.06	2.28			0.40	0.18	0.09	0.06	2.11	0.82	0.51	0.39	0.18		0.06
04-04		0.82	0.53		0.18	0.08	0.06	2.56	0.82	0.54	0.39	0.19	0.08	0.06	2.07	0.82	0.53	0.41	0.18	0.08	0.05
04-05	2.19	0.82	0.53		0.18	0.08	0.05	2.28	0.82	0.53		0.18	0.08	0.06	2.12	0.81	0.52	0.38	0.19	0.08	0.06
04-06	2.13	0.81	0.50		0.18	0.09		2.28	0.82	0.49		0.18	0.09	0.06	2.13	0.81	0.51	0.39	0.18	0.09	0.05
04-07		0.80			0.18	0.09	0.06	2.28	0.82		0.40	0.18	0.09	0.06	2.10	0.80		0.40			0.06
04-08		0.79			0.18	0.09	0.06	2.28	0.79			0.18	0.09	0.06	2.07	0.80			0.18	0.08	0.06
04-09		0.81	0.51	0.38	0.18	0.09	0.06	2.28	0.82			0.18	0.09	0.06	2.09	0.80		0.38	0.18		
04-10	1.97	0.77	0.49	0.37	0.18	0.09	0.06	2.05	0.79	0.50	0.39	0.18	0.09	0.06	1.99	0.77	0.49	0.36	0.18	0.09	0.06
04-11	1.90	0.75	0.48	0.35	0.17	0.09	0.06	1.86	0.76	0.49	0.35	0.17	0.09	0.06		0.75	0.48	0.35	0.17	0.09	0.06
05-01		0.75	0.48	0.35	0.17	0.09	0.06	2.05	0.76	0.50	0.35	0.17	0.09	0.06		0.76	0.48	0.35	0.17	0.09	0.06
05-02	1.90	0.82	0.51		0.18	0.09	0.06	2.05			0.39	0.18	0.09	0.06	1.86	0.80	0.51	0.37	0.18	0.09	0.05
05-03	2.09				0.18	0.09	0.06	2.28			0.40	0.19	0.08	0.06	2.16	0.81	0.53	0.41	0.18	0.08	
05-04	1.87	0.83	0.55		0.18	0.09	0.06	2.56	0.85	0.57	0.43	0.19	0.09	0.06	2.14	0.84	0.54	0.43	0.18	0.09	0.06
05-05	2.11	0.84	0.54		0.18	0.09	0.06	2.28	0.82			0.19	0.09	0.06	2.15	0.82	0.55	0.40	0.19	0.09	0.06
05-06	2.12	0.81	0.52		0.19	0.09	0.06	2.56	0.82			0.18	0.09	0.06	2.16	0.82			0.18	0.09	0.06
05-07	2.12	0.82	0.52	0.42	0.18	0.09	0.06	2.28	0.82			0.19	0.08	0.05	2.14	0.80		0.42	0.18	0.09	
05-08		0.79			0.18	0.09	0.06	2.28	0.79		0.46	0.18	0.08	0.05	2.12	0.75			0.18	0.08	0.05
05-09		0.80		0.45	0.19	0.09	0.06	2.28	0.79		0.42	0.19	0.08	0.06	2.07	0.78			0.19		0.05
05-10	2.07	0.80	0.52	0.38	0.18	0.09	0.06	2.05	0.79	0.55	0.37	0.18	0.09	0.06	2.11	0.80	0.53	0.39	0.18		0.06
05-11	1.95	0.76	0.49	0.36	0.18	0.09	0.06	2.05	0.76	0.49	0.36	0.18	0.09	0.06		0.76	0.49	0.36	0.18	0.09	0.06
06-01	1.98	0.75	0.49	0.36	0.18	0.09	0.06	2.05	0.79	0.57	0.37	0.18	0.09	0.06	1.96	0.77	0.49	0.37	0.18	0.09	0.06
06-02		0.79	0.50	0.36	0.18	0.09	0.06	2.05	0.82	0.53		0.18	0.09	0.06		0.78	0.50	0.36	0.18	0.09	0.06
06-03	2.05	0.80	0.52		0.18	0.09	0.06	2.05	0.79	0.57		0.18	0.09	0.06	2.11	0.80	0.50	0.37	0.18	0.09	0.05
06-04	2.04	0.82	0.51	0.38	0.18	0.09	0.06	2.28			0.39	0.18	0.09	0.06	2.05	0.82	0.52	0.38	0.18	0.09	0.06
06-05	2.14	0.80	0.51	0.40	0.19	0.09	0.06	2.28	0.82	0.54		0.19	0.09	0.06	2.13	0.80	0.53	0.39	0.18		0.06
06-06	2.08	0.81	0.54	0.40	0.19	0.09	0.06	2.28		0.54		0.18	0.09	0.06	2.11	0.81	0.54	0.40	0.19	0.09	0.06

Seismic Performance of Steel Moment-Resisting Frames with Spines Examined by Shake-Table Tests

06-07	2.08	0.79	0.51	0.39	0.19	0.09	0.06	2.28	0.85	0.53	0.43	0.19	0.09	0.06	2.14	0.80	0.52	0.39	0.19	0.07	0.06
06-08		0.81	0.55	0.41	0.18	0.09	0.06	2.28	0.85		0.44	0.19	0.09	0.06	2.15	0.81	0.54	0.39	0.19		0.06
06-09	2.09	0.81	0.54		0.18	0.08	0.06	2.05	0.85			0.18	0.08	0.06	2.15	0.81	0.53		0.19	0.09	0.06
06-10	2.15	0.81	0.52		0.18	0.09	0.06		0.85	0.51		0.18	0.09	0.06		0.81	0.52	0.39	0.18	0.09	
06-11	2.10	0.80	0.52		0.18	0.09	0.06	2.05	0.93	0.51	0.39	0.18	0.09	0.05	2.13	0.80	0.52	0.40	0.18	0.09	0.06
06-12	2.10	0.80	0.51	0.38	0.18	0.09	0.06	2.28	0.89		0.37	0.19	0.09	0.06	2.13	0.80	0.52	0.38	0.18	0.09	0.05
06-13	2.06	0.79	0.51	0.38	0.18	0.09	0.06	2.16	0.91	0.51	0.37	0.18	0.09	0.06	2.06	0.79	0.51	0.37	0.18	0.09	0.06
07-01	1.99	0.78	0.50	0.37	0.18	0.09	0.06	2.05	0.82	0.54	0.38	0.18	0.09	0.06	2.00	0.79	0.50	0.37	0.18	0.09	0.06
07-02	2.14	0.84	0.49	0.38	0.18	0.09	0.06	2.05	0.93	0.54	0.38	0.18	0.09	0.06	2.11	0.79	0.50	0.38	0.18	0.09	0.05
07-03	2.12	0.80	0.53	0.41	0.19	0.09	0.06	2.28	0.85	0.54	0.42	0.18	0.09	0.06	2.15	0.80	0.52		0.19	0.09	0.06
07-04	2.15	0.83	0.53	0.40	0.19	0.09	0.06	2.93	0.93	0.53		0.18	0.09	0.06	2.07	0.83	0.52	0.40	0.18	0.09	0.06
07-05	2.15	0.82			0.19	0.09	0.06	2.28	0.82	0.55	0.45	0.19	0.09	0.06	2.15	0.82		0.43	0.18	0.09	
07-06	2.16	0.83	0.54	0.41	0.19	0.09	0.06	2.28	0.85	0.54	0.41	0.19	0.09	0.06	2.18	0.82	0.56	0.42	0.19	0.09	0.05
07-07	2.15	0.80	0.54	0.41	0.19	0.09	0.06	2.56	0.85	0.54	0.43	0.20	0.09	0.06	2.19		0.53	0.41	0.19	0.09	0.06
07-08		0.82	0.56	0.43	0.18	0.09	0.06	2.56	0.82	0.54		0.20	0.09	0.06	2.22	0.82	0.55		0.18	0.08	0.06
07-09	2.01	0.83	0.55	0.41	0.18	0.09	0.06		0.89	0.54		0.19	0.09	0.06	2.20	0.84	0.54	0.41	0.18	0.08	0.06
07-10	2.18	0.83	0.55	0.41	0.19	0.09	0.06	2.28	0.89	0.53	0.42	0.19	0.09	0.06	2.16	0.83	0.55	0.41	0.19	0.09	0.06
07-11	2.07	0.83	0.52		0.19	0.09	0.06	2.28	0.85		0.40	0.19	0.09	0.06	2.14	0.82	0.52	0.40	0.19	0.08	0.05
07-12	2.12	0.81	0.53	0.40	0.19	0.09	0.06	2.28	0.85		0.39	0.19	0.09	0.06	2.10	0.81	0.52	0.40	0.19		
07-13	2.12	0.80	0.52	0.40	0.19	0.09	0.06	2.05	0.89	0.51	0.40	0.19	0.09	0.06	2.13	0.80	0.52	0.40	0.19	0.09	0.05
08-01	1.82	0.76	0.48	0.35	0.18	0.09	0.06	2.28	0.79	0.49	0.35	0.18	0.09	0.06			0.48		0.18	0.09	0.06
08-02		0.82	0.53	0.39	0.19	0.09		2.56	0.89	0.55		0.19	0.09	0.06		0.84	0.54	0.39	0.19		
08-03	2.23	0.83	0.54	0.41	0.19	0.09	0.06	2.05	0.85	0.50	0.41	0.18	0.09	0.06	2.11	0.84		0.41	0.19		
08-04	2.18	0.83	0.54	0.40	0.19	0.09	0.06	2.28	0.85	0.53	0.42	0.19	0.09	0.06	2.17	0.82	0.54	0.41	0.19	0.09	0.06
08-05	1.90	0.82	0.54	0.40	0.18	0.09	0.05		0.85			0.18	0.09	0.06	2.07	0.81	0.53	0.39	0.19	0.09	
08-06	2.12	0.81	0.52	0.38	0.18	0.09	0.06	2.93	0.79	0.51		0.18	0.09	0.06		0.81	0.52	0.38	0.18	0.09	0.06
08-07	1.94	0.80	0.52	0.38	0.18	0.09	0.06		0.85	0.54		0.18	0.09	0.06		0.81	0.50	0.38	0.18	0.09	
08-08	2.06	0.79	0.51	0.38	0.18	0.09	0.06	2.05	0.82	0.53	0.37	0.18	0.09	0.06	2.06	0.80	0.51	0.38	0.18	0.09	0.06
09-01		0.77	0.50	0.35	0.18	0.09	0.06		0.76			0.18	0.09	0.06			0.49		0.18	0.09	0.06
09-02		0.84	0.55	0.40	0.19	0.09	0.06	2.56	0.89	0.55		0.19	0.09	0.06		0.84	0.54		0.19	0.09	
09-03	2.25	0.86	0.55	0.42	0.19	0.09	0.06	2.28	0.85		0.41	0.18	0.09	0.06	2.24	0.86	0.56	0.42	0.19	0.09	0.06

Appendix H Full Results of Vibration Property Analysis: 08EDF

09-04	2.18	0.84	0.56	0.43	0.19	0.09	0.06	2.28	0.85		0.42	0.19	0.09	0.06		0.84	0.55	0.42	0.19	0.09	
09-05	2.09	0.83	0.54	0.40	0.19	0.09	0.06	2.56	0.82			0.18	0.09	0.06	2.00	0.82	0.55	0.41	0.19	0.09	0.05
09-06	2.17	0.83	0.53	0.39	0.18	0.09	0.06		0.79	0.53		0.19	0.09	0.06	2.20	0.83	0.53	0.39	0.18		0.05
09-07	2.08	0.81	0.52	0.40	0.19	0.09	0.06		0.89	0.54	0.39	0.19	0.09	0.06			0.52	0.39	0.18	0.09	0.06
09-08	2.11	0.81	0.52	0.39	0.18	0.09	0.06	2.28	0.82	0.53	0.39	0.18	0.09	0.06	2.11	0.81	0.52	0.39	0.18	0.09	0.05
12-01					0.18	0.09	0.06			0.48	0.35	0.18	0.09	0.06					0.18	0.09	
12-02		0.85			0.19	0.09	0.05	2.28	0.82			0.19	0.09	0.06					0.19		
12-03		0.83	0.53	0.39	0.19	0.09		2.28	0.82	0.51	0.40	0.18	0.09	0.06	2.08	0.83	0.53	0.39	0.19		
12-04		0.83	0.53		0.19	0.09	0.06		0.85	0.51	0.41	0.19	0.09	0.06	2.02	0.82	0.53	0.39	0.19	0.09	
12-05	2.15	0.83	0.54	0.40	0.19	0.09		2.28	0.89	0.55	0.41	0.18	0.09	0.06	2.18	0.84	0.55		0.19		
12-06	2.18	0.82			0.19		0.06	2.28	0.89	0.55	0.39	0.18	0.09	0.06	2.14	0.82	0.53		0.19		
12-07	2.15	0.82			0.19	0.09			0.85		0.42	0.20	0.09	0.06	2.16	0.83	0.52		0.19		
12-08		0.82	0.52		0.19			2.28	0.85	0.51		0.19	0.09	0.06	2.14	0.83	0.52		0.19	0.09	0.06
12-09	2.10	0.82	0.52					2.56	0.85			0.19	0.08	0.06	2.14	0.82					
12-10	2.11	0.81	0.52			0.09		2.28	0.85	0.53		0.18	0.09	0.06	2.15	0.81	0.52		0.19		
12-11		0.81			0.19	0.09	0.06		0.82	0.53	0.39	0.19	0.09	0.06	2.09		0.51		0.19		
12-12	2.02	0.79	0.51	0.38	0.18	0.09	0.06	2.05	0.79	0.50	0.37	0.18	0.09	0.06	2.04	0.79	0.50		0.18	0.09	
13-01					0.18	0.09	0.06				0.35	0.18	0.09	0.06		0.80				0.09	
13-02		0.81			0.19	0.09	0.06	2.28	0.82			0.19	0.09	0.06		0.83			0.19		
13-03		0.84	0.54		0.19		0.06	2.28			0.40	0.18	0.09	0.06			0.54	0.40	0.19		
13-04		0.84	0.55		0.19	0.09	0.06		0.85		0.41	0.19	0.09	0.06	2.05	0.84	0.54	0.40	0.19	0.09	0.06
13-05	2.16	0.84	0.56	0.42	0.19	0.09	0.06	2.28	0.89	0.57	0.42	0.19	0.09	0.06	2.19	0.86	0.56	0.42	0.19	0.09	0.06
13-06	2.15	0.84	0.55		0.19	0.09	0.06	2.28	0.89	0.55		0.18	0.09	0.06	2.19	0.85	0.54		0.19		
13-07	2.18	0.85	0.54	0.41	0.19				0.89	0.57	0.42	0.19	0.09	0.06	2.21	0.86	0.55	0.40	0.19	0.09	0.06
13-08	2.26	0.85			0.19	0.08		2.28	0.85	0.51		0.19	0.08	0.06	2.16	0.84	0.53	0.41	0.19		0.06
13-09		0.83	0.54		0.19	0.09		2.56	0.85	0.53		0.19	0.09	0.06	2.18	0.83	0.53		0.19	0.09	
13-10	2.19	0.83			0.19	0.09		2.28	0.85	0.53	0.40	0.18	0.09	0.06	2.18	0.82	0.53		0.19	0.08	0.06
13-11		0.81			0.19	0.09	0.06	2.28	0.85		0.41	0.18	0.09	0.06	2.14	0.81			0.19	0.09	0.06
13-12	2.04	0.81	0.52	0.38	0.18	0.09	0.06	2.05	0.79	0.50	0.39	0.18	0.09	0.06	2.08	0.80	0.52	0.38	0.18	0.09	0.06
14-01			0.48	0.35	0.18	0.09	0.05		0.79	0.48	0.36	0.18	0.09	0.06					0.18		
14-02		0.88	0.58	0.43	0.19	0.09		2.56	0.93	0.59		0.19	0.09	0.06			0.58	0.42		0.09	

Seismic Performance of Steel Moment-Resisting Frames with Spines Examined by Shake-Table Tests

14-03	2.29	0.91		0.42	0.19	0.09		2.56	0.89	0.55		0.19	0.09	0.06	2.36	0.91	0.58	0.44	0.19	0.09
14-04	2.11	0.88	0.57	0.42	0.19	0.09	0.05		0.85	0.62	0.42	0.19	0.09	0.06		0.87	0.57	0.43	0.19	0.09
14-05	2.26	0.86	0.55	0.43	0.19	0.09	0.06	2.28	0.85			0.18	0.09	0.06		0.85	0.56	0.42	0.19	0.09
14-06		0.87	0.56	0.42	0.19	0.09		2.56		0.54		0.19	0.09	0.06			0.56	0.41	0.19	
14-07		0.81	0.54	0.40	0.19	0.09		2.56	0.93	0.54	0.40	0.19	0.09	0.06		0.82	0.54	0.40	0.19	
14-08	2.18	0.83	0.54	0.41	0.19	0.09	0.06	2.28	0.89	0.54	0.41	0.19	0.09	0.06	2.18	0.84	0.54	0.41	0.19	0.09 0.06
17-01		0.77	0.48	0.35	0.18	0.09	0.06	2.05	0.76	0.49	0.35	0.18	0.09	0.06		0.77	0.49	0.35	0.18	0.09 0.06
17-02		0.83	0.50		0.19			2.05			0.40	0.18	0.09	0.06		0.83	0.50		0.19	
17-03					0.19	0.09	0.06	2.56				0.19	0.09	0.06	2.19		0.58	0.44	0.19	
17-04	2.41	0.89			0.20		0.06	2.93	0.89			0.09	0.08	0.06	2.41		0.56		0.19	
17-05		0.85			0.19			2.56	0.89	0.62	0.37	0.20	0.09	0.06	2.40	0.87	0.60	0.49		
17-06	2.21				0.19	0.09	0.06	2.56	0.85		0.50	0.20	0.09	0.06	2.29		0.57		0.19	0.06
17-07		0.84			0.20	0.08		2.28	0.89		0.49	0.19	0.08	0.06	2.16	0.86				
17-08	2.33	0.85				0.09			0.85	0.62	0.48	0.19	0.09	0.06	2.32	0.85	0.59	0.47		
17-09					0.19	0.09	0.06	2.28			0.48	0.20	0.09	0.06	2.17	0.82		0.48		0.06
17-10	2.14							2.28		0.59	0.47	0.20	0.08	0.06	2.18	0.82		0.46		
17-11	2.20	0.77	0.51		0.18	0.09	0.06	2.28	0.79	0.48	0.38	0.18	0.09	0.06	2.17		0.50	0.38	0.18	0.09 0.06
20-01	2.13	0.81	0.51		0.19	0.09	0.06	2.28	0.85	0.54	0.39	0.19	0.09	0.06	2.14	0.82	0.51	0.39	0.19	0.09 0.05
20-02	2.24	0.89			0.19	0.09	0.06	2.05	0.89	0.55	0.40	0.18	0.09	0.06	2.18				0.19	0.09 0.06
20-03	2.35	0.84			0.19	0.09		2.28	0.82			0.21	0.09	0.06	2.37	0.85		0.49	0.20	0.09
20-04		2.37	0.88	0.58	0.47	0.19	0.09	2.56	0.93			0.21	0.09	0.06		0.90	0.59	0.49	0.20	0.09
20-05	2.37	0.89			0.20	0.09		2.56	0.89		0.49	0.21	0.09	0.06	2.41	0.90			0.20	0.08
20-06	2.39	0.90	0.57		0.20	0.09		2.93			0.46	0.19	0.09	0.06	2.42	0.88	0.57	0.46	0.19	
20-07						0.09		2.93	0.93		0.53	0.20	0.10	0.06		0.96		0.52		
20-08		0.93			0.20			2.56	0.89			0.20	0.09	0.06	2.32	0.91		0.49	0.20	
20-09	2.46	0.90		0.51	0.20	0.10		2.56	0.93			0.19	0.10	0.09	2.46	0.91	0.62	0.51		
20-10		0.89			0.20				0.89		0.47	0.19	0.09	0.06	2.36	0.90		0.45	0.20	
20-11		0.87			0.20	0.10	0.06		0.89	0.57		0.20	0.10	0.06	2.25	0.88	0.57	0.42	0.20	0.06
20-12	2.31	0.88	0.54				0.06	2.56	0.89	0.55		0.20	0.10	0.06	2.29	0.87	0.55	0.43	0.20	
20-13	2.32	0.86		0.45	0.20	0.10	0.06	2.28	0.89			0.20	0.09	0.06	2.32	0.86	0.53	0.43	0.20	0.10
23-01	2.20	0.83	0.52	0.40	0.20	0.09	0.06	2.28	0.85	0.54	0.39	0.20	0.10	0.06	2.20	0.83	0.52	0.39	0.19	0.09

Appendix H Full Results of Vibration Property Analysis: 08EDF

23-02	2.26			0.20	0.10	0.06		0.89	0.51	0.42	0.22	0.10	0.07	2.17	0.86	0.53		0.20	0.10		
23-03	2.38			0.21	0.10	0.06	2.56	0.89		0.43	0.21	0.10	0.06	2.43	0.87	0.59		0.21			
23-04				0.21	0.10		2.56		0.57		0.21	0.10	0.06	2.38	0.90		0.44	0.20	0.10		
23-05		0.92		0.48	0.10		2.56	0.89		0.49	0.21	0.10	0.06	2.46	0.90		0.48	0.21			
23-06	2.35	0.89			0.20		0.07	2.93		0.57		0.19	0.10	0.06	2.39	0.88	0.57	0.44	0.20		
23-07					0.10			2.93	0.98		0.57	0.20	0.10	0.06		0.96	0.60		0.21		
23-08		0.94			0.10	0.06	2.56	0.89		0.47	0.20	0.10	0.06	2.30	0.92		0.46	0.20	0.10		
23-09	2.42	0.89			0.10	0.06	2.56	0.93			0.19	0.10	0.06	2.44	0.89			0.21	0.10		
23-10		0.92			0.20	0.10	0.06		0.93		0.46	0.21	0.10	0.06	2.38	0.91	0.55		0.20		
23-11		0.88			0.20	0.10		2.56	0.89	0.54	0.48	0.20	0.10	0.06	2.33	0.88		0.43	0.20		
23-12	2.33	0.87	0.54	0.41			2.56	0.89	0.54	0.39	0.20	0.10	0.06	2.29	0.88	0.55	0.41	0.20	0.06		
23-13	2.34	0.86			0.20	0.10	0.06	2.28	0.93	0.54	0.43	0.20	0.10	0.06	2.34	0.87	0.54	0.43	0.20	0.10	
26-01	2.26	0.85	0.53		0.20	0.10	0.06	2.56	0.85	0.54		0.20	0.10	0.06	2.28	0.85	0.53	0.40	0.19	0.10	
26-02		0.87			0.20	0.10			0.89	0.59	0.44	0.20	0.10	0.06	2.09	0.88		0.40	0.20	0.10	0.06
26-03		0.89			0.20	0.10	0.06	2.28	0.89	0.60		0.21	0.11	0.06	2.45	0.89	0.59	0.45	0.21	0.10	
26-04					0.21	0.10	0.06	2.56			0.45	0.21	0.10	0.06	2.39	0.91	0.61	0.45	0.21	0.10	
26-05			0.90	0.48	0.20		0.06	2.56	0.89		0.50	0.21	0.10	0.06	2.47	0.91		0.48	0.21		
26-06	2.28	0.90	0.58	0.44	0.20	0.10	0.06	2.93	0.89	0.60	0.47	0.19	0.10	0.06	2.39	0.89	0.59	0.43	0.20	0.10	
26-07						0.10	0.06	2.93	0.98		0.57	0.20	0.09	0.06		0.97			0.21		
26-08		0.96				0.10		2.56	0.89			0.20	0.10	0.06	2.29	0.94	0.60	0.48	0.21		
26-09	2.51	0.94			0.21			2.56	0.98			0.21	0.10	0.06	2.57	0.94			0.21		
26-10		0.90		0.45	0.21	0.10			0.93	0.62	0.48	0.22	0.10	0.06	2.45	0.91	0.56	0.44	0.21	0.10	
26-11		0.92	0.56		0.21	0.10		2.56	0.93	0.55		0.10	0.06		2.47	0.92	0.57		0.21	0.10	
26-12	2.38	0.91	0.55	0.43	0.20	0.10	0.06	2.56	0.89	0.55	0.44	0.20	0.10	0.06	2.44	0.91	0.55	0.43		0.10	
26-13	2.45	0.89	0.57	0.42	0.20	0.10	0.06	2.56	0.93	0.55	0.42	0.20	0.10	0.06	2.44	0.89	0.56	0.42	0.21	0.10	

H.3.2 Modal damping ratio

Figure H.7 plots the modal damping ratios in the ground motion cases of the 08EDF test estimated using the ARX, FDD and SS methods, respectively. Table H.6 summarizes the corresponding values in each segment.

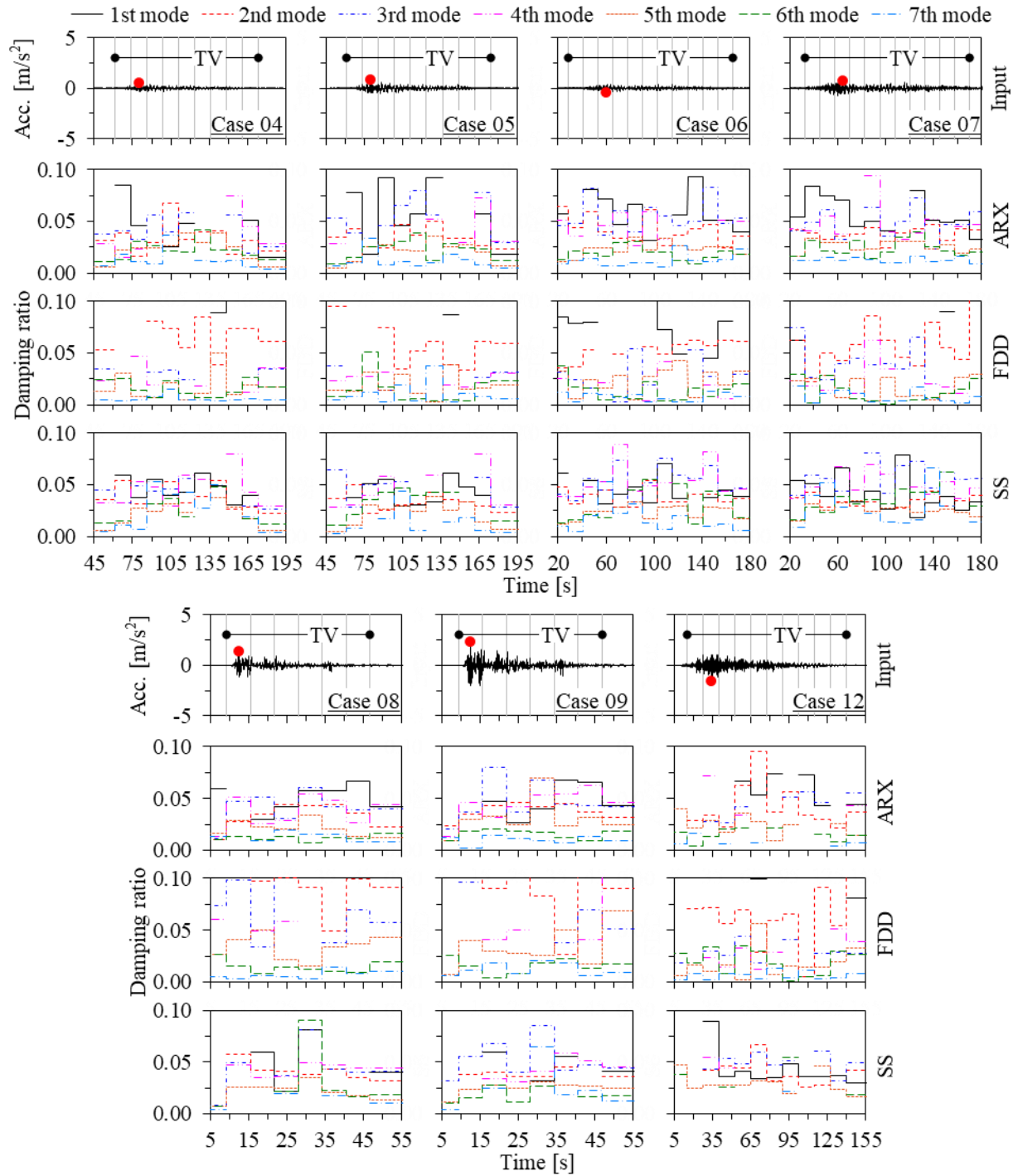


Figure H.7 Damping ratio in ground motion excitations of the 08EDF test

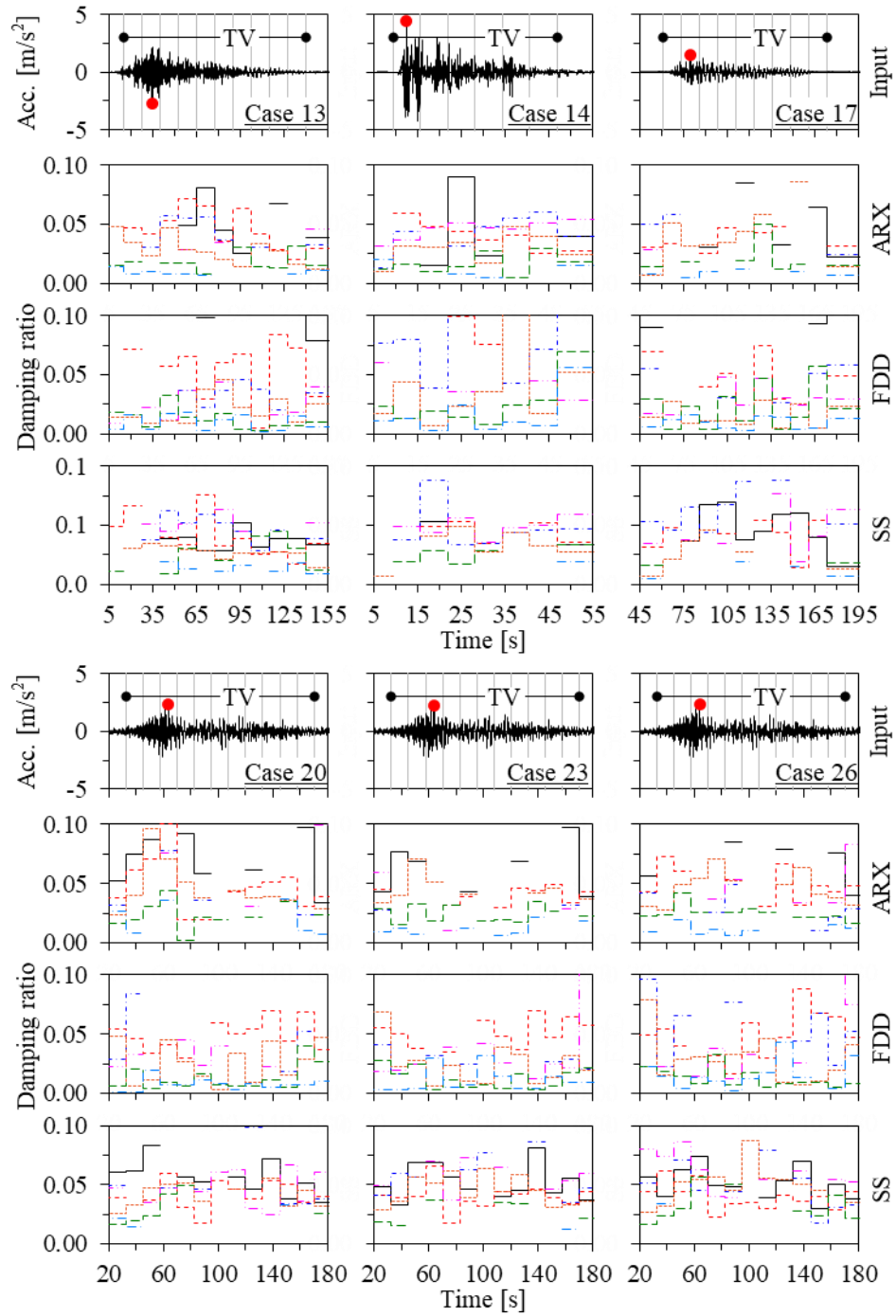


Figure H.7 (continued) Damping ratio in ground motion excitations of the 08EDF test

Table H.6 Damping ratio in ground motion excitations of the 08EDF test

Case-Segment	1st mode			2nd mode			3rd mode			4th mode			5th mode			6th mode			7th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01		0.03	0.04	0.03	0.01	0.01	0.01		0.05	0.03	0.02	0.01	0.02	0.00		0.04	0.04	0.03	0.01	0.01	0.00
04-02	0.08	0.04	0.04	0.01	0.02	0.01	0.02					0.03	0.02	0.00	0.06	0.05	0.03	0.03	0.01	0.02	0.01
04-03	0.05	0.02	0.03	0.02	0.02	0.03	0.01	0.14			0.05	0.01	0.01	0.01	0.04	0.03	0.05	0.05	0.03		0.01
04-04		0.03	0.06		0.02	0.03	0.01		0.08	0.03	0.02	0.01	0.01	0.01	0.06	0.03	0.04	0.05	0.02	0.03	0.05
04-05	0.03	0.07	0.04		0.03	0.02	0.04	0.14	0.07	0.03		0.02	0.02	0.01	0.04	0.05	0.05	0.03	0.03	0.04	0.04
04-06	0.05	0.03	0.06		0.04	0.02		0.17	0.05	0.02		0.01	0.01	0.01	0.04	0.03	0.06	0.06	0.02	0.02	0.04
04-07		0.04			0.03	0.04	0.01		0.08		0.02	0.01	0.01	0.01	0.06	0.04		0.05			0.03
04-08		0.04			0.04	0.02	0.01	0.09	0.04				0.05	0.01	0.00	0.05	0.05		0.04	0.05	0.05
04-09		0.02	0.06	0.07	0.02	0.02	0.01	0.12	0.07			0.01	0.01	0.01	0.03	0.03		0.08	0.03		
04-10	0.05	0.03	0.02	0.04	0.01	0.02	0.01	0.14	0.07	0.03	0.01	0.02	0.03	0.01	0.04	0.03	0.03	0.04	0.02	0.02	0.02
04-11	0.02	0.02	0.03	0.03	0.01	0.01	0.00	0.14	0.06	0.03	0.04	0.02	0.02	0.00		0.02	0.03	0.03	0.01	0.01	0.00
05-01		0.03	0.05	0.03	0.00	0.01	0.01		0.09	0.04	0.02	0.01	0.01	0.00		0.04	0.06	0.03	0.01	0.01	0.00
05-02	0.08	0.04	0.04		0.01	0.01	0.01				0.03	0.01	0.01	0.01	0.04	0.05	0.03	0.03	0.01	0.02	0.01
05-03	0.02				0.03	0.03	0.03	0.15			0.03	0.03	0.05	0.01	0.05	0.02	0.05	0.04	0.03	0.04	
05-04	0.09	0.02	0.05		0.03	0.03	0.01		0.07	0.01	0.03	0.01	0.02	0.00	0.06	0.03	0.05	0.06	0.03	0.04	0.02
05-05	0.05	0.05	0.07		0.02	0.03	0.01	0.14	0.03			0.03	0.00	0.02	0.03	0.03	0.05	0.05	0.03	0.05	0.04
05-06	0.06	0.04	0.08		0.02	0.04	0.01	0.20	0.05			0.01	0.01	0.00	0.03	0.03			0.02	0.04	0.01
05-07	0.09	0.05	0.06	0.05	0.04	0.01	0.01		0.06			0.00	0.00	0.04	0.03	0.04		0.06	0.04	0.04	
05-08		0.03			0.03	0.03	0.02	0.09	0.04		0.02	0.04	0.00	0.00	0.06	0.04			0.03	0.04	0.01
05-09		0.03		0.03	0.03	0.02	0.01		0.03		0.02	0.01	0.00	0.01	0.05	0.03			0.03		0.02
05-10	0.06	0.03	0.08	0.07	0.02	0.02	0.01	0.12	0.06	0.03	0.02	0.02	0.02	0.01	0.04	0.03	0.06	0.08	0.01		0.01
05-11	0.02	0.02	0.03	0.03	0.01	0.01	0.00		0.06	0.03	0.03	0.02	0.02	0.01		0.02	0.03	0.03	0.01	0.01	0.00
06-01	0.06	0.06	0.05	0.05	0.01	0.01	0.01	0.09	0.04	0.03	0.03	0.03	0.04	0.01	0.06	0.05	0.04	0.04	0.01	0.01	0.01
06-02		0.04	0.05	0.04	0.01	0.02	0.01	0.08	0.04	0.02		0.01	0.02	0.00		0.03	0.03	0.04	0.01	0.02	0.00
06-03	0.08	0.06	0.08		0.02	0.02	0.01	0.08	0.05	0.02		0.02	0.02	0.01	0.05	0.04	0.06	0.05	0.02	0.02	0.02
06-04	0.07	0.04	0.04	0.05	0.02	0.02	0.01				0.02	0.02	0.00	0.01	0.03	0.05	0.04	0.05	0.02	0.02	0.05
06-05	0.05	0.03	0.06	0.06	0.02	0.03	0.01	0.14	0.05	0.00		0.03	0.01	0.01	0.04	0.03	0.07	0.09	0.03		0.02
06-06	0.07	0.03	0.04	0.03	0.03	0.03	0.01	0.11		0.05		0.00	0.01	0.01	0.05	0.03	0.03	0.04	0.03	0.05	0.03

Appendix H Full Results of Vibration Property Analysis: 08EDF

06-07	0.03	0.06	0.06	0.06	0.02	0.02	0.01	0.11	0.06	0.02	0.00	0.03	0.02	0.00	0.03	0.05	0.05	0.05	0.02	0.05	0.01
06-08		0.03	0.06	0.03	0.03	0.02	0.01	0.07	0.05		0.04	0.03	0.00	0.01	0.07	0.03	0.08	0.05	0.03		0.01
06-09	0.06	0.04	0.05		0.03	0.01	0.03	0.05	0.06			0.04	0.00	0.00	0.04	0.03	0.06		0.03	0.05	0.02
06-10	0.09	0.04	0.05		0.02	0.01	0.01		0.06	0.05		0.01	0.02	0.00		0.04	0.06	0.05	0.02	0.01	
06-11	0.05	0.05	0.08		0.03	0.01	0.01	0.04	0.06	0.03	0.02	0.03	0.01	0.01	0.04	0.04	0.07	0.08	0.03	0.04	0.01
06-12	0.05	0.02	0.05	0.05	0.03	0.02	0.01	0.08	0.06		0.01	0.02	0.02	0.01	0.04	0.03	0.04	0.04	0.03	0.04	0.01
06-13	0.04	0.04	0.05	0.05	0.02	0.02	0.02		0.06	0.03		0.03	0.02	0.01	0.04	0.04	0.05	0.05	0.02	0.02	0.01
07-01	0.05	0.04	0.04	0.04	0.02	0.02	0.01	0.06	0.06	0.07	0.02	0.02	0.03	0.02	0.05	0.04	0.04	0.04	0.02	0.02	0.01
07-02	0.08	0.04	0.05	0.04	0.03	0.03	0.01		0.02	0.03	0.02	0.03	0.00	0.00	0.05	0.04	0.06	0.05	0.03	0.03	0.04
07-03	0.07	0.03	0.06	0.05	0.03	0.02	0.02		0.05	0.02	0.01	0.03	0.03	0.01	0.04	0.03	0.06		0.03	0.02	0.05
07-04	0.07	0.04	0.03	0.04	0.03	0.02	0.01		0.04	0.04		0.01	0.02	0.01	0.07	0.03	0.04	0.07	0.03	0.04	0.03
07-05	0.04	0.04			0.02	0.03	0.01		0.06	0.04	0.01	0.02	0.01	0.01	0.03	0.03		0.04	0.03	0.03	
07-06	0.05	0.04	0.06	0.09	0.02	0.01	0.01	0.15	0.09	0.04	0.06	0.00	0.00	0.00	0.04	0.03	0.08	0.07	0.02	0.04	0.02
07-07	0.04	0.02	0.04	0.04	0.03	0.02	0.01		0.06	0.04	0.03	0.03	0.00	0.00	0.03		0.04	0.06	0.03	0.03	0.01
07-08		0.04	0.05	0.03	0.03	0.03	0.01		0.05	0.03		0.01	0.00	0.00	0.08	0.03	0.07		0.03	0.05	0.01
07-09	0.08	0.04	0.07	0.04	0.03	0.04	0.02		0.04	0.07		0.01	0.00	0.02	0.02	0.04	0.07	0.05	0.02	0.04	0.02
07-10	0.05	0.05	0.02	0.05	0.02	0.02	0.01		0.08	0.02	0.02	0.01	0.01	0.01	0.03	0.04	0.05	0.04	0.02	0.04	0.07
07-11	0.05	0.03	0.05		0.03	0.02	0.01	0.09	0.06		0.02	0.01	0.01	0.00	0.04	0.03	0.06	0.05	0.03	0.06	0.03
07-12	0.05	0.04	0.05	0.04	0.02	0.02	0.02		0.04		0.01	0.03	0.01	0.00	0.03	0.03	0.06	0.04	0.02		
07-13	0.03	0.04	0.06	0.05	0.02	0.02	0.01	0.13	0.32			0.03	0.03	0.01	0.03	0.04	0.06	0.05	0.02	0.03	0.01
08-01	0.06	0.01	0.01	0.01	0.02	0.01	0.01		0.12	0.07	0.06	0.03	0.03	0.00			0.01		0.01	0.01	0.00
08-02		0.03	0.05	0.05	0.03	0.01			0.18	0.10		0.04	0.02	0.00		0.06	0.05	0.05	0.03		
08-03	0.03	0.04	0.05	0.03	0.02	0.01	0.01		0.10	0.03	0.05	0.05	0.01	0.01	0.06	0.04		0.03	0.03		
08-04	0.04	0.04	0.03	0.03	0.02	0.01	0.02		0.10	0.24	0.06	0.02	0.01	0.00	0.02	0.04	0.04	0.04	0.03	0.02	0.02
08-05	0.06	0.04	0.06	0.05	0.03	0.01	0.02		0.09			0.02	0.01	0.00	0.08	0.04	0.08	0.05	0.03	0.09	
08-06	0.06	0.04	0.04	0.05	0.02	0.01	0.02		0.05	0.04		0.03	0.01	0.01		0.04	0.04	0.05	0.02	0.02	0.02
08-07	0.07	0.04	0.04	0.03	0.01	0.01	0.01		0.10	0.07		0.04	0.01	0.01		0.04	0.04	0.04	0.02	0.02	
08-08	0.04	0.02	0.04	0.04	0.01	0.02	0.01		0.09	0.06		0.04	0.02	0.01	0.04	0.03	0.04	0.04	0.01	0.02	0.01
09-01		0.02	0.02	0.01	0.01	0.01	0.01		0.11			0.01	0.03	0.01			0.03		0.01	0.01	0.00
09-02		0.03	0.04	0.05	0.03	0.02	0.00		0.17	0.10		0.04	0.01	0.01		0.04	0.06		0.02	0.01	
09-03	0.05	0.04	0.08	0.03	0.03	0.02	0.01		0.09		0.04	0.03	0.00	0.02	0.06	0.04	0.07	0.03	0.03	0.03	0.02

Seismic Performance of Steel Moment-Resisting Frames with Spines Examined by Shake-Table Tests

09-04	0.03	0.05	0.04	0.04	0.02	0.02	0.01	0.11		0.05	0.03	0.01	0.01		0.04	0.04	0.03	0.02	0.01	
09-05	0.04	0.04	0.07	0.05	0.07	0.02	0.01	0.08			0.03	0.02	0.02	0.03	0.04	0.08	0.04	0.03	0.03	0.06
09-06	0.07	0.04	0.04	0.05	0.02	0.01	0.01	0.03	0.04		0.05	0.02	0.02	0.06	0.04	0.04	0.06	0.03		0.02
09-07	0.07	0.04	0.04	0.06	0.03	0.01	0.01	0.10	0.07	0.04	0.02	0.01	0.01			0.05	0.05	0.03	0.02	0.02
09-08	0.04	0.03	0.04	0.05	0.03	0.02	0.01	0.09	0.05	0.34	0.07	0.02	0.01	0.04	0.04	0.04	0.05	0.02	0.02	0.01
12-01					0.04	0.02	0.01		0.02	0.03	0.01	0.03	0.00					0.05	0.04	
12-02		0.03			0.02	0.00	0.02	0.07			0.02	0.02	0.01					0.02		
12-03		0.03	0.03	0.07	0.03	0.01		0.07	0.03	0.02	0.01	0.03	0.00	0.09	0.04	0.04	0.05	0.03		
12-04		0.02	0.03		0.02	0.02	0.01	0.06	0.03	0.01	0.02	0.01	0.01	0.04	0.05	0.05	0.04	0.03	0.03	
12-05	0.07	0.06	0.04	0.03	0.04	0.02		0.15	0.04	0.03	0.00	0.03	0.01	0.04	0.04	0.05		0.03		
12-06	0.05	0.09			0.03		0.01	0.10	0.02	0.01	0.06	0.03	0.00	0.03	0.07	0.06		0.03		
12-07	0.07	0.04			0.01	0.02		0.06		0.03	0.01	0.02	0.01	0.04	0.03	0.05		0.03		
12-08		0.06	0.05		0.02			0.07	0.04		0.02	0.00	0.00	0.05	0.04	0.05		0.02	0.05	0.02
12-09	0.07	0.03	0.06					0.17			0.01	0.01	0.01	0.04	0.03					
12-10	0.04	0.03	0.05			0.02		0.12	0.03		0.01	0.01	0.00	0.04	0.03	0.06		0.05		
12-11		0.02			0.03	0.01	0.00	0.05	0.03	0.05	0.02	0.03	0.00	0.04		0.03		0.02		
12-12	0.04	0.04	0.06	0.04	0.01	0.01	0.01	0.08	0.03	0.04	0.03	0.03	0.01	0.03	0.04	0.05		0.02	0.02	
13-01					0.05	0.02	0.01			0.01	0.01	0.02	0.00		0.05				0.01	
13-02		0.05			0.03	0.02	0.01	0.07			0.01	0.02	0.02		0.07			0.03		
13-03		0.04	0.03		0.02		0.01			0.02	0.01	0.01	0.00			0.04	0.05	0.03		
13-04		0.05	0.06		0.05	0.02	0.01	0.06		0.01	0.01	0.03	0.02	0.04	0.04	0.06	0.05	0.03	0.01	0.02
13-05	0.05	0.07	0.05	0.03	0.03	0.02	0.01	0.07	0.04	0.04	0.01	0.01	0.02	0.04	0.03	0.05	0.05	0.03	0.03	0.01
13-06	0.08	0.07	0.06		0.03	0.01	0.01	0.10	0.02		0.04	0.01	0.00	0.03	0.08	0.06		0.03		
13-07	0.04	0.04	0.04	0.03	0.02			0.06	0.04	0.04	0.05	0.02	0.01	0.03	0.03	0.05	0.06	0.02	0.02	0.01
13-08	0.03	0.06			0.01	0.03		0.07	0.05		0.02	0.00	0.02	0.05	0.04	0.04	0.04	0.03		0.02
13-09		0.04	0.03		0.03	0.01		0.15	0.04		0.01	0.00	0.00	0.03	0.03	0.03		0.03	0.04	
13-10	0.07	0.03			0.03	0.01		0.13	0.02	0.02	0.03	0.01	0.00	0.04	0.03	0.05		0.03	0.04	0.01
13-11		0.02			0.02	0.03	0.01	0.20		0.02	0.01	0.01	0.01	0.04	0.02			0.03	0.03	0.02
13-12	0.04	0.03	0.03	0.05	0.01	0.01	0.01	0.08	0.03	0.04	0.03	0.01	0.01	0.03	0.03	0.04	0.05	0.01	0.01	0.01
14-01			0.01	0.03	0.01	0.01	0.02	0.13	0.08	0.06	0.02	0.02	0.01					0.01		
14-02		0.06	0.04	0.04	0.03	0.02		0.18	0.08		0.04	0.01	0.01			0.04	0.05		0.02	

Appendix H Full Results of Vibration Property Analysis: 08EDF

14-03	0.02	0.05		0.05	0.03	0.01		0.12	0.04		0.01	0.02	0.00	0.05	0.05	0.09	0.04	0.04	0.03		
14-04	0.09	0.04	0.03	0.05	0.03	0.01	0.01	0.10	0.11	0.04	0.02	0.01	0.02		0.05	0.03	0.06	0.05	0.02		
14-05	0.02	0.04	0.05	0.05	0.02	0.03	0.01	0.08			0.04	0.01	0.00		0.03	0.03	0.04	0.03	0.03		
14-06		0.04	0.05	0.05	0.05	0.00			0.04		0.13	0.02	0.01			0.04	0.05	0.04			
14-07		0.03	0.06	0.05	0.04	0.03			0.07	0.04	0.02	0.03	0.01		0.05	0.05	0.05	0.03			
14-08	0.04	0.03	0.04	0.05	0.02	0.02	0.01	0.13	0.22	0.03	0.05	0.07	0.06	0.03	0.04	0.05	0.06	0.03	0.03	0.02	
17-01		0.03	0.05	0.03	0.01	0.01	0.01	0.09	0.07	0.06	0.02	0.01	0.03	0.01	0.03	0.05	0.04	0.01	0.02	0.00	
17-02		0.03	0.06		0.05						0.02	0.03	0.00	0.01	0.05	0.05		0.02			
17-03					0.03	0.02	0.00	0.18				0.01	0.02	0.02	0.04		0.07	0.04	0.04		
17-04	0.03	0.03			0.03		0.01	0.04				0.01	0.00	0.01	0.07		0.04		0.05		
17-05		0.05			0.04			0.05	0.03	0.05	0.01	0.03	0.00	0.07	0.04	0.07	0.03				
17-06	0.09				0.04	0.02	0.01	0.01		0.02	0.01	0.01	0.00	0.04		0.09		0.03		0.02	
17-07		0.04			0.06	0.05		0.07		0.05	0.03	0.05	0.02	0.05	0.05						
17-08	0.03	0.05				0.01		0.03	0.03	0.03	0.00	0.00	0.01	0.06	0.04	0.09	0.08				
17-09					0.09	0.02	0.01			0.02	0.02	0.01	0.00	0.06	0.01		0.02			0.02	
17-10	0.06							0.09	0.05	0.03	0.01	0.06	0.00	0.04	0.05		0.04				
17-11	0.02	0.03	0.02		0.01	0.02	0.01	0.11	0.05	0.06	0.03	0.02	0.02	0.01	0.01		0.05	0.06	0.01	0.02	0.01
20-01	0.05	0.04	0.03		0.02	0.02	0.03	0.05	0.03	0.02	0.05	0.01	0.00	0.06	0.04	0.05	0.05	0.03	0.02	0.02	
20-02	0.07	0.06			0.04	0.02	0.01	0.05	0.08	0.03	0.01	0.02	0.00	0.06				0.03	0.02	0.01	
20-03	0.09	0.07			0.10	0.03		0.03			0.01	0.00	0.02	0.08	0.04		0.04	0.05	0.02		
20-04		0.10	0.08	0.08	0.07	0.04	0.04	0.04			0.05	0.01	0.01		0.06	0.05	0.05	0.05	0.04		
20-05	0.09	0.02			0.05	0.00		0.03		0.05	0.03	0.01	0.01	0.06	0.03			0.05	0.05		
20-06	0.06	0.02	0.04		0.04	0.02				0.02	0.01	0.02	0.01	0.05	0.02	0.05	0.04	0.04			
20-07						0.02		0.06		0.04	0.00	0.01	0.01		0.05		0.06				
20-08		0.04			0.04			0.05			0.03	0.01	0.00	0.06	0.05		0.06	0.05			
20-09	0.06	0.05		0.03	0.04	0.02		0.05			0.01	0.01	0.01	0.05	0.04	0.10	0.03				
20-10		0.05			0.04			0.07		0.02	0.04	0.01	0.01	0.07	0.05		0.02	0.06			
20-11		0.05			0.04	0.03	0.04	0.04	0.02		0.02	0.01	0.01	0.04	0.05	0.03	0.07	0.03		0.03	
20-12	0.10	0.03	0.02				0.01	0.07	0.05		0.03	0.04	0.01	0.05	0.03	0.03	0.04	0.04			
20-13	0.03	0.04		0.10	0.03	0.02	0.01	0.04			0.05	0.03	0.01	0.04	0.04	0.04	0.06	0.03	0.03		
23-01	0.04	0.04	0.03	0.06	0.03	0.03	0.01	0.05	0.04	0.02	0.07	0.03	0.00	0.05	0.04	0.04	0.05	0.03	0.02		

Seismic Performance of Steel Moment-Resisting Frames with Spines Examined by Shake-Table Tests

23-02	0.08			0.04	0.02	0.01	0.05	0.02	0.02	0.02	0.02	0.00	0.03	0.04	0.06		0.04	0.02
23-03	0.07			0.07	0.03	0.01	0.04		0.02	0.03	0.02	0.00	0.07	0.04	0.06		0.06	
23-04				0.05	0.02			0.03		0.01	0.00	0.03	0.07	0.07		0.07	0.05	0.04
23-05		0.03		0.01			0.03		0.04	0.02	0.01	0.01	0.06	0.02		0.04	0.06	
23-06	0.04	0.04			0.04	0.01		0.02		0.01	0.00	0.01	0.05	0.04	0.06	0.07	0.04	
23-07					0.02		0.06		0.04	0.01	0.01	0.03		0.03	0.08		0.06	
23-08		0.03			0.02	0.01	0.05		0.03	0.04	0.00	0.01	0.04	0.04		0.07	0.04	0.03
23-09	0.07	0.05			0.02	0.01	0.04			0.03	0.02	0.00	0.05	0.04			0.06	0.03
23-10		0.04			0.04	0.03	0.02		0.01	0.01	0.00	0.00	0.08	0.05	0.09		0.04	
23-11		0.05			0.05	0.03		0.03	0.02	0.01	0.01	0.01	0.04	0.04		0.05	0.03	
23-12	0.10	0.03	0.03	0.03			0.06	0.03	0.02	0.03	0.02	0.01	0.05	0.03	0.04	0.05	0.03	0.01
23-13	0.04	0.04			0.04	0.02	0.02	0.02	0.22	0.02	0.02	0.01	0.04	0.04	0.05	0.06	0.04	0.02
26-01	0.06	0.04	0.04		0.03	0.02	0.01	0.10		0.08	0.02	0.02	0.06	0.04	0.05	0.08	0.03	0.02
26-02		0.07			0.03	0.02		0.02	0.02	0.01	0.02	0.01	0.04	0.06		0.07	0.03	0.02
26-03		0.06			0.05	0.04	0.02	0.07		0.03	0.01	0.00	0.06	0.04	0.07	0.09	0.05	0.03
26-04					0.05	0.03	0.01		0.01	0.03	0.01	0.01	0.07	0.06	0.05	0.06	0.05	0.06
26-05			0.03	0.04	0.07		0.01		0.04	0.03	0.03	0.01	0.05	0.02		0.04	0.06	
26-06	0.09	0.05	0.05	0.05	0.05	0.03	0.01	0.08	0.01	0.03	0.01	0.00	0.05	0.03	0.04	0.04	0.05	0.05
26-07						0.03	0.01		0.03	0.03	0.02	0.01		0.04			0.09	
26-08		0.03				0.03				0.05	0.00	0.02	0.04	0.03	0.08	0.05	0.05	
26-09	0.08	0.07			0.05					0.01	0.01	0.04	0.05	0.04			0.05	
26-10		0.03		0.03	0.06	0.02		0.04	0.03	0.01	0.01	0.01	0.07	0.04	0.05	0.06	0.05	0.03
26-11		0.05	0.01		0.04	0.02		0.07		0.01	0.01		0.03	0.04	0.02		0.05	0.02
26-12	0.08	0.04	0.02	0.03	0.04	0.02	0.02	0.02	0.12	0.02	0.01	0.00	0.05	0.03	0.03	0.05		0.04
26-13	0.04	0.05	0.03	0.08	0.03	0.02	0.01	0.05	0.05	0.07	0.04	0.01	0.03	0.04	0.04	0.03	0.05	0.04

H.3.3 Mode shapes

Figure H.8 to Figure H.10 plot the mode shapes in the ground motion cases of the 08EDF test estimated using the ARX, FDD and SS methods, respectively.

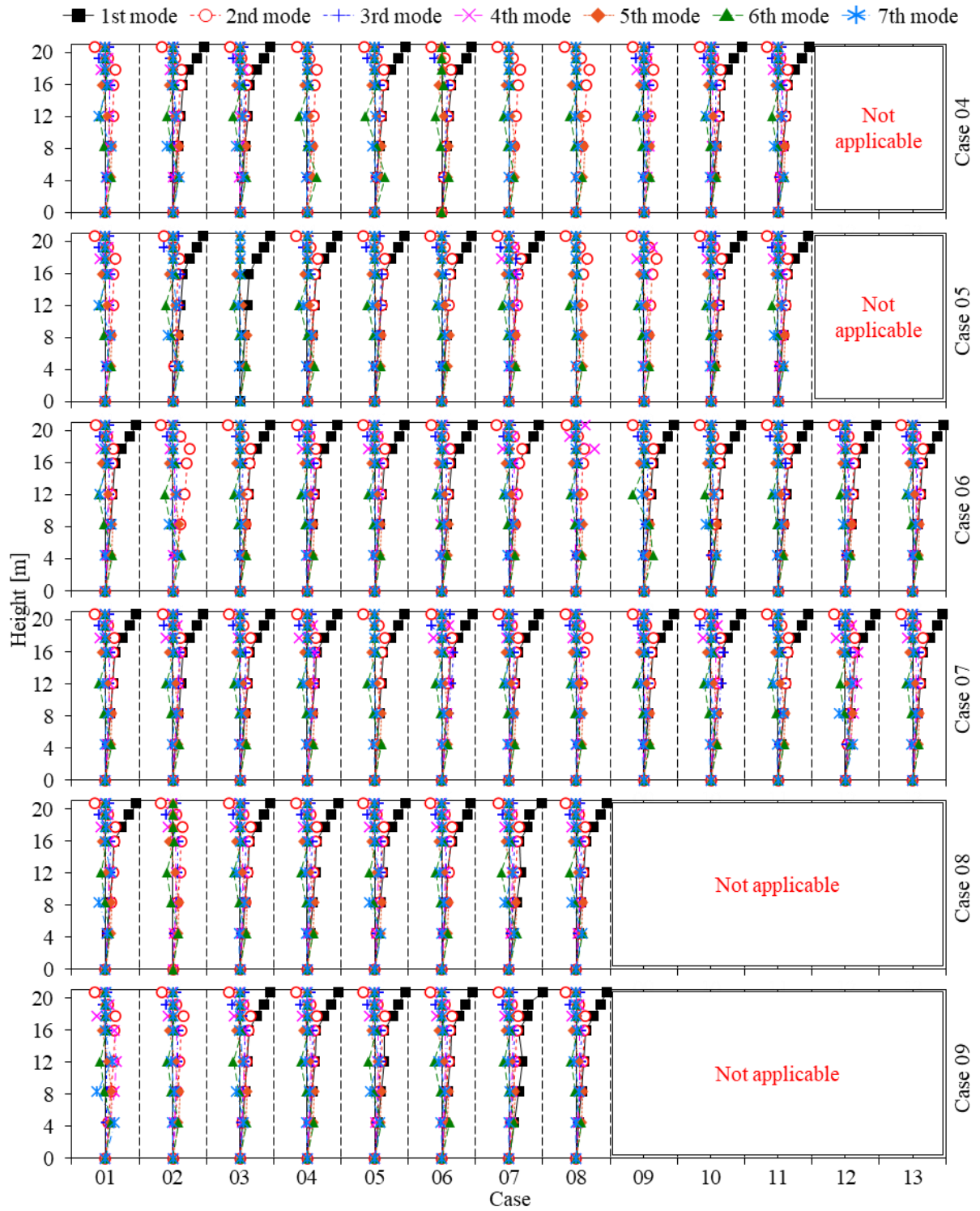


Figure H.8 Mode shapes in ground motion excitations of the 08EDF test (ARX)

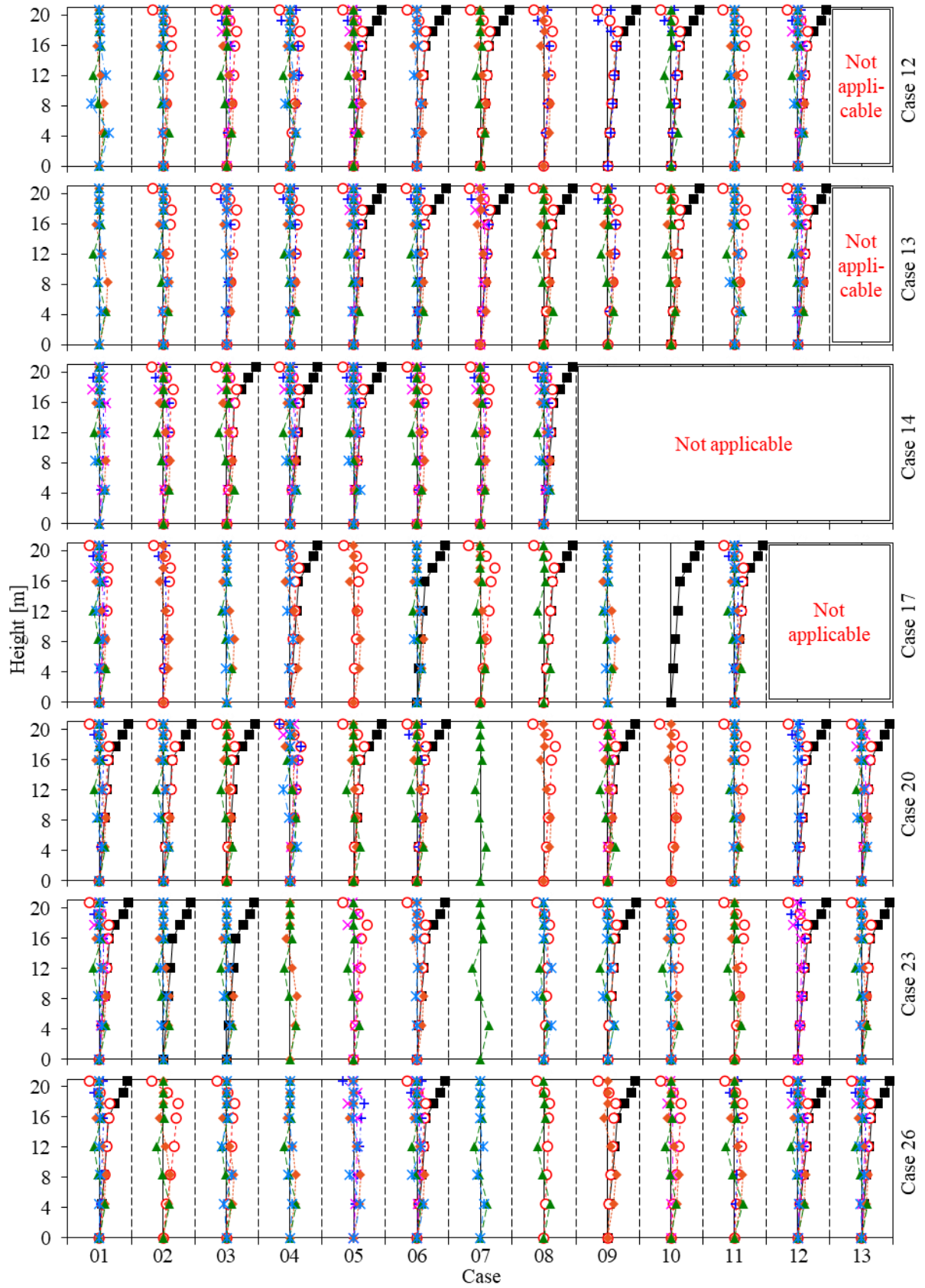


Figure H.8 (continued) Mode shapes in ground motion excitations of the 08EDF test (ARX)

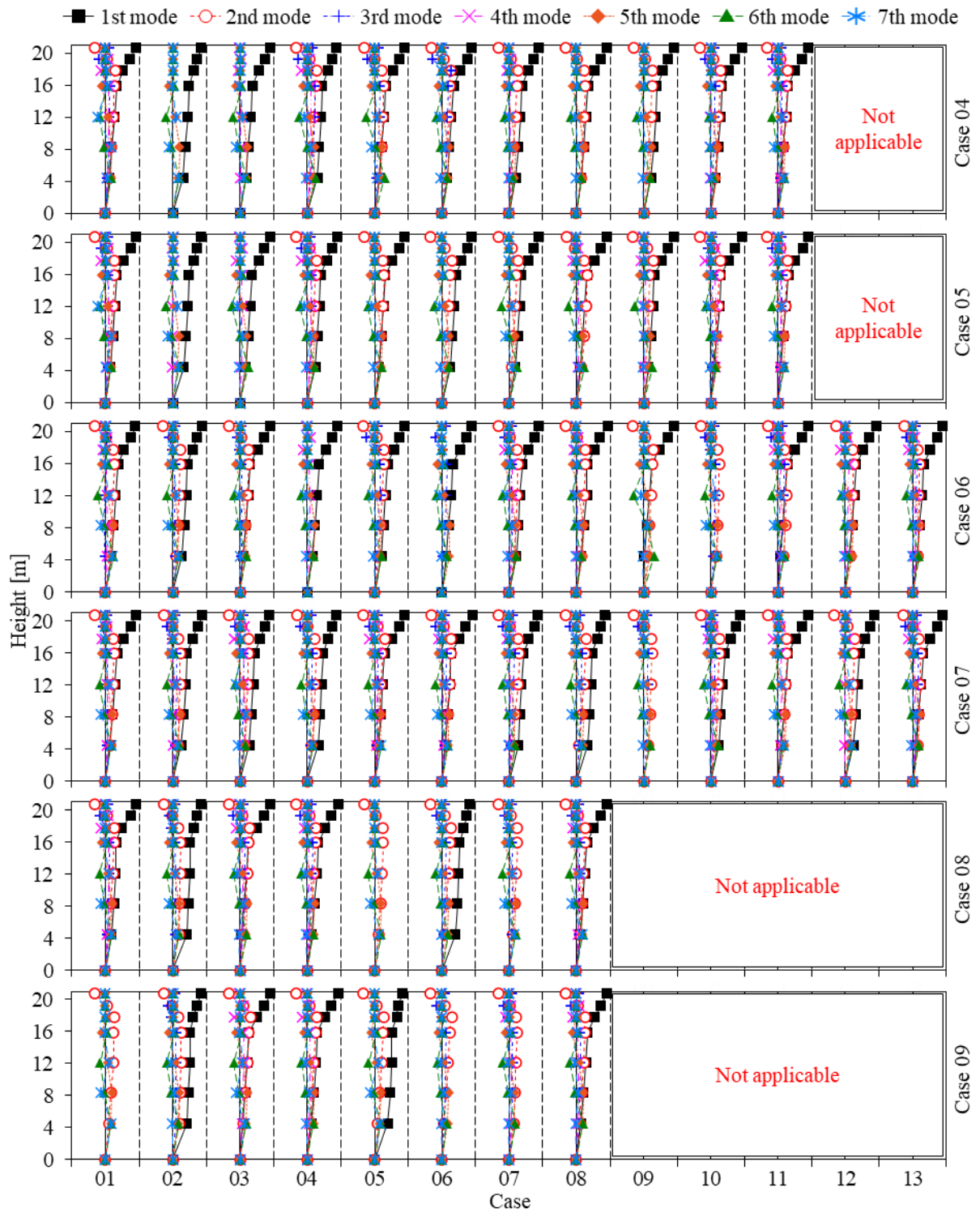


Figure H.9 Mode shapes in ground motion excitations of the 08EDF test (FDD)

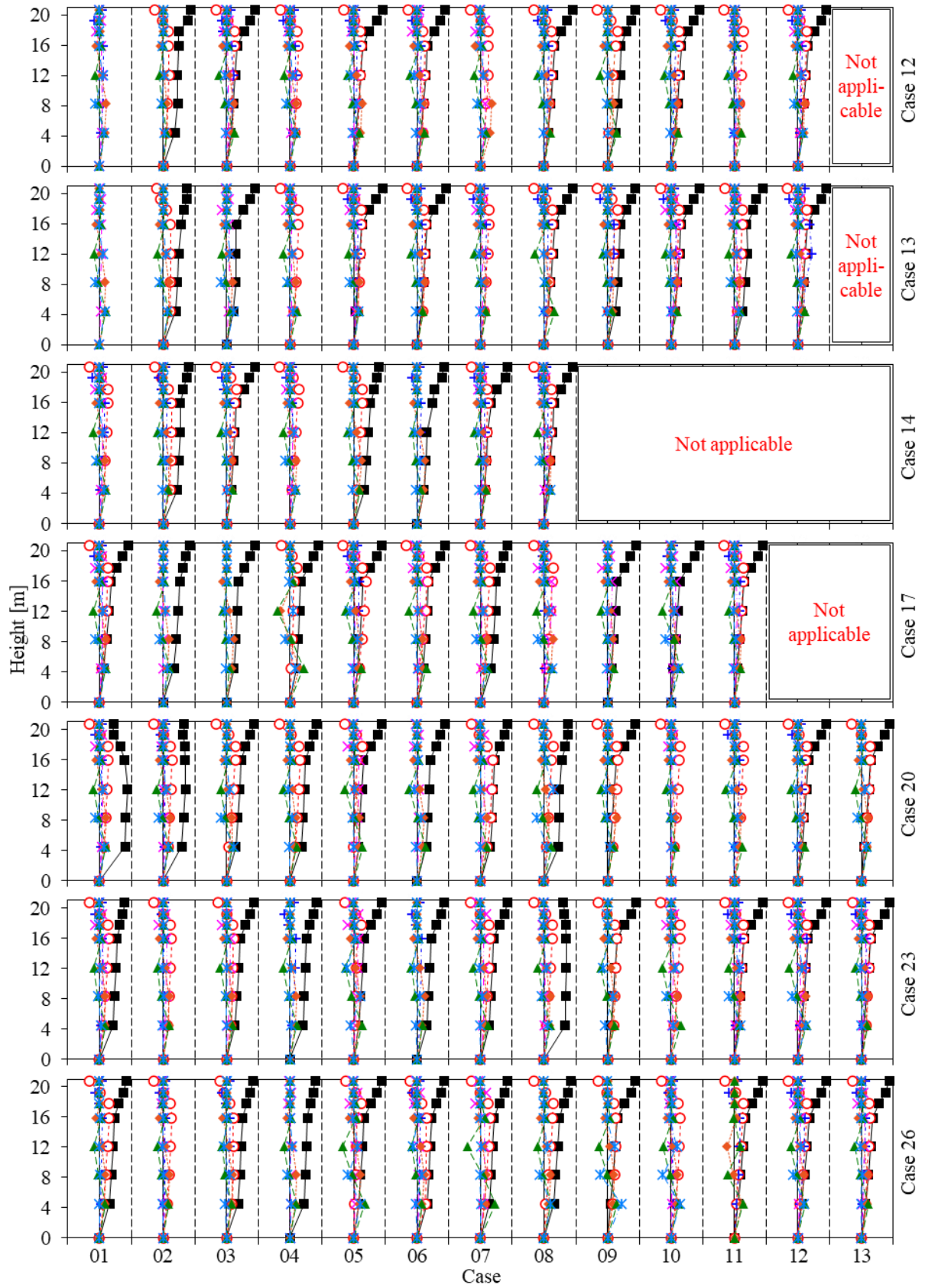


Figure H.9 (continued) Mode shapes in ground motion excitations of the 08EDF test (FDD)

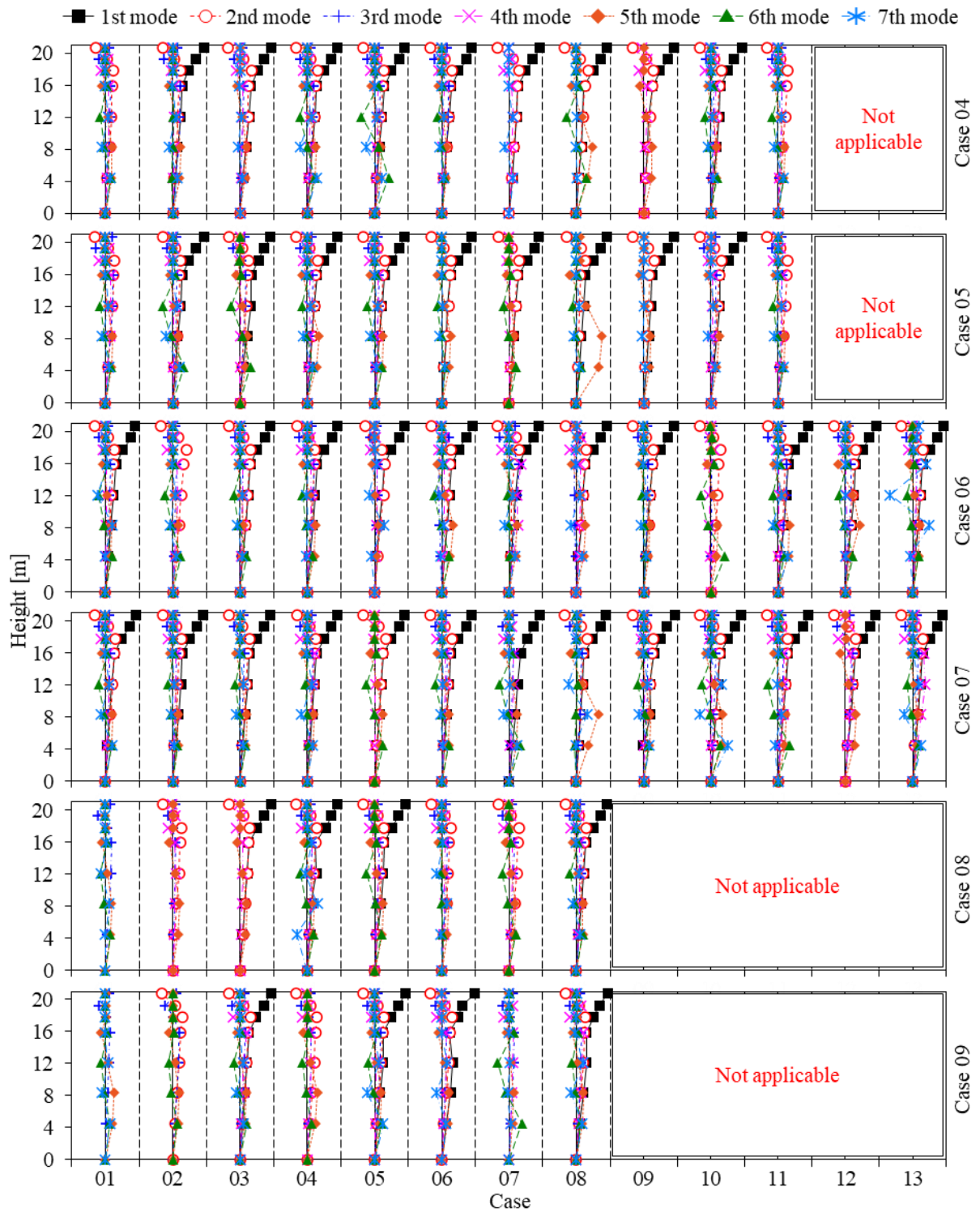


Figure H.10 Mode shapes in ground motion excitations of the 08EDF test (SS)

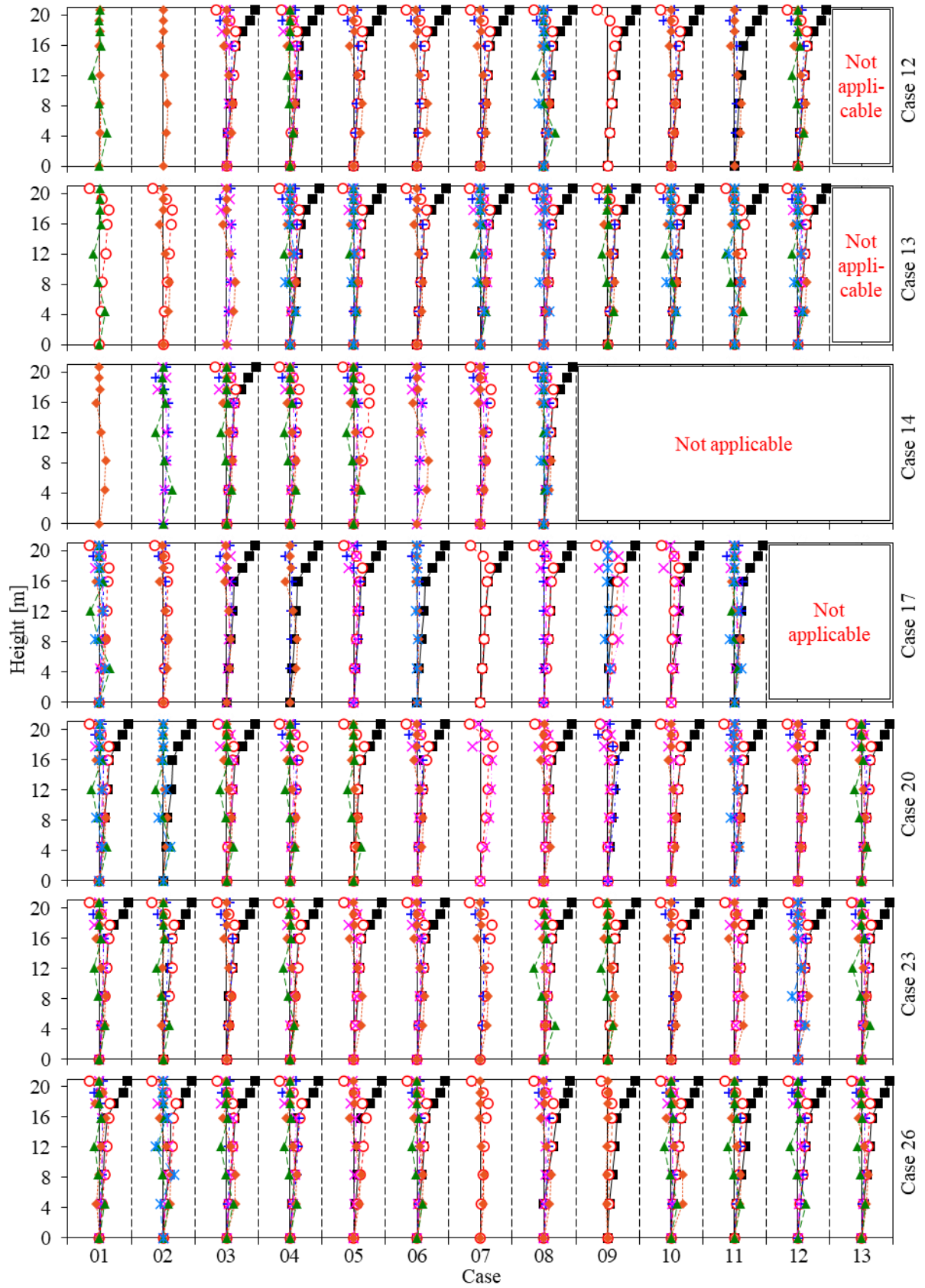


Figure H.10 (continued) Mode shapes in ground motion excitations of the 08EDF test (SS)

H.3.4 Modal Effective Mass (MEM)

Figure H.11 plots the natural period in the ground motion cases of the 08EDF test estimated using the ARX, FDD and SS methods, respectively. Table H.7 summarizes the corresponding values in each segment.

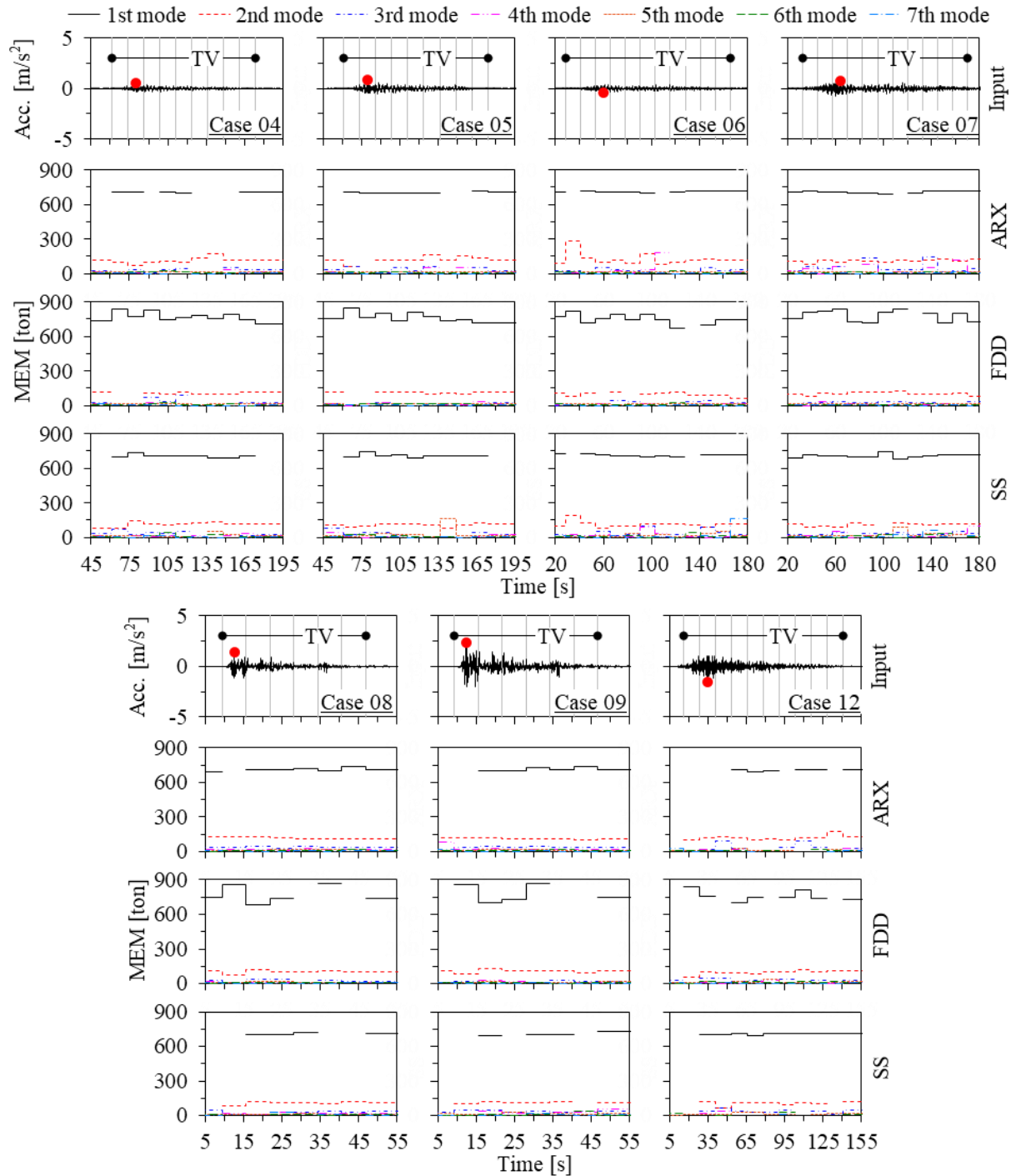


Figure H.11 MEMs in ground motion excitations of the 08EDF test

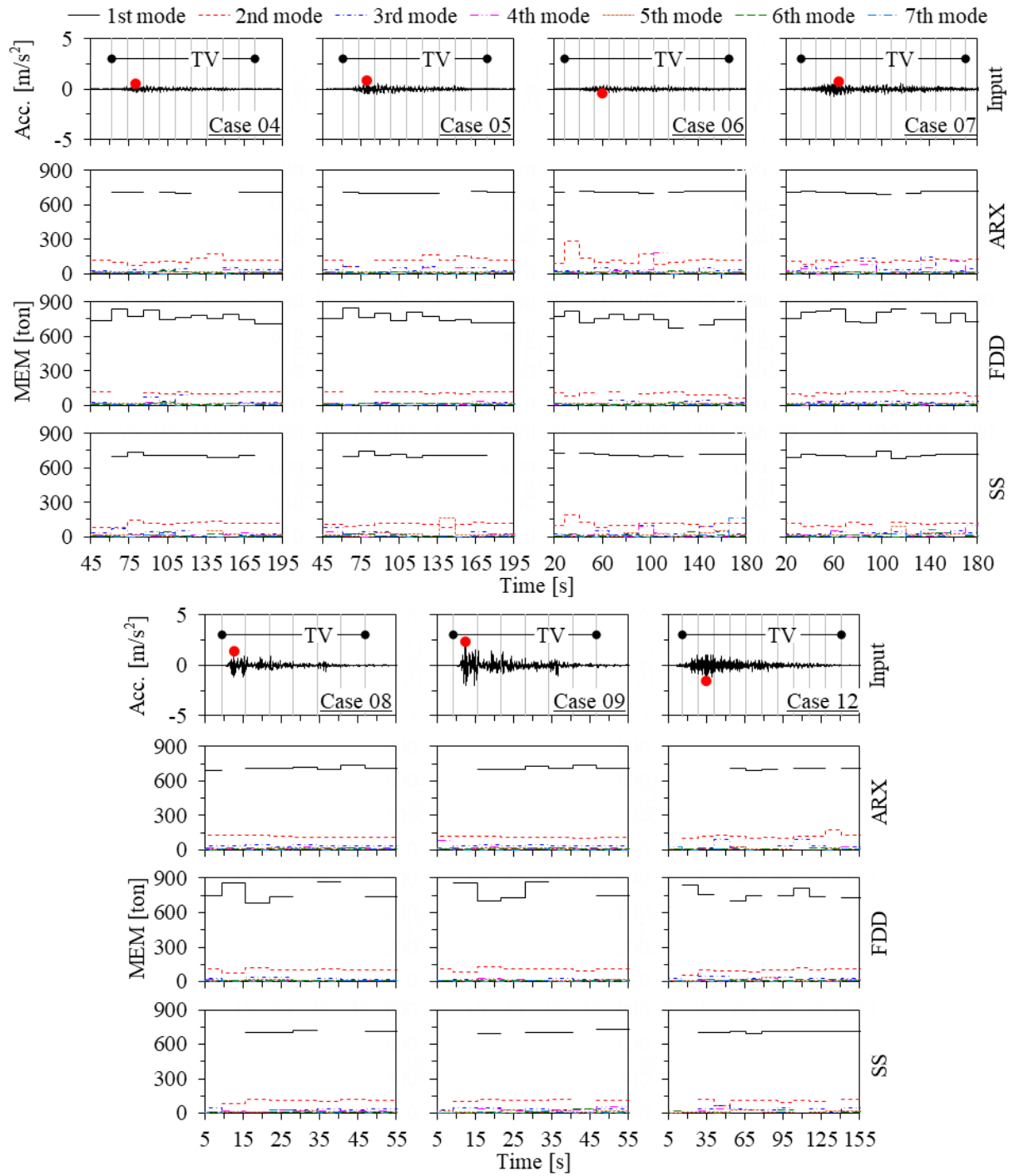


Figure H.11 (continued) MEMs in ground motion excitations of the 08EDF test

Table H.7 MEMs in ground motion excitations of the 08EDF test [Unit: ton]

Case-Segment	1st mode			2nd mode			3rd mode			4th mode			5th mode			6th mode			7th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
04-01		0.75	0.48	0.35	0.17	0.09	0.06	1.86	0.76	0.48	0.35	0.17	0.09	0.06		0.75	0.47	0.35	0.17	0.09	0.06
04-02	1.79	0.80	0.49	0.37	0.18	0.09	0.06	2.05				0.18	0.09	0.06	1.78	0.80	0.50	0.37	0.18	0.09	0.06
04-03	1.96	0.81	0.52	0.42	0.18	0.09	0.06	2.28			0.40	0.18	0.09	0.06	2.11	0.82	0.51	0.39	0.18		0.06
04-04		0.82	0.53		0.18	0.08	0.06	2.56	0.82	0.54	0.39	0.19	0.08	0.06	2.07	0.82	0.53	0.41	0.18	0.08	0.05
04-05	2.19	0.82	0.53		0.18	0.08	0.05	2.28	0.82	0.53		0.18	0.08	0.06	2.12	0.81	0.52	0.38	0.19	0.08	0.06
04-06	2.13	0.81	0.50		0.18	0.09		2.28	0.82	0.49		0.18	0.09	0.06	2.13	0.81	0.51	0.39	0.18	0.09	0.05
04-07		0.80			0.18	0.09	0.06	2.28	0.82		0.40	0.18	0.09	0.06	2.10	0.80		0.40			0.06
04-08		0.79			0.18	0.09	0.06	2.28	0.79			0.18	0.09	0.06	2.07	0.80			0.18	0.08	0.06
04-09		0.81	0.51	0.38	0.18	0.09	0.06	2.28	0.82			0.18	0.09	0.06	2.09	0.80		0.38	0.18		
04-10	1.97	0.77	0.49	0.37	0.18	0.09	0.06	2.05	0.79	0.50	0.39	0.18	0.09	0.06	1.99	0.77	0.49	0.36	0.18	0.09	0.06
04-11	1.90	0.75	0.48	0.35	0.17	0.09	0.06	1.86	0.76	0.49	0.35	0.17	0.09	0.06		0.75	0.48	0.35	0.17	0.09	0.06
05-01		0.75	0.48	0.35	0.17	0.09	0.06	2.05	0.76	0.50	0.35	0.17	0.09	0.06		0.76	0.48	0.35	0.17	0.09	0.06
05-02	1.90	0.82	0.51		0.18	0.09	0.06	2.05			0.39	0.18	0.09	0.06	1.86	0.80	0.51	0.37	0.18	0.09	0.05
05-03	2.09				0.18	0.09	0.06	2.28			0.40	0.19	0.08	0.06	2.16	0.81	0.53	0.41	0.18	0.08	
05-04	1.87	0.83	0.55		0.18	0.09	0.06	2.56	0.85	0.57	0.43	0.19	0.09	0.06	2.14	0.84	0.54	0.43	0.18	0.09	0.06
05-05	2.11	0.84	0.54		0.18	0.09	0.06	2.28	0.82			0.19	0.09	0.06	2.15	0.82	0.55	0.40	0.19	0.09	0.06
05-06	2.12	0.81	0.52		0.19	0.09	0.06	2.56	0.82			0.18	0.09	0.06	2.16	0.82			0.18	0.09	0.06
05-07	2.12	0.82	0.52	0.42	0.18	0.09	0.06	2.28	0.82			0.19	0.08	0.05	2.14	0.80		0.42	0.18	0.09	
05-08		0.79			0.18	0.09	0.06	2.28	0.79		0.46	0.18	0.08	0.05	2.12	0.75			0.18	0.08	0.05
05-09		0.80		0.45	0.19	0.09	0.06	2.28	0.79		0.42	0.19	0.08	0.06	2.07	0.78			0.19		0.05
05-10	2.07	0.80	0.52	0.38	0.18	0.09	0.06	2.05	0.79	0.55	0.37	0.18	0.09	0.06	2.11	0.80	0.53	0.39	0.18		0.06
05-11	1.95	0.76	0.49	0.36	0.18	0.09	0.06	2.05	0.76	0.49	0.36	0.18	0.09	0.06		0.76	0.49	0.36	0.18	0.09	0.06
06-01	1.98	0.75	0.49	0.36	0.18	0.09	0.06	2.05	0.79	0.57	0.37	0.18	0.09	0.06	1.96	0.77	0.49	0.37	0.18	0.09	0.06
06-02		0.79	0.50	0.36	0.18	0.09	0.06	2.05	0.82	0.53		0.18	0.09	0.06		0.78	0.50	0.36	0.18	0.09	0.06
06-03	2.05	0.80	0.52		0.18	0.09	0.06	2.05	0.79	0.57		0.18	0.09	0.06	2.11	0.80	0.50	0.37	0.18	0.09	0.05
06-04	2.04	0.82	0.51	0.38	0.18	0.09	0.06	2.28			0.39	0.18	0.09	0.06	2.05	0.82	0.52	0.38	0.18	0.09	0.06
06-05	2.14	0.80	0.51	0.40	0.19	0.09	0.06	2.28	0.82	0.54		0.19	0.09	0.06	2.13	0.80	0.53	0.39	0.18		0.06
06-06	2.08	0.81	0.54	0.40	0.19	0.09	0.06	2.28		0.54		0.18	0.09	0.06	2.11	0.81	0.54	0.40	0.19	0.09	0.06

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06-07	2.08	0.79	0.51	0.39	0.19	0.09	0.06	2.28	0.85	0.53	0.43	0.19	0.09	0.06	2.14	0.80	0.52	0.39	0.19	0.07	0.06
06-08		0.81	0.55	0.41	0.18	0.09	0.06	2.28	0.85		0.44	0.19	0.09	0.06	2.15	0.81	0.54	0.39	0.19		0.06
06-09	2.09	0.81	0.54		0.18	0.08	0.06	2.05	0.85			0.18	0.08	0.06	2.15	0.81	0.53		0.19	0.09	0.06
06-10	2.15	0.81	0.52		0.18	0.09	0.06		0.85	0.51		0.18	0.09	0.06		0.81	0.52	0.39	0.18	0.09	
06-11	2.10	0.80	0.52		0.18	0.09	0.06	2.05	0.93	0.51	0.39	0.18	0.09	0.05	2.13	0.80	0.52	0.40	0.18	0.09	0.06
06-12	2.10	0.80	0.51	0.38	0.18	0.09	0.06	2.28	0.89		0.37	0.19	0.09	0.06	2.13	0.80	0.52	0.38	0.18	0.09	0.05
06-13	2.06	0.79	0.51	0.38	0.18	0.09	0.06	2.16	0.91	0.51	0.37	0.18	0.09	0.06	2.06	0.79	0.51	0.37	0.18	0.09	0.06
07-01	1.99	0.78	0.50	0.37	0.18	0.09	0.06	2.05	0.82	0.54	0.38	0.18	0.09	0.06	2.00	0.79	0.50	0.37	0.18	0.09	0.06
07-02	2.14	0.84	0.49	0.38	0.18	0.09	0.06	2.05	0.93	0.54	0.38	0.18	0.09	0.06	2.11	0.79	0.50	0.38	0.18	0.09	0.05
07-03	2.12	0.80	0.53	0.41	0.19	0.09	0.06	2.28	0.85	0.54	0.42	0.18	0.09	0.06	2.15	0.80	0.52		0.19	0.09	0.06
07-04	2.15	0.83	0.53	0.40	0.19	0.09	0.06	2.93	0.93	0.53		0.18	0.09	0.06	2.07	0.83	0.52	0.40	0.18	0.09	0.06
07-05	2.15	0.82			0.19	0.09	0.06	2.28	0.82	0.55	0.45	0.19	0.09	0.06	2.15	0.82		0.43	0.18	0.09	
07-06	2.16	0.83	0.54	0.41	0.19	0.09	0.06	2.28	0.85	0.54	0.41	0.19	0.09	0.06	2.18	0.82	0.56	0.42	0.19	0.09	0.05
07-07	2.15	0.80	0.54	0.41	0.19	0.09	0.06	2.56	0.85	0.54	0.43	0.20	0.09	0.06	2.19		0.53	0.41	0.19	0.09	0.06
07-08		0.82	0.56	0.43	0.18	0.09	0.06	2.56	0.82	0.54		0.20	0.09	0.06	2.22	0.82	0.55		0.18	0.08	0.06
07-09	2.01	0.83	0.55	0.41	0.18	0.09	0.06		0.89	0.54		0.19	0.09	0.06	2.20	0.84	0.54	0.41	0.18	0.08	0.06
07-10	2.18	0.83	0.55	0.41	0.19	0.09	0.06	2.28	0.89	0.53	0.42	0.19	0.09	0.06	2.16	0.83	0.55	0.41	0.19	0.09	0.06
07-11	2.07	0.83	0.52		0.19	0.09	0.06	2.28	0.85		0.40	0.19	0.09	0.06	2.14	0.82	0.52	0.40	0.19	0.08	0.05
07-12	2.12	0.81	0.53	0.40	0.19	0.09	0.06	2.28	0.85		0.39	0.19	0.09	0.06	2.10	0.81	0.52	0.40	0.19		
07-13	2.12	0.80	0.52	0.40	0.19	0.09	0.06	2.05	0.89	0.51	0.40	0.19	0.09	0.06	2.13	0.80	0.52	0.40	0.19	0.09	0.05
08-01	1.82	0.76	0.48	0.35	0.18	0.09	0.06	2.28	0.79	0.49	0.35	0.18	0.09	0.06			0.48		0.18	0.09	0.06
08-02		0.82	0.53	0.39	0.19	0.09		2.56	0.89	0.55		0.19	0.09	0.06		0.84	0.54	0.39	0.19		
08-03	2.23	0.83	0.54	0.41	0.19	0.09	0.06	2.05	0.85	0.50	0.41	0.18	0.09	0.06	2.11	0.84		0.41	0.19		
08-04	2.18	0.83	0.54	0.40	0.19	0.09	0.06	2.28	0.85	0.53	0.42	0.19	0.09	0.06	2.17	0.82	0.54	0.41	0.19	0.09	0.06
08-05	1.90	0.82	0.54	0.40	0.18	0.09	0.05		0.85			0.18	0.09	0.06	2.07	0.81	0.53	0.39	0.19	0.09	
08-06	2.12	0.81	0.52	0.38	0.18	0.09	0.06	2.93	0.79	0.51		0.18	0.09	0.06		0.81	0.52	0.38	0.18	0.09	0.06
08-07	1.94	0.80	0.52	0.38	0.18	0.09	0.06		0.85	0.54		0.18	0.09	0.06		0.81	0.50	0.38	0.18	0.09	
08-08	2.06	0.79	0.51	0.38	0.18	0.09	0.06	2.05	0.82	0.53	0.37	0.18	0.09	0.06	2.06	0.80	0.51	0.38	0.18	0.09	0.06
09-01		0.77	0.50	0.35	0.18	0.09	0.06		0.76			0.18	0.09	0.06			0.49		0.18	0.09	0.06
09-02		0.84	0.55	0.40	0.19	0.09	0.06	2.56	0.89	0.55		0.19	0.09	0.06		0.84	0.54		0.19	0.09	
09-03	2.25	0.86	0.55	0.42	0.19	0.09	0.06	2.28	0.85		0.41	0.18	0.09	0.06	2.24	0.86	0.56	0.42	0.19	0.09	0.06

Appendix H Full Results of Vibration Property Analysis: 08EDF

09-04	2.18	0.84	0.56	0.43	0.19	0.09	0.06	2.28	0.85		0.42	0.19	0.09	0.06		0.84	0.55	0.42	0.19	0.09	
09-05	2.09	0.83	0.54	0.40	0.19	0.09	0.06	2.56	0.82			0.18	0.09	0.06	2.00	0.82	0.55	0.41	0.19	0.09	0.05
09-06	2.17	0.83	0.53	0.39	0.18	0.09	0.06		0.79	0.53		0.19	0.09	0.06	2.20	0.83	0.53	0.39	0.18		0.05
09-07	2.08	0.81	0.52	0.40	0.19	0.09	0.06		0.89	0.54	0.39	0.19	0.09	0.06			0.52	0.39	0.18	0.09	0.06
09-08	2.11	0.81	0.52	0.39	0.18	0.09	0.06	2.28	0.82	0.53	0.39	0.18	0.09	0.06	2.11	0.81	0.52	0.39	0.18	0.09	0.05
12-01					0.18	0.09	0.06			0.48	0.35	0.18	0.09	0.06					0.18	0.09	
12-02		0.85			0.19	0.09	0.05	2.28	0.82			0.19	0.09	0.06					0.19		
12-03		0.83	0.53	0.39	0.19	0.09		2.28	0.82	0.51	0.40	0.18	0.09	0.06	2.08	0.83	0.53	0.39	0.19		
12-04		0.83	0.53		0.19	0.09	0.06		0.85	0.51	0.41	0.19	0.09	0.06	2.02	0.82	0.53	0.39	0.19	0.09	
12-05	2.15	0.83	0.54	0.40	0.19	0.09		2.28	0.89	0.55	0.41	0.18	0.09	0.06	2.18	0.84	0.55		0.19		
12-06	2.18	0.82			0.19		0.06	2.28	0.89	0.55	0.39	0.18	0.09	0.06	2.14	0.82	0.53		0.19		
12-07	2.15	0.82			0.19	0.09			0.85		0.42	0.20	0.09	0.06	2.16	0.83	0.52		0.19		
12-08		0.82	0.52		0.19			2.28	0.85	0.51		0.19	0.09	0.06	2.14	0.83	0.52		0.19	0.09	0.06
12-09	2.10	0.82	0.52					2.56	0.85			0.19	0.08	0.06	2.14	0.82					
12-10	2.11	0.81	0.52			0.09		2.28	0.85	0.53		0.18	0.09	0.06	2.15	0.81	0.52		0.19		
12-11		0.81			0.19	0.09	0.06		0.82	0.53	0.39	0.19	0.09	0.06	2.09		0.51		0.19		
12-12	2.02	0.79	0.51	0.38	0.18	0.09	0.06	2.05	0.79	0.50	0.37	0.18	0.09	0.06	2.04	0.79	0.50		0.18	0.09	
13-01					0.18	0.09	0.06				0.35	0.18	0.09	0.06		0.80				0.09	
13-02		0.81			0.19	0.09	0.06	2.28	0.82			0.19	0.09	0.06		0.83			0.19		
13-03		0.84	0.54		0.19		0.06	2.28			0.40	0.18	0.09	0.06			0.54	0.40	0.19		
13-04		0.84	0.55		0.19	0.09	0.06		0.85		0.41	0.19	0.09	0.06	2.05	0.84	0.54	0.40	0.19	0.09	0.06
13-05	2.16	0.84	0.56	0.42	0.19	0.09	0.06	2.28	0.89	0.57	0.42	0.19	0.09	0.06	2.19	0.86	0.56	0.42	0.19	0.09	0.06
13-06	2.15	0.84	0.55		0.19	0.09	0.06	2.28	0.89	0.55		0.18	0.09	0.06	2.19	0.85	0.54		0.19		
13-07	2.18	0.85	0.54	0.41	0.19				0.89	0.57	0.42	0.19	0.09	0.06	2.21	0.86	0.55	0.40	0.19	0.09	0.06
13-08	2.26	0.85			0.19	0.08		2.28	0.85	0.51		0.19	0.08	0.06	2.16	0.84	0.53	0.41	0.19		0.06
13-09		0.83	0.54		0.19	0.09		2.56	0.85	0.53		0.19	0.09	0.06	2.18	0.83	0.53		0.19	0.09	
13-10	2.19	0.83			0.19	0.09		2.28	0.85	0.53	0.40	0.18	0.09	0.06	2.18	0.82	0.53		0.19	0.08	0.06
13-11		0.81			0.19	0.09	0.06	2.28	0.85		0.41	0.18	0.09	0.06	2.14	0.81			0.19	0.09	0.06
13-12	2.04	0.81	0.52	0.38	0.18	0.09	0.06	2.05	0.79	0.50	0.39	0.18	0.09	0.06	2.08	0.80	0.52	0.38	0.18	0.09	0.06
14-01			0.48	0.35	0.18	0.09	0.05		0.79	0.48	0.36	0.18	0.09	0.06					0.18		
14-02		0.88	0.58	0.43	0.19	0.09		2.56	0.93	0.59		0.19	0.09	0.06			0.58	0.42		0.09	

Seismic Performance of Steel Moment-Resisting Frames with Spines Examined by Shake-Table Tests

14-03	2.29	0.91		0.42	0.19	0.09		2.56	0.89	0.55		0.19	0.09	0.06	2.36	0.91	0.58	0.44	0.19	0.09
14-04	2.11	0.88	0.57	0.42	0.19	0.09	0.05		0.85	0.62	0.42	0.19	0.09	0.06		0.87	0.57	0.43	0.19	0.09
14-05	2.26	0.86	0.55	0.43	0.19	0.09	0.06	2.28	0.85			0.18	0.09	0.06		0.85	0.56	0.42	0.19	0.09
14-06		0.87	0.56	0.42	0.19	0.09		2.56		0.54		0.19	0.09	0.06			0.56	0.41	0.19	
14-07		0.81	0.54	0.40	0.19	0.09		2.56	0.93	0.54	0.40	0.19	0.09	0.06		0.82	0.54	0.40	0.19	
14-08	2.18	0.83	0.54	0.41	0.19	0.09	0.06	2.28	0.89	0.54	0.41	0.19	0.09	0.06	2.18	0.84	0.54	0.41	0.19	0.09 0.06
17-01		0.77	0.48	0.35	0.18	0.09	0.06	2.05	0.76	0.49	0.35	0.18	0.09	0.06		0.77	0.49	0.35	0.18	0.09 0.06
17-02		0.83	0.50		0.19			2.05			0.40	0.18	0.09	0.06		0.83	0.50		0.19	
17-03					0.19	0.09	0.06	2.56				0.19	0.09	0.06	2.19		0.58	0.44	0.19	
17-04	2.41	0.89			0.20		0.06	2.93	0.89			0.09	0.08	0.06	2.41		0.56		0.19	
17-05		0.85			0.19			2.56	0.89	0.62	0.37	0.20	0.09	0.06	2.40	0.87	0.60	0.49		
17-06	2.21				0.19	0.09	0.06	2.56	0.85		0.50	0.20	0.09	0.06	2.29		0.57		0.19	0.06
17-07		0.84			0.20	0.08		2.28	0.89		0.49	0.19	0.08	0.06	2.16	0.86				
17-08	2.33	0.85				0.09			0.85	0.62	0.48	0.19	0.09	0.06	2.32	0.85	0.59	0.47		
17-09					0.19	0.09	0.06	2.28			0.48	0.20	0.09	0.06	2.17	0.82		0.48		0.06
17-10	2.14							2.28		0.59	0.47	0.20	0.08	0.06	2.18	0.82		0.46		
17-11	2.20	0.77	0.51		0.18	0.09	0.06	2.28	0.79	0.48	0.38	0.18	0.09	0.06	2.17		0.50	0.38	0.18	0.09 0.06
20-01	2.13	0.81	0.51		0.19	0.09	0.06	2.28	0.85	0.54	0.39	0.19	0.09	0.06	2.14	0.82	0.51	0.39	0.19	0.09 0.05
20-02	2.24	0.89			0.19	0.09	0.06	2.05	0.89	0.55	0.40	0.18	0.09	0.06	2.18				0.19	0.09 0.06
20-03	2.35	0.84			0.19	0.09		2.28	0.82			0.21	0.09	0.06	2.37	0.85		0.49	0.20	0.09
20-04		2.37	0.88	0.58	0.47	0.19	0.09	2.56	0.93			0.21	0.09	0.06		0.90	0.59	0.49	0.20	0.09
20-05	2.37	0.89			0.20	0.09		2.56	0.89		0.49	0.21	0.09	0.06	2.41	0.90			0.20	0.08
20-06	2.39	0.90	0.57		0.20	0.09		2.93			0.46	0.19	0.09	0.06	2.42	0.88	0.57	0.46	0.19	
20-07						0.09		2.93	0.93		0.53	0.20	0.10	0.06		0.96		0.52		
20-08		0.93			0.20			2.56	0.89			0.20	0.09	0.06	2.32	0.91		0.49	0.20	
20-09	2.46	0.90		0.51	0.20	0.10		2.56	0.93			0.19	0.10	0.09	2.46	0.91	0.62	0.51		
20-10		0.89			0.20				0.89		0.47	0.19	0.09	0.06	2.36	0.90		0.45	0.20	
20-11		0.87			0.20	0.10	0.06		0.89	0.57		0.20	0.10	0.06	2.25	0.88	0.57	0.42	0.20	0.06
20-12	2.31	0.88	0.54				0.06	2.56	0.89	0.55		0.20	0.10	0.06	2.29	0.87	0.55	0.43	0.20	
20-13	2.32	0.86		0.45	0.20	0.10	0.06	2.28	0.89			0.20	0.09	0.06	2.32	0.86	0.53	0.43	0.20	0.10
23-01	2.20	0.83	0.52	0.40	0.20	0.09	0.06	2.28	0.85	0.54	0.39	0.20	0.10	0.06	2.20	0.83	0.52	0.39	0.19	0.09

Appendix H Full Results of Vibration Property Analysis: 08EDF

23-02	2.26			0.20	0.10	0.06		0.89	0.51	0.42	0.22	0.10	0.07	2.17	0.86	0.53		0.20	0.10		
23-03	2.38			0.21	0.10	0.06	2.56	0.89		0.43	0.21	0.10	0.06	2.43	0.87	0.59		0.21			
23-04				0.21	0.10		2.56		0.57		0.21	0.10	0.06	2.38	0.90		0.44	0.20	0.10		
23-05		0.92		0.48	0.10		2.56	0.89		0.49	0.21	0.10	0.06	2.46	0.90		0.48	0.21			
23-06	2.35	0.89			0.20		0.07	2.93		0.57		0.19	0.10	0.06	2.39	0.88	0.57	0.44	0.20		
23-07					0.10			2.93	0.98		0.57	0.20	0.10	0.06		0.96	0.60		0.21		
23-08		0.94			0.10	0.06	2.56	0.89		0.47	0.20	0.10	0.06	2.30	0.92		0.46	0.20	0.10		
23-09	2.42	0.89			0.10	0.06	2.56	0.93			0.19	0.10	0.06	2.44	0.89			0.21	0.10		
23-10		0.92			0.20	0.10	0.06		0.93		0.46	0.21	0.10	0.06	2.38	0.91	0.55		0.20		
23-11		0.88			0.20	0.10		2.56	0.89	0.54	0.48	0.20	0.10	0.06	2.33	0.88		0.43	0.20		
23-12	2.33	0.87	0.54	0.41			2.56	0.89	0.54	0.39	0.20	0.10	0.06	2.29	0.88	0.55	0.41	0.20	0.06		
23-13	2.34	0.86			0.20	0.10	0.06	2.28	0.93	0.54	0.43	0.20	0.10	0.06	2.34	0.87	0.54	0.43	0.20	0.10	
26-01	2.26	0.85	0.53		0.20	0.10	0.06	2.56	0.85	0.54		0.20	0.10	0.06	2.28	0.85	0.53	0.40	0.19	0.10	
26-02		0.87			0.20	0.10			0.89	0.59	0.44	0.20	0.10	0.06	2.09	0.88		0.40	0.20	0.10	0.06
26-03		0.89			0.20	0.10	0.06	2.28	0.89	0.60		0.21	0.11	0.06	2.45	0.89	0.59	0.45	0.21	0.10	
26-04					0.21	0.10	0.06	2.56			0.45	0.21	0.10	0.06	2.39	0.91	0.61	0.45	0.21	0.10	
26-05			0.90	0.48	0.20		0.06	2.56	0.89		0.50	0.21	0.10	0.06	2.47	0.91		0.48	0.21		
26-06	2.28	0.90	0.58	0.44	0.20	0.10	0.06	2.93	0.89	0.60	0.47	0.19	0.10	0.06	2.39	0.89	0.59	0.43	0.20	0.10	
26-07						0.10	0.06	2.93	0.98		0.57	0.20	0.09	0.06		0.97			0.21		
26-08		0.96				0.10		2.56	0.89			0.20	0.10	0.06	2.29	0.94	0.60	0.48	0.21		
26-09	2.51	0.94			0.21			2.56	0.98			0.21	0.10	0.06	2.57	0.94			0.21		
26-10		0.90		0.45	0.21	0.10			0.93	0.62	0.48	0.22	0.10	0.06	2.45	0.91	0.56	0.44	0.21	0.10	
26-11		0.92	0.56		0.21	0.10		2.56	0.93	0.55		0.10	0.06		2.47	0.92	0.57		0.21	0.10	
26-12	2.38	0.91	0.55	0.43	0.20	0.10	0.06	2.56	0.89	0.55	0.44	0.20	0.10	0.06	2.44	0.91	0.55	0.43		0.10	
26-13	2.45	0.89	0.57	0.42	0.20	0.10	0.06	2.56	0.93	0.55	0.42	0.20	0.10	0.06	2.44	0.89	0.56	0.42	0.21	0.10	

H.3.5 PSD peaks

Figure H.12 plots the peak values of the Power Spectra Density (PSD) in the white noise cases of the 08EDF test identified using the FDD method. Table H.8 summarizes the corresponding values.

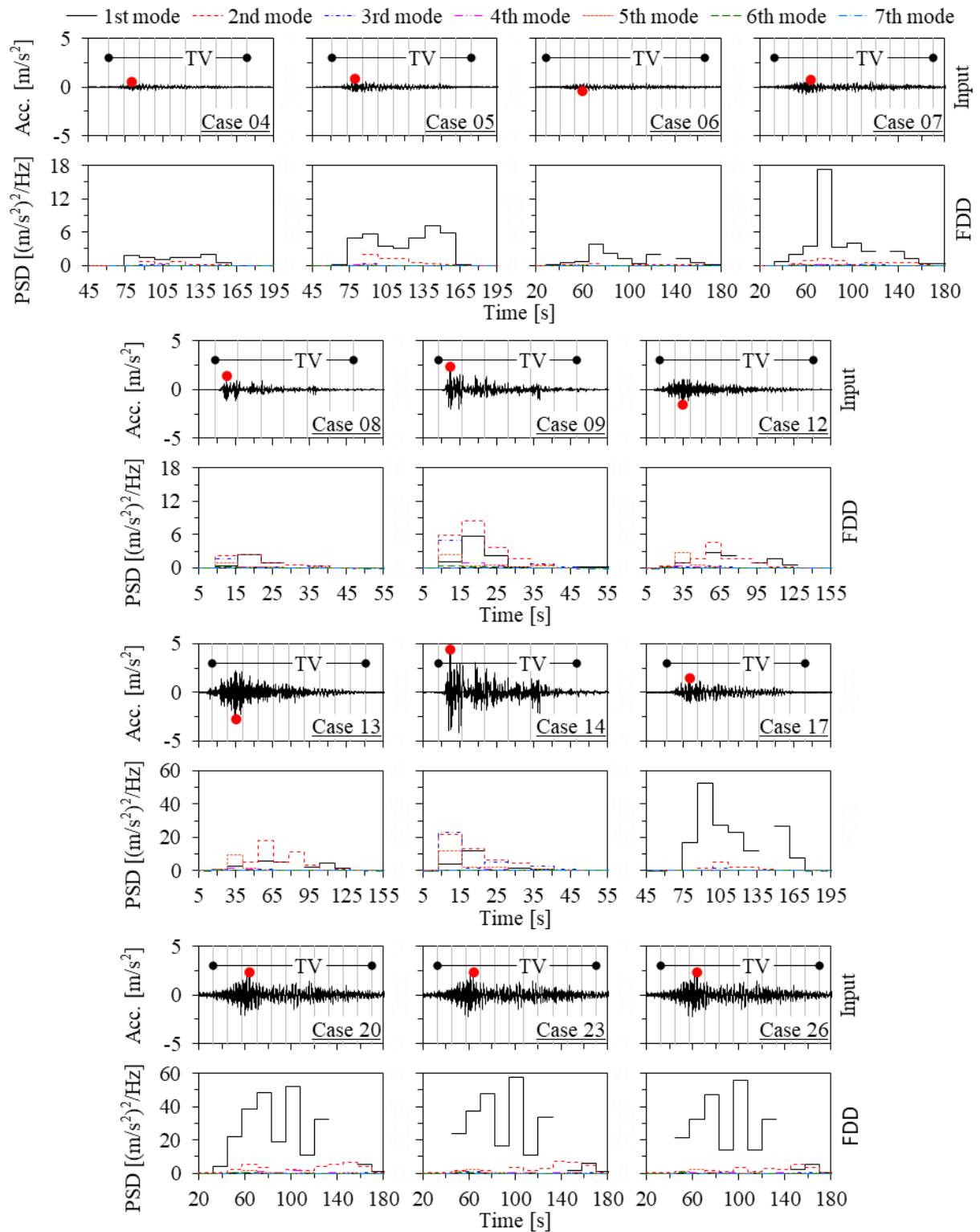


Figure H.12 PSD peaks in ground motion excitations of the 08EDF test

Table H.8 PSD peaks in ground motion excitations of the 08EDF test [Unit: (m/s²)²/Hz]

Case-Segment	Mode						
	1st	2nd	3rd	4th	5th	6th	7th
04-01	0.000	0.000	0.000	0.000	0.002	0.000	0.000
04-02	0.024				0.003	0.000	0.000
04-03	1.812			0.010	0.012	0.005	0.014
04-04	1.378	0.726	0.037	0.003	0.008	0.010	0.002
04-05	1.105	0.376	0.040		0.005	0.007	0.006
04-06	1.368	0.590	0.005		0.002	0.002	0.003
04-07	1.391	0.143		0.003	0.003	0.003	0.006
04-08	1.945	0.112			0.002	0.004	0.005
04-09	0.571	0.039			0.002	0.002	0.002
04-10	0.004	0.002	0.001	0.000	0.001	0.000	0.000
04-11	0.000	0.000	0.000	0.000	0.001	0.000	0.001
05-01	0.000	0.000	0.000	0.000	0.001	0.000	0.000
05-02	0.067			0.001	0.011	0.001	0.001
05-03	4.958			0.047	0.031	0.013	0.005
05-04	5.708	1.956	0.237	0.017	0.042	0.017	0.017
05-05	3.385	1.161			0.013	0.008	0.006
05-06	3.161	1.200			0.013	0.006	0.011
05-07	4.966	0.461			0.008	0.014	0.015
05-08	7.214	0.319		0.009	0.010	0.012	0.007
05-09	5.860	0.160		0.003	0.017	0.021	0.007
05-10	0.199	0.006	0.002	0.000	0.003	0.000	0.001
05-11	0.001	0.000	0.000	0.000	0.002	0.000	0.000
06-01	0.008	0.002	0.000	0.000	0.001	0.000	0.000
06-02	0.041	0.004	0.001		0.003	0.001	0.001
06-03	0.505	0.063	0.004		0.015	0.008	0.001
06-04	0.748			0.006	0.060	0.004	0.003
06-05	3.763	0.324	0.013		0.009	0.007	0.008
06-06	2.097		0.019		0.003	0.005	0.004
06-07	1.262	0.034	0.013	0.002	0.006	0.004	0.001
06-08	0.375	0.214		0.002	0.005	0.002	0.006
06-09	1.985	0.180			0.005	0.006	0.006
06-10		0.125	0.015		0.010	0.003	0.001
06-11	1.238	0.082	0.011	0.001	0.002	0.003	0.003
06-12	0.446	0.080		0.000	0.001	0.001	0.001
06-13	0.210	0.003	0.001	0.000	0.001	0.001	0.000
07-01	0.032	0.011	0.002	0.000	0.002	0.001	0.000
07-02	0.616	0.013	0.005	0.002	0.011	0.010	0.001
07-03	2.017	0.299	0.076	0.007	0.129	0.021	0.003
07-04	3.527	0.783	0.157		0.156	0.029	0.015
07-05	17.279	1.205	0.069	0.037	0.023	0.027	0.016
07-06	3.320	0.926	0.041	0.007	0.048	0.016	0.009
07-07	3.996	0.081	0.051	0.035	0.026	0.013	0.013
07-08	2.453	0.491	0.094		0.014	0.011	0.067
07-09		0.457	0.070		0.017	0.007	0.003
07-10	2.465	0.490	0.014	0.010	0.030	0.012	0.011
07-11	1.144	0.476		0.003	0.014	0.001	0.006

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07-12	0.350	0.068		0.001	0.003	0.002	0.002
07-13	0.375	0.010	0.002	0.000	0.002	0.001	0.001
08-01	0.000	0.000	0.000	0.000	0.001	0.000	0.000
08-02	0.386	2.118	1.605		0.929	0.168	0.018
08-03	2.316	2.380	0.184	0.229	0.115	0.076	0.013
08-04	0.930	0.951	0.111	0.067	0.147	0.066	0.010
08-05		0.537			0.014	0.003	0.008
08-06	0.039	0.290	0.306		0.159	0.003	0.001
08-07		0.009	0.034		0.003	0.001	0.000
08-08	0.041	0.003	0.001	0.000	0.001	0.000	0.000
09-01		0.000			0.000	0.000	0.000
09-02	1.055	5.868	5.012		2.378	0.410	0.049
09-03	5.775	8.507		0.953	0.451	0.123	0.067
09-04	2.138	3.724		0.420	0.562	0.126	0.025
09-05	0.107	1.658			0.115	0.014	0.009
09-06		0.628	0.761		0.295	0.003	0.004
09-07		0.015	0.159	0.011	0.003	0.003	0.002
09-08	0.107	0.013	0.004	0.001	0.001	0.001	0.000
12-01			0.000	0.000	0.000	0.000	0.000
12-02	0.089	0.380			0.268	0.006	0.001
12-03	0.829	0.908	0.204	0.400	2.829	0.018	0.005
12-04		1.598	0.174	0.123	0.553	0.011	0.006
12-05	2.770	4.654	0.301	0.137	0.146	0.013	0.005
12-06	2.203	1.576	0.093	0.013	0.027	0.003	0.005
12-07		1.742		0.016	0.012	0.009	0.006
12-08	0.861	0.846	0.021		0.008	0.002	0.003
12-09	1.640	0.145			0.005	0.004	0.008
12-10	0.561	0.183	0.009		0.004	0.001	0.001
12-11		0.019	0.002	0.000	0.001	0.000	0.000
12-12	0.044	0.000	0.000	0.000	0.001	0.000	0.000
13-01				0.000	0.000	0.000	0.000
13-02	0.213	1.046			1.144	0.024	0.002
13-03	2.650			1.257	9.210	0.031	0.027
13-04		5.464		0.687	1.228	0.024	0.010
13-05	5.882	18.317	1.121	0.383	0.419	0.057	0.069
13-06	5.122	5.364	0.306		0.096	0.022	0.012
13-07		11.150	0.173	0.112	0.052	0.027	0.011
13-08	2.109	3.116	0.035		0.010	0.005	0.010
13-09	4.398	0.359	0.035		0.012	0.006	0.007
13-10	1.454	0.625	0.021	0.003	0.005	0.007	0.002
13-11	0.437	0.086		0.001	0.002	0.003	0.011
13-12	0.104	0.001	0.000	0.000	0.002	0.001	0.000
14-01		0.000	0.000	0.000	0.000	0.000	0.000
14-02	3.814	21.783	23.187		12.011	0.518	0.107
14-03	11.737	13.416	1.725		1.857	0.143	0.029
14-04		6.720	5.419	0.970	2.149	0.322	0.011
14-05	1.438	4.848			0.189	0.082	0.014
14-06	1.115		2.774		0.820	0.036	0.013
14-07	0.156	0.045	1.155	0.064	0.026	0.007	0.001

Appendix H Full Results of Vibration Property Analysis: 08EDF

14-08	0.482	0.038	0.025	0.003	0.005	0.001	0.001
17-01	0.000	0.000	0.000	0.000	0.001	0.000	0.000
17-02	0.315			0.002	0.007	0.002	0.001
17-03	17.087				0.044	0.016	0.013
17-04	52.859	1.292			0.018	0.032	0.020
17-05	27.549	5.322	1.653	0.036	0.052	0.029	0.034
17-06	23.296	2.318		0.157	0.030	0.037	0.034
17-07	12.235	1.907		0.084	0.019	0.020	0.082
17-08		0.990	0.088	0.019	0.012	0.006	0.010
17-09	26.950			0.178	0.021	0.017	0.027
17-10	7.670		0.019	0.067	0.005	0.011	0.005
17-11	0.025	0.000	0.000	0.000	0.002	0.000	0.000
20-01	0.092	0.159	0.011	0.008	0.012	0.009	0.000
20-02	3.877	0.260	0.016	0.022	0.038	0.036	0.012
20-03	22.043	2.198			0.484	0.326	0.027
20-04	38.306	5.282			1.724	0.140	0.083
20-05	48.668	3.239		1.039	0.342	0.028	0.020
20-06	18.899			0.184	0.235	0.038	0.016
20-07	52.416	2.112		1.859	0.103	0.043	0.021
20-08	11.074	1.754			0.300	0.045	0.009
20-09	32.547	3.962			0.051	0.029	0.048
20-10		5.640		0.130	0.076	0.013	0.035
20-11		6.523	0.078		0.203	0.010	0.010
20-12	5.283	4.147	0.225		0.047	0.008	0.013
20-13	0.788	0.146			0.005	0.002	0.001
23-01	0.066	0.186	0.012	0.013	0.010	0.005	0.000
23-02		0.332	0.026	0.121	0.055	0.064	0.005
23-03	23.800	1.664		0.057	0.901	0.321	0.019
23-04	37.253		0.895		2.352	0.190	0.019
23-05	47.599	1.904		0.731	0.399	0.045	0.034
23-06	16.333		0.119		0.145	0.011	0.009
23-07	57.505	3.385		0.415	0.104	0.058	0.027
23-08	10.764	1.391		0.390	0.258	0.098	0.010
23-09	33.704	2.687			0.066	0.072	0.027
23-10		7.098		0.145	0.073	0.023	0.020
23-11	1.653	6.575	0.166	0.174	0.181	0.016	0.002
23-12	6.267	4.950	0.214	0.084	0.020	0.010	0.010
23-13	0.866	0.158	0.025	0.014	0.006	0.004	0.001
26-01	0.060	0.147	0.011		0.010	0.006	0.000
26-02		0.304	0.051	0.030	0.048	0.053	0.009
26-03	21.328	1.173	0.244		0.783	0.154	0.036
26-04	32.406			0.569	2.174	0.100	0.020
26-05	47.213	1.522		0.739	0.424	0.015	0.050
26-06	13.995	1.156	0.130	0.238	0.096	0.030	0.004
26-07	55.946	3.341		0.361	0.114	0.018	0.034
26-08	14.082	0.811			0.261	0.037	0.015
26-09	32.680	2.624			0.038	0.060	0.027
26-10		2.412	0.055	0.132	0.035	0.020	0.008
26-11	2.495	5.396	0.290		0.027	0.014	

26-12	5.095	3.759	0.211	0.096	0.024	0.015	0.004
26-13	0.622	0.192	0.030	0.013	0.005	0.005	0.001

H.4 EFFECTIVE MODAL PSEUDO-ACCELERATION (EMPA)

Figure H.13 plots the peak EMPA values identified in the ground motion cases of the 08EDF test, including the elastic pseudo-acceleration spectra with 2% damping of the measured excitations.

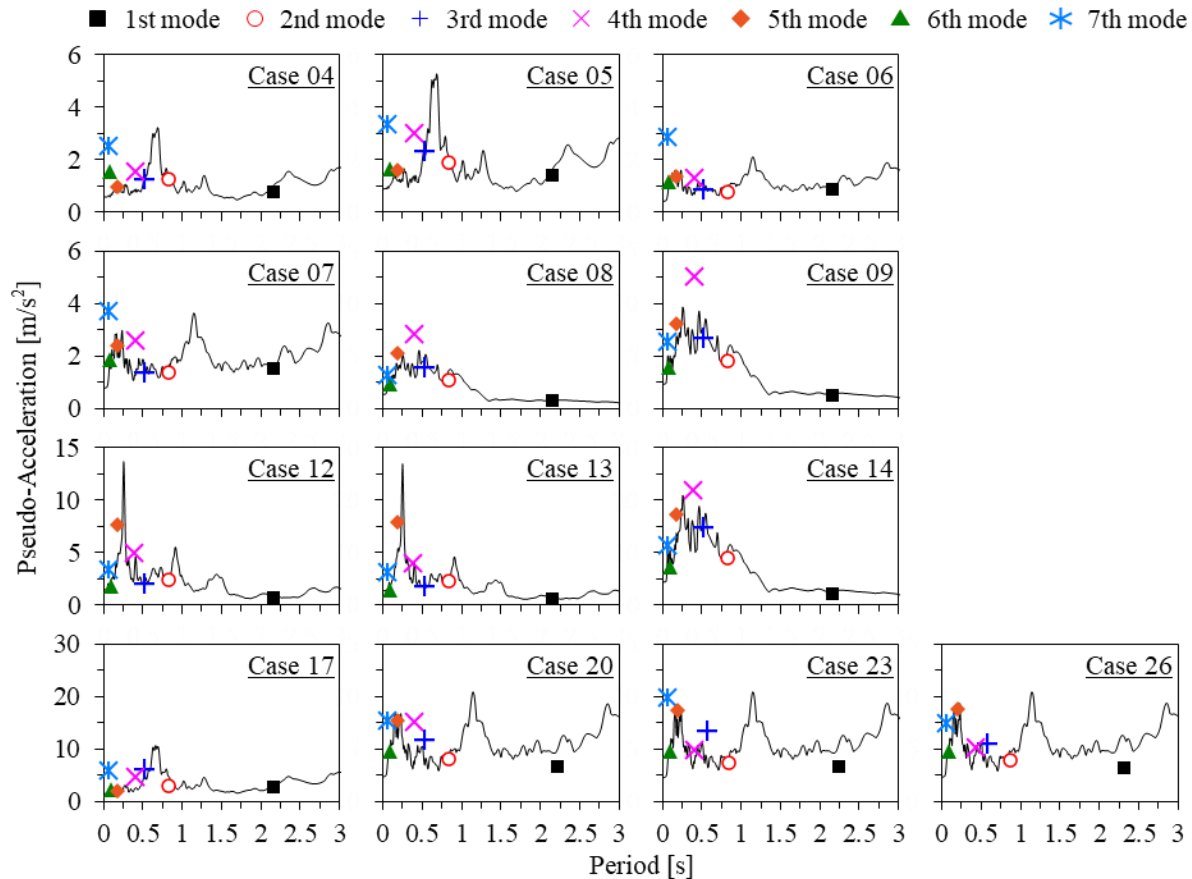


Figure H.13 Peak EMPA values in ground motion excitations of the 08EDF test

H.5 TIME-FREQUENCY RESULTS FROM GROUND MOTION EXCITATIONS

H.5.1 Magnitude of wavelet coefficients and instantaneous periods

Figure H.14 plots the magnitude of wavelet coefficients on the time-frequency plane and the identified instantaneous periods in the ground motion cases of the 08EDF test.

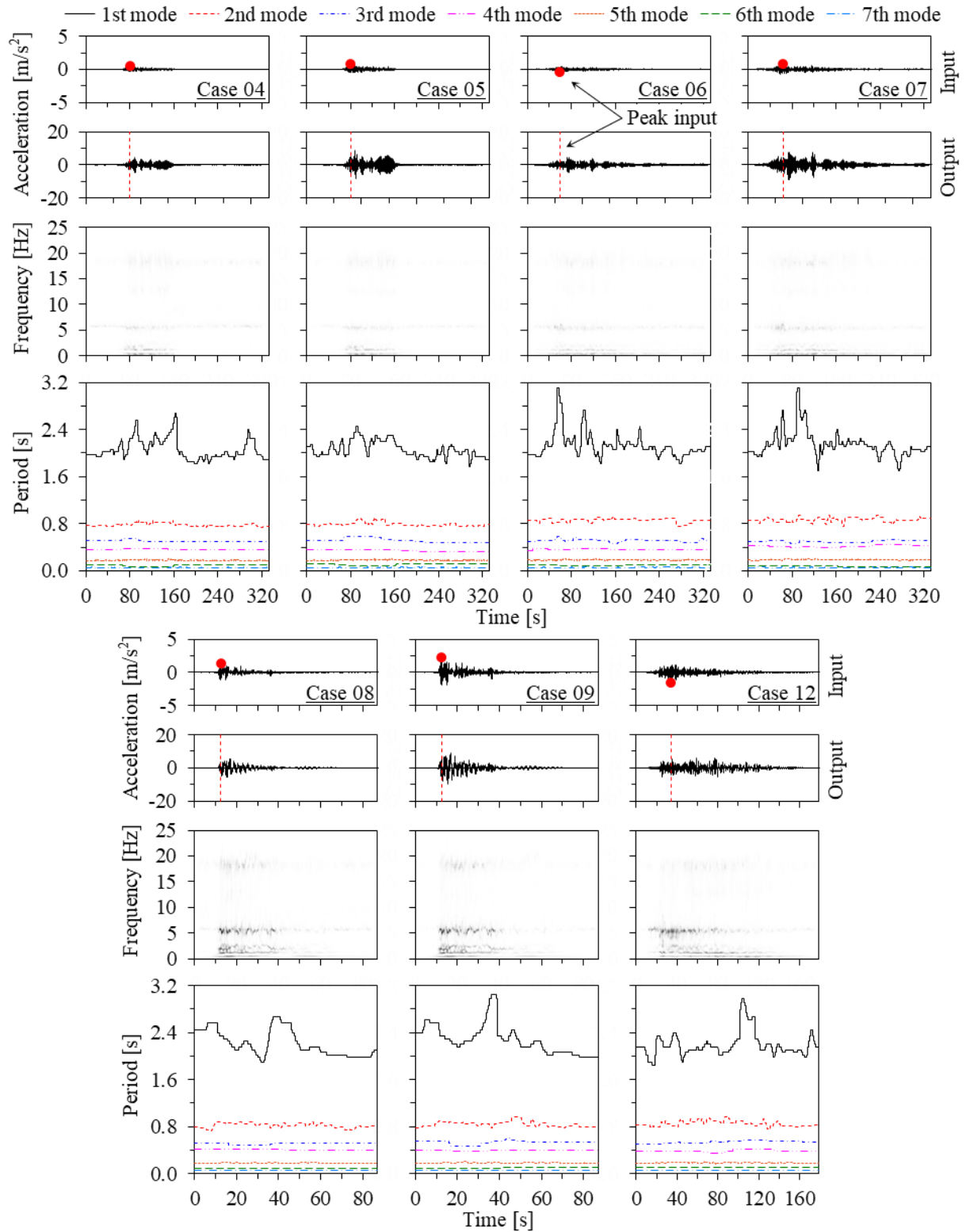


Figure H.14 Magnitude of wavelet coefficients in ground motion excitations of the 08EDF test

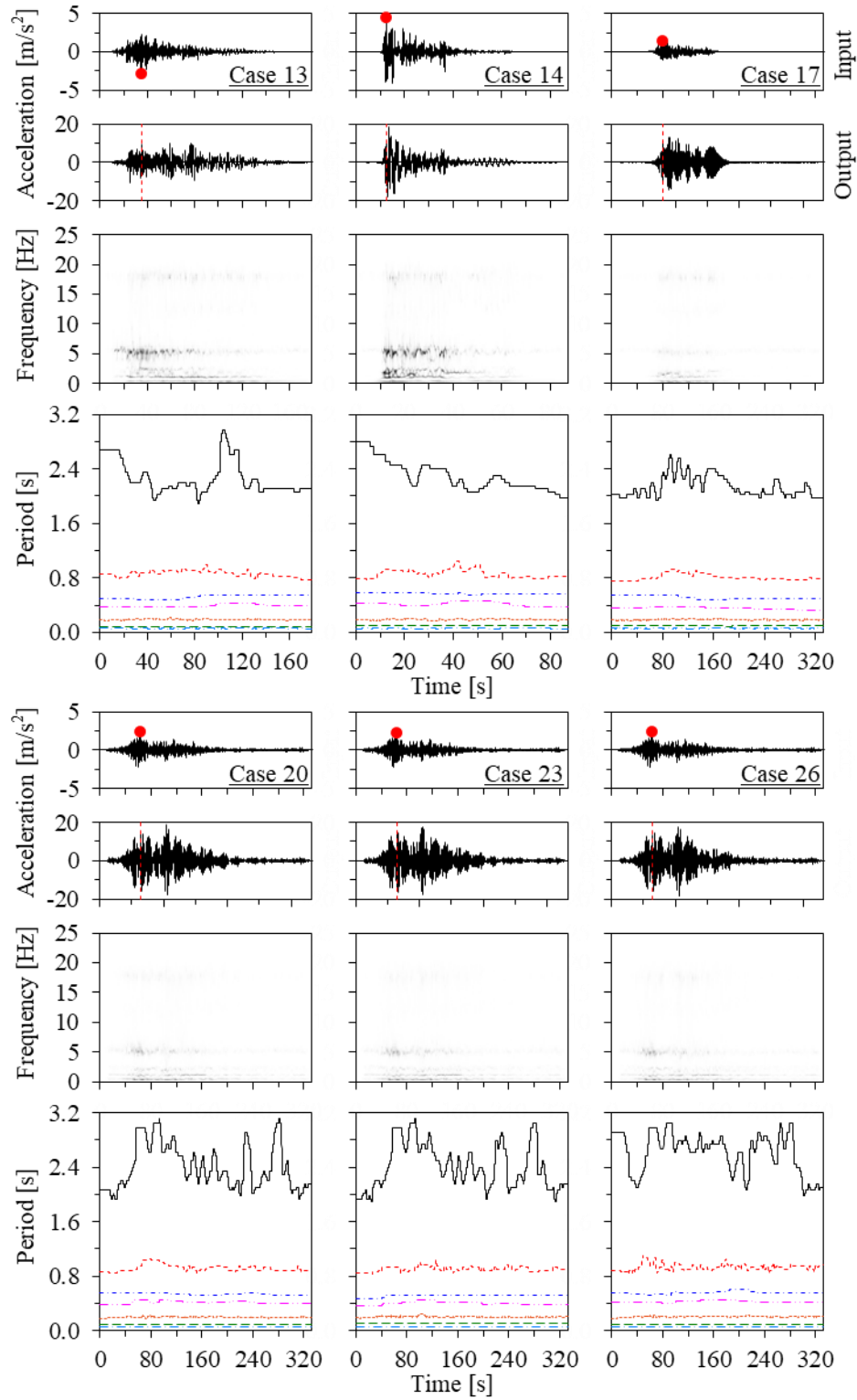


Figure H.14 (continued) Magnitude of wavelet coefficients in ground motion excitations of the 08EDF test

H.5.2 Magnitude of ridge wavelet coefficient

Figure H.15 plots the magnitude of wavelet coefficient on the wavelet ridges in the ground motion cases of the 08EDF test.

H.5.3 Reconstructed modal components

Figure H.16 plots the time-domain modal components reconstructed from the SSWT results and ridge identification in the ground motion cases of the 08EDF test.

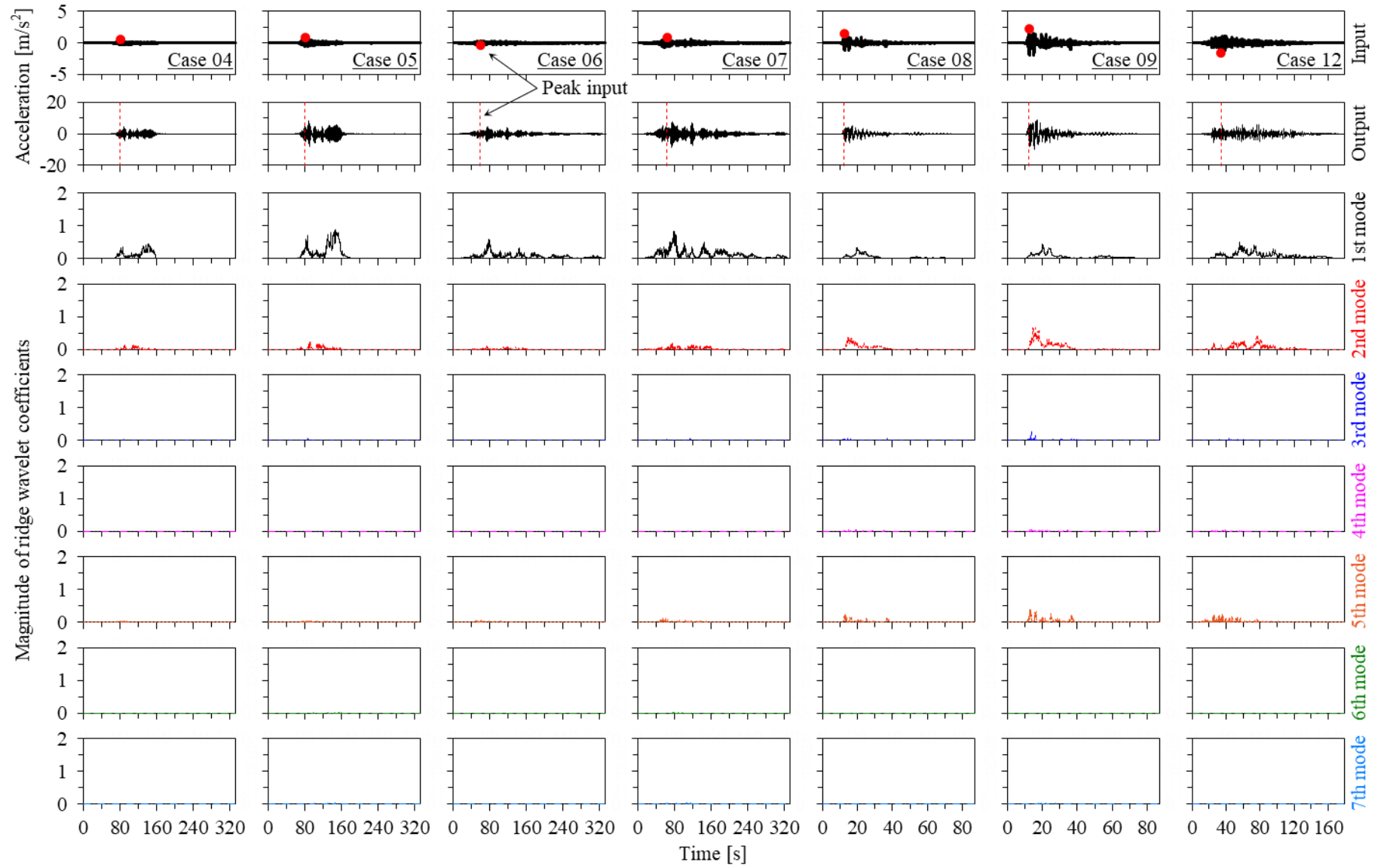


Figure H.15 Magnitude on wavelet ridges in ground motion excitations of the 08EDF test

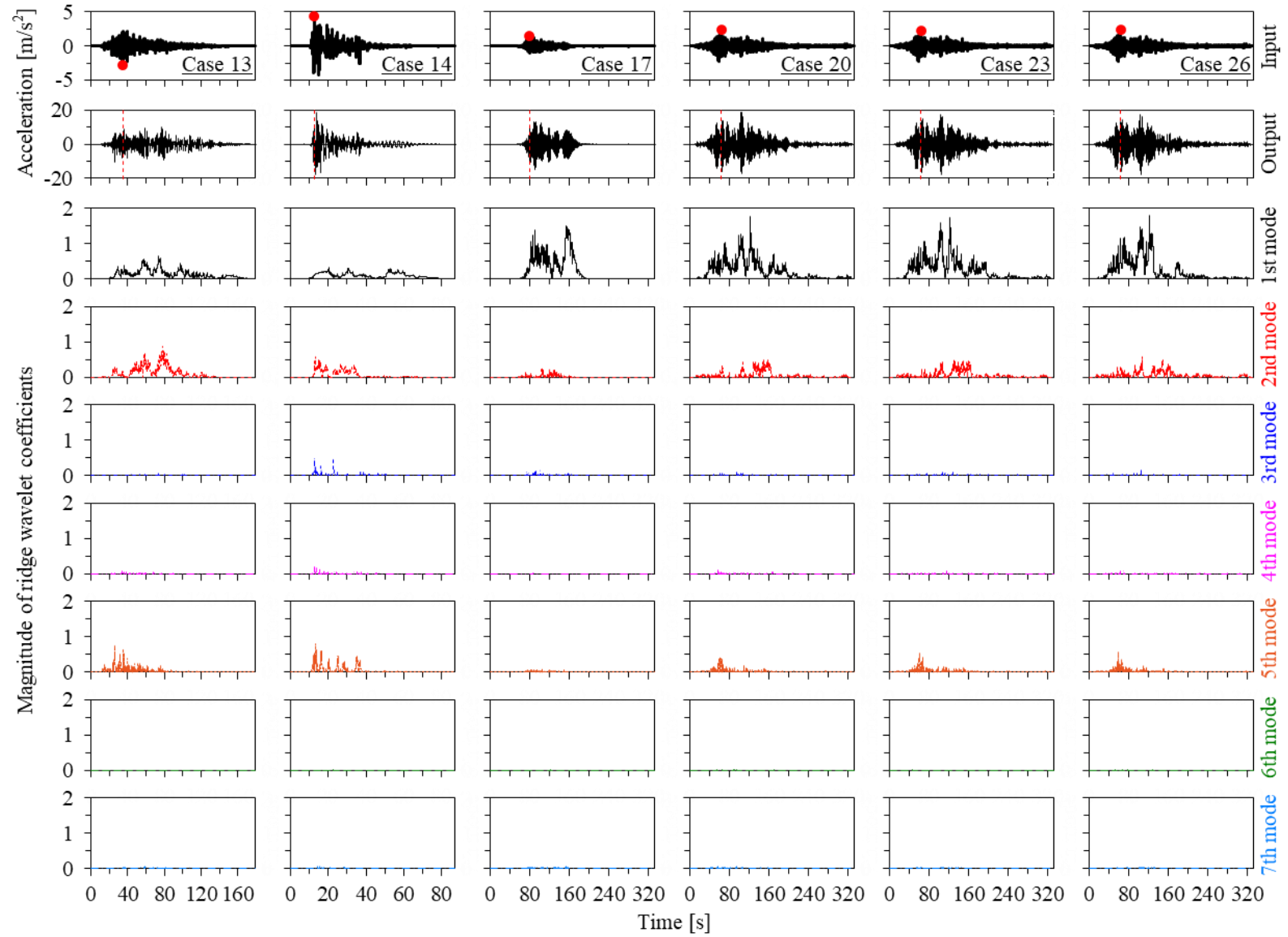


Figure H.15 (continued) Magnitude on wavelet ridges in ground motion excitations of the 08EDF test



Figure H.16 Magnitude on wavelet ridges in ground motion excitations of the 08EDF test

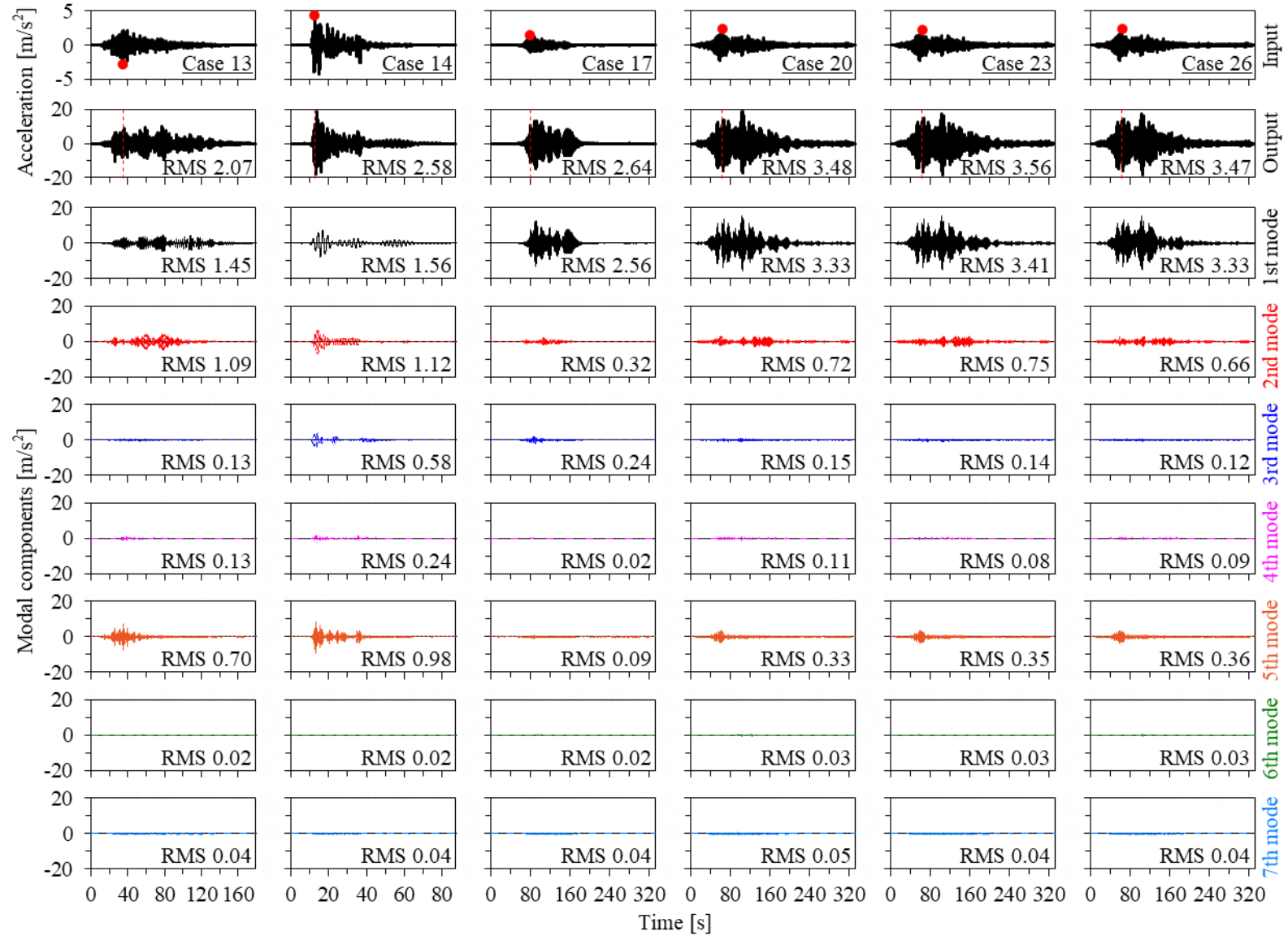


Figure H.16 (continued) Magnitude on wavelet ridges in ground motion excitations of the 08EDF test

APPENDIX I. FULL RESULTS OF VIBRATION PROPERTY ANALYSIS: 14UCSD

I.1 GENERAL

This appendix provides results of the vibration property analysis on the 14UCSD test. The results are grouped by different tests, each including results from all analysis methods. The following results are provided in this appendix:

- 1) **Modal parameters:** natural periods, modal damping ratios, mode shapes, Modal Effective Mass (MEM) and PSD peaks from modal analysis methods.
- 2) **Spectral results:** peak values of Effective Modal Pseudo-Acceleration (EMPA).
- 3) **Time-frequency results:** magnitude of wavelet coefficients, instantaneous periods, magnitude of ridge wavelet coefficient and reconstructed modal components.

In the 14UCSD test, 19 white noise and 22 ground motion cases in the East-West direction were selected for analysis. The structural system was changed from Flat Slab-Rocking Wall-FLC to Flat Slab-Rocking Wall between Cases 25 and 26, and further to Flat Slab-Shear Wall between Cases 35 and 36.

I.2 MODAL PARAMETERS FROM WHITE NOISE EXCITATIONS

I.2.1 Natural period

Figure I.1 plots the natural period in the white noise cases of the 14UCSD test identified using the ARX, FDD and SS methods, respectively. Table I.1 summarizes the corresponding values.

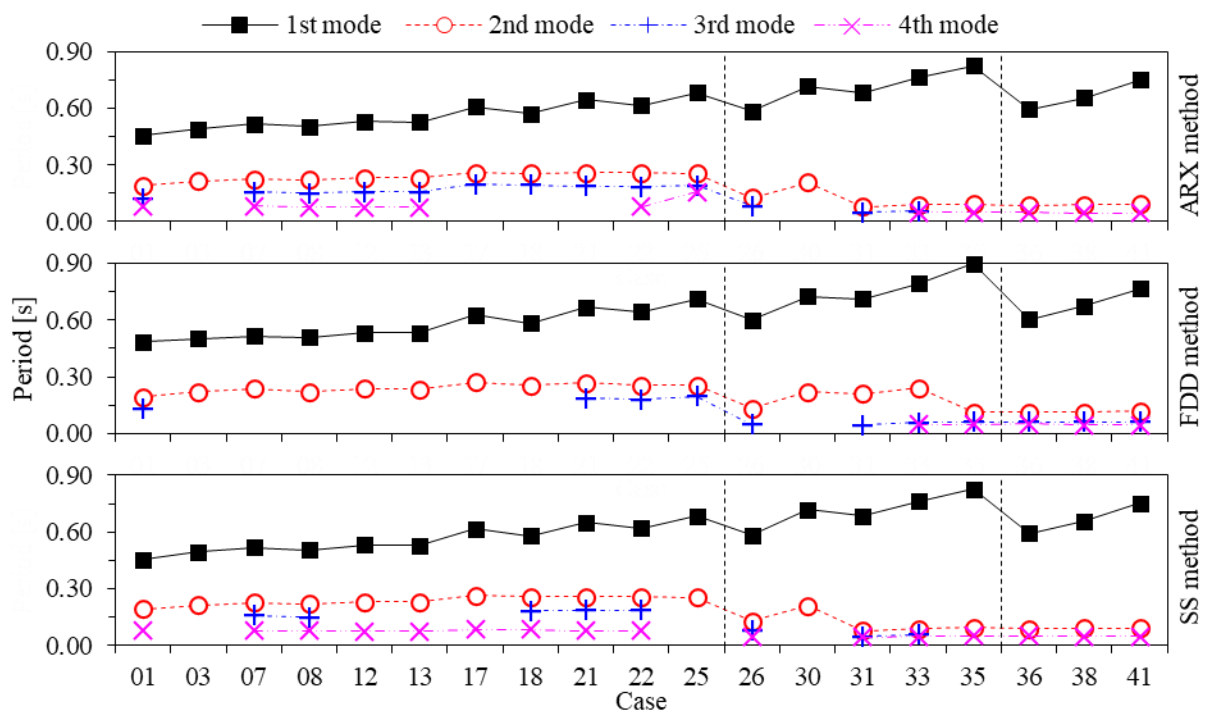


Figure I.1 Natural periods in white noise excitations of the 14UCSD test

Table I.1 Natural periods in white noise excitations of the 14UCSD test [Unit: s]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.45	0.48	0.45	0.19	0.20	0.19	0.12	0.13		0.08		0.08
03	0.49	0.50	0.49	0.21	0.22	0.21						
07	0.52	0.52	0.52	0.23	0.24	0.23	0.16		0.16	0.08		0.08
08	0.50	0.51	0.50	0.22	0.22	0.22	0.15		0.15	0.08		0.08
12	0.53	0.53	0.53	0.23	0.24	0.23	0.16			0.07		0.08
13	0.53	0.53	0.53	0.23	0.24	0.23	0.16			0.07		0.08
17	0.61	0.63	0.62	0.26	0.27	0.26	0.20					0.08
18	0.57	0.58	0.58	0.26	0.26	0.26	0.19		0.18			0.08
21	0.65	0.67	0.65	0.26	0.27	0.26	0.19	0.19	0.19			0.08
22	0.62	0.64	0.62	0.26	0.26	0.26	0.18	0.18	0.18	0.08		0.08
25	0.68	0.71	0.68	0.26	0.25	0.26	0.19	0.20		0.16		
26	0.58	0.60	0.58	0.13	0.13	0.13	0.08	0.05	0.08			0.04
30	0.72	0.73	0.72	0.21	0.22	0.21						
31	0.68	0.71	0.68	0.08	0.21	0.08	0.05	0.04	0.05			0.04
33	0.77	0.79	0.76	0.09	0.24	0.09	0.06	0.06	0.06	0.05	0.05	0.05
35	0.83	0.90	0.83	0.09	0.11	0.09		0.06		0.05	0.05	0.05
36	0.59	0.60	0.59	0.09	0.11	0.09		0.06		0.05	0.05	0.05
38	0.66	0.68	0.66	0.09	0.11	0.09		0.06		0.05	0.04	0.05
41	0.75	0.77	0.75	0.09	0.12	0.09		0.06		0.05	0.04	0.05

I.2.2 Modal damping ratio

Figure I.2 plots the modal damping ratios in the white noise cases of the 14UCSD test identified using the ARX, FDD and SS methods, respectively. Table I.2 summarizes the corresponding values.

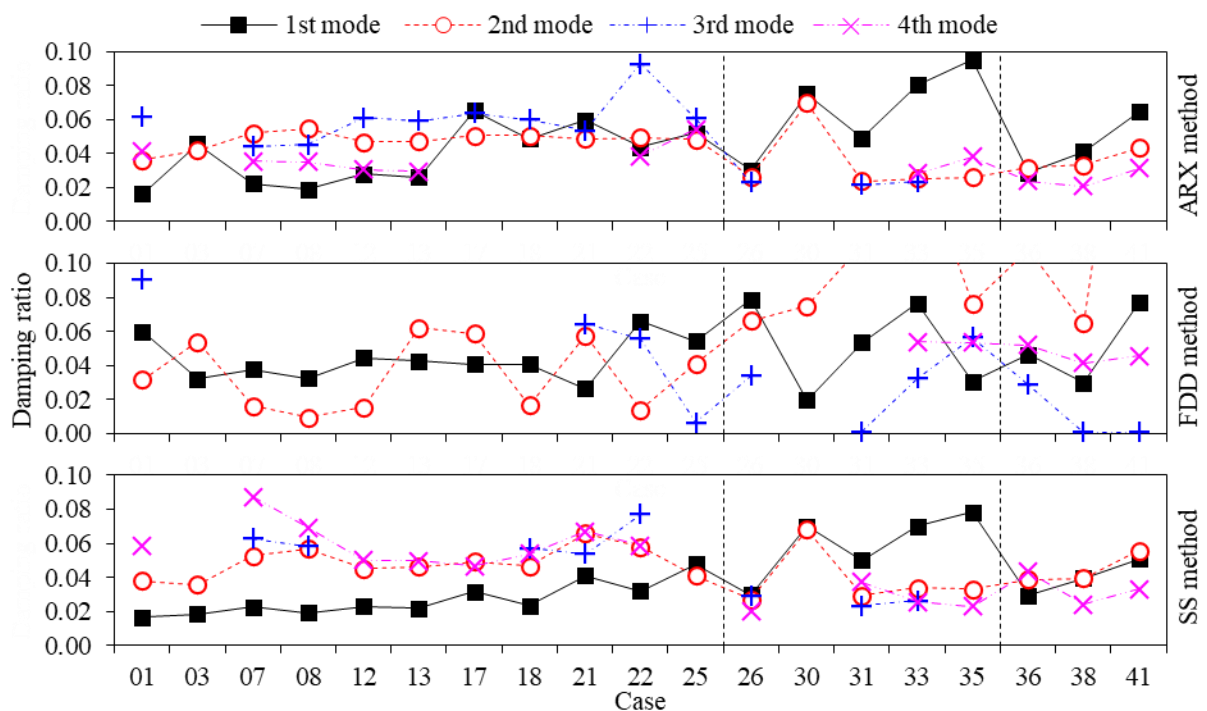


Figure I.2 Damping ratios in white noise excitations of the 14UCSD test

Table I.2 Damping ratios in white noise excitations of the 14UCSD test

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	0.02	0.06	0.02	0.04	0.03	0.04	0.06	0.09		0.04		0.06
03	0.05	0.03	0.02	0.04	0.05	0.04						
07	0.02	0.04	0.02	0.05	0.02	0.05	0.04		0.06	0.04		0.09
08	0.02	0.03	0.02	0.05	0.01	0.06	0.05		0.06	0.04		0.07
12	0.03	0.04	0.02	0.05	0.02	0.05	0.06			0.03		0.05
13	0.03	0.04	0.02	0.05	0.06	0.05	0.06			0.03		0.05
17	0.07	0.04	0.03	0.05	0.06	0.05	0.06					0.05
18	0.05	0.04	0.02	0.05	0.02	0.05	0.06		0.06			0.05
21	0.06	0.03	0.04	0.05	0.06	0.07	0.05	0.06	0.05			0.07
22	0.04	0.07	0.03	0.05	0.01	0.06	0.09	0.06	0.08	0.04		0.06
25	0.05	0.05	0.05	0.05	0.04	0.04	0.06	0.01		0.05		
26	0.03	0.08	0.03	0.03	0.07	0.03	0.02	0.03	0.03			0.02
30	0.08	0.02	0.07	0.07	0.07	0.07						
31	0.05	0.05	0.05	0.02	0.11	0.03	0.02	0.00	0.02			0.04
33	0.08	0.08	0.07	0.03	0.17	0.03	0.02	0.03	0.03	0.03	0.05	0.03
35	0.09	0.03	0.08	0.03	0.08	0.03		0.06		0.04	0.05	0.02
36	0.03	0.05	0.03	0.03	0.11	0.04		0.03		0.02	0.05	0.04
38	0.04	0.03	0.04	0.03	0.06	0.04		0.00		0.02	0.04	0.02
41	0.06	0.08	0.05	0.04	0.28	0.06		0.00		0.03	0.05	0.03

I.2.3 Mode shapes

Figure I.3 plots the mode shapes in the white noise cases of the 14UCSD test identified using the ARX, FDD and SS methods, respectively. The plotted mode shapes are normalized by the modal participation factor Γ_n .

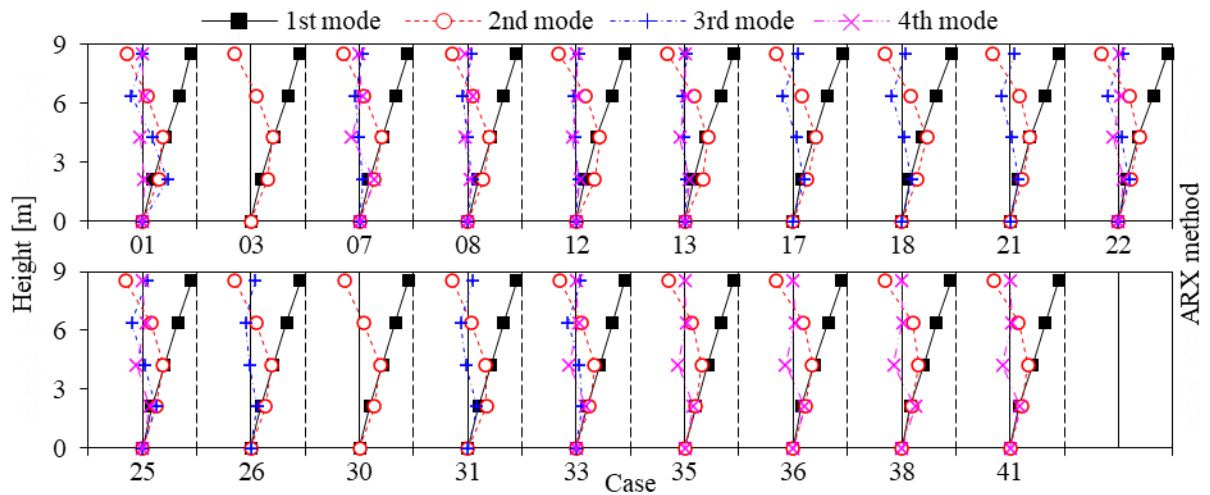


Figure I.3 Mode shapes in white noise excitations of the 14UCSD test

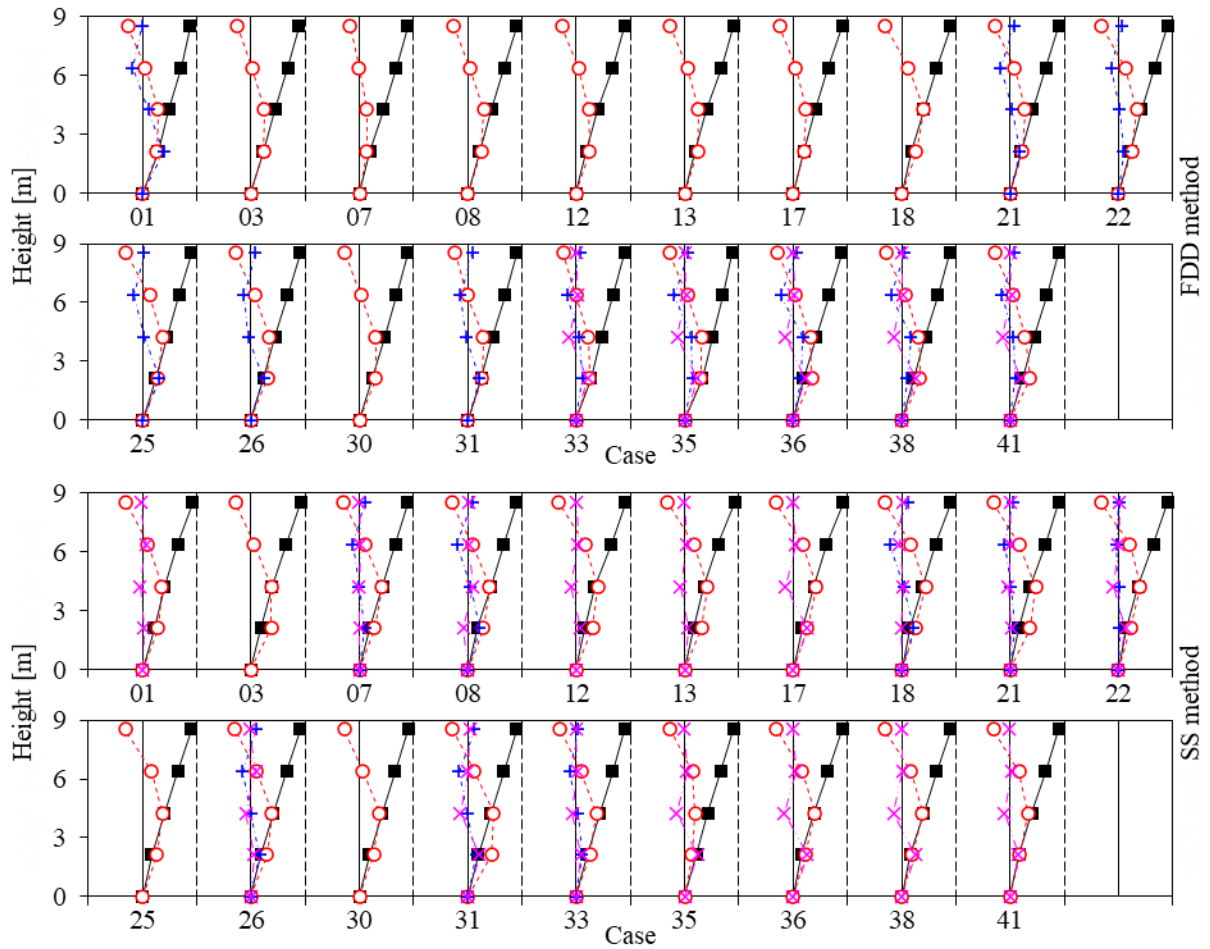


Figure I.3 (continued) Mode shapes in white noise excitations of the 14UCSD test

I.2.4 Modal Effective Mass (MEM)

Figure I.4 plots the Modal Effective Mass (MEM) in the white noise cases of the 14UCSD test identified using the ARX, FDD and SS methods, respectively. Table I.3 summarizes the corresponding values of the MEMs. The total mass of the structure is 217 tons, with Floor 1 to 3 being 55 tons and Floor 4 being 52 tons.

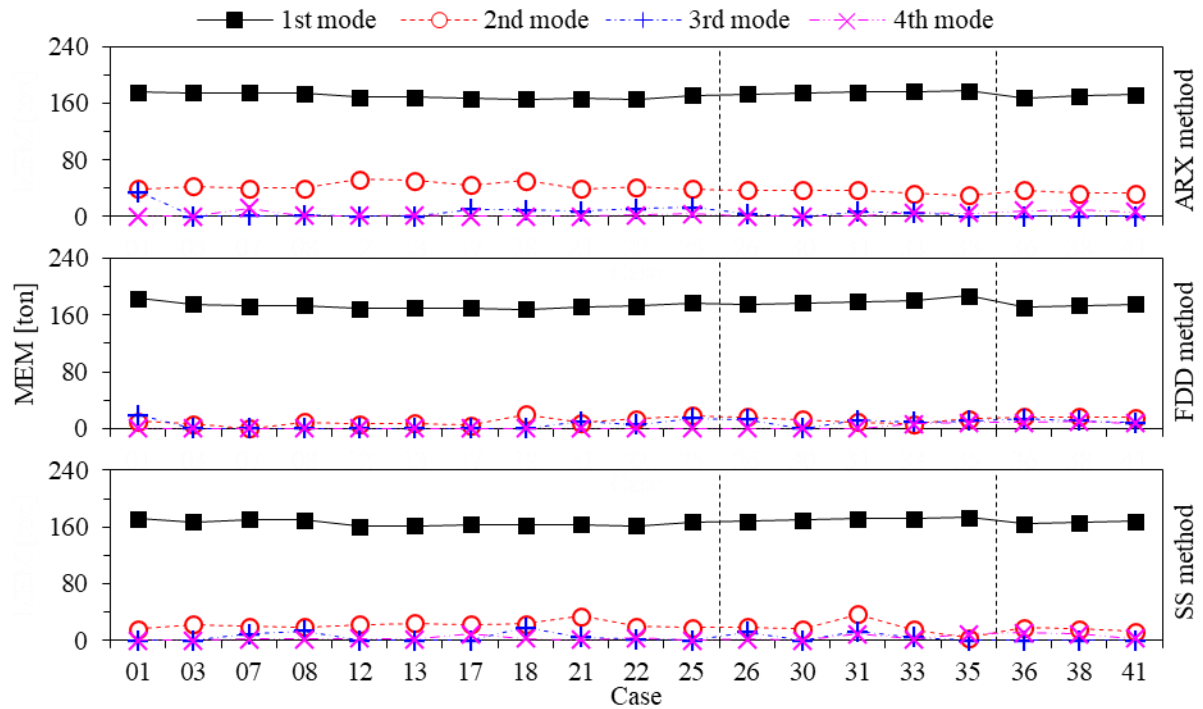


Figure I.4 MEMs in white noise excitations of the 14UCSD test

Table I.3 MEMs in white noise excitations of the 14UCSD test [Unit: ton]

Case	1st mode			2nd mode			3rd mode			4th mode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
01	176.0	183.6	171.3	39.0	9.9	16.4	35.0	19.6		0.5		0.0
03	175.3	175.8	166.8	42.6	6.2	22.3						
07	175.0	172.6	171.1	40.3	0.6	19.9	1.6		9.1	12.6		1.4
08	174.4	173.3	169.8	40.5	9.5	19.1	2.6		13.8	1.6		2.0
12	169.0	168.7	160.6	52.6	6.6	22.2	0.8			1.2		2.3
13	168.8	170.1	161.9	51.4	7.7	24.5	0.6			1.9		1.3
17	166.9	169.9	163.5	44.9	5.1	22.7	10.6					8.7
18	166.0	167.9	162.2	51.0	19.6	23.5	9.1		18.0			2.3
21	167.0	171.8	163.4	39.6	8.4	34.0	7.2	10.5	4.6			0.4
22	166.0	172.8	161.9	42.0	14.1	20.1	11.4	5.9	1.3	2.5		3.9
25	171.4	177.0	167.2	39.5	18.1	18.4	13.2	14.6		4.2		
26	173.5	175.4	168.2	37.2	16.0	18.5	3.6	12.8	11.2			1.3
30	175.3	177.4	169.8	37.6	12.9	15.9						
31	176.1	178.8	171.4	37.5	9.4	36.8	6.5	11.7	12.9			9.3
33	177.1	180.6	171.8	33.1	6.1	15.7	5.6	9.5	4.3	6.3	6.8	1.4
35	177.1	186.9	173.4	29.9	13.4	3.8		11.2		4.9	8.1	8.9
36	168.0	170.9	164.3	37.7	16.0	18.4		13.8		8.1	7.8	10.5
38	170.5	174.1	166.0	32.8	16.0	16.5		10.9		10.4	10.1	9.2
41	172.0	175.7	167.6	33.1	15.2	12.1		7.9		6.5	7.4	2.8

I.2.5 PSD peaks

Figure I.5 plots the peak values of the Power Spectra Density (PSD) in the white noise cases identified using the FDD method. Table I.4 summarizes the corresponding values.

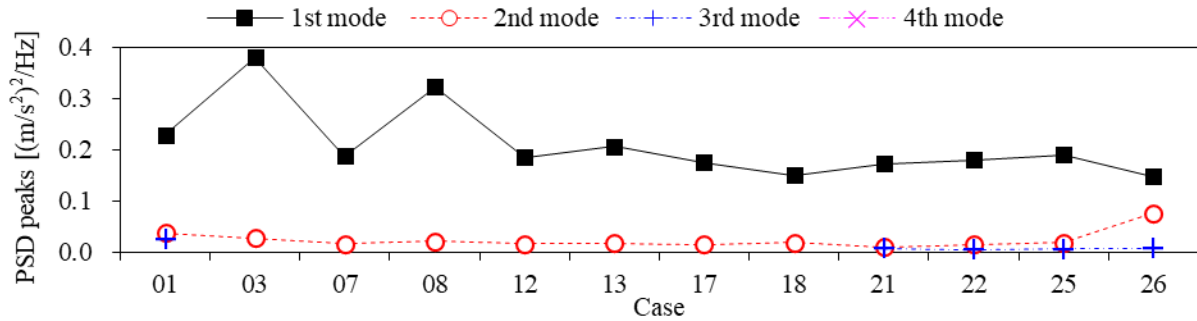


Figure I.5 PSD peaks in white noise excitations of the 14UCSD test

Table I.4 PSD peaks in white noise excitations of the 14UCSD test [Unit: $(m/s^2)^2/Hz$]

Case	Mode			
	1st	2nd	3rd	4th
01	0.229	0.038	0.027	
03	0.381	0.027		
07	0.188	0.016		
08	0.323	0.021		
12	0.186	0.017		
13	0.206	0.018		
17	0.175	0.015		
18	0.151	0.019		
21	0.173	0.010	0.007	
22	0.180	0.015	0.006	
25	0.191	0.019	0.006	
26	0.148	0.077	0.008	
30	0.148	0.017		
31	0.216	0.006	0.011	
33	0.109	0.005	0.013	0.011
35	0.116	0.021	0.011	0.019
36	0.148	0.018	0.039	0.026
38	0.175	0.026	0.044	0.021
41	0.144	0.013	0.025	0.012

I.3 MODAL PARAMETERS FROM GROUND MOTION EXCITATIONS

I.3.1 Natural period

Figure I.6 plots the natural period in the ground motion cases of the 14UCSD test estimated using the ARX, FDD and SS methods, respectively. Table I.5 summarizes the corresponding values in each segment.

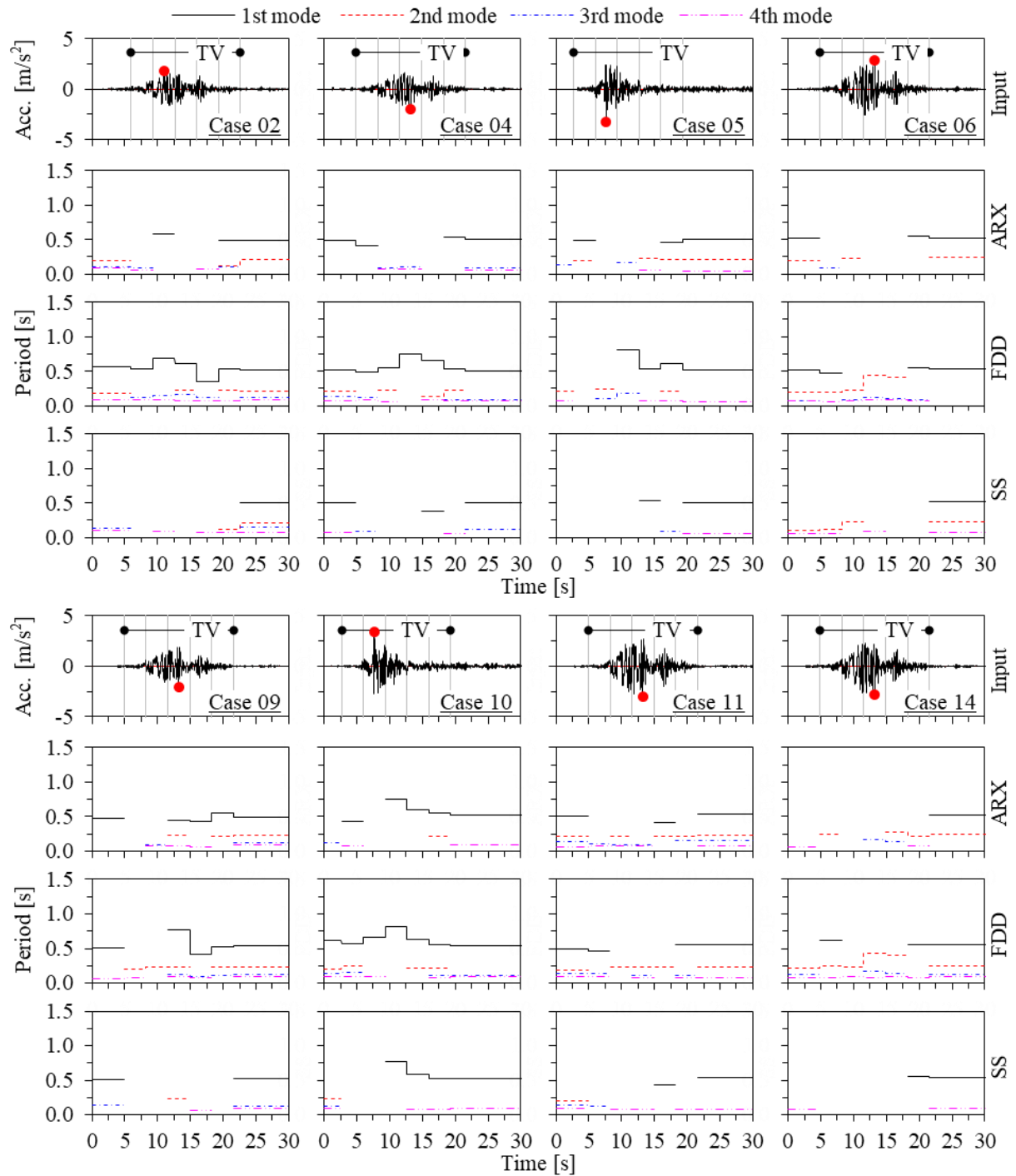


Figure I.6 Natural periods in ground motion excitations of the 14UCSD test

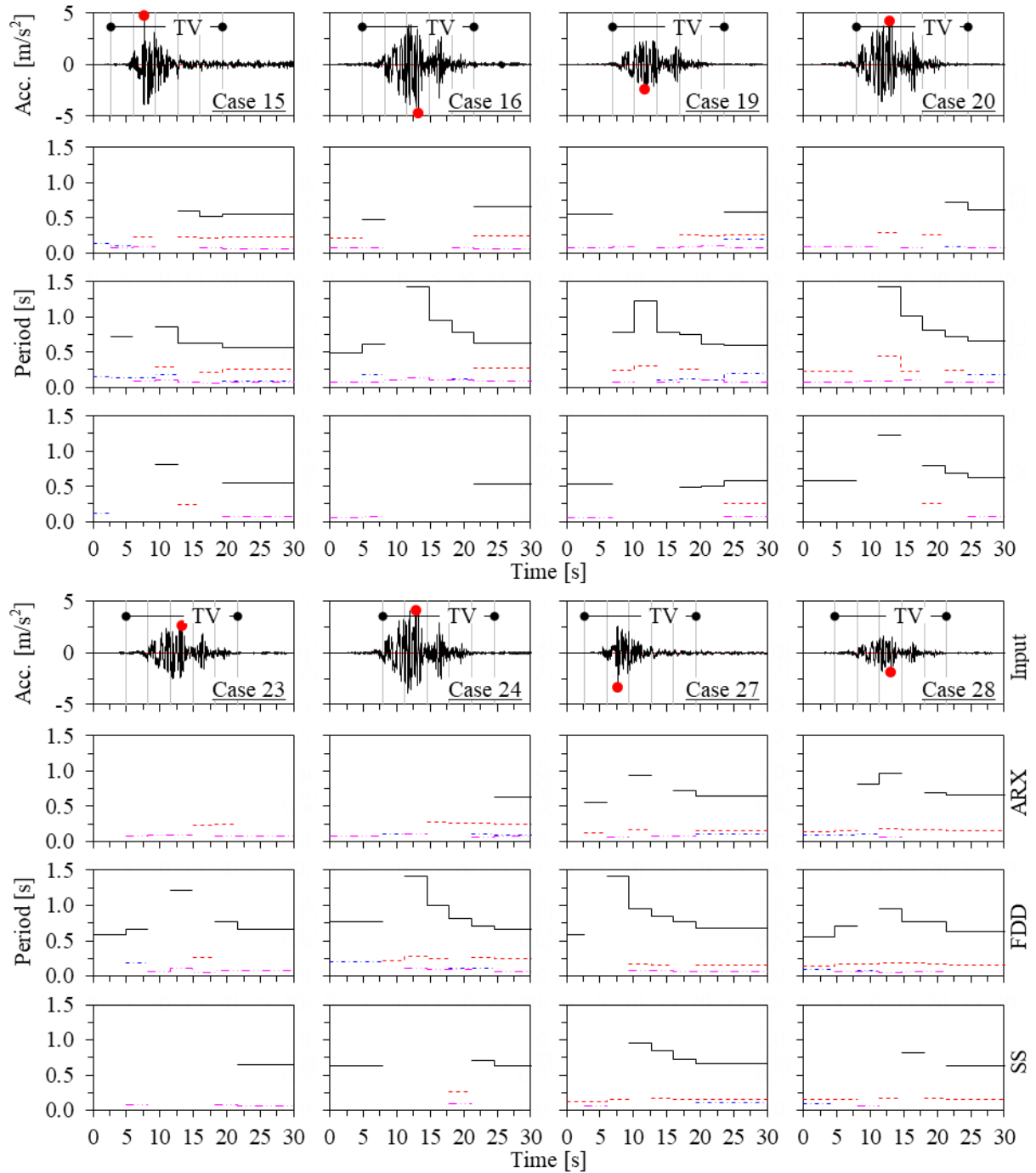


Figure I.6 (continued) Natural periods in ground motion excitations of the 14UCSD test

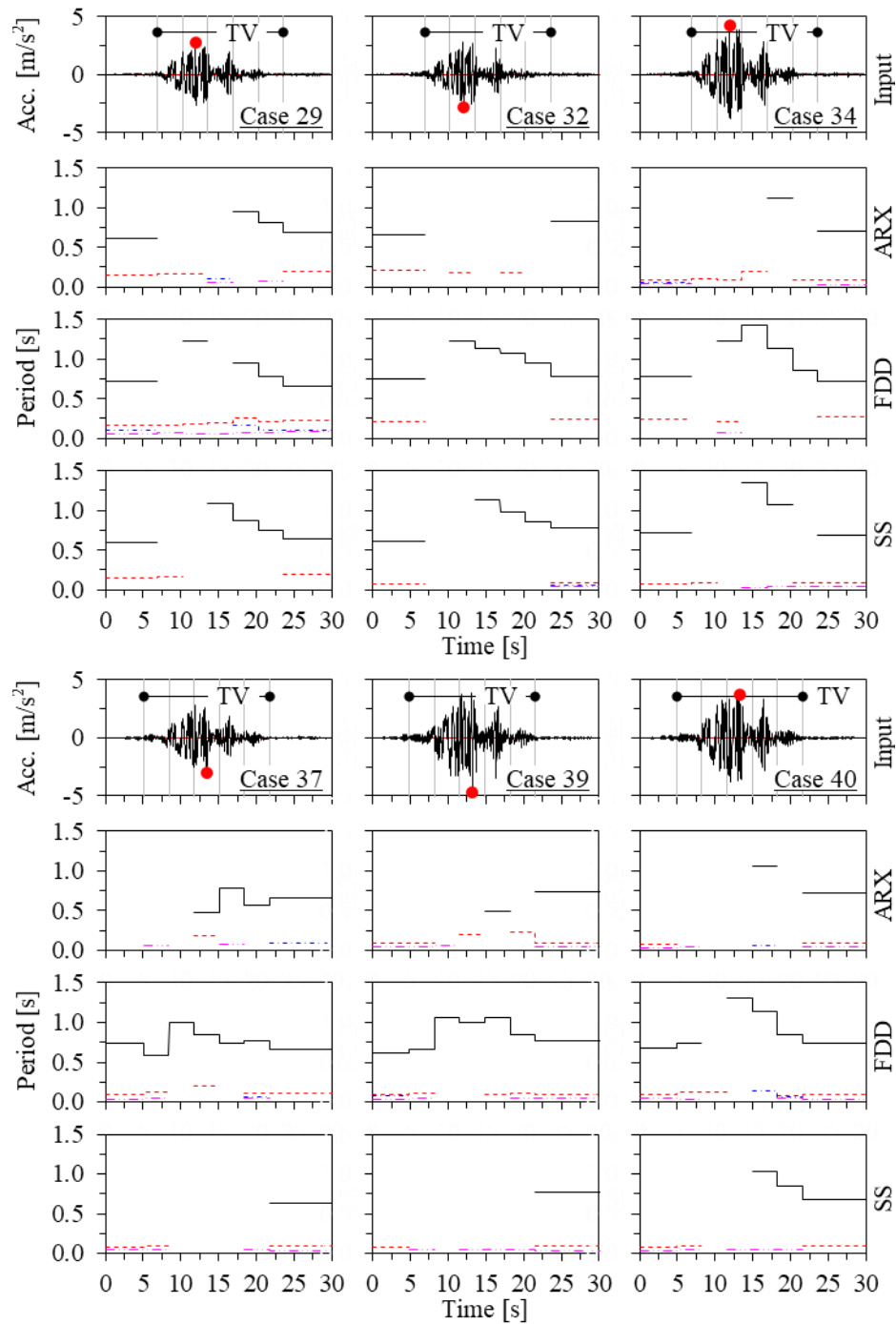


Figure I.6 (continued) Natural periods in ground motion excitations of the 20EDF test

Table I.5 Natural periods in ground motion excitations of the 14UCSD test [Unit: s]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01		0.57		0.20	0.18		0.10		0.14	0.08	0.08	0.10
02-02		0.53					0.09	0.11		0.06	0.08	
02-03	0.58	0.68						0.15			0.09	0.09
02-04		0.61			0.23			0.16			0.08	
02-05		0.36						0.13		0.08	0.08	0.08
02-06	0.49	0.53		0.12	0.23	0.12	0.10				0.08	0.08
02-07	0.49	0.52	0.50	0.21	0.22	0.21		0.12	0.15		0.08	0.08

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04-01	0.49	0.52	0.51		0.20		0.14		0.08	0.08
04-02	0.42	0.49					0.11	0.09	0.08	
04-03		0.55			0.23	0.09			0.07	0.06
04-04		0.74				0.10			0.08	
04-05		0.66	0.38		0.14					0.09
04-06	0.53	0.53			0.22		0.09		0.08	0.08
04-07	0.51	0.50	0.51			0.09	0.09	0.11	0.07	0.07
05-01					0.21	0.14			0.07	
05-02	0.49			0.20						
05-03					0.25		0.10			
05-04		0.81				0.16	0.17			
05-05		0.53	0.53	0.22					0.06	0.07
05-06	0.45	0.61		0.22	0.21			0.09	0.07	
05-07	0.50	0.52	0.50	0.21					0.06	0.06
06-01	0.51	0.52		0.20	0.19	0.10	0.07		0.07	0.05
06-02		0.47			0.20	0.12	0.08		0.06	0.06
06-03				0.22	0.23	0.23	0.09		0.08	
06-04					0.45		0.12		0.09	0.09
06-05					0.41		0.10		0.09	
06-06	0.55	0.55					0.09		0.07	
06-07	0.51	0.53	0.52	0.24		0.23				0.08
09-01	0.47	0.50	0.51					0.13	0.07	
09-02					0.20				0.07	
09-03					0.23	0.09			0.07	
09-04	0.45	0.78		0.22	0.23	0.23	0.13		0.08	0.09
09-05	0.42	0.42					0.09		0.07	0.08
09-06	0.56	0.52		0.22	0.23		0.11			
09-07	0.50	0.53	0.53	0.23	0.23		0.12	0.12	0.12	0.10
10-01		0.61			0.20	0.23	0.12	0.15	0.12	0.09
10-02	0.44	0.57			0.24			0.15	0.09	0.09
10-03		0.66								0.08
10-04	0.76	0.81	0.77							
10-05	0.60	0.63	0.59		0.21					0.07
10-06	0.55	0.55	0.52	0.22	0.22		0.10		0.08	0.08
10-07	0.52	0.53	0.52				0.11		0.10	0.09
11-01	0.50	0.49		0.21	0.19	0.21	0.14	0.14	0.14	0.07
11-02		0.46					0.11	0.14	0.13	0.08
11-03				0.22	0.23		0.09			0.08
11-04					0.23		0.10	0.11		0.08
11-05	0.42		0.43	0.22	0.23					
11-06		0.55		0.21			0.16	0.11		0.07
11-07	0.54	0.55	0.54	0.24	0.23		0.15		0.07	0.07
14-01					0.22		0.12		0.07	0.07
14-02		0.61		0.25	0.25					0.07
14-03					0.23					0.09
14-04					0.43		0.17	0.17		0.07
14-05				0.27	0.41		0.14	0.14		0.08
14-06		0.55	0.55	0.22					0.08	0.08
14-07	0.53	0.55	0.55	0.25	0.24		0.12			0.10

15-01							0.14	0.15	0.13			
15-02		0.71					0.11	0.13		0.07		
15-03				0.22				0.13		0.09	0.09	
15-04		0.85	0.81		0.28			0.17			0.10	
15-05	0.59	0.63		0.23		0.24					0.07	
15-06	0.52	0.63		0.22	0.21					0.08	0.05	
15-07	0.55	0.57	0.54	0.22	0.25			0.10		0.06	0.07	0.08
16-01		0.49		0.21						0.07	0.07	0.07
16-02	0.48	0.61						0.17		0.07	0.07	0.07
16-03											0.11	
16-04		1.42									0.14	
16-05		0.95									0.10	
16-06		0.78						0.11		0.08	0.10	
16-07	0.65	0.63	0.54	0.25	0.27					0.06	0.08	
19-01	0.55		0.54							0.07		0.07
19-02		0.78			0.24					0.09	0.08	
19-03		1.22			0.30							
19-04		0.78						0.11		0.08	0.08	
19-05		0.74	0.49	0.26	0.25			0.12		0.09		
19-06		0.61	0.51	0.24				0.11		0.10	0.10	
19-07	0.57	0.59	0.58	0.26		0.26	0.19	0.20		0.08	0.08	0.08
20-01			0.58		0.23					0.08	0.08	
20-02										0.10	0.09	
20-03		1.42	1.23	0.28	0.45						0.09	
20-04		1.00			0.23					0.07	0.10	
20-05		0.81	0.79	0.25		0.26						
20-06	0.72	0.71	0.69		0.24		0.08				0.08	
20-07	0.61	0.66	0.63					0.17		0.07	0.07	0.07
23-01		0.59										
23-02		0.66						0.18		0.08		0.08
23-03										0.10	0.06	
23-04		1.22								0.09	0.11	
23-05				0.25	0.26						0.05	
23-06		0.78		0.25						0.07	0.08	0.08
23-07		0.66	0.64							0.08	0.07	0.07
24-01		0.78	0.63					0.19		0.08		
24-02					0.22		0.11					
24-03		1.42			0.28					0.11	0.11	
24-04		1.00		0.28	0.24						0.09	
24-05		0.81		0.26		0.26		0.11			0.10	0.09
24-06		0.71	0.71	0.26	0.26		0.11	0.10		0.07		
24-07	0.62	0.66	0.63	0.24	0.24		0.09			0.08	0.06	
27-01		0.59				0.12						
27-02	0.56			0.13		0.13						0.07
27-03		1.42				0.16				0.07		
27-04	0.94	0.95	0.95	0.17	0.17						0.08	
27-05		0.85	0.84		0.16	0.17				0.07	0.07	
27-06	0.72	0.78	0.72			0.16				0.07	0.06	
27-07	0.64	0.68	0.65	0.15	0.16	0.15	0.10		0.10		0.06	

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28-01		0.55		0.14	0.14	0.15	0.10	0.09	0.10		
28-02		0.71		0.15	0.17	0.15	0.10			0.06	
28-03	0.82				0.17		0.11	0.07		0.06	0.06
28-04	0.96	0.95		0.19	0.18	0.17			0.06	0.05	
28-05		0.78	0.82	0.17	0.18					0.06	
28-06	0.69	0.78		0.16	0.17	0.16				0.06	
28-07	0.66	0.63	0.64	0.15	0.16	0.15					
29-01	0.61	0.71	0.60	0.15	0.16	0.15		0.10		0.05	
29-02				0.17	0.17	0.17				0.07	
29-03		1.22		0.17	0.19					0.06	
29-04			1.08		0.19		0.10		0.07	0.06	
29-05	0.95	0.95	0.87		0.26			0.16		0.07	
29-06	0.81	0.78	0.75		0.22			0.10	0.07	0.07	
29-07	0.68	0.66	0.64	0.20	0.22	0.20		0.10		0.08	
32-01	0.66	0.74	0.61	0.20	0.21	0.08					
32-02											
32-03		1.22		0.18							
32-04		1.14	1.13								
32-05		1.07	0.97	0.18							
32-06		0.95	0.86								
32-07	0.83	0.78	0.78		0.24	0.09		0.06		0.05	
34-01		0.78	0.71	0.09	0.24	0.09	0.06		0.05		
34-02				0.10		0.09					
34-03		1.22		0.09	0.21					0.07	
34-04		1.42	1.34	0.19							0.04
34-05	1.11	1.14	1.08								0.05
34-06		0.85		0.09		0.09					0.05
34-07	0.70	0.71	0.69	0.09	0.27	0.09			0.05		0.05
37-01		0.74			0.09	0.09				0.05	0.05
37-02		0.59			0.12	0.09			0.07	0.05	0.05
37-03		1.00									
37-04	0.47	0.85		0.19	0.19						
37-05	0.79	0.74							0.08		
37-06	0.57	0.78			0.11			0.07		0.05	0.05
37-07	0.66	0.66	0.63		0.10	0.09	0.09				0.04
39-01		0.61		0.09	0.10	0.09		0.08	0.05	0.04	
39-02		0.66		0.09	0.12				0.05	0.05	0.05
39-03		1.07							0.06		
39-04		1.00		0.20							0.04
39-05	0.50	1.07			0.09						0.05
39-06		0.85		0.23	0.11					0.05	0.05
39-07	0.74	0.78	0.77	0.09	0.09	0.09			0.04	0.05	0.04
40-01		0.68		0.09	0.09	0.09			0.04	0.05	0.04
40-02		0.74			0.13	0.09			0.04	0.04	0.04
40-03					0.13						
40-04		1.31									0.04
40-05	1.07	1.14	1.04				0.06	0.13			0.05
40-06		0.85	0.85		0.08			0.06		0.05	0.05
40-07	0.72	0.74	0.68	0.09	0.09	0.09			0.04	0.04	

I.3.2 Modal damping ratio

Figure I.7 plots the modal damping ratios in the ground motion cases of the 14UCSD test estimated using the ARX, FDD and SS methods, respectively. Table I.6 summarizes the corresponding values in each segment.

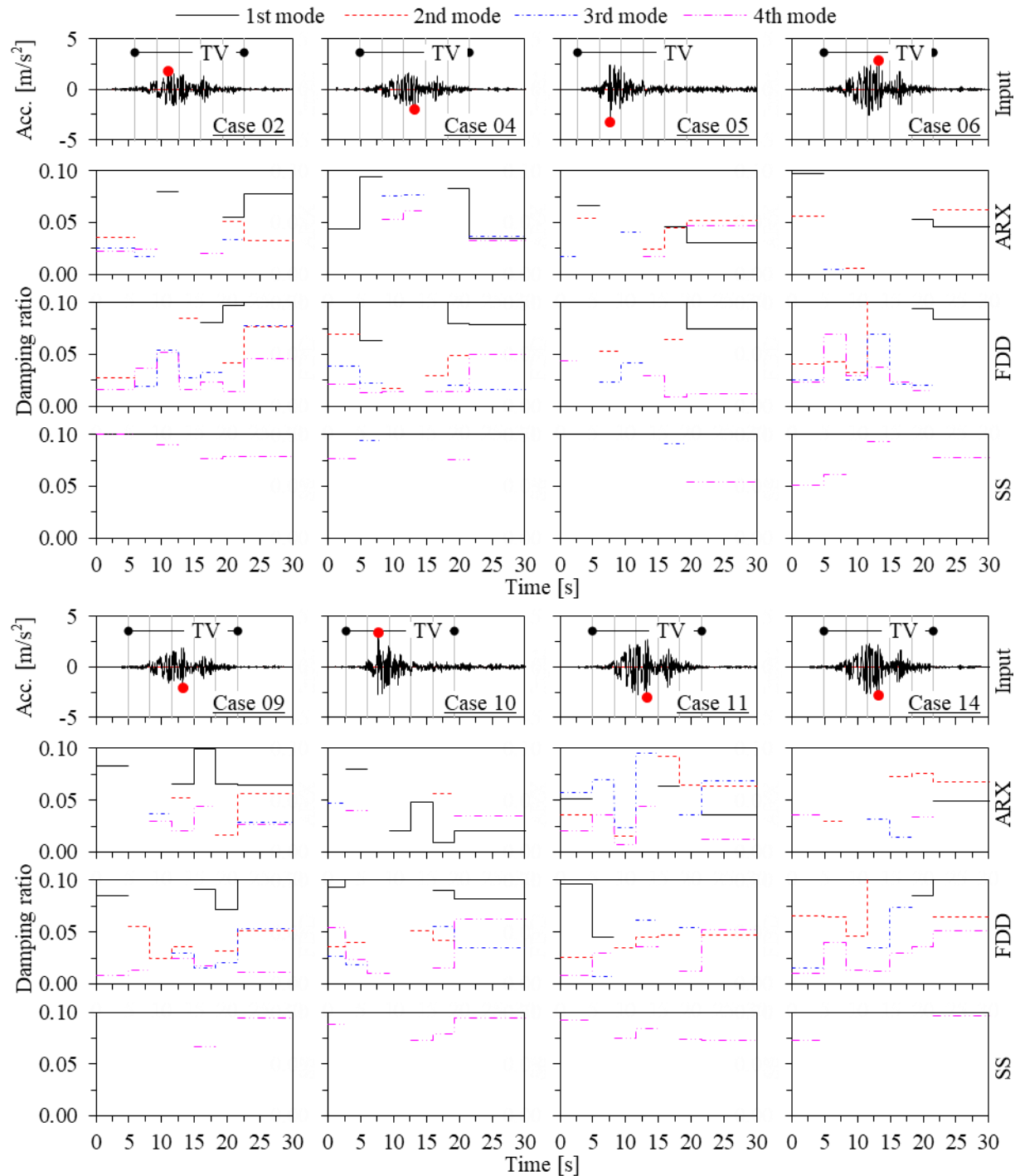


Figure I.7 Damping ratio in ground motion excitations of the 14UCSD test

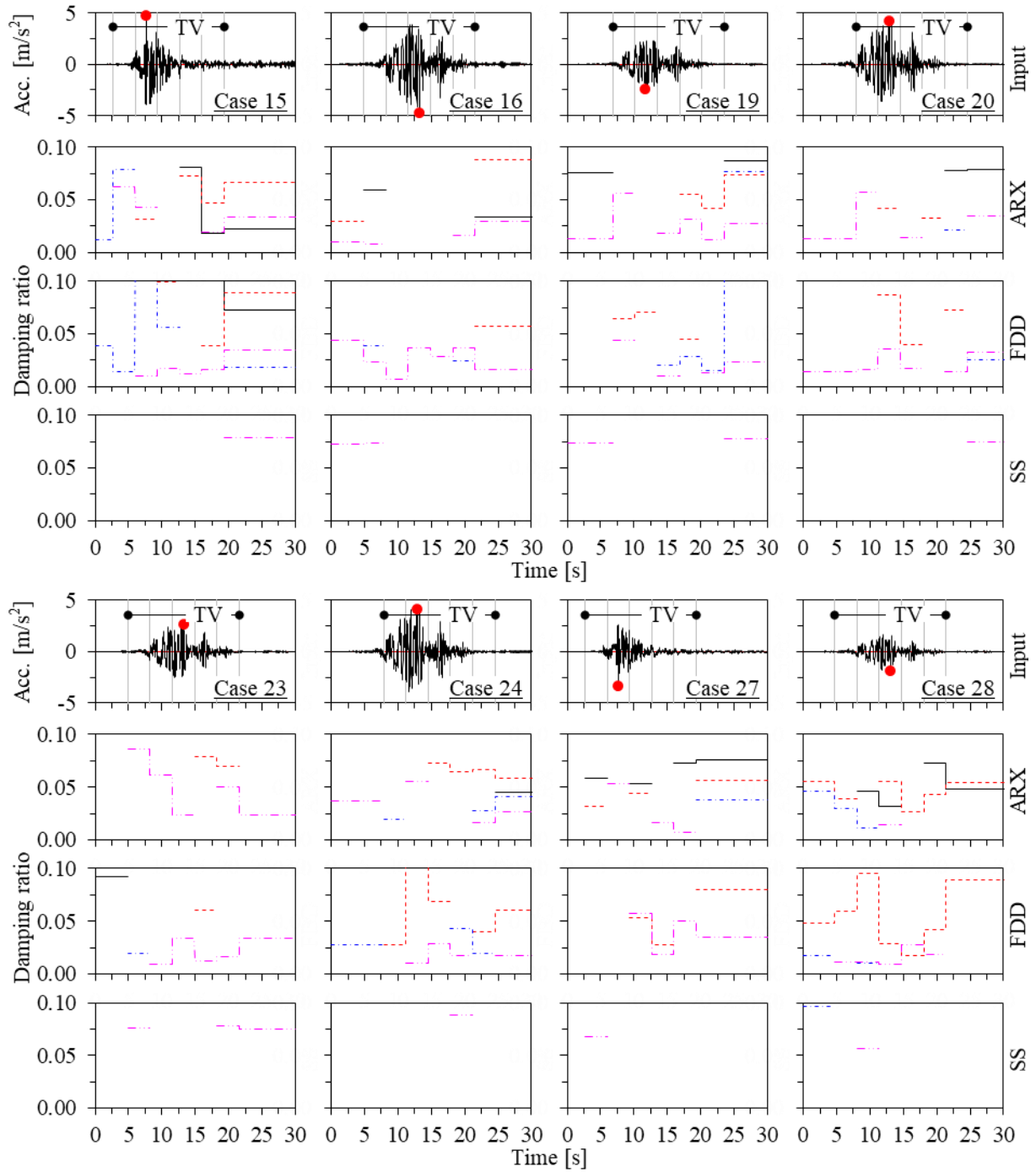


Figure I.7 (continued) Damping ratio in ground motion excitations of the 14UCSD test

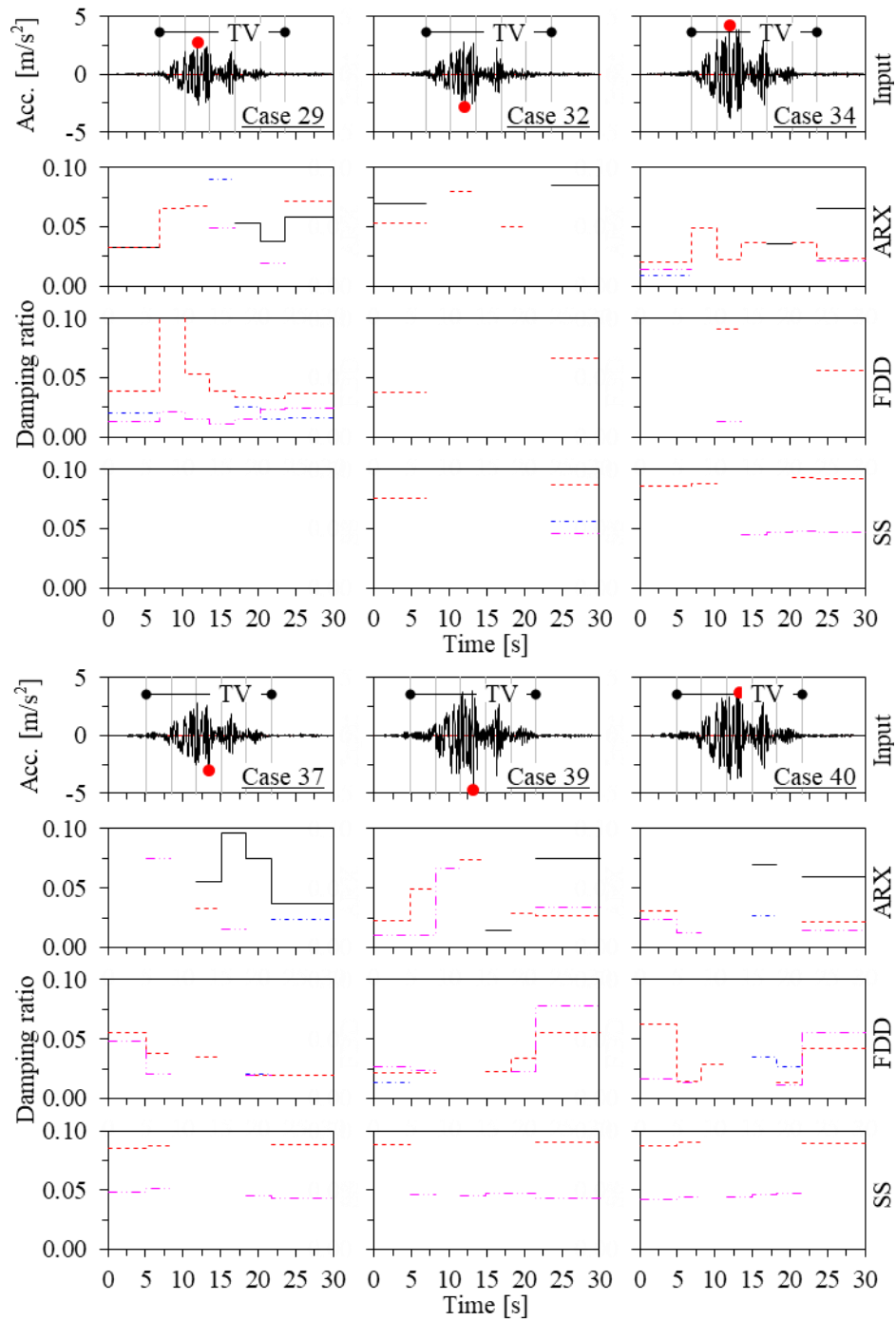


Figure I.7 (continued) Damping ratio in ground motion excitations of the 14UCSD test

Table I.6 Damping ratio in ground motion excitations of the 14UCSD test

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01		0.17		0.04	0.03		0.03		0.14	0.02	0.02	0.10
02-02		0.18					0.02	0.02		0.02	0.04	
02-03	0.08							0.05			0.05	0.09
02-04					0.09			0.03			0.02	
02-05		0.08						0.03		0.02	0.02	0.08
02-06	0.06	0.10		0.05	0.04	0.12	0.03				0.01	0.08
02-07	0.08	0.10	0.50	0.03	0.08	0.21		0.08	0.15		0.05	0.08

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04-01	0.04	0.13	0.51	0.07			0.04			0.02	0.08		
04-02	0.09	0.06				0.02			0.09	0.01			
04-03				0.02		0.08		0.05			0.01		
04-04						0.08		0.06					
04-05	0.10		0.38	0.03						0.01			
04-06	0.08	0.08	0.05			0.02				0.01	0.08		
04-07	0.03	0.08	0.51				0.04	0.02	0.11	0.03	0.05		
05-01				0.67		0.02			0.04				
05-02	0.07	0.05											
05-03				0.05		0.02							
05-04						0.04	0.04						
05-05			0.53	0.02				0.02			0.03		
05-06	0.05	0.14	0.04	0.06				0.09		0.01			
05-07	0.03	0.07	0.50	0.05				0.05			0.01	0.05	
06-01	0.10	0.14	0.06		0.04	0.10	0.03			0.02	0.05		
06-02				0.04		0.12	0.00				0.07	0.06	
06-03				0.01	0.03	0.23	0.03			0.03			
06-04				0.17		0.07			0.04	0.09			
06-05				0.12		0.02			0.02				
06-06	0.05	0.09				0.02			0.01				
06-07	0.05	0.08	0.52	0.06	0.23						0.08		
09-01	0.08	0.09	0.51				0.13			0.01			
09-02				0.06					0.01				
09-03				0.02		0.04		0.03					
09-04	0.07	0.05			0.04	0.23	0.03			0.02	0.03		
09-05	0.10	0.09				0.02			0.04	0.02	0.07		
09-06	0.07	0.07	0.02		0.03	0.02							
09-07	0.06	0.10	0.53	0.06	0.05	0.03		0.05	0.12	0.03	0.01	0.09	
10-01	0.09		0.04		0.23	0.05	0.03	0.12	0.05			0.09	
10-02	0.08	0.16	0.04		0.02			0.04		0.02			
10-03										0.01			
10-04	0.02	0.77											
10-05	0.05	0.59			0.05					0.07			
10-06	0.01	0.09	0.52	0.06	0.04	0.05			0.02			0.08	
10-07	0.02	0.08	0.52				0.03		0.04		0.06	0.09	
11-01	0.05	0.10	0.04		0.03	0.21	0.06	0.14		0.02	0.01	0.09	
11-02	0.05					0.07		0.01	0.13	0.04	0.03		
11-03				0.02	0.04	0.02			0.01			0.08	
11-04				0.05		0.10		0.06	0.04			0.04	0.08
11-05	0.06	0.43		0.09	0.05								
11-06				0.06	0.04			0.05	0.01			0.07	
11-07	0.04	0.54		0.06	0.05	0.07			0.01			0.05	0.07
14-01				0.07		0.02			0.04			0.01	0.07
14-02				0.03	0.06							0.04	
14-03				0.05								0.01	
14-04				0.16		0.03		0.04				0.01	
14-05				0.07	0.14	0.01		0.07				0.03	
14-06	0.08		0.55	0.08				0.03			0.04		
14-07	0.05	0.11	0.55	0.07	0.06	0.11						0.05	0.10

15-01						0.01	0.04	0.13		
15-02						0.08	0.01		0.06	
15-03				0.03			0.18		0.04	0.01
15-04			0.81		0.10		0.06			0.02
15-05	0.08			0.07		0.24				0.01
15-06	0.02	0.15		0.05	0.04				0.02	0.02
15-07	0.02	0.07	0.54	0.07	0.09		0.02		0.03	0.03
16-01		0.12		0.03					0.01	0.04
16-02	0.06						0.04		0.01	0.02
16-03										0.01
16-04										0.04
16-05										0.03
16-06							0.02		0.02	0.04
16-07	0.03		0.54	0.09	0.06				0.03	0.02
19-01	0.08		0.54						0.01	0.07
19-02					0.06				0.06	0.04
19-03					0.07					
19-04							0.02		0.02	0.01
19-05			0.49	0.06	0.04		0.03		0.03	
19-06			0.51	0.04			0.02		0.01	0.01
19-07	0.09		0.58	0.07		0.26	0.08	0.31	0.03	0.02
20-01			0.58		0.15				0.01	0.01
20-02									0.06	0.02
20-03			1.23	0.04	0.09					0.04
20-04					0.04				0.01	0.02
20-05			0.79	0.03		0.26				
20-06	0.08		0.69		0.07		0.02			0.01
20-07	0.08		0.63				0.03		0.03	0.03
23-01		0.09								
23-02							0.02		0.09	0.08
23-03									0.06	0.01
23-04									0.02	0.03
23-05				0.08	0.06					0.01
23-06				0.07					0.05	0.02
23-07			0.64						0.02	0.03
24-01			0.63				0.03		0.04	
24-02					0.03		0.02			
24-03					0.12				0.05	0.01
24-04				0.07	0.07					0.03
24-05				0.06		0.26		0.04		0.02
24-06			0.71	0.07	0.04		0.03	0.02	0.02	
24-07	0.05		0.63	0.06	0.06		0.04		0.03	0.02
27-01						0.12				
27-02	0.06			0.03		0.13				0.07
27-03						0.16			0.05	
27-04	0.05		0.95	0.04	0.05					0.06
27-05			0.84		0.03	0.17			0.02	0.02
27-06	0.07		0.72			0.16			0.01	0.05
27-07	0.08		0.65	0.06	0.08	0.15	0.04	0.10		0.03

Appendix I Full Results of Vibration Property Analysis: 14UCSD

28-01			0.06	0.05	0.15	0.05	0.02	0.10		
28-02			0.04	0.06	0.15	0.03			0.01	
28-03	0.05			0.10		0.01	0.01		0.01	0.06
28-04	0.03		0.06	0.03	0.17			0.01	0.01	
28-05		0.82	0.03	0.02					0.03	
28-06	0.07		0.04	0.04	0.16				0.02	
28-07	0.05	0.64	0.05	0.09	0.15					
29-01	0.03	0.60	0.03	0.04	0.15		0.02		0.01	
29-02			0.07	0.17	0.17				0.02	
29-03			0.07	0.05					0.01	
29-04		1.08		0.04		0.09		0.05	0.01	
29-05	0.05	0.87		0.03			0.03		0.02	
29-06	0.04	0.75		0.03			0.02	0.02	0.02	
29-07	0.06	0.64	0.07	0.04	0.20		0.02		0.02	
32-01	0.07	0.61	0.05	0.04	0.08					
32-02										
32-03			0.08							
32-04		1.13								
32-05		0.97	0.05							
32-06		0.86								
32-07	0.08	0.78		0.07	0.09		0.06			0.05
34-01		0.71	0.02	0.11	0.09	0.01		0.01		
34-02			0.05		0.09					
34-03			0.02	0.09					0.01	
34-04		1.34	0.04							0.04
34-05	0.04	1.08								0.05
34-06			0.04		0.09					0.05
34-07	0.07	0.69	0.02	0.06	0.09			0.02		0.05
37-01				0.06	0.09				0.05	0.05
37-02				0.04	0.09			0.08	0.02	0.05
37-03										
37-04	0.06		0.03	0.03						
37-05	0.10							0.02		
37-06	0.07			0.02			0.02		0.02	0.05
37-07	0.04	0.63		0.02	0.09	0.02				0.04
39-01			0.02	0.02	0.09		0.01	0.01	0.03	
39-02			0.05	0.02				0.01	0.02	0.05
39-03								0.07		
39-04			0.07							0.04
39-05	0.01			0.02						0.05
39-06			0.03	0.03					0.02	0.05
39-07	0.07	0.77	0.03	0.06	0.09			0.03	0.08	0.04
40-01			0.03	0.06	0.09			0.02	0.02	0.04
40-02				0.01	0.09			0.01	0.01	0.04
40-03				0.03						
40-04										0.04
40-05	0.07	1.04				0.03	0.03			0.05
40-06		0.85		0.01			0.03		0.01	0.05
40-07	0.06	0.68	0.02	0.04	0.09			0.01	0.06	

I.3.3 Mode shapes

Figure I.8 to Figure I.10 plot the mode shapes in the ground motion cases of the 14UCSD test estimated using the ARX, FDD and SS methods, respectively.

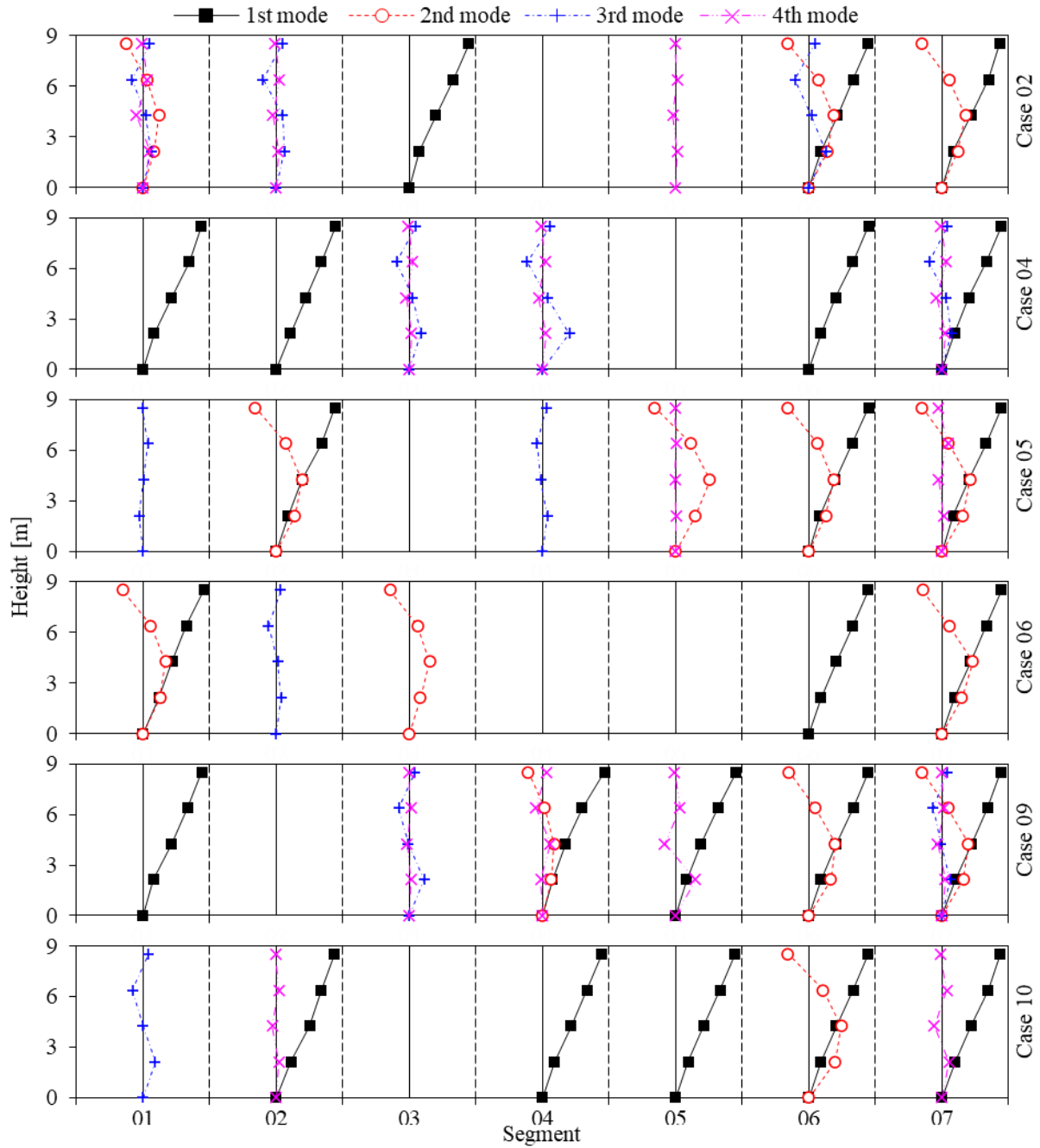


Figure I.8 Mode shapes in ground motion excitations of the 14UCSD test (ARX)

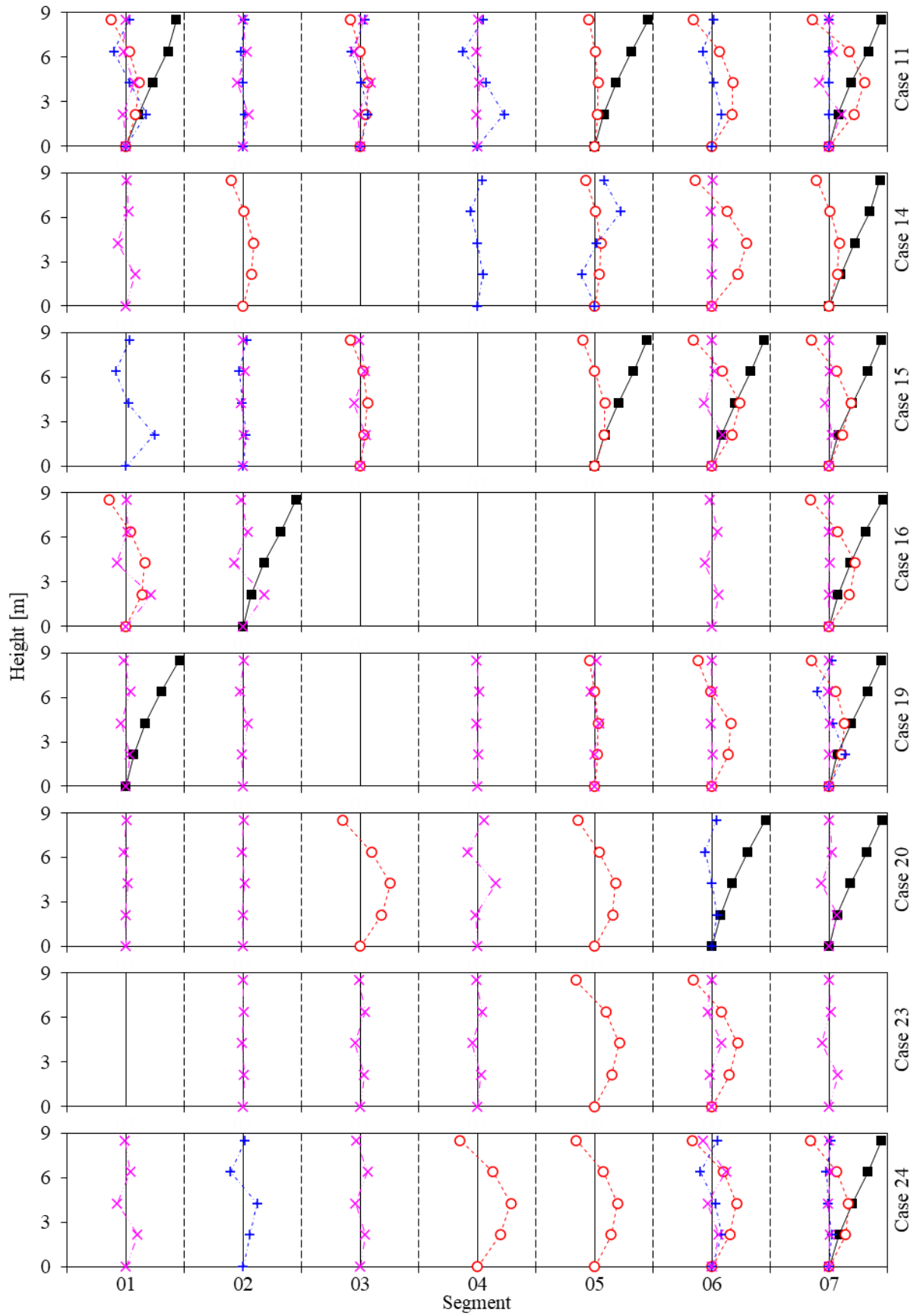


Figure I.8 (continued) Mode shapes in ground motion excitations of the 14UCSD test (ARX)

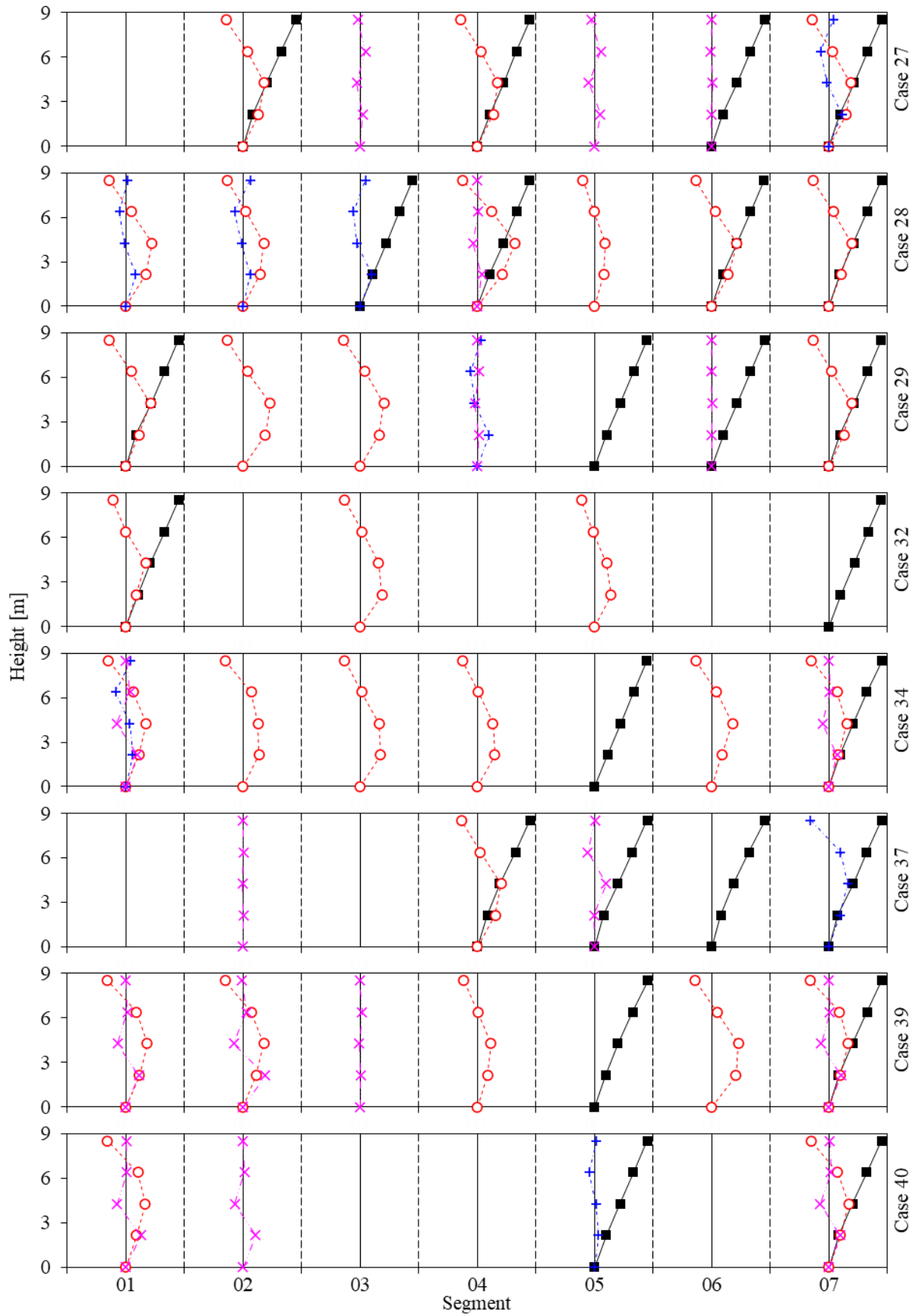


Figure I.8 (continued) Mode shapes in ground motion excitations of the 14UCSD test (ARX)

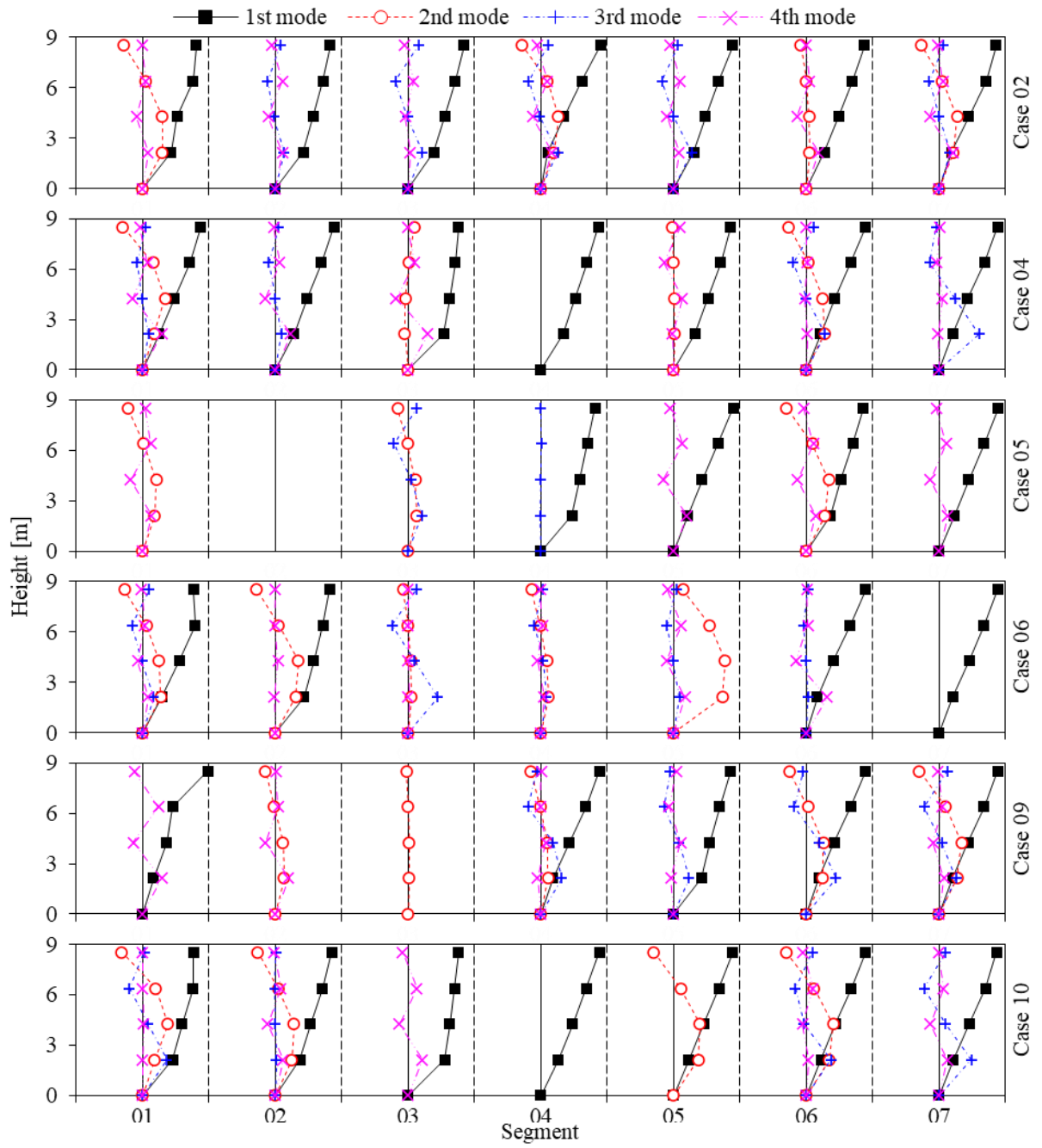


Figure I.9 Mode shapes in ground motion excitations of the 14UCSD test (FDD)

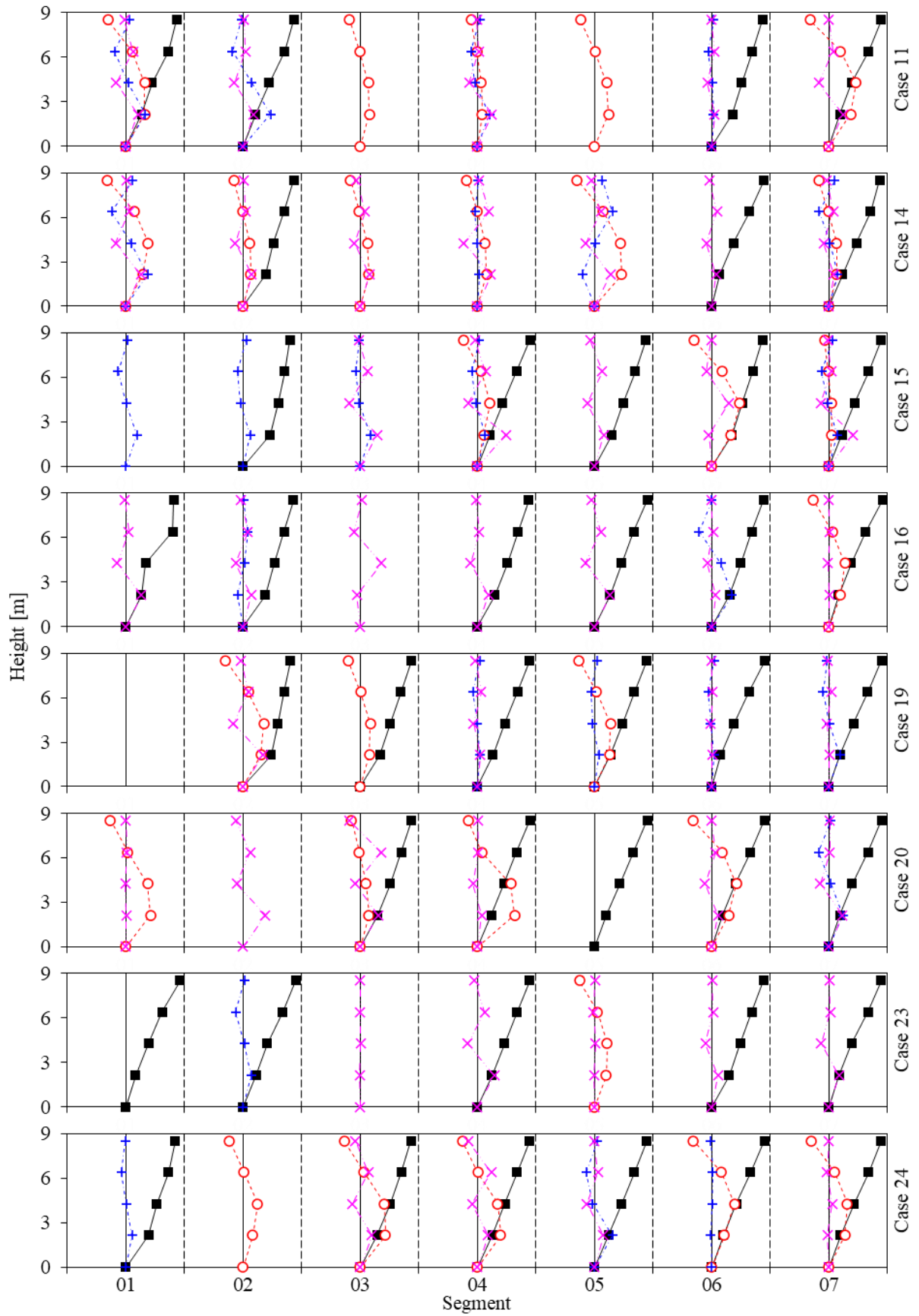


Figure I.9 (continued) Mode shapes in ground motion excitations of the 14UCSD test (FDD)

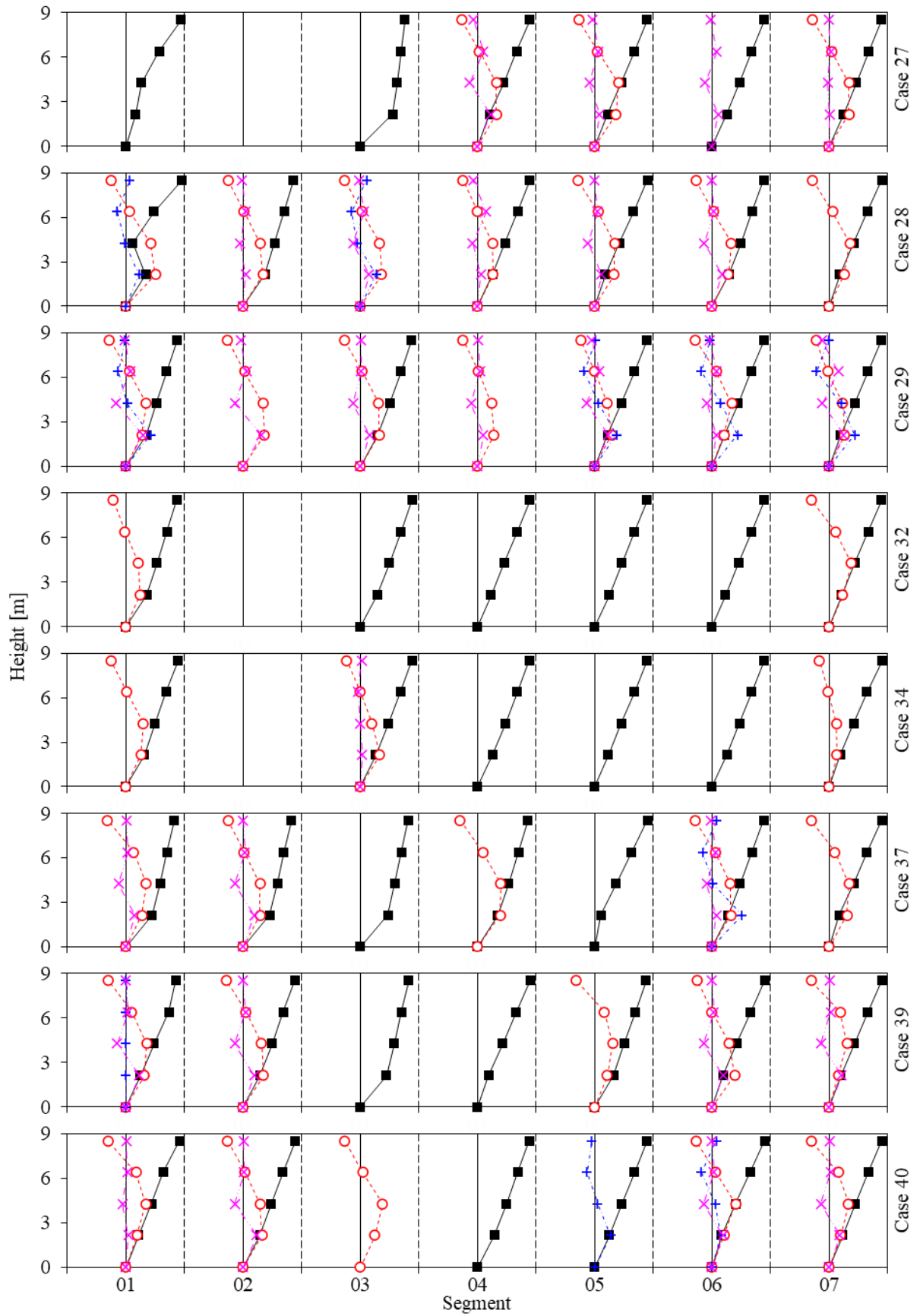


Figure I.9 (continued) Mode shapes in ground motion excitations of the 14UCSD test (FDD)

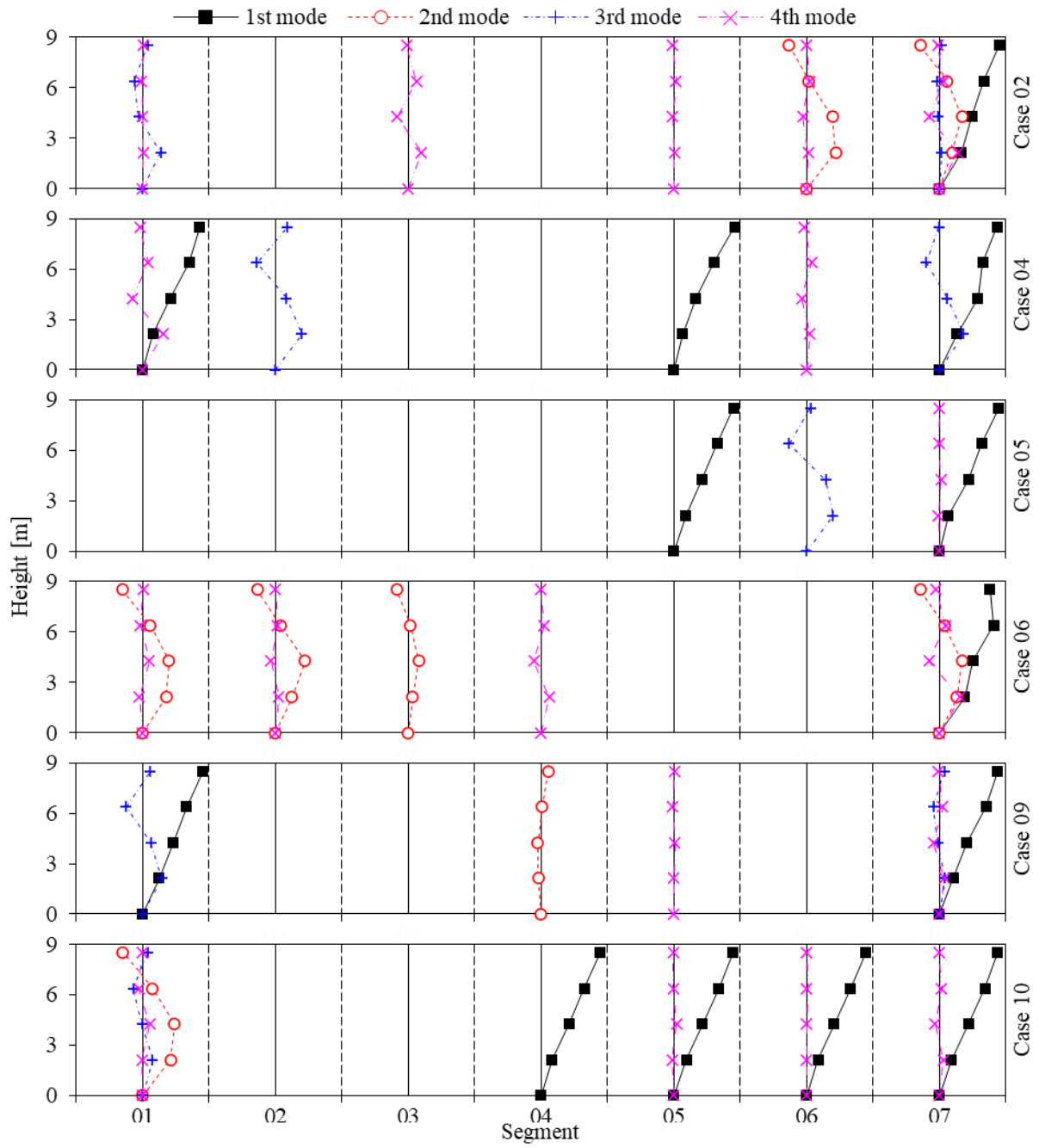


Figure I.10 Mode shapes in ground motion excitations of the 14UCSD test (SS)

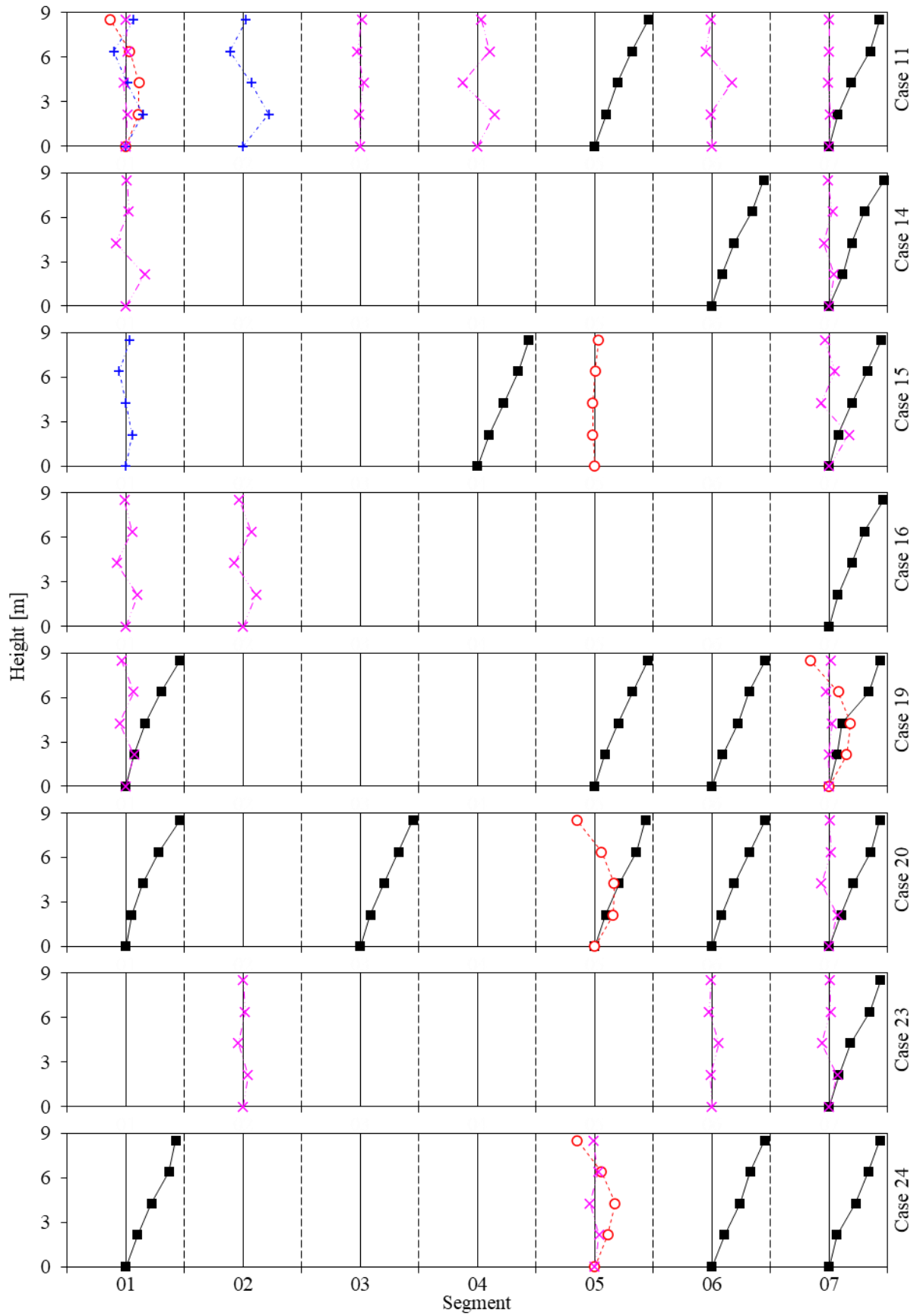


Figure I.10 (continued) Mode shapes in ground motion excitations of the 14UCSD test (SS)

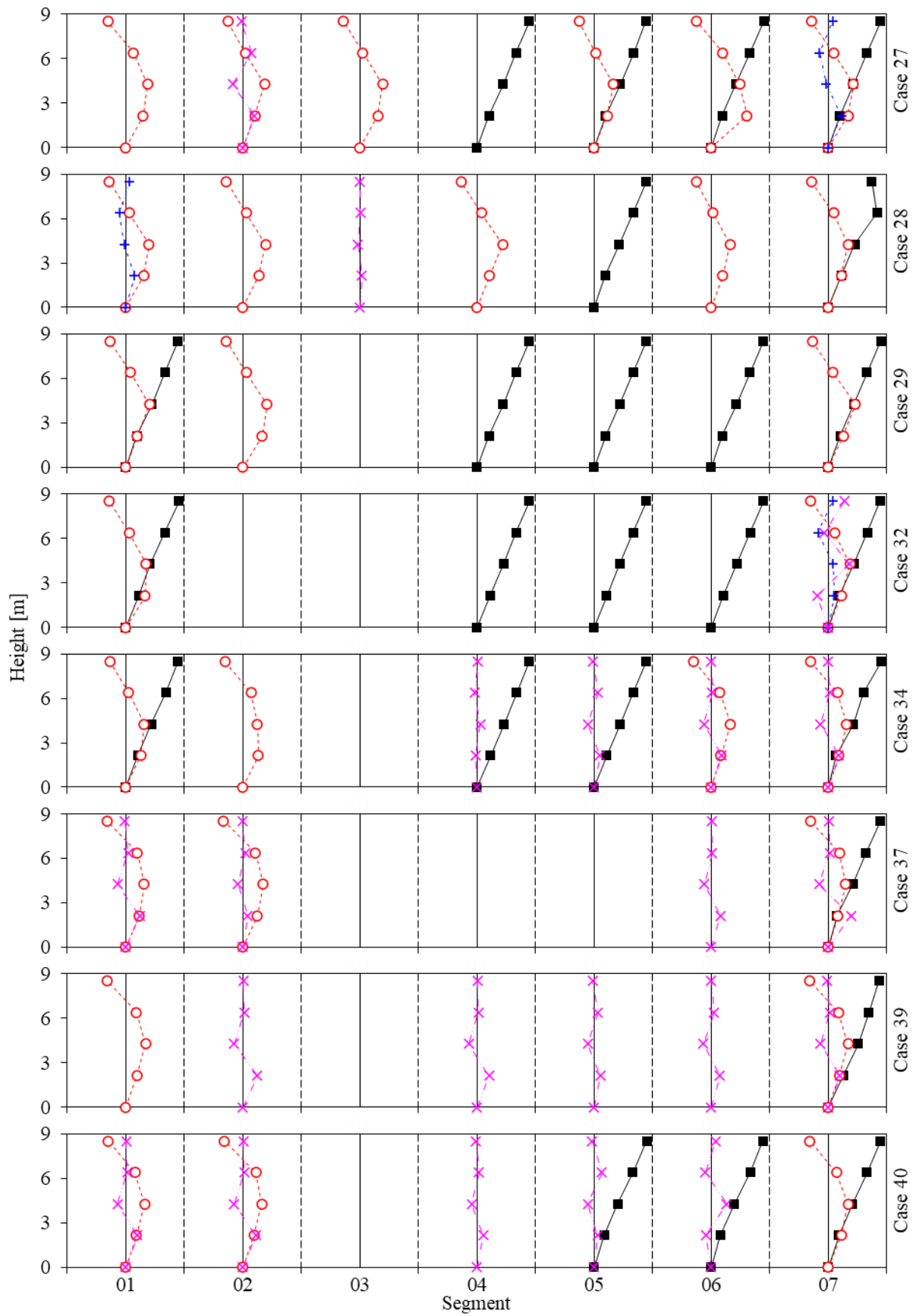


Figure I.10 (continued) Mode shapes in ground motion excitations of the 14UCSD test (SS)

I.3.4 Modal Effective Mass (MEM)

Figure I.11 plots the natural period in the ground motion cases of the 14UCSD test estimated using the ARX, FDD and SS methods, respectively. Table I.7 summarizes the corresponding values in each segment.

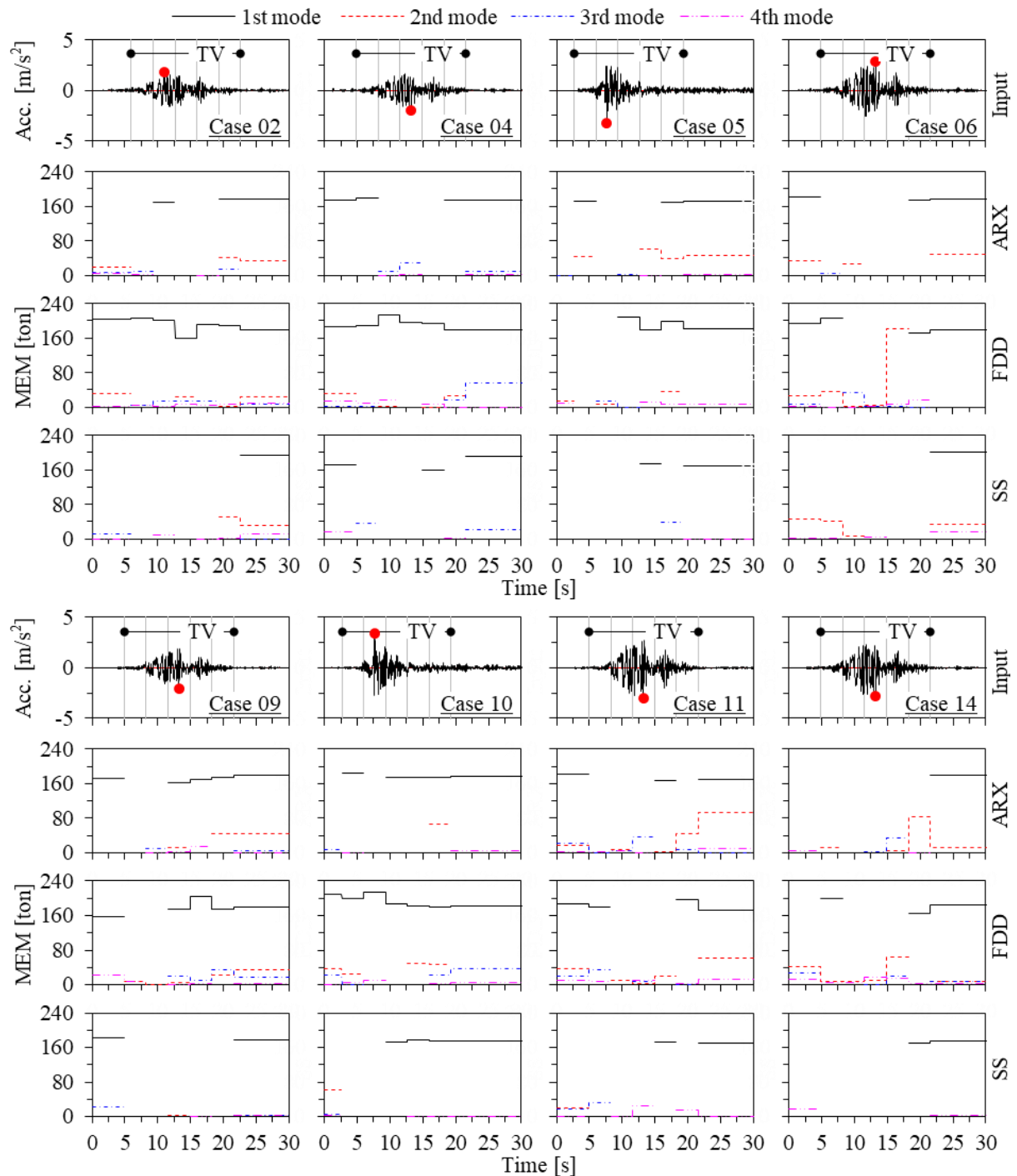


Figure I.11 MEMs in ground motion excitations of the 14UCSD test

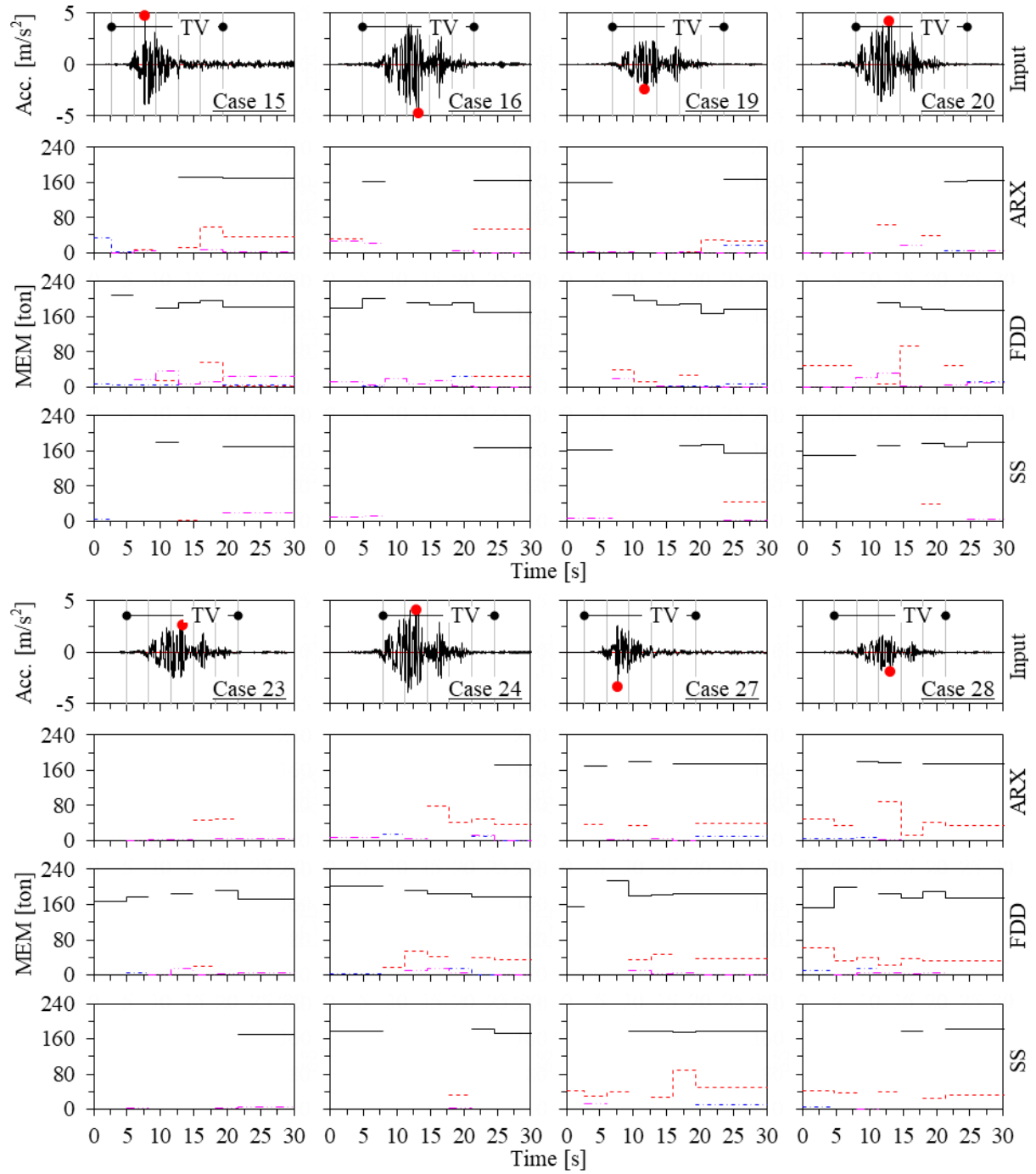


Figure I.11 (continued) MEMs in ground motion excitations of the 14UCSD test

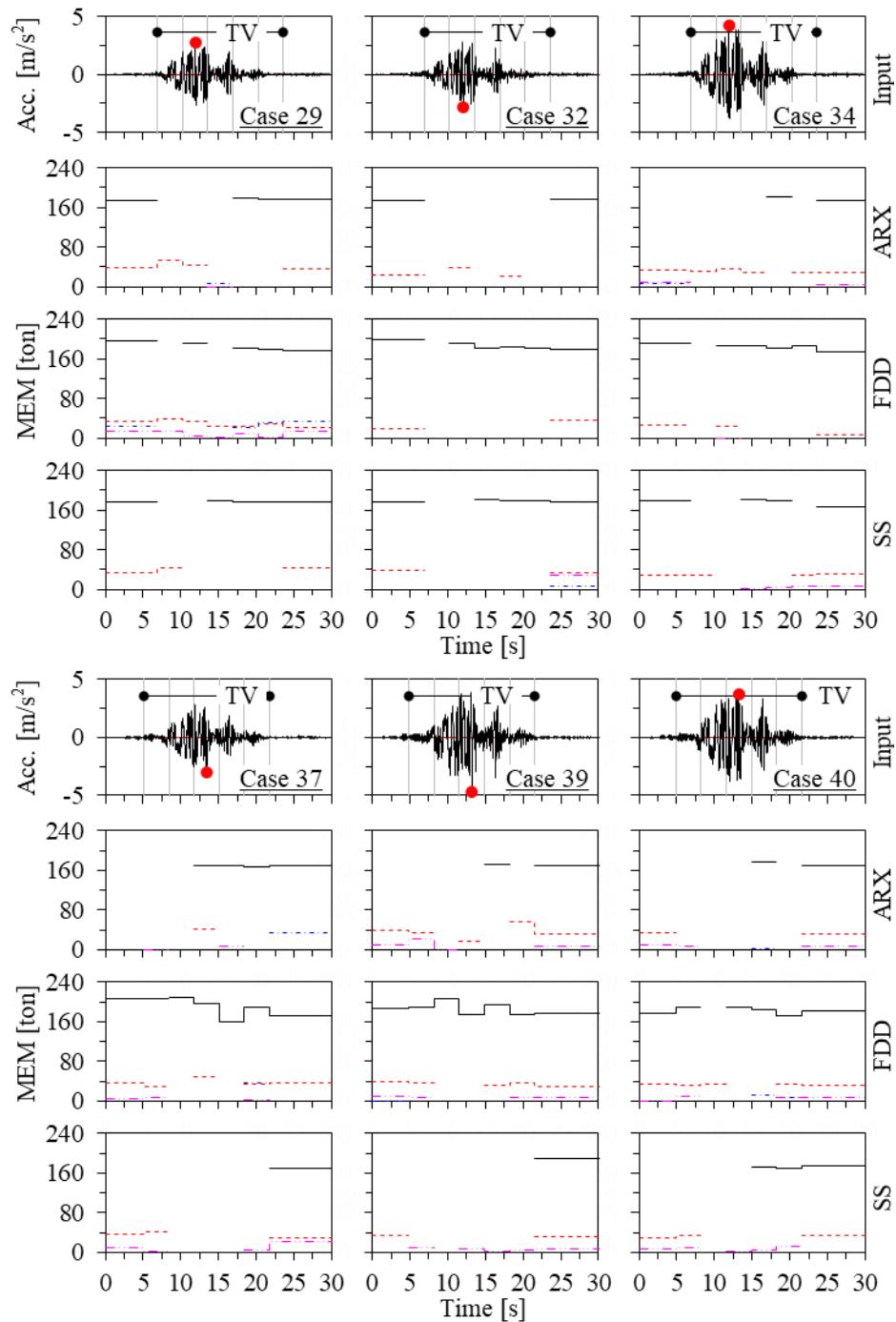


Figure I.11 (continued) MEMs in ground motion excitations of the 14UCSD test

Table I.7 MEMs in ground motion excitations of the 14UCSD test [Unit: ton]

Case-Segment	1stmode			2ndmode			3rdmode			4thmode		
	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS	ARX	FDD	SS
02-01		203.8		17.9	30.9		6.5		12.0	3.2	2.1	0.1
02-02		204.9					9.2	4.5		0.8	4.5	
02-03	169.1	201.9						13.1			1.5	10.1
02-04		158.6			24.0			13.7			6.7	
02-05		190.2						13.2		0.4	3.4	0.2
02-06	175.4	188.7		40.8	1.3	50.3	14.7				6.9	0.7
02-07	175.7	178.8	192.6	34.4	23.7	30.4		6.5	0.4		8.2	12.7

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04-01	173.9	185.4	172.1		31.3		2.0		14.0	15.3
04-02	179.8	188.1					2.5	36.4	10.1	
04-03		213.4			1.5	9.7			0.7	16.5
04-04		196.2				29.3			1.0	
04-05		194.7	159.7		0.1					5.8
04-06	173.8	178.4			25.3		16.0		0.1	1.7
04-07	174.6	178.3	191.3			8.2	54.8	22.4	1.7	0.5
05-01					15.2		1.1			8.5
05-02	172.1			42.6						
05-03					6.6		13.5			
05-04		208.5				2.0	0.0			
05-05		178.8	173.9	61.1					0.0	10.4
05-06	171.0	198.5		39.4	35.4			38.7		6.4
05-07	172.3	181.6	169.2	45.9					1.7	5.9
06-01	180.6	194.6		34.5	25.7	47.3	7.5			1.6
06-02		205.9			35.4	41.0	3.0			0.3
06-03				26.3	1.2	7.4		34.3		0.0
06-04					5.2			2.3		0.7
06-05					180.5			2.6		7.9
06-06	174.0	171.0					0.4			15.3
06-07	175.8	179.7	199.9	49.1		33.2				16.6
09-01	171.6	156.7	183.3					21.5		21.7
09-02					6.3					7.8
09-03					0.1		9.9		0.2	
09-04	162.6	175.7		11.4	5.2	1.7	20.7		3.5	1.0
09-05	168.7	203.4					10.3		14.9	2.4
09-06	174.0	174.9		44.3	23.1		33.5			0.1
09-07	179.1	178.6	177.4	44.0	35.2		4.9	16.1	2.6	1.0
10-01		208.2			37.5	62.8	7.7	21.9	6.1	0.0
10-02	184.0	199.5			25.4			0.0		0.8
10-03		213.9								10.8
10-04	175.2	186.8	172.4							
10-05	175.0	182.3	176.6		48.8					0.3
10-06	173.8	179.9	173.6	66.1	47.9		21.6			2.1
10-07	178.2	181.7	175.9				37.7		3.7	4.6
11-01	181.5	186.4		16.7	37.6	20.9	21.1	19.0	17.9	3.1
11-02		180.7					0.5	34.0	32.5	2.7
11-03				6.5	9.5		5.7			5.9
11-04					2.8		36.1	7.1		0.3
11-05	166.1		172.5	1.8	18.8					10.0
11-06		197.1		44.9			6.7	0.8		1.3
11-07	169.0	173.1	168.8	92.3	61.1		0.0		9.2	11.6
14-01					42.4			26.7		5.6
14-02		200.1		11.6	6.7					12.2
14-03					7.9					5.2
14-04					9.2		3.3	0.3		5.6
14-05				5.0	63.5		33.6	19.1		18.2
14-06		164.9	170.9	84.5					0.1	13.9
14-07	178.6	184.5	174.8	11.6	8.2			7.2		2.8
										2.6
										2.2

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15-01						33.5	6.8	3.6			
15-02		209.2				1.4	3.5		0.3		
15-03				5.9			4.6		3.4	16.8	
15-04		178.6	178.6		13.8		3.3			35.4	
15-05	172.2	191.1		11.8		0.7				7.5	
15-06	171.1	197.2		58.2	56.6				6.1	11.6	
15-07	170.0	180.4	169.3	36.1	1.3		5.2		0.9	23.9	18.3
16-01		179.3		32.3					26.1	12.0	9.3
16-02	164.1	200.3					1.9		20.7	5.2	11.9
16-03										18.1	
16-04		191.9								6.3	
16-05		185.2								13.4	
16-06		191.8					23.8		5.1	1.3	
16-07	164.3	167.9	165.7	53.1	22.7				0.0	0.1	
19-01	159.1		160.7						2.2		6.9
19-02		209.2			38.6				1.2	18.8	
19-03		195.6			11.7						
19-04		186.7					1.1		0.2	1.5	
19-05		187.7	171.7	1.6	25.6		1.4		1.3		
19-06		166.7	174.6	29.2			0.9		0.0	0.1	
19-07	166.9	175.8	153.4	25.8		42.9	15.6	5.7	0.0	0.4	0.7
20-01			149.9		47.6				0.3	0.1	
20-02									0.1	22.4	
20-03		191.6	171.4	64.4	6.5					31.9	
20-04		182.2			93.1				17.5	1.4	
20-05		176.1	175.6	38.4		38.2					
20-06	162.9	173.9	168.5		48.3		3.3			3.7	
20-07	165.3	173.7	178.1					11.4	4.7	9.7	4.8
23-01		167.8									
23-02		177.9					4.6		0.1		1.9
23-03									2.4	0.0	
23-04		184.6							2.4	15.7	
23-05				48.8	18.5					0.1	
23-06		190.8		49.5					4.2	2.9	2.1
23-07		171.1	169.7						4.6	5.8	4.7
24-01		201.0	178.4				2.1		8.4		
24-02					17.1		14.6				
24-03		191.7			53.2				4.0	10.3	
24-04		185.3		77.6	41.4					14.7	
24-05		183.6		42.6		33.0		14.3		5.3	1.8
24-06		176.3	181.6	50.4	39.5		9.5	0.1	11.8		
24-07	171.7	177.1	171.4	37.1	33.5		0.8		0.1	0.6	
27-01		155.4				41.1					
27-02	170.3			35.7		30.0					11.5
27-03		214.1				39.8			1.8		
27-04	179.5	178.7	178.3	33.4	34.6					10.1	
27-05		181.4	178.0		45.7	27.6			4.3	2.4	
27-06	175.8	185.5	175.5			88.6			0.1	4.2	
27-07	175.8	184.4	177.0	38.2	37.3	48.3	9.4		9.5	0.1	

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28-01		151.6		49.0	61.4	40.7	5.0	9.2	4.7		
28-02		199.2		35.1	32.6	37.2	6.1				0.8
28-03	179.1				38.3		7.8	13.7			5.1 0.3
28-04	178.4	185.6		87.7	23.4	38.4			1.4		4.5
28-05		173.7	176.5	11.8	37.8						3.6
28-06	175.5	189.7		42.3	31.8	25.1					5.8
28-07	173.7	174.1	183.4	33.7	33.1	31.0					
29-01	173.8	196.8	176.1	38.0	33.7	34.2		24.2			14.3
29-02				53.2	38.5	43.7					15.3
29-03		191.3		43.6	33.2						4.8
29-04			178.5		23.9		7.3		0.4		2.9
29-05	178.0	180.7	177.4		22.8			21.9			10.3
29-06	176.2	179.6	175.6		29.8			31.1	0.0		2.6
29-07	175.9	177.6	177.6	35.8	21.2	44.7		34.7			14.3
32-01	176.0	197.5	177.5	23.9	18.2	37.6					
32-02											
32-03		190.4		38.1							
32-04		181.8	181.1								
32-05		183.9	178.8	21.1							
32-06		182.3	178.2								
32-07	177.6	179.5	176.5		36.4	34.9		6.4			29.3
34-01		191.9	177.8	33.5	26.9	29.1	6.5		8.7		
34-02				30.9		29.5					
34-03		186.5		35.4	24.8					0.4	
34-04		186.3	182.6	27.9							0.7
34-05	180.3	182.0	179.8								3.4
34-06		185.5		28.8		29.9					6.2
34-07	174.3	174.8	167.6	27.7	7.1	30.2			3.7		6.5
37-01		207.0			37.7	35.7					4.6 10.0
37-02		207.4			28.5	41.0			0.0	7.0	1.9
37-03		208.6									
37-04	169.7	197.0		40.9	48.8						
37-05	169.6	160.2							6.6		
37-06	167.0	189.0			35.5			36.3		2.4	4.8
37-07	170.0	173.2	170.3		38.0	30.0	34.1				21.8
39-01		186.4		38.9	40.0	34.4		0.0	8.7	9.3	
39-02		190.3		34.9	36.1				20.9	6.9	10.5
39-03		206.9							0.2		
39-04		174.9		16.7							8.0
39-05	173.2	193.5			31.7						3.2
39-06		174.9		57.2	37.2					6.9	5.3
39-07	171.2	176.3	188.3	33.1	30.3	33.7			7.6	8.2	7.2
40-01		177.4		34.2	33.3	30.5			10.8	0.7	7.4
40-02		188.7			31.1	34.6			8.4	8.8	9.0
40-03					34.3						
40-04		190.3									2.7
40-05	177.1	184.2	172.3				1.8	11.9			4.1
40-06		171.1	170.2		34.5			8.4		7.1	11.8
40-07	170.8	180.9	173.8	32.9	33.0	34.3			7.4	6.6	

I.3.5 PSD peaks

Figure I.12 plots the peak values of the Power Spectra Density (PSD) in the white noise cases of the 14UCSD test identified using the FDD method. Table I.8 summarizes the corresponding values.

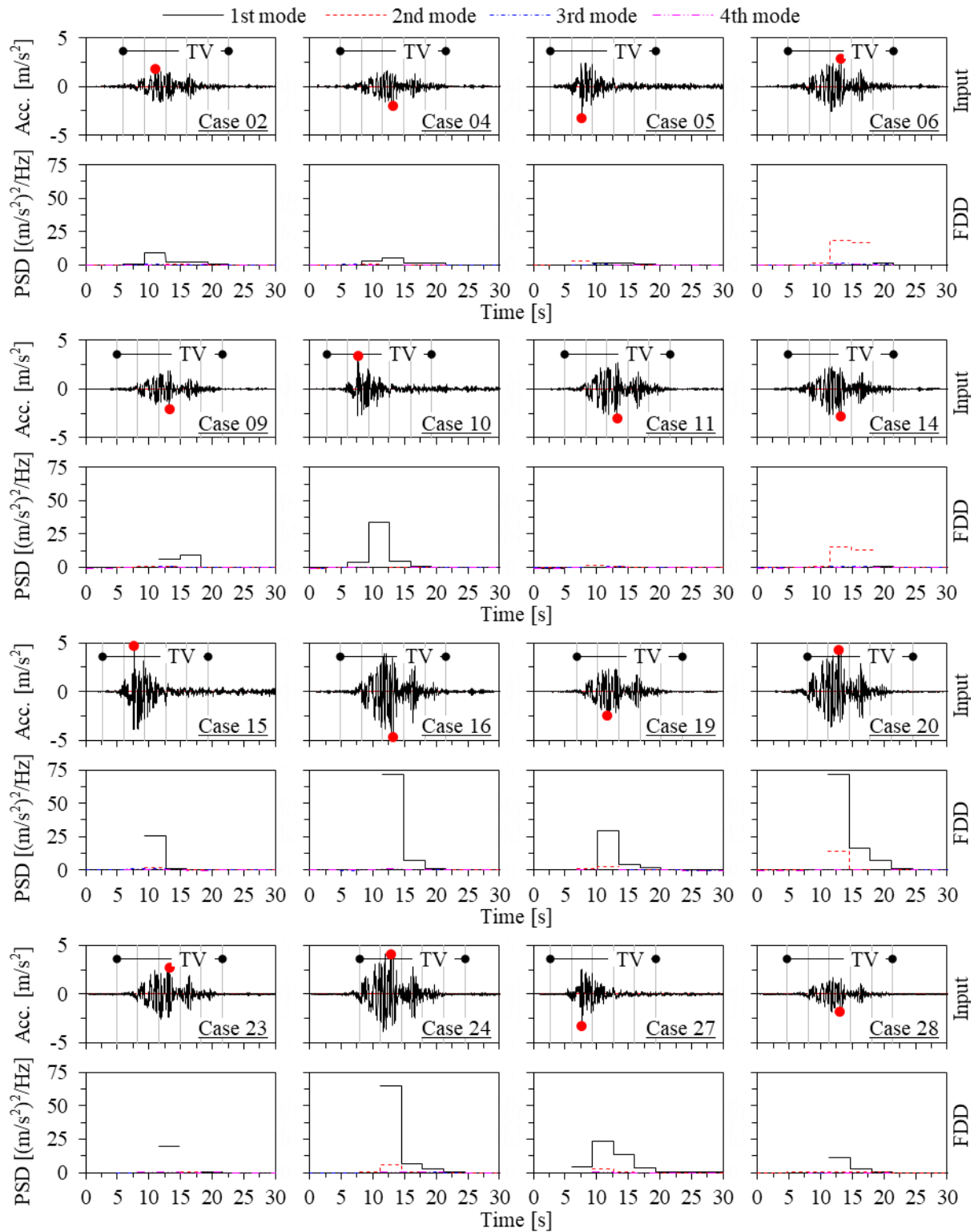


Figure I.12 PSD peaks in ground motion excitations of the 14UCSD test

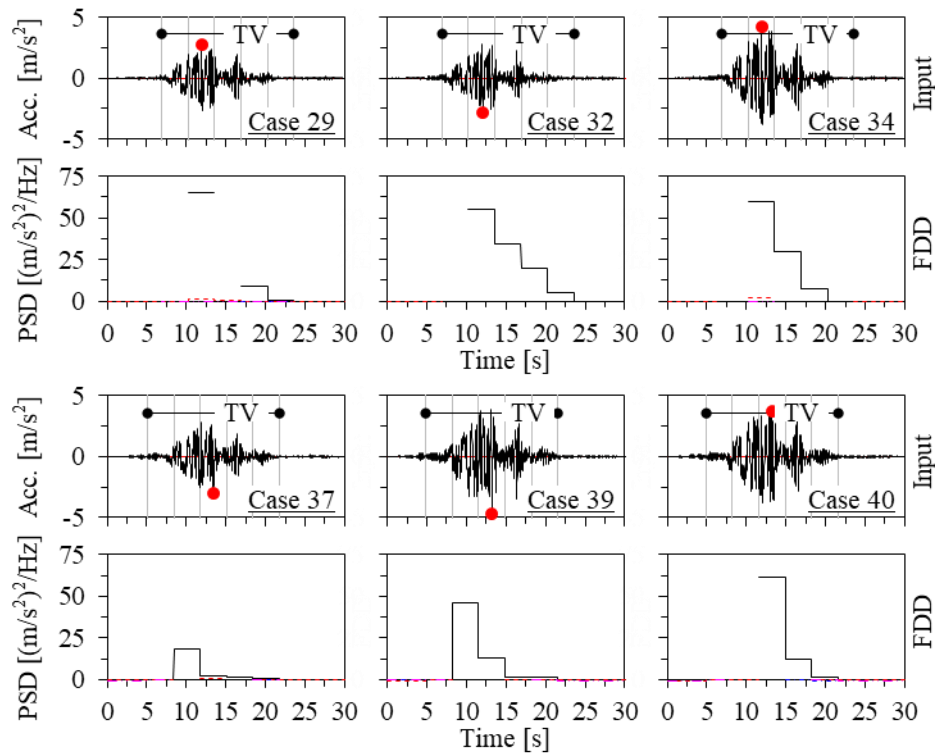


Figure I.12 (continued) PSD peaks in ground motion excitations of the 14UCSD test

 Table I.8 PSD peaks in ground motion excitations of the 14UCSD test [Unit: $(\text{m/s}^2)^2/\text{Hz}$]

Case-Segment	Mode			
	1st	2nd	3rd	4th
02-01	0.00	0.00		0.00
02-02	0.15		0.03	0.06
02-03	9.09		0.29	0.08
02-04	1.99	0.29	0.13	0.06
02-05	2.24		0.03	0.15
02-06	0.17	0.01		0.05
02-07	0.00	0.00	0.00	0.00
04-01	0.00	0.00	0.00	0.00
04-02	0.01		0.18	0.01
04-03	2.71	0.40		0.04
04-04	5.42			
04-05	1.41	0.12		0.12
04-06	1.13	0.07	0.07	0.08
04-07	0.01		0.00	0.00
05-01		0.00		0.00
05-02				
05-03		2.91	0.20	
05-04	1.43		0.20	
05-05	1.33			0.10
05-06	0.16	0.06		0.02
05-07	0.03			0.00
06-01	0.00	0.00	0.00	0.00
06-02	0.01	0.01		0.01

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06-03		1.09	0.13	0.12
06-04		18.22	1.18	0.77
06-05		16.58	0.22	0.25
06-06	1.12		0.21	0.06
06-07	0.02			
09-01	0.00			0.00
09-02		0.00		0.01
09-03		0.61		
09-04	5.95	0.65	0.40	0.24
09-05	8.93		0.18	0.15
09-06	0.23	0.05	0.03	
09-07	0.00	0.00	0.00	0.00
10-01	0.00	0.00	0.00	0.00
10-02	0.00	0.00	0.01	0.00
10-03	3.94			0.12
10-04	33.75			
10-05	4.85	0.03		
10-06	1.04	0.08	0.07	0.06
10-07	0.03		0.02	0.01
11-01	0.00	0.00	0.00	0.00
11-02	0.01		0.02	0.01
11-03		1.13		
11-04		0.79	0.70	0.24
11-05		0.29		
11-06	0.12		0.13	0.03
11-07	0.01	0.00		0.00
14-01		0.00	0.00	0.00
14-02	0.01	0.03		0.01
14-03		0.65		0.09
14-04		15.34	0.37	0.10
14-05		13.23	0.37	0.16
14-06	0.59			0.06
14-07	0.01	0.00	0.00	0.00
15-01			0.00	
15-02	0.01		0.01	
15-03			0.77	0.24
15-04	25.45	1.64	1.09	0.12
15-05	0.84			0.02
15-06	0.37	0.05		0.00
15-07	0.08	0.01	0.03	0.01
16-01	0.00			0.00
16-02	0.03		0.01	0.01
16-03				0.22
16-04	71.94			0.65
16-05	7.38			0.17
16-06	1.28		0.10	0.09
16-07	0.01	0.00		0.00
19-01				
19-02	0.56	0.90		0.11

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19-03	29.76	2.51		
19-04	4.30		0.09	0.03
19-05	1.87	0.37	0.04	
19-06	0.14		0.02	0.05
19-07	0.00		0.00	0.00
20-01		0.00		0.00
20-02				0.18
20-03	71.51	14.23		0.36
20-04	15.99	0.20		0.10
20-05	7.22			
20-06	0.74	0.02		0.01
20-07	0.01		0.00	0.00
23-01	0.00			
23-02	0.01		0.01	
23-03				0.02
23-04	19.54			0.53
23-05		0.26		0.01
23-06	0.51			0.02
23-07	0.01			0.00
24-01	0.00		0.00	
24-02		0.30		
24-03	64.63	5.65		0.64
24-04	6.67	0.28		0.14
24-05	2.37		0.12	0.06
24-06	0.23	0.00	0.01	
24-07	0.00	0.00		0.00
27-01	0.00			
27-02				
27-03	4.33			
27-04	23.67	2.47		0.06
27-05	13.71	0.02		0.01
27-06	3.11			0.00
27-07	0.05	0.00		0.00
28-01	0.00	0.00	0.00	
28-02	0.00	0.03		0.00
28-03		0.10	0.01	0.00
28-04	11.15	0.28		0.01
28-05	3.03	0.06		0.01
28-06	0.41	0.07		0.00
28-07	0.00	0.00		
29-01	0.00	0.01	0.00	0.00
29-02		0.12		0.01
29-03	65.13	1.24		0.12
29-04		0.15		0.01
29-05	8.98	0.06	0.02	0.01
29-06	0.24	0.00	0.00	0.00
29-07	0.00	0.00	0.00	0.00
32-01	0.00	0.00		
32-02				

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32-03	55.43			
32-04	34.14			
32-05	20.10			
32-06	5.07			
32-07	0.05	0.00		
34-01	0.00	0.00		
34-02				
34-03	59.57	2.22		0.13
34-04	29.59			
34-05	7.17			
34-06	0.05			
34-07	0.00	0.00		
37-01	0.00	0.00		0.00
37-02	0.01	0.03		0.00
37-03	18.35			
37-04	2.50	0.54		
37-05	1.58			
37-06	0.67	0.12	0.00	0.00
37-07	0.00	0.00		
39-01	0.00	0.00	0.00	0.00
39-02	0.03	0.29		0.00
39-03	46.49			
39-04	13.21			
39-05	1.48	0.32		
39-06	1.88	0.16		0.01
39-07	0.02	0.01		0.00
40-01	0.00	0.00		0.00
40-02	0.02	0.05		0.00
40-03		0.21		
40-04	61.75			
40-05	12.24		0.16	
40-06	1.47	0.10	0.00	0.01
40-07	0.00	0.01		0.00

I.4 EFFECTIVE MODAL PSEUDO-ACCELERATION (EMPA)

Figure I.13 plots the peak EMPA values identified in the ground motion cases of the 14UCSD test, including the elastic pseudo-acceleration spectra with 2% damping of the measured excitations.

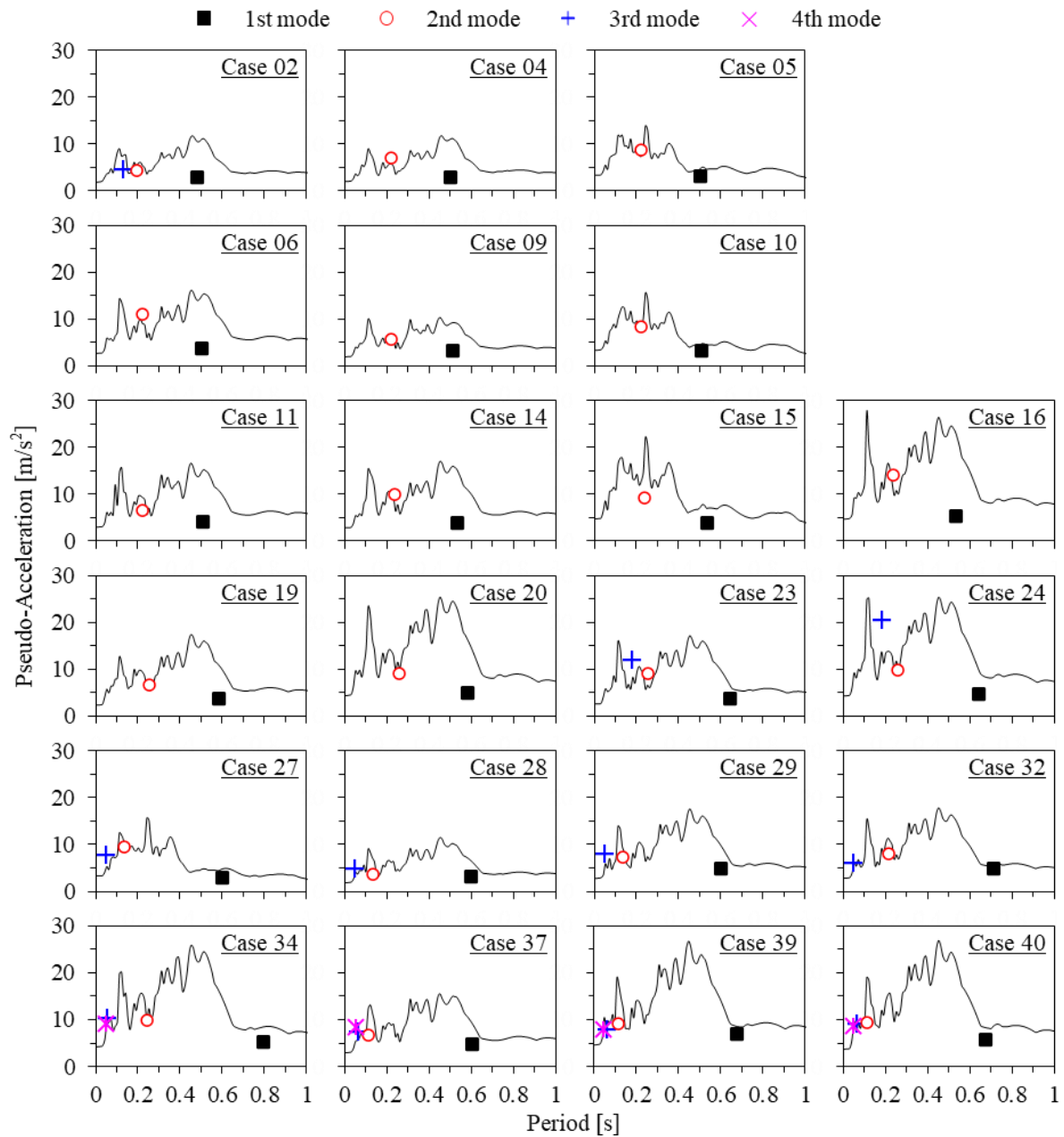


Figure I.13 Peak EMPA values in ground motion excitations of the 14UCSD test

I.5 TIME-FREQUENCY RESULTS FROM GROUND MOTION EXCITATIONS

I.5.1 Magnitude of wavelet coefficients and instantaneous periods

Figure I.14 plots the magnitude of wavelet coefficients on the time-frequency plane and the identified instantaneous periods in the ground motion cases of the 14UCSD test.

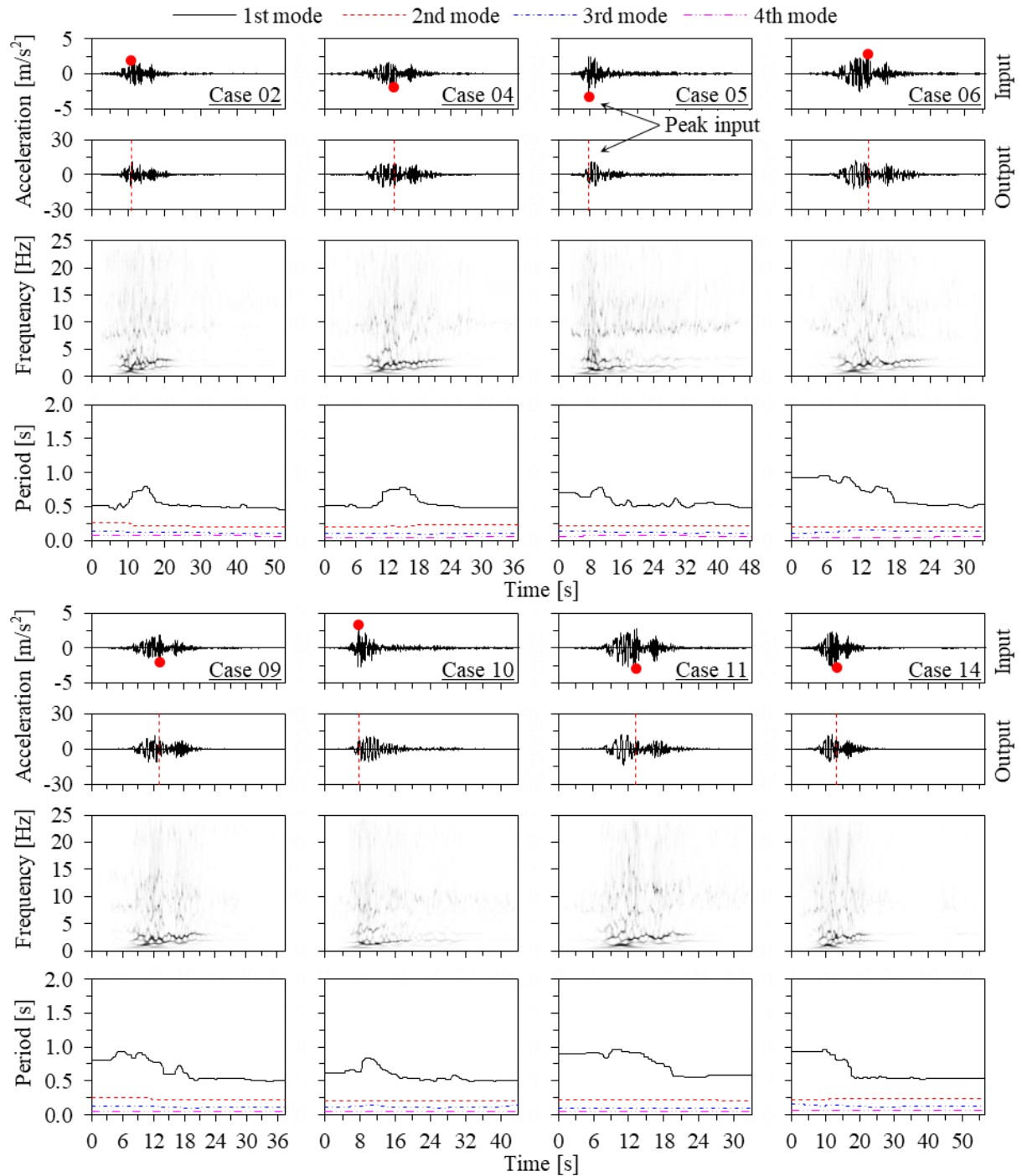


Figure I.14 Magnitude of wavelet coefficients in ground motion excitations of the 14UCSD test

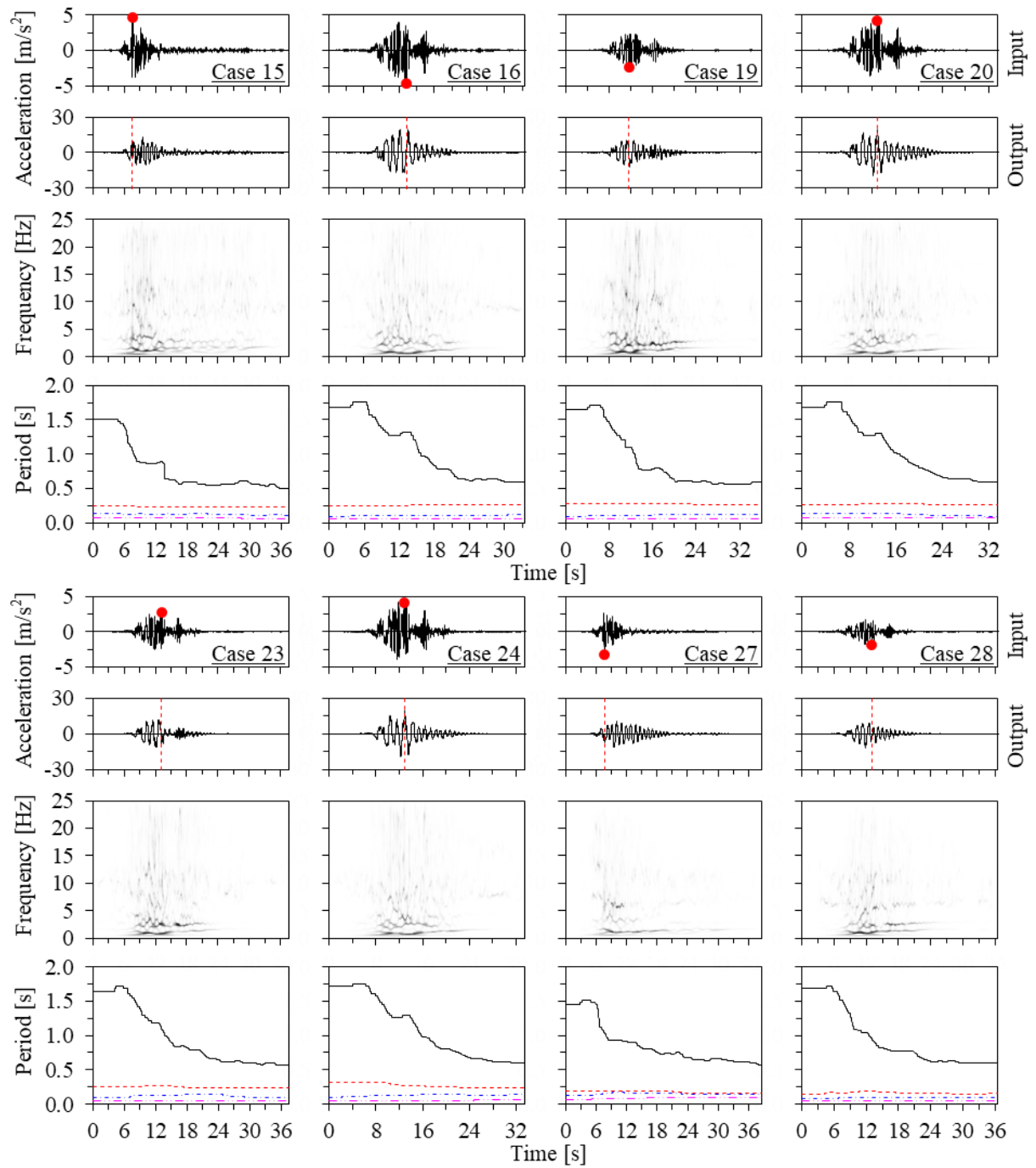


Figure I.14 (continued) Magnitude of wavelet coefficients in ground motion excitations of the 14UCSD test

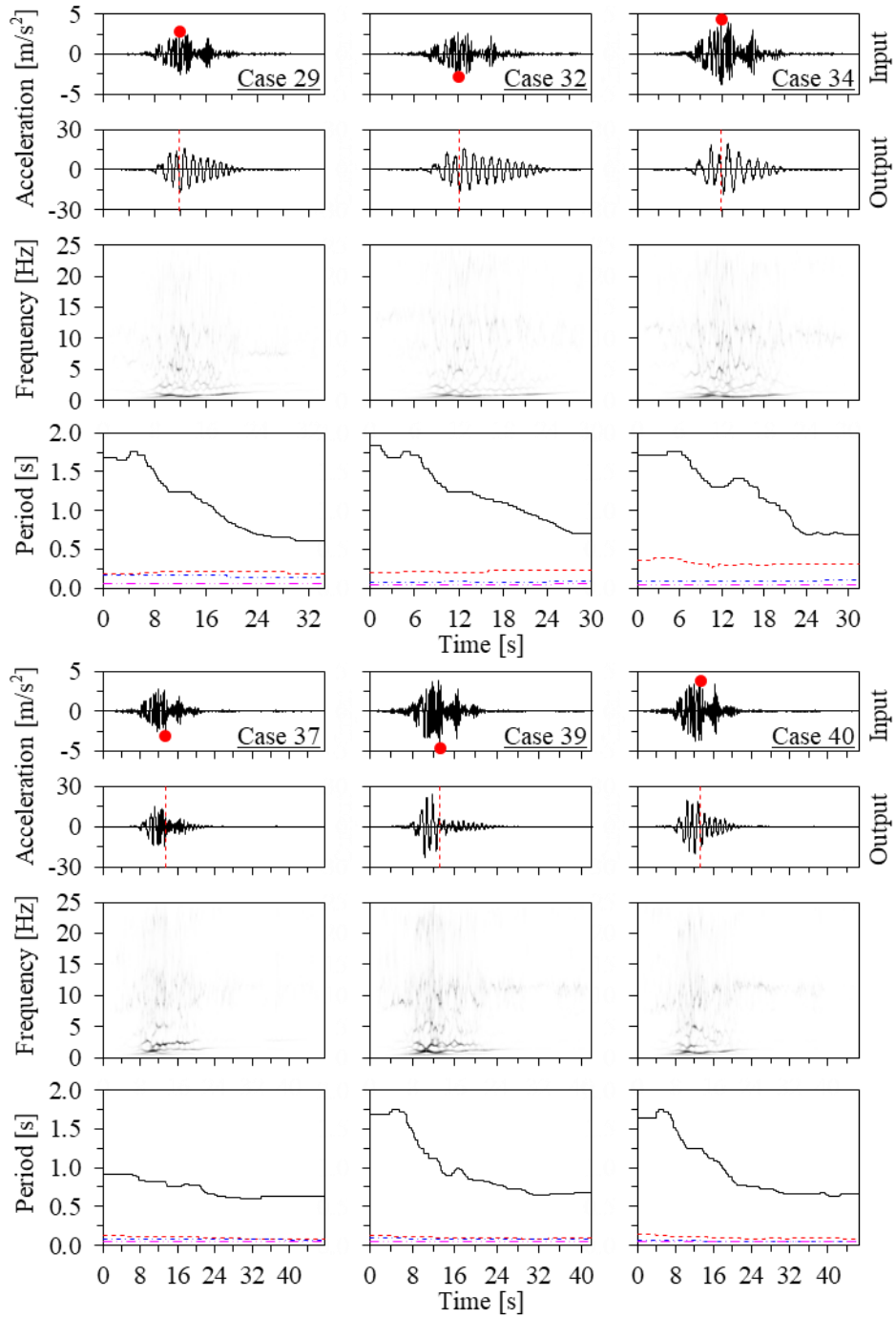


Figure I.14 (continued) Magnitude of wavelet coefficients in ground motion excitations of the 14UCSD test

I.5.2 Magnitude of ridge wavelet coefficient

Figure I.15 plots the magnitude of wavelet coefficient on the wavelet ridges in the ground motion cases of the 14UCSD test.

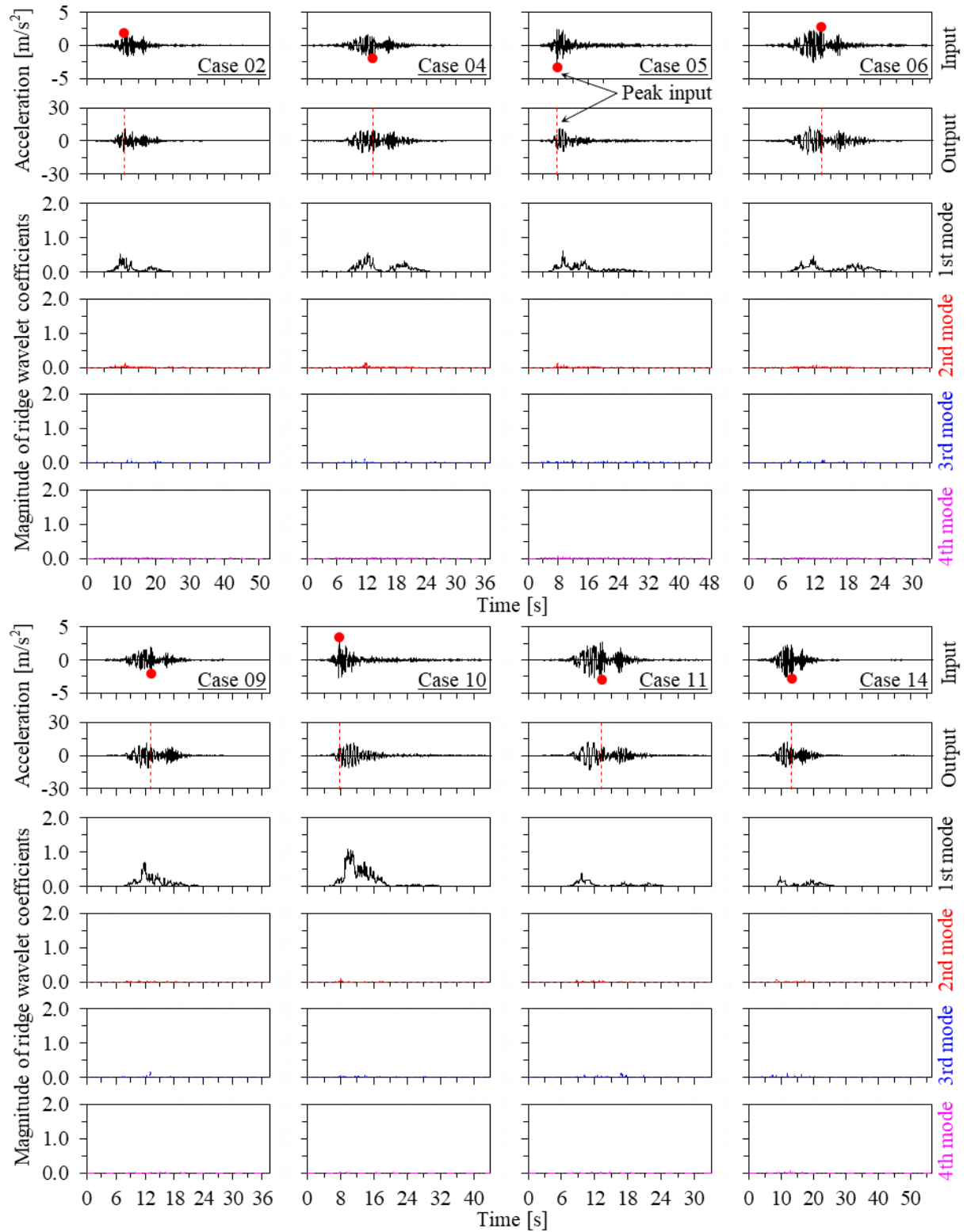


Figure I.15 Magnitude on wavelet ridges in ground motion excitations of the 14UCSD test

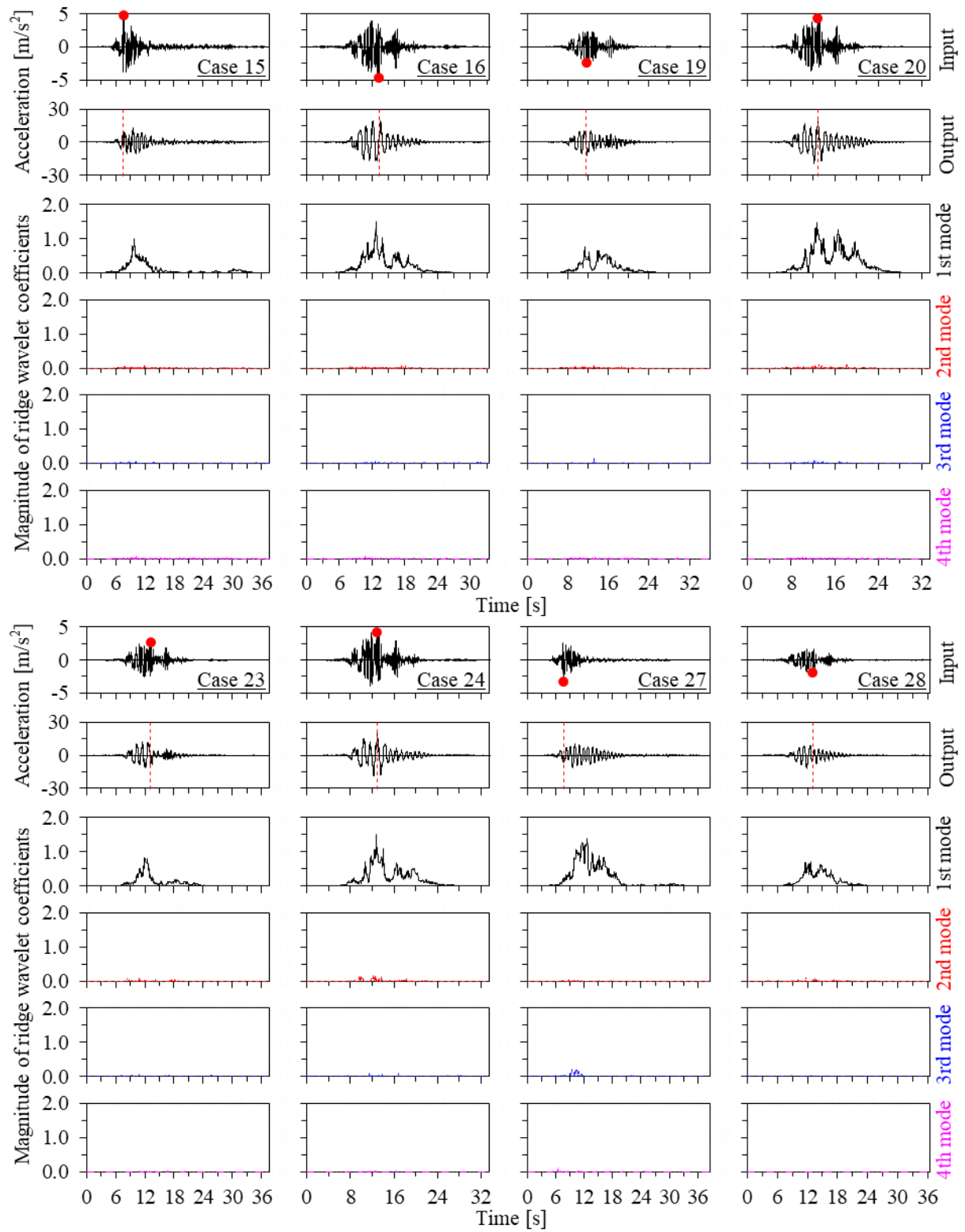


Figure I.15 (continued) Magnitude on wavelet ridges in ground motion excitations of the 14UCSD test

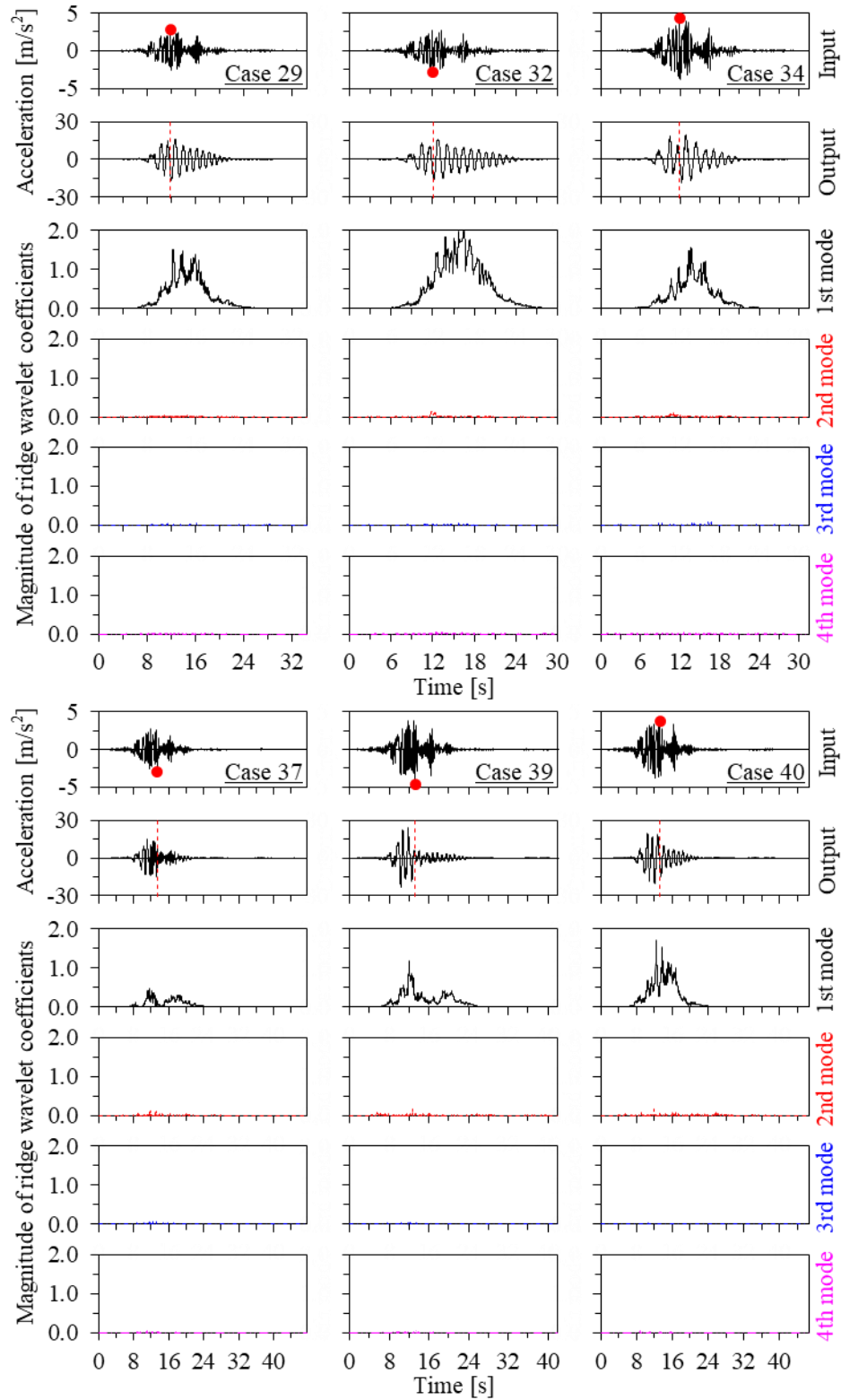


Figure I.15 (continued) Magnitude on wavelet ridges in ground motion excitations of the 14UCSD test

I.5.3 Reconstructed modal components

Figure I.16 plots the time-domain modal components reconstructed from the SSWT results and ridge identification in the ground motion cases of the 14UCSD test.

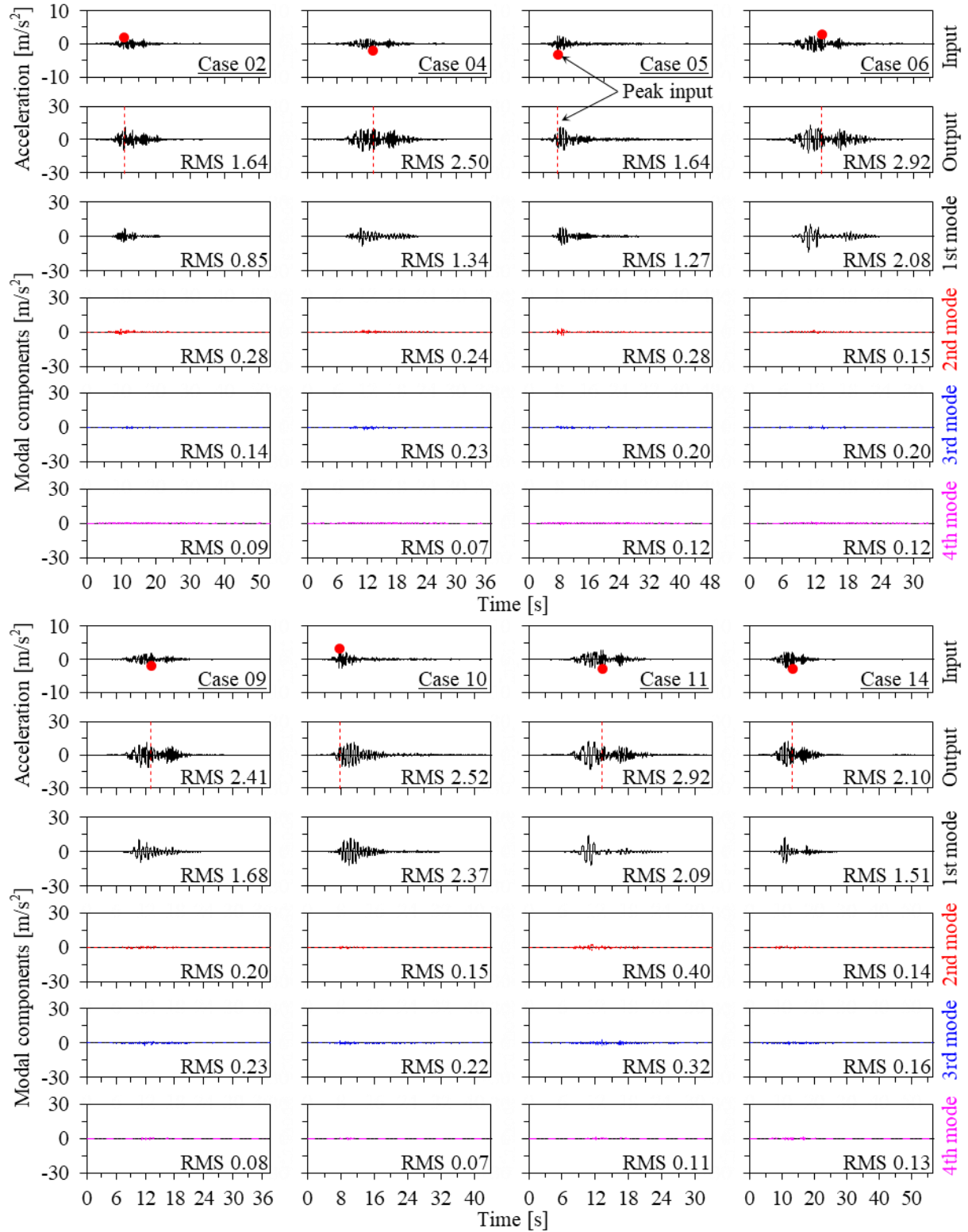


Figure I.16 Modal components in ground motion excitations of the 14UCSD test

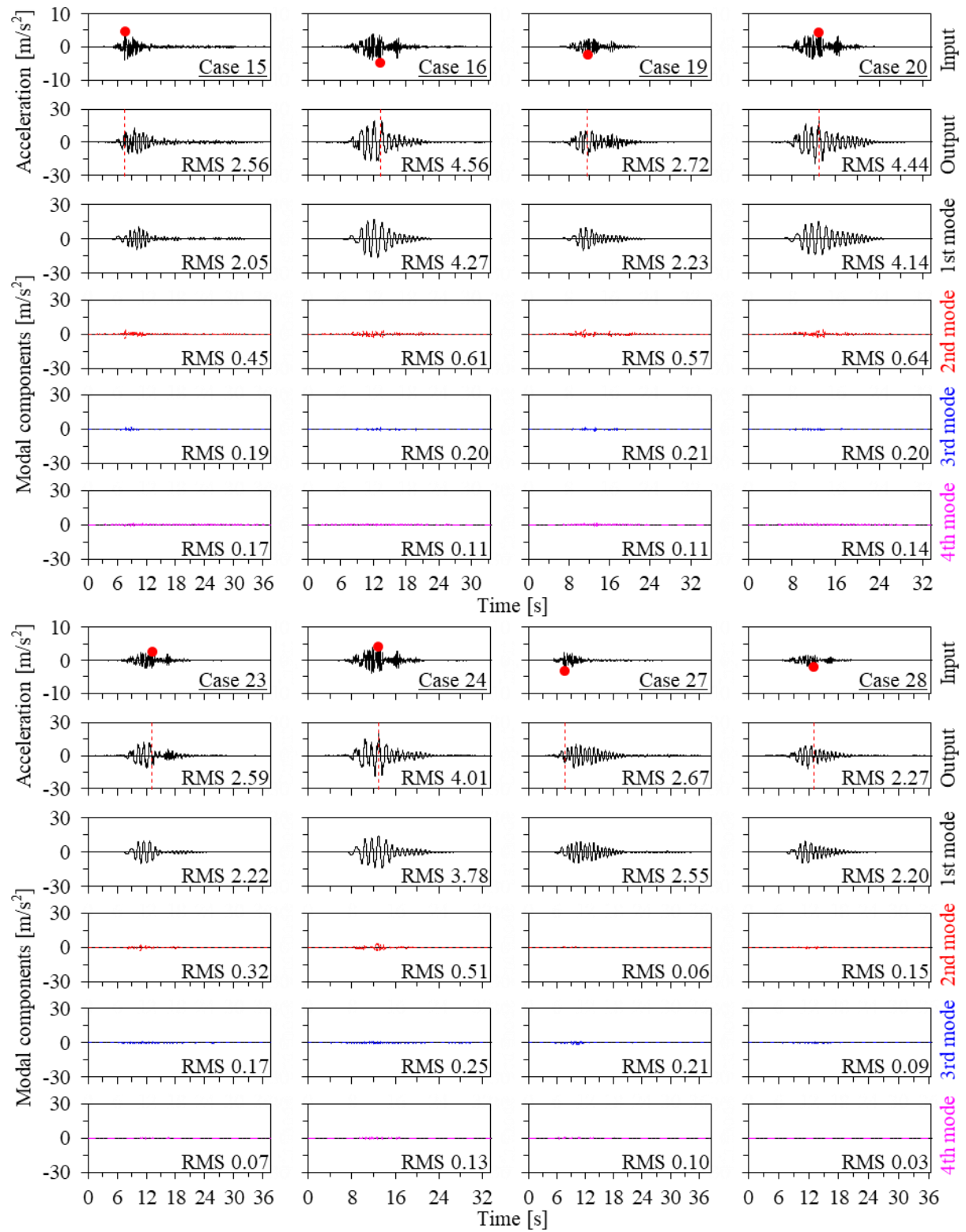


Figure I.16 (continued) Modal components in ground motion excitations of the 14UCSD test

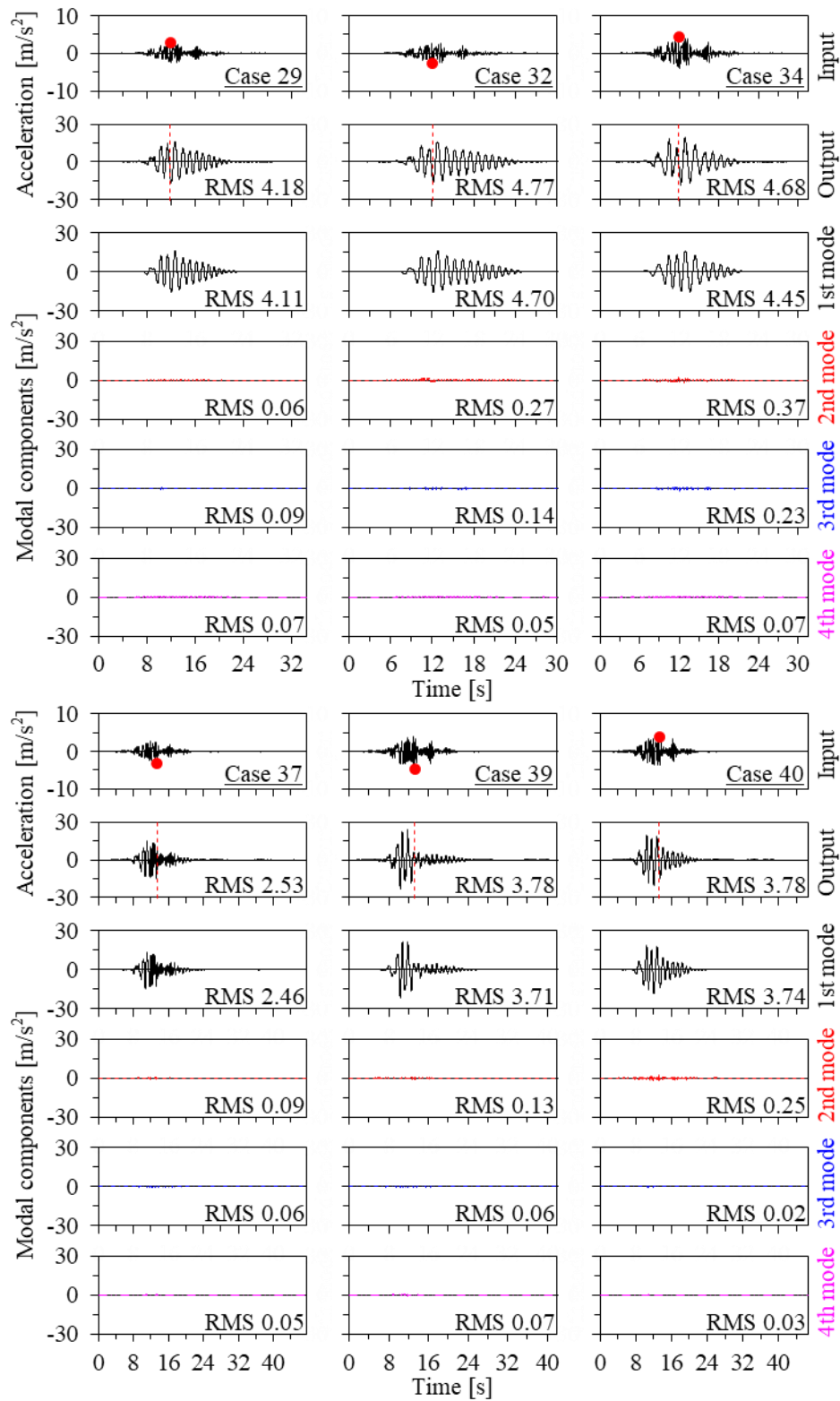


Figure I.16 (continued) Modal components in ground motion excitations of the 14UCSD test