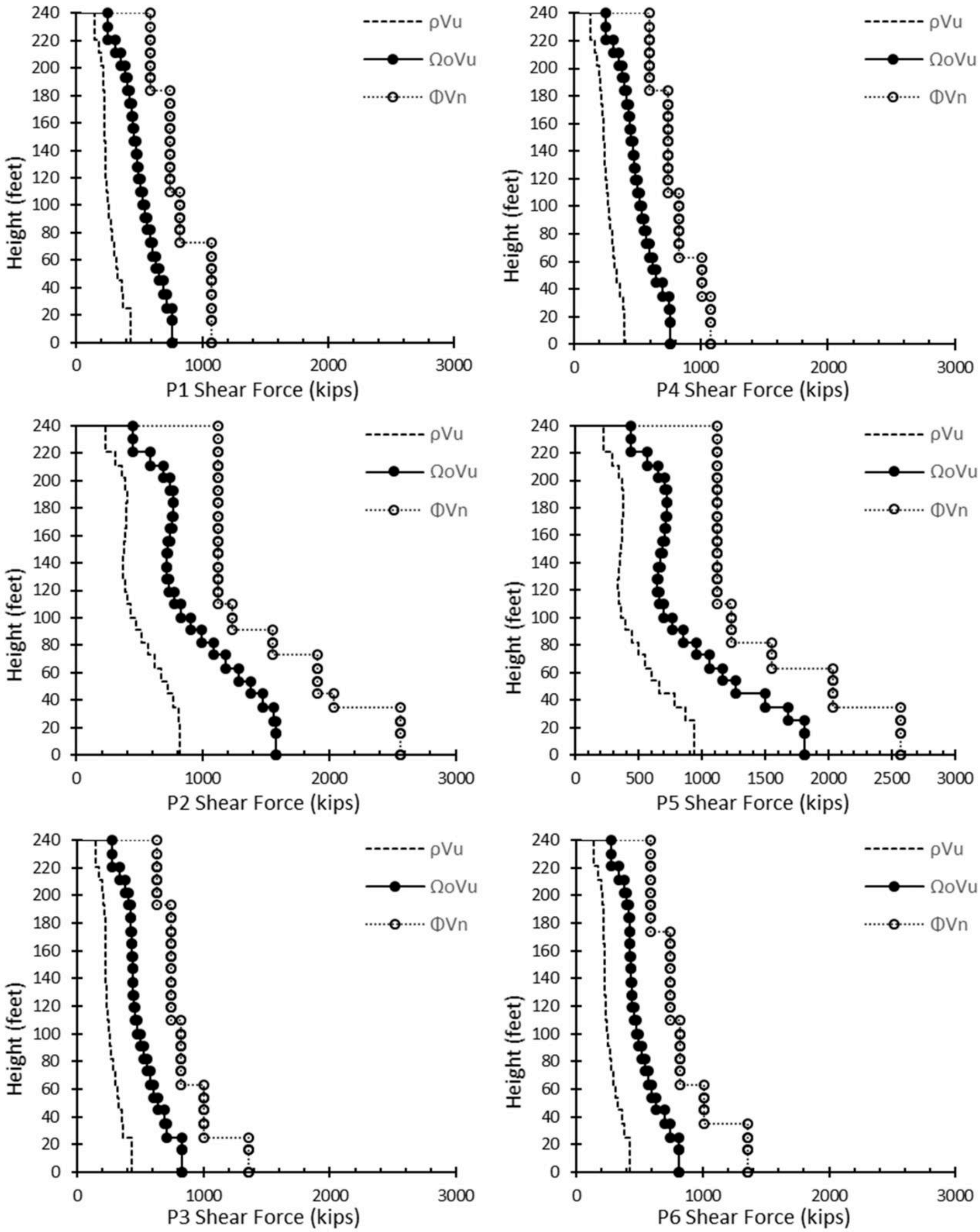

Figure 5— Building-A maximum wall pier shear demand and capacity at code design level (1 kips = 4.448 Kn, 1 foot = 0.3048 m)

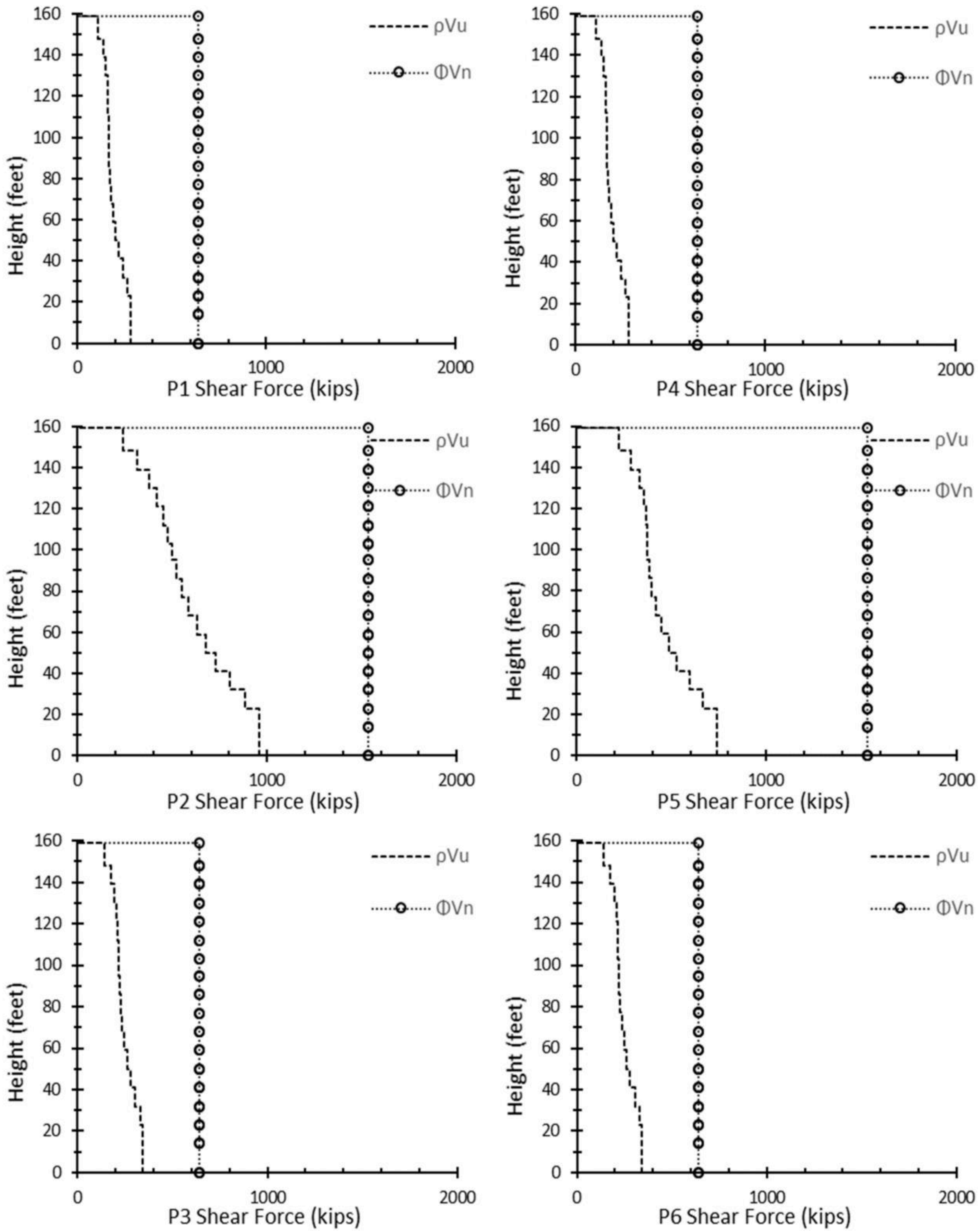

Figure 6— Building-B maximum wall pier shear demand and capacity at code design level (1 kips = 4.448 Kn, 1 foot = 0.3048 m)

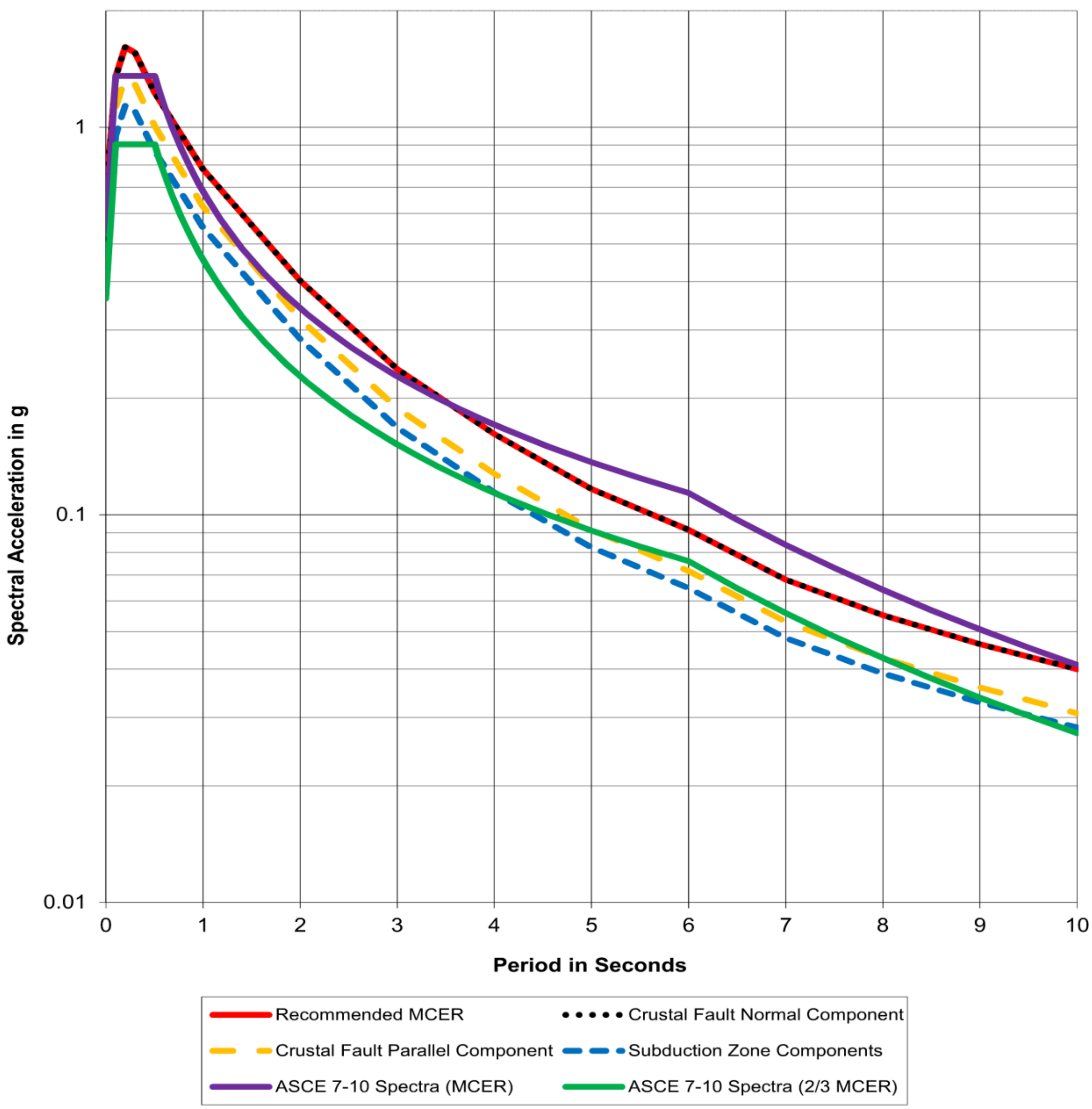

Figure 7— Target spectra for ground motion scaling

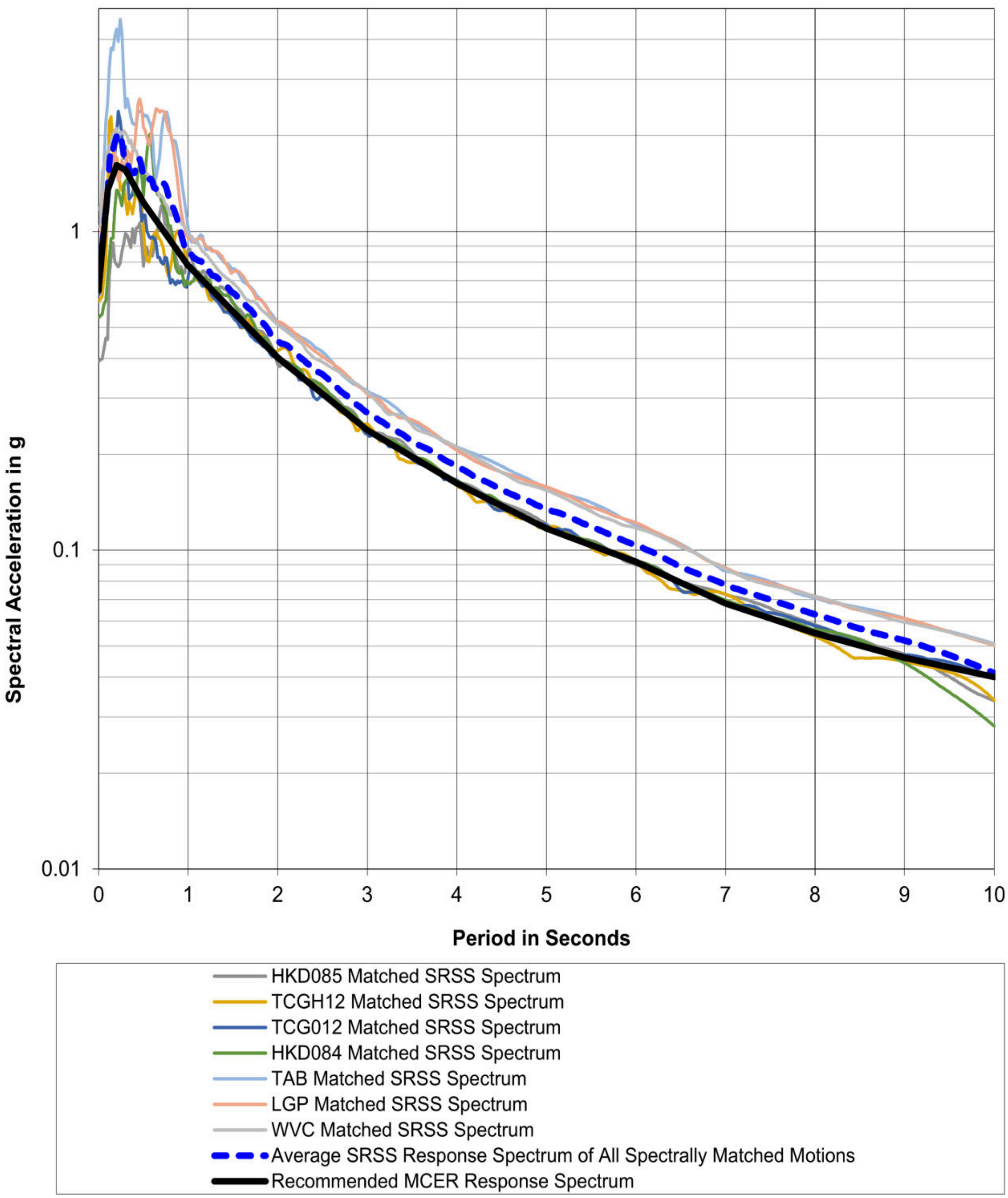

Figure 8— Comparison of average SRSS ground motions to MCE_R spectrum

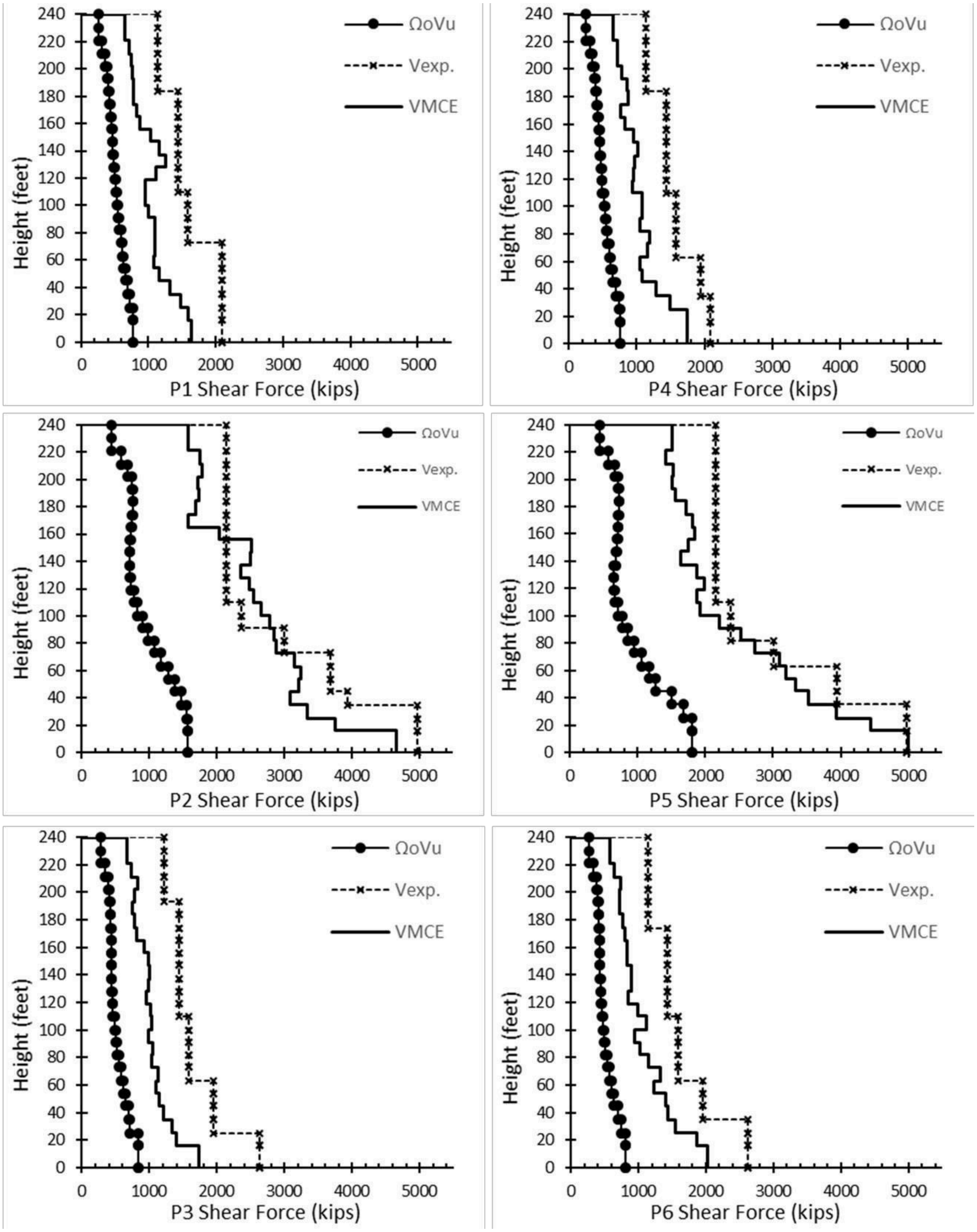

Figure 9— Building-A wall pier shear demand and capacity at MCE level (1 kips = 4.448 Kn, 1 foot = 0.3048 m)

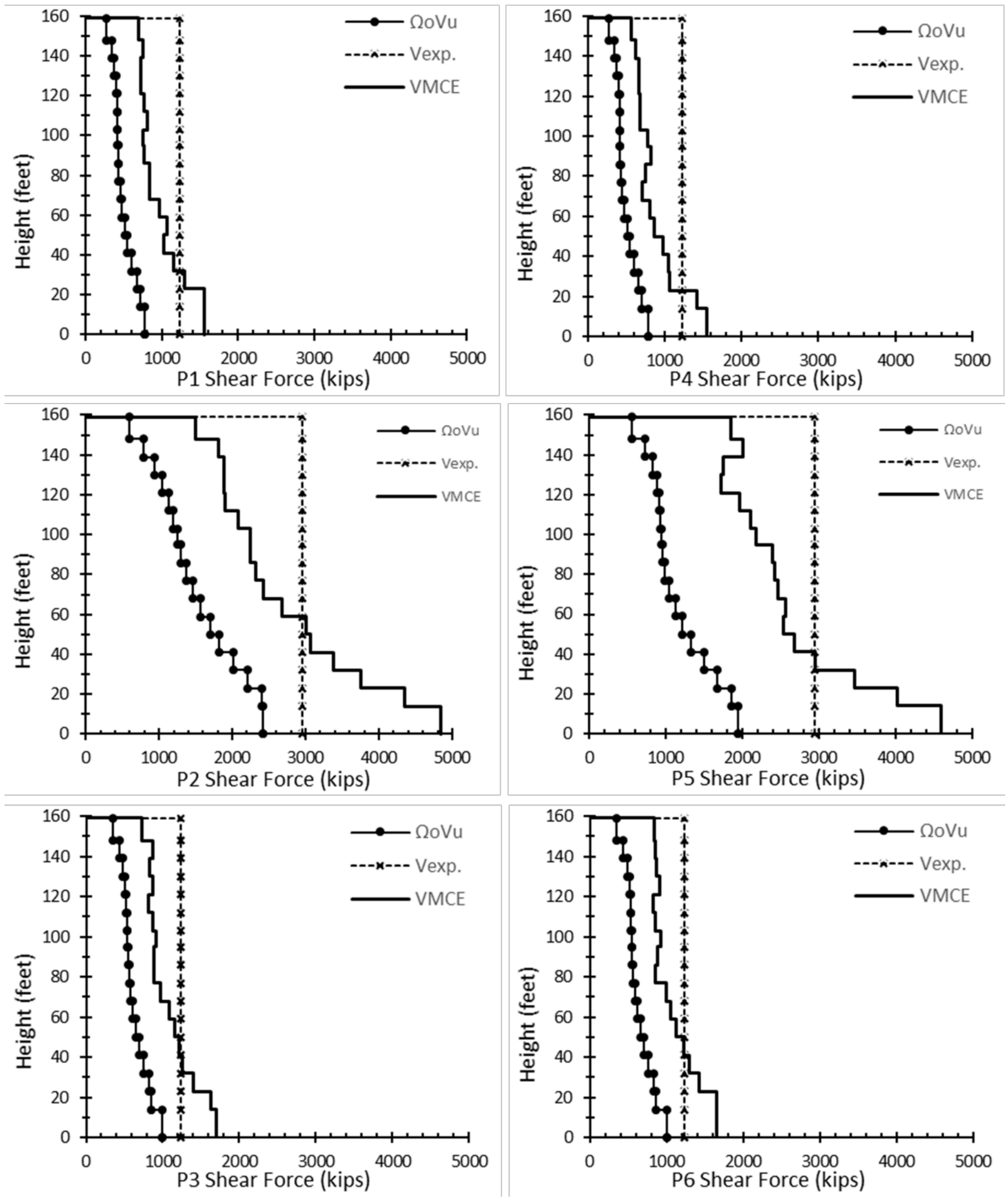

Figure 10— Building-B wall pier shear demand and capacity at MCE level (1 kips = 4.448 Kn, 1 foot = 0.3048 m)

Trends in Demands for Concrete Performance-Based Seismic Design Towers

Kevin Aswegan and Ian McFarlane

Synopsis: The use of a Performance-Based Seismic Design (PBSD) approach to design buildings whose heights exceed 240 ft (73 m) has become common in many West Coast cities. This paper studies trends across 14 special reinforced concrete shear wall PBSD towers designed within the last 5 years. The primary purpose of evaluating these trends is to compare demands calculated using a linear elastic design approach (i.e. for Design Earthquake or Service Level shaking) to the demands (average results from 7 or 11 ground motions) determined through nonlinear analysis (i.e. for Maximum Considered Earthquake shaking). The specific demands evaluated include core wall shears and foundation overturning moments. The paper also demonstrates that shear and moment amplification are significant phenomena for concrete buildings, and are believed to be primarily due to nonlinear behavior, material over-strength, higher mode effects, and damping and stiffness assumptions. The results present a useful range of trends to provide an engineer guidance on the expected demands and the level of variability between projects. The paper highlights some of the reasons for the variability in these trends, and provides general proportioning recommendations.

Keywords: concrete shear walls, PBSD, performance-based seismic design

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INTRODUCTION

The use of a Performance-Based Seismic Design (PBSD) approach to design buildings whose heights exceed 240 ft (73 m) has become common in many west coast cities. The authors have experience with concrete shear wall towers designed using PBSD methodology in Seattle, San Francisco, Oakland, and Los Angeles. The purpose of this paper is to document and explore trends observed for these towers, specifically drawing on results from 14 recent building designs by Magnusson Klemencic Associates (MKA), and an in-depth case study.

History

Performance-based seismic design was first implemented in the United States in the late 1990s for the design of the 322 ft (98 m) Key Center in Bellevue, Washington. For this building, the methodology was used to demonstrate code-equivalency for the special reinforced concrete core wall lateral system which exceeded the 240 ft height limit found in Table 12.2-1 of ASCE 7 (2010). The design fell outside of the prescriptive requirements of the building code, which would have otherwise mandated a full moment frame system or a dual system consisting of a combination of special reinforced concrete shear walls and special moment frames. For a typical building the special moment frames have several significant drawbacks, including added cost, additional time to construct, and aesthetic/architectural impacts. For the majority of projects the benefits of the elimination of the moment frames drastically outweigh the additional time and effort required for the more advanced analysis.

Over the past twenty years developers, architects, contractors, and engineers have become more and more familiar with the PBSD process. This increased familiarity and understanding has led to PBSD becoming the de facto starting point for virtually all tall buildings designed and constructed on the West Coast of the United States. This includes several recently constructed high-rise structures, such as the 1,070 ft (326 m) Salesforce Tower in San Francisco, the 1,100 ft (335 m) Wilshire Grand Center in Los Angeles, and the trio of 520 ft (158 m) Amazon headquarters towers in Seattle. Fig. 1 shows a view of the rapidly growing South of Market neighborhood in San Francisco, in which nearly all of the towers have been designed using a PBSD approach.

Figure 1— PBSD towers in San Francisco south of market neighborhood (c. 2016)

Design Process

The performance-based seismic design process is documented in various industry reference documents, including the PEER Tall Buildings Initiative (PEER TBI, 2010), PEER TBI Version 2 (2017), and the Los Angeles Tall Buildings Structural Design Council consensus document (LATBSDC, 2015). The process typically involves two or three steps, as outlined below.

1) Code-level Elastic Analysis and Design: initial structural design for the code-prescribed Design Earthquake (DE) shaking. It is during this step that the building is designed to conform to all provisions of the building code, with the standard exception of the lateral system height limits and other specific building code provisions which the design team lists as code exceptions. This step is not required by either PEER TBI or the LATBSDC guidelines, although certain jurisdictions (San Francisco, for example) mandate it.

2) Serviceability Analysis: verification of primary structural response behavior (e.g. story drift) under the Service Level Earthquake (SLE) 43-year return period shaking. This step provides an additional minimum level of building strength, with the intent of maintaining a serviceable structure under frequent seismic demands by requiring “essentially elastic” building behavior.

3) Nonlinear Response History Analysis: verification of structural performance under the risk-targeted Maximum Considered Earthquake (MCE_R) shaking. The intent of this step is to demonstrate a low probability of collapse when the building is subjected to an extremely rare, large magnitude earthquake. This step requires the development of a detailed nonlinear structural analysis model which is subjected to a suite of ground motions.

In terms of level of effort and time involved, Steps 1 and 2 above are relatively simple and straightforward, and fit easily within a typical project design timeline. Step 3 requires substantial time to develop the nonlinear model, run the ground motion simulations, and post-process results. Depending on the complexity of the building, the time to complete Step 3 can take several months, with additional time required for review and approval by the expert peer reviewers. Due to the level of effort and time required for the nonlinear response history analysis (NLRHA), it is usually the goal of the structural engineer to perform Step 3 only once for a given project. To achieve this, it is critical that the building is well-proportioned and designed for the DE and SLE demands, such that the NLRHA is truly a verification stage, rather than a “design by trial and error” stage. As a result of the nonlinear nature of the analysis, any changes to the analytical model during Step 3 (wall reinforcing, coupling beam depths, wall thickness, etc.) have the potential to trigger re-running the entire suite of ground motions.

The authors have found that designing solely for the elastic DE and SLE demands will usually result in structural designs which do not meet all of the acceptance criteria, requiring revisions and additional analysis. This is especially true for wall shear forces and foundation overturning moments, which experience significant amplifications due to nonlinear behavior, flexural over-strength, higher mode effects, etc. For example, a wall shear stress corresponding to $4\sqrt{f'_c}$ under DE-level demands may be amplified under MCE demands to $12\sqrt{f'_c}$ (amplification factor of 3.0) which no longer meets the code-prescribed maximum of $10\sqrt{f'_c}$ (ACI 318-14, section 18.10.4.4). To avoid unwanted surprises during the MCE verification phase, it is critical that the engineer has a thorough understanding of the expected demand amplifications prior to proportioning and designing the structure. Common practice is to assume that MCE demands for force-controlled actions will be approximately 2.0 to 4.0 times the DE- or SLE-level demands. This paper seeks to validate and expand on the assumed amplification factors.

GLOBAL RESULTS – AMPLIFICATION FACTORS

In an effort to verify the shear and moment amplification factors used for preliminary proportioning of high-rise concrete core wall structures, the authors and others within their firm reviewed and collected results from recent projects. The primary purpose of this effort was to assemble a database of information related to buildings designed using a performance-based seismic design methodology. Data points include items such as building height, site class, location, primary use, and foundation system. Also included for each building are the base shear and overturning moment results for the SLE, DE, and MCE analyses (average results of 7 or 11 ground motions).

The tables below contain a subset of the information in the database. Table 1 presents the base shear results for 14 different concrete core wall projects; Table 2 contains the foundation overturning moments for the same buildings. The foundation overturning moments are affected by the number of basement levels (i.e. backstay effect), which varies from a minimum of 1 to a maximum of 7 levels. In general, if the peak moment occured at the grade-level (transfer) diaphragm, the peak is carried down to the foundation for design. Except for Building No. 4 (in Los Angeles), the buildings are roughly evenly split between Seattle and San Francisco (international projects have been excluded from this paper). The building heights range from approximately 295 ft (90 m) to 605 ft (184 ft), with an average of 449 ft (137 m). As shown in the tables, the buildings are primarily residential, office, or hotel use.

Figs. 2 and 3 show the amplification factors for base shear and base moment, respectively. For each building, the amplification factors were calculated as the average factor of the two orthogonal directions. For example, for Building No. 1, the MCE/SLE base shear amplification factor was determined by the following steps:

1) Divide the x-dir MCE base shear by the x-dir SLE base shear: 20,400 k / 4,450 k = 4.58
2) Divide the y-dir MCE base shear by the y-dir SLE base shear: 13,850 k / 3,400 k = 4.07
3) Average the x-dir and y-dir amplification factors: 0.5 x (4.58 + 4.07) = 4.33.

The individual amplification factors show no clear trend with respect to building height.