Evaluating Existing and Proposing New Seismic Design Provisions for Rigid Wall – Flexible Diaphragm Buildings

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Abstract

Buildings with heavy concrete or masonry walls supported by flexible wood or steel deck roof diaphragms are ubiquitous across the United States and the rest of North America. The current seismic design approach is based on the equivalent lateral force (ELF) method whose underlying assumptions significantly differ from the actual dynamic response of these buildings. The seismic behavior of rigid wall-flexible roof diaphragm (RWFD) buildings is dominated by the diaphragm's response instead of the walls' in-plane response. Furthermore, the diaphragm's ductility and overstrength capacity is unique to its own construction. Yet the current design methodology employed by practitioners directly ties the diaphragm shears and overstrength to the characteristics of the seismic force-resisting system's (SFRS) *vertical* elements.

Past problems in these buildings have been the repeated failures of the walls' anchorage to the diaphragm, and through a series of "trial and error" iterations, the current design provisions have evolved. Current wall anchorage forces for RWFD buildings are believed to now be near maximum expected force levels with little necessary reliance on connector ductility; however, solving the wall anchorage issue may result in new failures within the diaphragm itself.

Using a dedicated numerical modeling framework coupled with a FEMA P-695 collapse capacity evaluation process, a research study was conducted to evaluate performance for a variety of RWFD archetypes conforming to ASCE/SEI 7-10, as well as redesigned archetypes conforming to a new design methodology. Furthermore, a review of the predicted wall anchorage forces in RWFD buildings was also compared with existing design provisions.

A new RWFD design methodology is proposed providing a rational approach to improve performance in these unique buildings.

Introduction

The commonly used equivalent lateral force (ELF) procedure in the current building code represents a seismic response based on a classical model that is quite different from the actual seismic behavior of low-rise buildings with large flexible roof diaphragms supported laterally by rigid walls or stiff frames. The past seismic performance of these rigid wall-flexible roof diaphragm (RWFD) buildings has been troublesome, and the code requirements for these buildings have evolved mostly as reactions to observed damage with little consideration of how these buildings respond differently to earthquakes than multi-story buildings or one-story buildings with rigid diaphragms. These buildings have diaphragms that dominate the building behavior; yet due to their complex inelastic response, past attempts in accurate modeling have typically been time consuming and elusive. With a numerical modeling framework developed specifically for this building type and that balances numerical efficiency and accuracy, the development of new seismic design methodologies for RWFD buildings may be possible to provide a more rational design approach that is still simple to apply.

Building Description

Structures containing rigid walls with flexible roof diaphragms are ubiquitous across our urban environment. Often labeled as “big-box” buildings, these structures are the mainstay for retail, storage and distribution facilities for North America’s largest companies. These buildings are favored by developers and owners for providing the most cost effective approach to enclosing large floor spaces while providing durable and secure perimeters.

RWFD buildings incorporate concrete or masonry walls, which are considered rigid in-plane, with flexible horizontal in-plane steel or wood roof diaphragm systems. The rigid walls act as shear walls to provide seismic shear resistance. Concrete wall systems are most often tilt-up concrete, a unique form of site-cast precast concrete [ACI, 2010]. These highly efficient and versatile enclosures are common across North America, including in high seismic zones. Plant-cast precast concrete walls and concrete block masonry are also very popular perimeter shear wall systems enclosing these structures. These rigid wall systems inherently carry large perimeter seismic weights relative to the roof diaphragm weight.

Roof diaphragms in these buildings consist either of a steel deck diaphragm or a wood structural panel diaphragm depending upon the regional preferences. Steel deck diaphragms are most popular in Canada, Mexico, as well as the Eastern, Central and Southern United States. In this system, the corrugated steel decking is fastened to supporting steel joists (open-web trusses) with welds or screws, and sometimes with an assortment of proprietary fasteners. The in-plane shear strength and stiffness of these diaphragms are a function of the steel deck gage, joist spacing, and fastener type and spacing [SDI, 2004]. Unlike composite steel decking topped with concrete, a popular floor and roof system in multistory buildings, untopped steel deck diaphragms are relatively flexible in-plane.

Diaphragms consisting of wood structural panels in a modular or “panelized” arrangement are very common in the Western and Southwestern United States, especially in high seismic regions. Plywood, or more often oriented strand board (OSB), is fastened with nails to wood framing to provide a structural diaphragm as well as a roofing substrate. More commonly encountered today, these wood structural panels are fastened to wood nailer plates that are factory installed on top of a steel joist and joist-girder roof support structure. The speed and cost-efficiency of combining the wood-based panelized diaphragm with a steel support structure make this “hybrid” system very popular in RWFD buildings in California, Oregon, Washington and Nevada. The in-plane shear strength and stiffness of these diaphragms are a function of the wood structural panel thickness and grade, as well as nail size and spacing schedule [Lawson, 2013]. Similar to steel deck systems, wood structural panel diaphragms are also relatively flexible and lightweight compared with the surrounding rigid walls.

Seismic Performance History

Historically, the seismic performance of RWFD buildings has been poor. These buildings typically suffer from the poor performance of the out-of-plane anchorage that attaches the heavy walls to the lightweight roof diaphragms. Observed damages and instrument records from the 1971 San Fernando, 1984 Morgan Hill, 1989 Loma Prieta and 1994 Northridge earthquakes have

continually led to improved building code provisions for wall anchorage [SEAONC, 2001]. Based on observations following the Northridge earthquake, the current wall anchorage provisions referenced by the 2012 International Building Code [ICC, 2012] are contained in ASCE/SEI 7-10 Section 12.11.2 [ASCE, 2010] and prescribe maximum expected design forces without allowing reliance on connection ductility [SEAOC, 1999]. These design force levels and detailing requirements for out-of-plane wall anchorage have remained largely unchanged since they were introduced into the 1997 Uniform Building Code (ICBO, 1997). Since that time, the current practice and force levels of anchoring heavy walls to the flexible diaphragms have not been tested by a strong earthquake event.

Earthquake damages to the in-plane rigid shear walls or the main flexible roof diaphragm have been rare, except for collateral damage from the out-of-plane wall anchorage issues. The perimeter shear walls often consist of largely solid wall portions with relatively few penetrations, resulting in excessive lateral strength. This inherent overstrength of the in-plane shear walls combined with the new wall anchorage and collector design forces factored up to maximum expected levels is now expected to transfer the inelastic building behavior into the diaphragm, making this the more critical element in the SFRS. Unfortunately, because diaphragm performance is primarily attributed to the performance of the fasteners installed from above, damage is often hidden without invasive access through the roofing assembly following a significant earthquake. It is important to consider that the out-of-plane detachment of the heavy walls from the diaphragm in the past may have been protecting the diaphragm from experiencing the shear forces which could have led to global failure.

Existing Seismic Design Provisions

While the out-of-plane wall anchorage provisions have dramatically evolved after each damaging earthquake, code complying design methods have remained fairly consistent. For RWFD buildings, the past and current practices are to engineer the SFRS using the ELF procedure of ASCE/SEI 7-10 Section 12.8. This procedure assumes the predominant structural response is closely associated with the vertical elements of the SRFS. Under the ELF procedure, seismic forces are a function of evaluating lumped masses of various story levels supported on flexible elements, which represent the lateral stiffness of shear walls or frames traditionally defining the SFRS. The structural system’s period, which is based on the structure’s height, is the key to determine the code-based seismic forces.

Short Comings of Existing Seismic Provisions

The simplistic model assumed by the ELF procedure fails to capture the actual behavior of RWFD buildings. The ELF procedure assumes that the seismic response consists primarily of deforming vertical elements and that the horizontal diaphragm is rigid, i.e. deformation of the diaphragm is not considered. However, for most RWFD structures the primary seismic response is governed by the deformation of the horizontal flexible diaphragm instead of the rigid vertical walls. A more accurate structural model would need to capture the flexible diaphragm dominating the response.

The ELF procedure also inappropriately assumes that the primary inelastic response is in the vertical wall or frame system, instead of the roof diaphragm. The seismic response modification factor, R, and the overstrength factor Ω in ASCE/SEI 7-10 Table 12.2-1 are used to estimate the strength demands and capacities on systems that are designed using linear methods while responding in the nonlinear range. Currently, the selection of R and Ω for design is solely based on the performance characteristics of the SFRS’s vertical elements. In reality, the inelastic response is likely to be in the horizontal diaphragm and the current ELF procedure fails to characterize this diaphragm property adequately. Currently, wood and steel flexible diaphragms and rigid concrete diaphragms are all designed as if they have the same seismic response.

Because RWFD buildings typically have excessive strength in the shear walls as compared with the diaphragm, it can be unrealistic to expect (or require) the failure mode to be in the walls instead of the diaphragm; despite the fact that the response modification factor R is selected based on that assumption. Past failures have typically included out-of-plane wall detachments. However, as that failure mode becomes more under control, it is expected that diaphragm damage will be the next dominant form of inelastic behavior, which cannot be captured by the current ELF procedure.

Development of a New Seismic Design Procedure

With seismic response dominated by the horizontal diaphragm in RWFD structures, it is more realistic that design forces are developed around the diaphragm's behavior. Furthermore, because the inelastic behavior in a RWFD building is typically located in the diaphragm, it is more realistic that the seismic system capacity is developed around the diaphragm's overstrength and ductility instead of the vertical elements of the SFRS. A new approach based on the diaphragm's response would be more realistic and has the potential to better evaluate a structure's margin against collapse.

A more accurate method of establishing seismic design loads for buildings dominated by diaphragm response is to consider the diaphragm's period relative to the design spectral acceleration. As an example, ASCE 41-06 [ASCE, 2006] provides the *Linear Static Procedure* to rehabilitate existing one-story RWFD buildings by estimating the diaphragm-dominated building period and then establishing a pseudo-lateral force on the system. A number of other sources have proposed other methods of estimating flexible diaphragm periods and their corresponding pseudo-lateral force [Freeman et. al., 2002; PEER, 2004; SEAONC, 2001]. With a more accurate period of the dominating response, the force-based procedures of ASCE/SEI 7 can be used more appropriately. Furthermore, the use of an ELF approach with a response modification factor R and overstrength factor Ω related to the diaphragm construction and detailing instead of the vertical elements of the SFRS is expected to produce more rational results.

Both the inelastically acting horizontal diaphragm and the SRFS's vertical elements need a unified design methodology. The fact that RWFD buildings have a flexible upper portion supported in series with a rigid lower portion make them ideally suited to be designed with a two-staged ELF procedure similar to what already exists within ASCE/SEI 7-10 Section 12.2.3.2. This approach often used for podium type structures allows the two portions to be treated independently or together as appropriate. As the seismic forces are handed off from one portion to another, they are adjusted either up or down to reflect the next portion's expected seismic performance influenced by its period and stipulated R-factor. For simplicity, it is reasonable to develop a methodology for RWFD structures that can fit within the existing code framework already familiar to practitioners, yet providing a more rational approach than currently exists.

Validating a New Design Procedure

Historically, validation of seismic design methodologies was simply based on field reconnaissance following major earthquakes. While learning from earthquakes is invaluable when validating current design practices, a new proposed design approach needs a more systematic form of validation. The methodology contained in FEMA P-695 [FEMA, 2009] is intended to provide a means to evaluate a SFRS proposed for adoption into building codes, but can also be used to evaluate proposed design methodologies.

The FEMA P-695 methodology is used to reliably quantify building system performance and provide guidance in the selection of appropriate design criteria when ASCE/SEI 7 linear design methods are applied. The primary objectives of FEMA P-695 are to obtain an acceptably low probability of collapse of the SFRS under maximum considered earthquake (MCE) ground motions, and to provide a uniform protection against collapse across various structural systems. An appropriate P-695 evaluation must develop a representative nonlinear model that includes both detailed design information of the system as well as comprehensive test data on the post-yield performance of system components and subassemblies.

A proposed structural system, or as in this case a proposed design methodology of an existing system, is evaluated through the use of collapse fragility curves with collapse margin ratios, defined as the median seismic intensity causing a collapse probability of 50% divided by the corresponding MCE seismic intensity. Furthermore, uncertainties judged to be within the evaluation process are considered in the FEMA P-695 procedure. Incremental Dynamic Analyses (IDA) [Vamvatsikos & Cornell, 2002] on a representative ensemble of nonlinear numerical building models (or archetypes) spanning the design space are conducted to build the fragility curves using a pre-determined ensemble of earthquake ground motions. The number of archetypes selected is based on constructing an appropriate representation of the typical

RWFD structure including the range of variation reasonably expected and likely to affect performance.

Using professional engineers and researchers with expertise in the design of RWFD structures across North America, a list of significant parameters was established for this study. Two of the most significant parameters have already been discussed: steel deck diaphragms and structural wood panel diaphragms. Within the steel deck parameter, several varieties of fasteners are commonly used, each with different nonlinear properties. Depending upon the geographic region, welds, screws, pins, button punches, and various proprietary seam attachments are utilized to varying degrees. Other important parameters are diaphragm size and aspect ratio. Diaphragms with a horizontal clear span of up to 400-ft and aspect ratios up to 2:1 were considered. In practice, buildings larger than this located in areas subjected to higher seismicity generally have an interior shear element introduced creating several smaller adjacent diaphragms to keep the diaphragm shears manageable.

Combining the different parameters into various performance groups created the basis for a number of archetypes to be evaluated for their collapse capacity under current code and potential future design methodologies. Because the RWFD building stock is common across North America, ground motions associated with both Seismic Design Categories (SDC) D_{max} and C_{max} were evaluated. It was judged by the authors that ground motions for SDC A and B regions of North America were of minor significance to the study because these engineered designs are usually governed by wind loadings.

Eleven archetype performance groups were identified, as illustrated in Table 1, then engineered in conformance with the 2012 IBC, ASCE/SEI 7-10 and standard industry practices. Wood and steel deck diaphragms were evaluated. Nail fasteners were judged appropriate for the wood structural panel diaphragms. Recognizing that steel deck diaphragm performance will be different depending upon the deck-to-framing connectors as well as the side-lap seam connectors, several performance sub-groups were studied involving whether welds, pins, screws, punches, or combinations of them. The engineered designs for the various performance groups were numerically modeled for P-695 evaluation.

Numerical Model Development

Accurately modeling large flexible diaphragms is a very complex and numerically demanding process. With the large numbers of fasteners interacting with the decking, each at a different inelastic state, the tracking of individual fastener performance and its impact on collapse performance is a monumental task. Adding to the burden is the suite of time-histories being evaluated incrementally towards system failure for each archetype performance group. Conducting a P-695 validation study

Performance Group	1	2	3	4	5	6	7	8	9	10	11
Diaphragm Construction[1]	Wood Structural Panel				Steel Deck						
Seismic Design Load Level[2]	SDC D_{max}		SDC C_{max}		SDC D_{max}				SDC C_{max}		
Building Size[3]	Large	Small	Large	Small	Large		Small		Large		Small
Connectors[4]	Nail	Nail	Nail	Nail	Weld/BP	PAF's & Screws/ Screws	Weld/BP	PAF's & Screws/ Screws	PAF's & Screws/ Screws	Weld/Weld	PAF's & Screws/ Screws

Table 1. Archetype Performance Groups

1. B-type steel deck and oriented strand board wood structural panels considered. Wood framed and "hybrid" framed roof structures are expected to have similar seismic response and thus share performance groups.
2. Seismic design load levels are in conjunction with ASCE/SEI 7-10 Seismic Design Categories.
3. Building sizes vary from 100-ft to 400-ft diaphragm horizontal clear spans. Large Buildings: 400ft x 200ft (2:1), 200ft x 400ft (1:2), 400ft x 400 ft (1:1); Small Buildings: 200ft x 100ft (2:1), 100ft x 200ft (1:2), 100ft x 100 ft (1:1)
4. Steel deck is fastened with a combination of "frame/side-lap" connectors indicated. Button punch (BP) side-laps are replaced with welded side-laps in diaphragm zones where shear demands are high. PAF's indicate the use of power actuated fasteners.

using a traditional finite element model on a full structure is impractical considering the huge amount of computational time it would require.

To streamline the process, a two dimensional numerical framework for nonlinear dynamic analyses was developed using a three step sub-structuring approach [Koliou, 2014, Koliou et. al., 2014]. This approach begins with assembling the hysteretic responses of the diaphragm connectors, then building an inelastic model of the diaphragm discretized into horizontal segments with the connectors, and lastly assembling the various diaphragm segments into an overall simplified building model complete with in-plane and out-of-plane inelastic wall responses, and second order (P-Δ) effects. This model was found to be in very good correlation with experimental and analytical studies available in the literature, capturing the nonlinear response of RWFD buildings. This numerical framework is simplified enough for researchers to efficiently conduct a large number of nonlinear time-history analyses in a timely manner, and has the potential for practitioners to better investigate new and existing RWFD buildings.

Building Period Determination

Free-vibration eigenvalue analyses were conducted on the simplified RWFD building model in the RUAUMOKO2D platform [Carr, 2007] to determine the natural periods of the archetypes. While multiple modes of vibration were identified, the 1st mode of vibration in each direction was determined to clearly dominate nearly the entire response.

The results from the simplified building model used in this study confirmed that RWFD buildings have fundamental periods that are significantly longer than currently estimated by the ASCE/SEI 7-10 approach. The large flexibility of the horizontal diaphragm dominates the overall building response. However, most often ASCE/SEI 7-10 Equation 12.8-7 is used by practitioners to compute period, and this approach fails to consider diaphragm response. Illustrated below is the use of Equation 12.8-7 for a single-story shear wall building with 30-foot roof elevation matching this study's archetypes.

$$T_a = C_t h_n^x = 0.02(30)^{0.75} = 0.26\, sec$$

where C_t and x are period parameters per AISC 7-10 Table 12.8-2 and h_n is the height to the roof structure.

The fundamental elastic periods from the eigenvalue analyses for this study's archetypes are shown in Table 2, and it is observed that the 0.26-second code approximation most often underestimates the buildings' fundamental period by a significant amount.

Wood diaphragm span length	Period for high-seismic archetypes	Period for moderate-seismic archetypes
400 ft	0.85 to 0.87 sec	0.90 to 0.92 sec
200 ft	0.49 to 0.54 sec	0.55 to 0.58 sec
100 ft	0.36 to 0.38 sec	0.43 to 0.45 sec
Steel diaphragm span length		
400 ft	0.49 to 0.56 sec	0.61 to 0.73 sec
200 ft	0.35 to 0.42 sec	0.51 to 0.59 sec
100 ft	0.21 to 0.26 sec	0.28 to 0.33 sec

Table 2. Summary of elastic periods for archetypes

Because a structure's seismic force experienced is directly related to its period of vibration, it was important in this study to produce simple fundamental period formulas that are more accurate than the current ASCE/SEI 7-10 approximation approach.

Using the elastic periods computed from the eigenvalue analyses, semi-empirical formulas were developed for the fundamental period of each of the high-seismic wood diaphragm archetypes and the high-seismic steel deck diaphragm archetypes.

The following formulas are proposed for computing the fundamental period of the wood and steel deck diaphragm buildings with concrete or masonry shear walls.

$$T_{wood} = \frac{0.0019}{\sqrt{C_w}} h_n + 0.002L \qquad \text{(Equation 1)}$$

$$T_{steel} = \frac{0.0019}{\sqrt{C_w}} h_n + 0.001L \qquad \text{(Equation 2)}$$

where the first term $\frac{0.0019}{\sqrt{C_w}} h_n$ is the ASCE/SEI 7-10 approximate fundamental period Equation 12.8-9 permitted for masonry or concrete shear wall structures to capture the period contribution of the vertical walls. ASCE/SEI 7-10, Equation 12.8- 10 defines C_w:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

where

A_B = area of base of structure, ft^2
A_i = web area of shear wall i in ft^2
D_i = length of shear wall i in ft
h_i = height of shear wall i in ft
x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

In Equation 1 and Equation 2, the 0.002L and 0.001L terms for wood and steel deck diaphragms respectfully capture the diaphragm's fundamental period contribution where L is the diaphragm span in feet between vertical elements of the SFRS. The linear relationship of period associated with the diaphragm span indicates that these structural elements undergo primarily shear deformation as opposed to flexural deformation

The proposed period equations are plotted along with the periods determined from the simplified building model analyses for the wood diaphragm archetypes in Figure 1 and the steel diaphragm archetypes in Figure 2. Although the periods are derived from analysis of only single-span diaphragms, the deformations were shear dominated, and thus the period equations are considered to still be valid for multi-span diaphragms in buildings with interior shear elements.

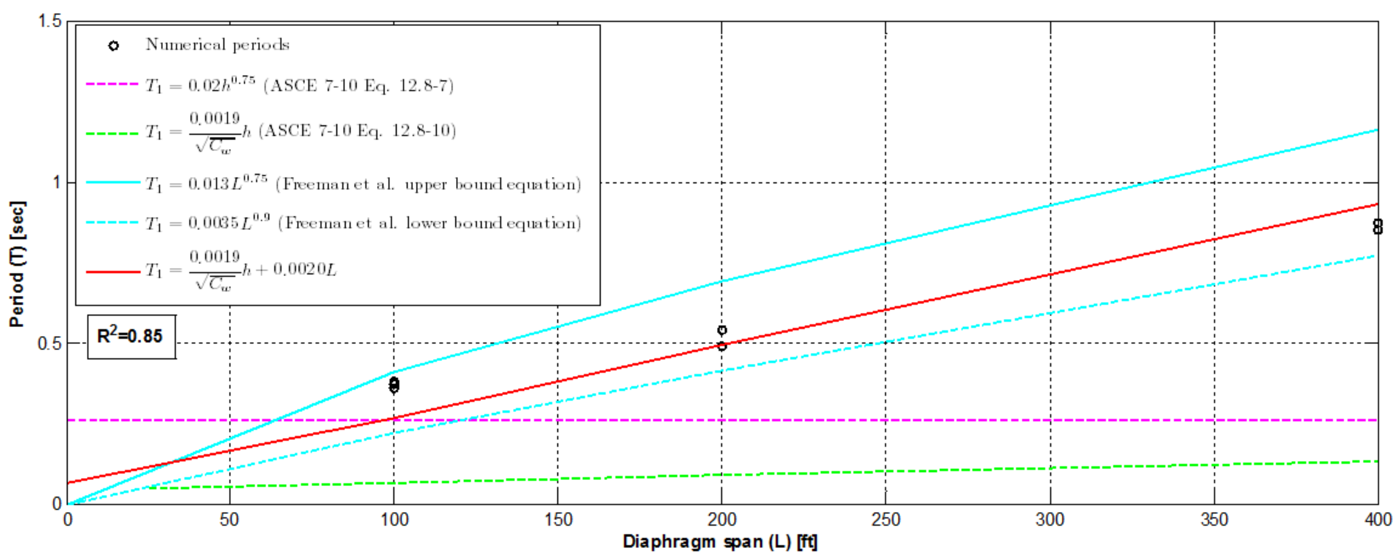

Figure 1. Comparison of fundamental periods from analyses of wood panel archetypes to those predicted by the proposed formula (Eq. 1), ASCE 7-10 equations, and those proposed by Freeman et. al [2002].

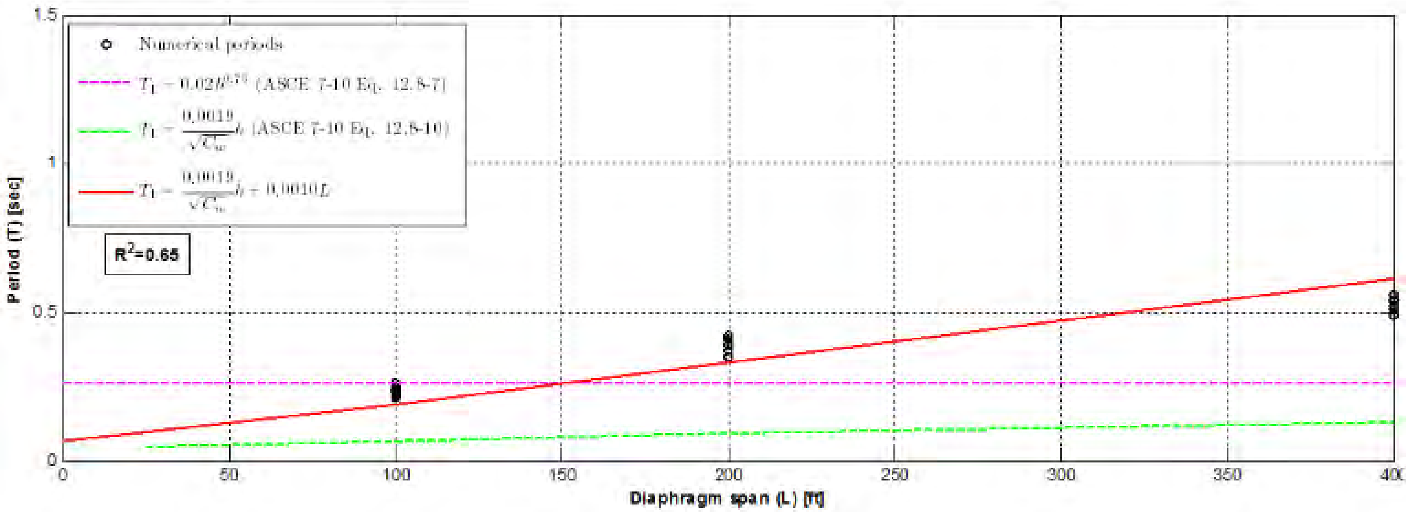

Figure 2. Comparison of fundamental periods from analyses of steel deck archetypes to those predicted by the proposed formula (Eq. 2) and ASCE 7-10 equations.

Results of the FEMA P-695 Study

After completing 44,000 non-linear time history dynamic analyses across the ASCE/SEI 7-10 design archetype performance groups, some interesting results have been observed. For the archetypes studied, the results indicate that inelastic seismic response of the modeled RWFD buildings is clearly within the diaphragm instead of the in-plane walls, but more importantly the inelastic behavior is concentrated adjacent to the diaphragm boundaries near the parallel shear walls. Even though the diaphragm design capacity was intentionally stepped down to efficiently follow the shear demand reduction towards the diaphragm interior as is often done in practice, the analysis results indicate that the inelastic response of the diaphragm is still concentrated adjacent to its shear wall boundary. The ductility distribution of the conventional ASCE/SEI 7-10 diaphragm design in Figure 3a illustrates this behavior.

Because the inelastic behavior is not well distributed across the diaphragm, the localized inelastic response near the boundary is quickly overwhelmed by the limited ability of the connectors to dissipate large amounts of energy, and leads to global building failure. This phenomenon was observed in both the steel and wood deck diaphragm models, and the FEMA P-695 collapse margin ratio evaluations were often problematic in meeting the proposed target values [Koliou, 2014].

Encouraging Distributed Inelastic Behavior

A potential design methodology to improve performance is to intentionally weaken the diaphragm's interior areas below current code-based force demands to better distribute the inelastic behavior. The results of the numerical modeling of this weakening approach are promising. This approach of relative weakening assists in protecting the perimeter boundary areas from excessive inelastic demand and increases the margin against collapse. A comparison of the inelastic distribution between a current ASCE/SEI 7-10 code-based diaphragm design and a weakened diaphragm design is shown in Figures 3a and 3b. The inelastic response of the diaphragm is better distributed along the length of the diaphragm for the weakened diaphragm design. The resulting collapse margin ratios were found to improve significantly with this intentional weakening.

One problematic issue with this approach is the necessary weakening of the diaphragm would be difficult to enforce in practice. Often in smaller buildings with low shear demands a minimum fastener size at maximum spacing is provided uniformly across the entire diaphragm, thus not accessible to intentional weakening. In these situations, an option to consider is providing overstrength to the boundary areas to mitigate the isolated inelastic behavior here. In this approach, the diaphragm boundary areas would be designed for a lower R-value compared with the

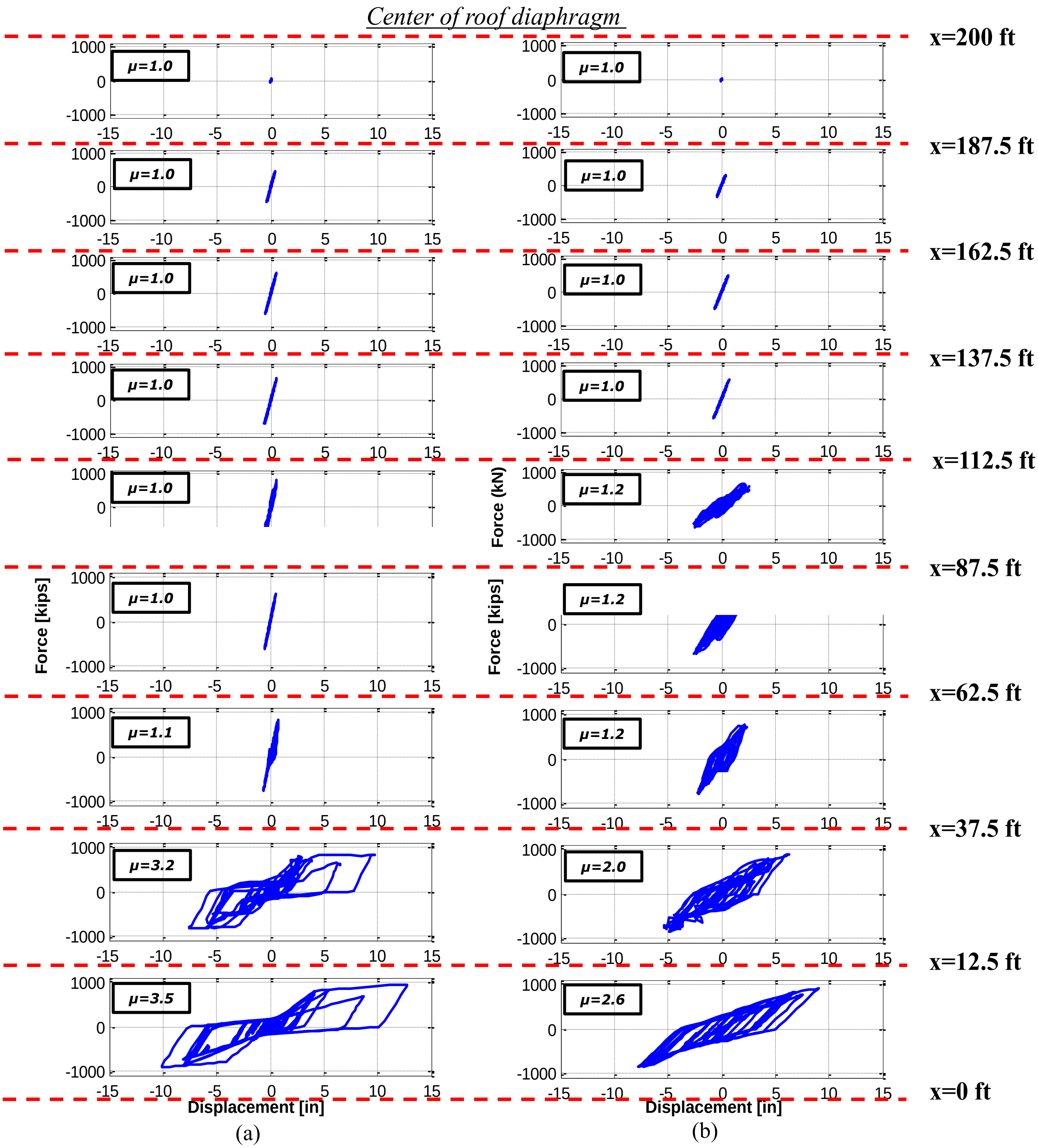

Figure 3. Hysteretic diaphragm response case study for Friuli, Italy (1976) Record at MCE intensity – excitation parallel to diaphragm short direction: (a) conventional (ASCE 7-10) diaphragm design and (b) weakened diaphragm interior design [Koliou, 2014].

interior areas, or alternatively the boundary areas are designed with an applied overstrength factor. In other words, instead of relative weakening of the interior areas, a strengthening of the boundary areas is pursued.

Another series of 47,520 non-linear time history dynamic analyses were conducted on performance groups using new archetypes following this new design approach, which consisted of designing for the spectral acceleration associated with the estimated period incorporating the diaphragm (Equation 1 or 2 of this paper), weakening the interior diaphragm portions with a response modification factor R_{diaph} = 4.5 (instead of 4), and strengthening the diaphragm boundaries (a width of 10% of the diaphragm span at each end) with 50% more capacity. With this new proposed design approach, the P-695 evaluation showed a significant improvement in the collapse capacity. Table 3 illustrates the significant increase in the adjusted collapse margin ratio (ACMR) of five steel deck performance groups under existing code-based design practices compared with this new proposed design approach. As defined by FEMA P-695, the ACMR is the metric used to judge the acceptability of a SFRS, or in this case a design methodology.

Proposed Design Procedure

Low-rise concrete or masonry buildings with lightweight flexible wood or steel diaphragms are a very common building stock in North America; and are common projects for a typical engineering office to encounter. Use of an ELF procedure is the most straightforward, simple approach to seismic design of these buildings. A primary objective of this project was to find a more rational design methodology that captures the actual behavior and performance of the RWFD building type, yet remain relatively simple to execute.

Following the philosophy of ASCE/SEI 7-10's current ELF procedure, the structure's base shear is a function of the building's fundamental period T and the response modification factor R. As previously mentioned, a semi-empirical period equation was developed for this building type based on a simplified numerical model, and an appropriate response modification factor was estimated based on a FEMA P-695 investigation. However, it is important to recognize that to this point, the T and R are more representative of the diaphragm portion of the structure and not the vertical walls.

Conceptually, a RWFD building is simply a flexible diaphragm structure supported by a more rigid wall structure. In regards to the seismic load path, the flexible diaphragm is in series with the rigid walls; and this is similar to today's common podium buildings with a flexible SFRS at the upper floors supported on a rigid SFRS at the lower floors. A proposed rational approach is to take the current two-stage analysis procedure of ASCE/SEI 7-10 Section 12.2.3.2 that is in common practice today on podium structures and adapt it to RWFD buildings. This approach supports the use of two different

Performance group ID	Design Configuration						Collapse margin parameters		Pass/Fail
	Design (E:existing N:new)	Building Size	Diaphragm Construction	Seismic SDC	Framing connectors	Sidelap connectors	ACMR	Acceptable ACMR	
PG-5	E	Large	Steel (WR Deck)	Dmax	Welds	Welds & button punches	1.71	2.30	Fail
	N	Large	Steel (WR Deck)	Dmax			2.73		Pass
PG-6	E	Large	Steel (WR Deck)	Dmax	PAF/screws	Screws	1.87	2.30	Fail
	N	Large	Steel (WR Deck)	Dmax			2.75		Pass
PG-7	E	Small	Steel (WR Deck)	Dmax	Welds	Button punches	2.23	2.30	Fail
	N	Small	Steel (WR Deck)	Dmax			2.82		Pass
PG-8	E	Small	Steel (WR Deck)	Dmax	PAF/screws	Screws	1.92	2.30	Fail
	N	Small	Steel (WR Deck)	Dmax			2.40		Pass

Table 3. Comparison of P-695 Results of the Current Code-based Design Approach with a New Proposed Design Approach for Select Performance Groups [Koliou, 2014]

periods T and two different response modification factors R in series within the same building to determine the applied seismic design forces. Furthermore, two separate overstrength factors Ω may be utilized when appropriate.

The first stage of the analysis involves the diaphragm. The diaphragm's seismic force is computed using the building's fundamental period based on Equation 1 or 2 presented in this paper, in conjunction with the proposed response modification factor R = 4.5. In order to encourage greater distribution of the inelastic behavior, the boundary regions on each end of the diaphragm span (10% of the span) are designed for 50% higher seismic forces (See Figure 4).

The second stage of the analysis involves the heavy in-plane shear walls, whose period does not involve the diaphragm component, and is thus comparatively small. Furthermore, the response modification factor will be as currently specified within ASCE/SEI 7-10 for that shear wall system.

Investigation into Wall Anchorage Forces

Using the numerical model framework developed for this project, a separate side study was also conducted to investigate the magnitude of the wall anchorage forces for various seismic intensities defined as the median (across the earthquake ground motions ensemble) spectral accelerations at the fundamental period of the building. Data was collected from the model indicating the attachment force between the out-of-plane wall panels and the diaphragm. The walls as modeled were simply supported at the roof and bottom and were permitted to crack and yield out-of-plane, but another study was also conducted to observe the effects if the simply supported walls remained rigid. Results for a large rectangular building incorporating a steel roof diaphragm are shown in Figure 5 and clearly demonstrate the softening of the diaphragm caused by the inelastic response at high spectral accelerations. Higher wall anchorage forces at the short ends of the rectangular building were observed when excited in the long direction. This phenomenon is likely caused by the diaphragm's inherent longitudinal overstrength due to the transverse loaded shear design.

The current ASCE/SEI 7-10 wall anchorage design force is also shown in Figure 5 as the straight diagonal line. The intent of this force level is to design the anchorage for maximum expected force levels without reliance on anchorage system ductility [SEAOC, 1999]. These ASCE/SEI 7-10 anchorage force levels are appropriate in the long direction and conservative in the short direction of excitation based on this large building archetype example.

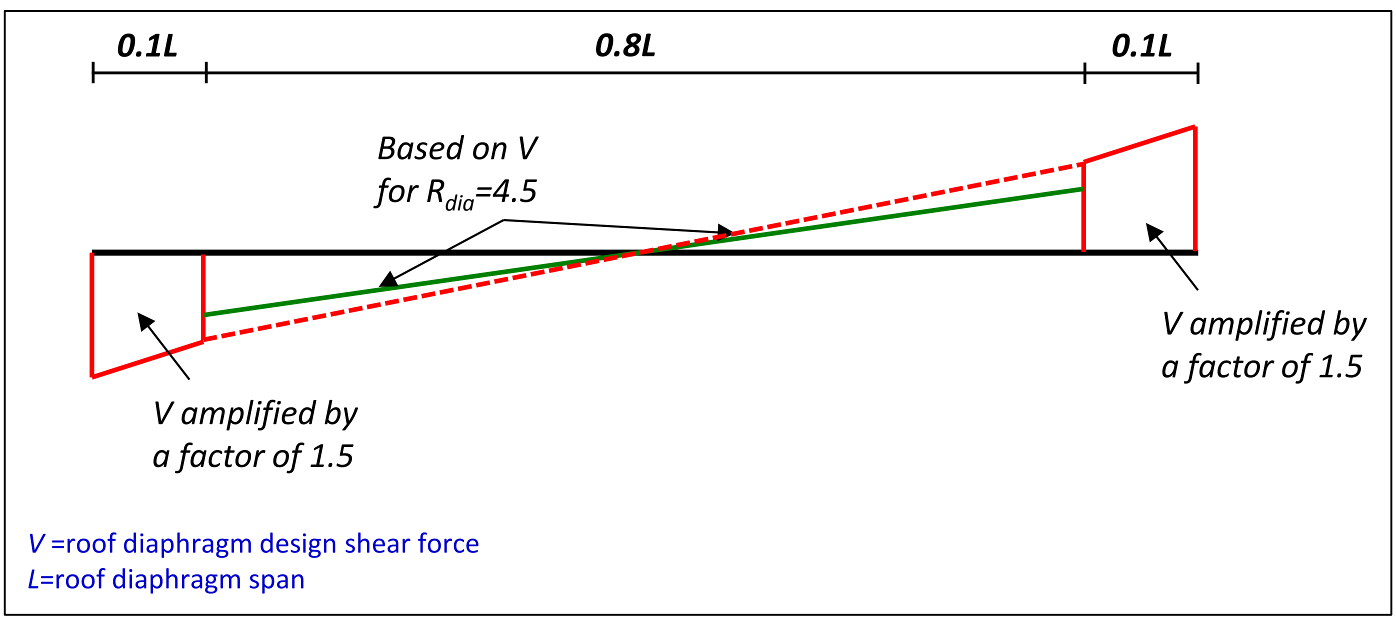

Figure 4. The diaphragm's shear diagram for the proposed design methodology.

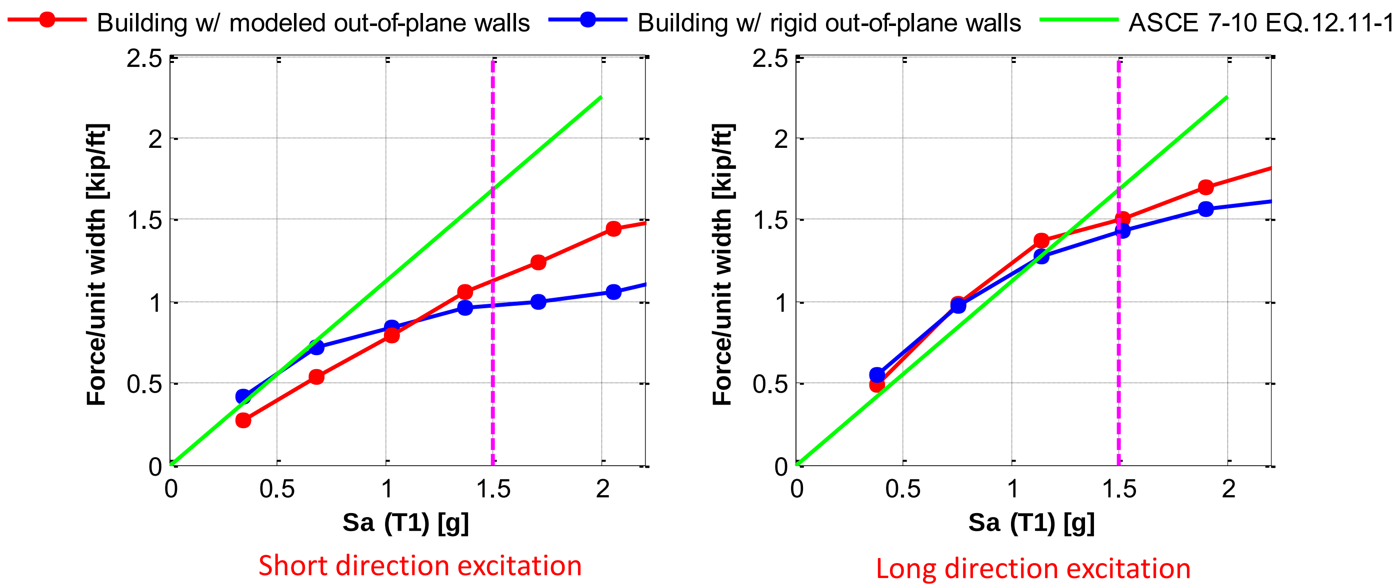

Figure 5. Wall anchorage forces at diaphragm mid-span associated with Performance Group 1 [Table 1]. Walls are 9¼-inch thick concrete, 30-feet high to roof with 3-foot parapet. The building dimensions are 200-ft (short direction) by 400-ft (long direction).

An important observation is that the wall anchorage forces increase when this building is excited in the long direction compared to the short direction. Alternatively stated, the wall anchorage forces are higher for diaphragms that have shorter spans compared with longer spans. As illustrated in Figure 5 for a rectangular building archetype, for median $S_a = 1.5$g the wall anchorage force is approximately 1.15 kips/foot for a 400-foot wide diaphragm span compared with 1.5 kips/foot for a 200-foot wide diaphragm span, indicating that shorter diaphragm spans have higher wall anchorage forces. This is contradictory to the current ASCE/SEI 7-10 wall anchorage provisions in Section 12.11.2.1

ASCE/SEI 7-10 requires that the anchorage of walls and transfer of forces into the diaphragm be designed for an out-of-plane wall force in accordance with the following equations:

$$\boldsymbol{F_p = 0.4S_{DS}k_aI_eW_p \geq 0.2k_aI_eW_p} \quad \text{(EQ 12.11-1)}$$

$$\boldsymbol{k_a = 1.0 + \frac{L_f}{100} \leq 2.0} \quad \text{(EQ 12.11-2)}$$

In these formulas, k_a is an amplification factor for diaphragm flexibility, and L_f is the span, in feet, of a flexible diaphragm between resisting walls or rigid frames. In a broader sense, k_a and L_f account for diaphragm flexibility. Per ASCE/SEI 7-10, if a diaphragm is rigid, L_f equals 0 but if it is flexible, L_f equals the span length. For many buildings with rigid walls and flexible roof diaphragms, the diaphragm span between supporting walls or frames will be longer than 100 ft so k_a will often be equal to 2.0. For diaphragms in which k_a equals 2.0, the acceleration parameter used to compute the out-of-plane wall anchorage force is $0.8S_{DS}$, which is 80% of the maximum design spectral acceleration parameter. As already mentioned, the intent is that the wall anchorage force is resisted elastically for a force level computed using the maximum design spectral acceleration. The 0.8 factor is included to recognize that some connection and member overstrength may be relied upon to resist the top of wall anchorage force [SEAOC, 1999].

ASCE/SEI 7-10 Equations 12.11-1 and 12.11-2 allow for top of wall anchorage forces to be less at smaller diaphragm spans (under 100 feet) than those with larger spans (over 100 feet). This reduction is inconsistent with the expectation that a shorter span diaphragm has a shorter period and higher accelerations. But more importantly in rectangular buildings, often there is significant overstrength when excited in the long direction that can result in greater stiffness and thus greater forces

developing at the top of wall support. This condition is visible when comparing the two graphs in Figure 5.

Additional research is currently underway using the dedicated numerical framework described in this paper to investigate a number of wall anchorage conditions and will be the subject of a future publication.

Conclusions

Despite how common RWFD structures are, their seismic behavior is not well represented within the current building code provisions. Past earthquake damages have led to improved code provisions for wall anchorage; but it is feared that the inelastic response will now transfer into the main diaphragm where there may be limited ability to accommodate the necessary inelastic demand. A two dimensional numerical framework and methodology validation was developed to efficiently evaluate the unique seismic performance of RWFD buildings and evaluate potential design methodologies to mitigate poor collapse margins driven by the seismic response of the flexible roof diaphragms.

Adopting a two-stage analysis procedure where the diaphragm's own response and overstrength capacity is recognized separately, and inelastic behavior in the diaphragm is encouraged to spread out, are rational and promising ideas to improve RWFD building performance. Furthermore, potential deficiencies in obtaining wall anchorage design forces were observed in the current methodology, and it is recommended that the current reduction in forces for diaphragms less than 100-ft wide be evaluated further and reconsidered.

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