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AN ALTERNATIVE PROCEDURE FOR THE SEISMIC DESIGN OF BUILDINGS WITH RIGID WALLS / FLEXIBLE DIAPHRAGMS: FEMA P-1026

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Abstract

Evidence indicates that the dynamic behavior of Rigid Wall - Flexible Diaphragm (RWFD) buildings is dominated by the flexible roof diaphragm's response instead of the walls' response, and this is a significant departure from the underlying assumptions of the widely used equivalent lateral static force methods in current building codes. RWFD buildings are common in the Americas and other parts of the world, and incorporate rigid in-plane concrete or masonry walls and flexible in-plane wood or steel roof diaphragms. In an effort to improve the seismic performance of this common building type, the National Institute of Building Sciences (NIBS) directed this study to develop a new design methodology that is simpler and more rational than current practice. With the use of a nonlinear numerical computer modeling framework developed specifically for this type of building, an investigation following procedures of the Federal Emergency Management Agency (FEMA) publication P695 was conducted across a wide range of RWFD archetype designs developed under the *2012 International Building Code* and the *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, for both moderate and high seismic exposures.

In this project, a new design procedure was developed that improves building performance by acknowledging the different vibrational periods and levels of available ductility in rigid walls and flexible roof diaphragms. Additionally, through encouraging distributed yielding across the diaphragm span, the performance of flexible wood roof diaphragms was found to improve their collapse resistance. This new procedure's methodology is proposed to be implemented into the design codes of the United States (US) as an alternative seismic design approach for this type of structure, and is included in Part 3 of the 2015 National Earthquake Hazard Reduction Program (NEHRP) *Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA P-1050-2). Additionally, the FEMA P1026 publication that resulted from this project provides background, illustrative commentary and design examples of this alternative procedure.

Keywords: Flexible roof diaphragms; FEMA P1026; two-stage analysis; dynamic behavior

1. Introduction

In March of 2015, the United States' Federal Emergency Management Agency (FEMA) published document P1026, *Seismic Design of Rigid Wall - Flexible Diaphragm Buildings: An Alternate Procedure* [1], which demonstrates an alternative approach to seismically designing a special class of buildings characterized by its stiff vertical seismic force resisting system (SFRS) supporting a horizontal flexible roof diaphragm. These Rigid Wall – Flexible Diaphragm (RWFD) buildings perform differently than most other classes of structural building types in earthquakes. One significant difference is that the RWFD buildings' dynamic behavior is dominated by its roof diaphragm instead of its walls, a concept that is not easily accommodated with current building code provisions for new construction.

The primary purpose of FEMA P1026 is to present an alternate seismic design procedure for RWFD buildings that better captures actual building behavior, and in particular accounts for yielding of the diaphragm being predominant, rather than yielding in the vertical elements of the SFRS. The intent of FEMA P1026 is to convey design principles to practicing engineers that they may use in their practice to improve the seismic performance of RWFD buildings.

The development of this alternative procedure began with a FEMA P695 [2] assessment of this class of buildings' seismic performance when designed under current building codes of the United States (US), then identifying the weaknesses in the buildings' structural design as well as weaknesses in the building code's own design methodology. With this information, the authors explored a number of different design approaches with the intent of finding a simple procedure that can be easily incorporated into existing seismic design practices. This paper presents an alternative design procedure for improved seismic performance of single-story RWFD buildings that was developed and evaluated using the FEMA P695 methodology to confirm that it would meet the intent of the current US building code seismic provisions. This alternative procedure is targeted for RWFD buildings with flexible roof diaphragms consisting of wood structural panels; however, many of the concepts are also thought to be applicable to RWFD buildings with untopped steel deck diaphragms.

2. The RWFD Building

Buildings with a perimeter using a stiff concrete or masonry wall system supporting a lightweight steel or wood roof diaphragm are commonplace in many parts of the world, especially in the US. The relative stiffness of the vertical SFRS compared to the horizontal diaphragm is what primarily characterizes a RWFD building, and contributes to its unique dynamic behavior.

2.1 Description of RWFD buildings

RWFD buildings are inexpensive to build and are the primary choice of design for warehouse structures, and other buildings devoted to light industrial uses. Depending upon regional preference, wall systems are typically constructed of precast tilt-up concrete or concrete block masonry. The limited window and door penetrations around their perimeters contribute to their inherent in-plane wall stiffness and significant shear wall overstrength. In contrast, the lightweight roof systems support very little live loads and are flexible in-plane. Roof diaphragms in these buildings consist either of a wood structural panel diaphragm or a steel deck diaphragm depending upon the regional preference. FEMA P1026 is focused primarily on wood diaphragms, which are very common in the western US, especially in high seismic regions.

Wood structural panel diaphragms consist of either plywood or oriented strand board (OSB), and are fastened with nails to wood framing to create the 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 open-web steel joist and joist-girder roof support structure. The speed and cost-efficiency of combining the wood-based diaphragm with a steel support structure make this "hybrid" system very popular in RWFD buildings in California, Oregon, Washington, Nevada and Arizona. The in-plane shear strength and stiffness of these diaphragms are primarily a function of the nail size and spacing, as well as the thickness and quality of the wood structural panel. Wood structural panel diaphragms are relatively flexible, weak, and lightweight compared with the surrounding stiff, strong, and heavy walls.

The use of concrete or masonry walls in conjunction with the economical long-span roof system make RWFD buildings a favorite by developers and owners for providing the most cost effective approach to enclosing large open floor spaces while providing durable and secure perimeters.

2.2 The seismic performance of RWFD buildings

Over the last half century, RWFD buildings have often performed poorly in the US during strong earthquakes with damage primarily occuring due to inadequate out-of-plane wall anchorage capacity at the roof diaphragm [3]. Earthquake damages to the in-plane rigid shear walls or the 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 in-plane lateral strength. This inherent overstrength of the shear walls is expected to cause the inelastic building behavior to move into the diaphragm. While most past damages have been associated with out-of-plane wall to roof detachment, it has been theorized that the accidental loss of the out-of-plane walls has been protecting the diaphragm from experiencing the full potential shear forces, which otherwise could have led to roof diaphragm failure [4]. The wall anchorage design force levels have increased dramatically in the recent editions of the US building codes, and this combined with the overstrength of the in-plane walls potentially will make diaphragm yielding and global response more critical for RWFD buildings in future earthquakes. Resolving potential issues with the global response of the roof diaphragm is the primary issue addressed by FEMA P1026.

3. Evaluation of RWFD Buildings designed under Current Code Procedures

Currently, the model building code in the US is the 2012 *International Building Code* (IBC) [5], and it makes reference to *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10) [6] for the development of seismic design forces and procedures. In order to properly assess the adequacy of current US code design procedures for RWFD buildings, a comprehensive study was undertaken for both wood and steel deck diaphragms within these buildings. Using a special numerical model and a large set of building archetypes, the collapse risk was evaluated under the FEMA P695 methodology because it is an accepted procedure for evaluating the adequacy of a SFRS for inclusion into ASCE 7.

3.1 Creation of the building archetypes

To represent the typical design space for RWFD buildings, 50 building archetypes of Risk Category II buildings designed under both current US practice and the FEMA P1026 alternative procedure were numerically modeled for FEMA P695 evaluation. Both high seismic and moderate seismic intensities associated with Seismic Design Categories D_{max} and C_{max} [6], respectfully, were considered in the design of six different building sizes with various rectangular aspect ratios (100-ft x 100-ft, 100-ft x 200-ft, 200-ft x 100-ft, 200-ft x 400-ft, 400-ft x 200-ft, 400-ft x 400-ft, where 1-ft = 0.305m). Reinforced precast (tilt-up) concrete wall panels 33-ft (10m) high and 25-ft (7.62m) wide act as shear walls around the perimeter. Both wood and steel deck diaphragms were investigated; however the focus of this paper is the 17 archetypes with a 15/32-inch (12mm) thick wood structural panel roof diaphragm consisting of a nailed, panelized construction [7] at an elevation of 30-ft (9.14m). Wall panels 9.25-inch (0.235m) thick for the high seismic archetypes and 7.25-inch (0.184m) thick for the moderate seismic archetypes were selected. This use of a wood panel roof diaphragm in conjunction with precast concrete tilt-up walls is the most popular RWFD building in the western US.

3.2 Development of a dedicated numerical model

Due to the necessary number of building archetypes needed and the number of expected nonlinear incremental dynamic analyses (IDAs) [8] associated with the FEMA P695 procedure, a very efficient numerical modeling procedure was developed without compromising its ability to respond to inelastic behavior and second order (P-Δ) stability effects. The numerical model's approach is briefly described as follows.

A three step numerical framework for the nonlinear dynamic response analyses, introduced by Koliou et.al. [9, 10], was used to conduct the collapse evaluation studies of the RWFD building archetypes designed with current US provisions as well as evaluating the proposed alternative design procedure for FEMA P1026.

The first step of the framework constructed a hysteretic response database of wood roof diaphragm connectors including common nails. An inelastic roof diaphragm analytical model was developed in MATLAB [11], in step 2 of the numerical framework, where each deck panel was modeled as a deep shear beam, and the hysteretic model developed in Step 1 was used to represent the inelastic cyclic response of each roof diaphragm connector. A constant cyclic force applied at the center of the roof was considered for the analysis on the diaphragm model to induce constant shear forces in the roof. The total in-plane flexible roof diaphragm displacement was computed as the sum of: (i) the elastic shear deformation of each individual panel, (ii) the inelastic deformations (slippage) of connectors and (iii) the elastic flexural deformations of the chord members. In the last step of the numerical framework, a two dimensional building model was generated in the general-purpose RUAUMOKO2D software [12], representing the three dimensional building without accounting for torsion. The building model considers the in-plane and out-of-plane vertical wall responses, the second order (P-Δ) effects and in-plane diaphragm springs developed in the second phase to account for the global hysteretic roof diaphragm response. The elastic flexure and shear response of the in-plane wall panels was modeled by a single horizontal elastic spring, while the out-of-plane walls were modeled as vertical beam elements simply supported at the top and bottom with four lumped masses and nonlinear springs along their height. A low value of initial stiffness Rayleigh damping equal to 2% of critical was selected as a representative value for RWFD buildings that include fewer nonstructural components than do conventional frame-and-wall buildings, while minimizing potential damping ratio overshoots that could occur as a result of the initial stiffness proportional damping formulation during yielding of the system.

Steps 2 and 3 of the numerical framework were validated with analytical and experimental studies available in other publications by the authors [9, 13].

3.3 Evaluation of current US practice

Utilizing 44,000 non-linear time history dynamic analyses, 50 RWFD building archetypes incorporating both wood and steel roof diaphragms of different geometry and connector configurations were evaluated for their probability of collapse given a maximum considered earthquake (MCE) event. The seismic responses were evaluated for their margins against collapse using the FEMA P695 methodology [10].

Using the numerical framework developed, IDAs reported median collapse intensity and ground motion spectral demand at MCE and were used to compute the FEMA P695 collapse margin ratio (CMR) of each building archetype, as well as the results of a pushover analyses (yield and ultimate diaphragm drift ratios) used to compute the period based ductility ratio of each archetype. The FEMA P695 adjusted collapse margin ratio (ACMR) for each archetype was determined from the CMR, period based ductility ratio and the spectral shape factor. For acceptable performance, each individual archetype and its performance group developed under FEMA P695 must exhibit satisfactory collapse performance as judged by the ACMR.

The results of the study indicate that current US code provisions and practice used to design new buildings that incorporate rigid walls and flexible roof diaphragms do not satisfy the collapse objectives of FEMA P695 under MCE ground motions [10]. This is due to the provisions being based on assumed yielding of the in-plane walls rather than of the diaphragm where the yielding actually occurs.

More specifically for 17 archetypes incorporating wood roof diaphragms, satisfactory probability of collapse was observed for large buildings when located either in moderate or high seismicity zones. However, small RWFD buildings with wood roof diaphragms only satisfied the 20% probability of collapse for individual archetypes and failed to meet the 10% probability of collapse criterion of FEMA P695 for the average of the performance group. This result may be attributed to the shorter periods of smaller building archetypes that attract higher seismic demands. Also for these small buildings, the period of the diaphragm is closer to the period of the in-plane walls, which causes additional amplification of the diaphragm displacement. Based on the results obtained, the current seismic design requirements for RWFD buildings with wood roof diaphragms would not meet the FEMA P695 acceptance criteria as a new SFRS system. Similar results were found during the study for building archetypes incorporating steel deck roof diaphragms [10].

4. Development of an Alternative Design Procedure for Wood Diaphragms

In order to mitigate the unacceptable risk of collapse found in the FEMA P695 study, the authors explored a number of different design approaches with the intent of finding a simple procedure that can be easily incorporated into existing seismic design practice. A new procedure was developed that addresses: (1) the differences in diaphragm period compared with the SFRS period, (2) the consideration for inelastic behavior in the diaphragm instead of the vertical SFRS, and (3) the encouragement of overall greater diaphragm ductility.

4.1 Development of an alternative period formula

An important aspect of predicting seismic forces in RWFD buildings is the computation of a realistic building period. Modern building codes encourage the use of simplified equations that estimate the building period as a function of building height, and this period is used to compute the seismic base shear force for design. In RWFD buildings, building period is usually dominated by the large and flexible diaphragm's behavior, and not the relatively short and stiff shear walls as the building code assumes, making this approach potentially inappropriate. As part of this study, an alternative period formula was developed for RWFD buildings.

Using the numerical framework developed for the dynamic analyses, it was found that the 0.26 second (for building height of 30-ft) period approximated by current US practice underestimates the wood diaphragm archetypes' building periods significantly. Computed periods from the numerical model ranged between 0.36s and 0.87s [10]. Moreover, the computed period of RWFD buildings was found to be proportional to the span of the roof diaphragm. Therefore, a semi-empirical period formula is proposed. This new formula combines mechanics based assumptions and extends the existing US code based period formula for vertical elements in order to provide a reasonably accurate estimate of the fundamental period of RWFD buildings, while being easily implementable in design situations. For RWFD buildings incorporating wood roof diaphragms and perimeter concrete or masonry shear walls as vertical elements of the SFRS, the fundamental period (T_1) taking into account diaphragm response can be approximated by the semi-empirical Eq. (1) provided in [10]:

$$T_1 = \frac{0.0019}{\sqrt{C_w}} h + 0.002L \qquad (1)$$

where, L is the roof diaphragm span in ft. (1-ft. = 0.3048m), h is the height of the in-plane shear walls in ft. (1-ft. = 0.3048m), and C_w is calculated according to Eq, (2) from ASCE 7-10:

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

where, A_B is the area of the base of the building in ft^2 ($1\text{-}ft^2 = 0.0929m^2$), A_i is the web area of the in-plane shear wall segment i in ft^2 ($1\text{-}ft^2 = 0.0929m^2$), D_i is the length of the in-plane shear wall segment i in ft. (1-ft. = 0.3048m), h_i is the height of the in-plane shear wall segment i in ft. (1-ft. = 0.3048m), and x is the number of in-plane shear wall segments.

4.2 Establishing a Response Modification Factor for the roof diaphragm

The inherent overstrength of the vertical shear walls causes the inelastic building behavior to migrate into the horizontal diaphragm; however, the current US building codes are based on the reliance of ductility and energy absorption characteristics of the vertical SFRS. This conflict between the likely behavior of a RWFD building and the behavior assumed by the building code is potentially resulting in unsafe building designs.

The alternative seismic design procedure of FEMA P1026 introduces a specific response modification coefficient (R-factor) for the roof diaphragm design in addition to the R-factor currently used in US seismic provisions for the design of the vertical elements of the SFRS. Additionally, a diaphragm force multiplier is introduced to increase the in-plane design shear forces in the diaphragm edges to favor the distribution of inelastic response over the interior regions of the diaphragm span as is further discussed later in this paper. This proposed design approach is currently considered only appropriate for RWFD buildings with rectangular shaped diaphragms incorporating ductile connectors, such as that found with wood sheathing panels nailed to wood framing members. A similar approach to steel deck diaphragms has not been established at this point due to a lack of reliability in connector properties for numerical modeling.

The main requirements of the proposed seismic design approach are summarized as follows [1]: The roof diaphragm design shear forces should be determined using an R-factor for the roof diaphragm (R_{dia}) equal to 4.5. This R-value for the diaphragm is slightly larger than the R-value currently specified in ASCE 7-10 for intermediate precast concrete shear walls (R=4), often used for concrete tilt-up wall buildings. Other RWFD buildings often incorporate special reinforced masonry shear walls (R=5) or special reinforced concrete shear walls (R=5). Historically, the concept of a separate diaphragm-dependent R-factor has not been a part of US model building codes. An exception is the 1994 and 1997 editions of the Uniform Building Code [14] (ICBO 1994, 1997) where a diaphragm R-factor of 4.0 was introduced for flexible diaphragms in concrete and masonry buildings; however, this code provision disappeared when the 2000 *International Building Code* [5] (IBC 2000) replaced the three US model building codes at that time. This study's choice of an R-Factor of 4.5 for the roof diaphragm design will be justified later in this paper through the evaluation of the proposed design procedure using the FEMA P695 methodology.

4.3 Using a two-stage analysis procedure

Recognizing that the horizontal diaphragm and vertical shear walls in a RWFD building each have their own unique period and ductility, it is reasonable to design each of these building components under separate stages of the analysis. ASCE 7-10 recognizes a two-stage analysis procedure when designing two vertical stacking SFRS's for the purpose of addressing different periods and R-factors, and this concept was deemed appropriate to incorporate into FEMA P1026's procedure for RWFD buildings. The perimeter vertical elements of the SFRS, such as concrete or masonry walls, should be designed for in-plane forces using their approximate period and recognized R-factor for the type of SFRS being engineered, while the diaphragm is designed for its approximate period using the new proposed formula and the R_{dia} = 4.5 discussed previously. Based on capacity design concepts, however, this R-factor for the vertical elements of the SFRS should not exceed R_{dia} so that roof diaphragm yielding occurs before yielding in the vertical elements. For vertical special reinforced concrete shear walls, for example, the R-factor used should not be greater than 4.5. As the diaphragm loads are transferred to the shear walls, they are factored by the ratio R_{SFRS} / R_{dia}. A comparison of the single degree of freedom vibration models is shown in Fig. 1.

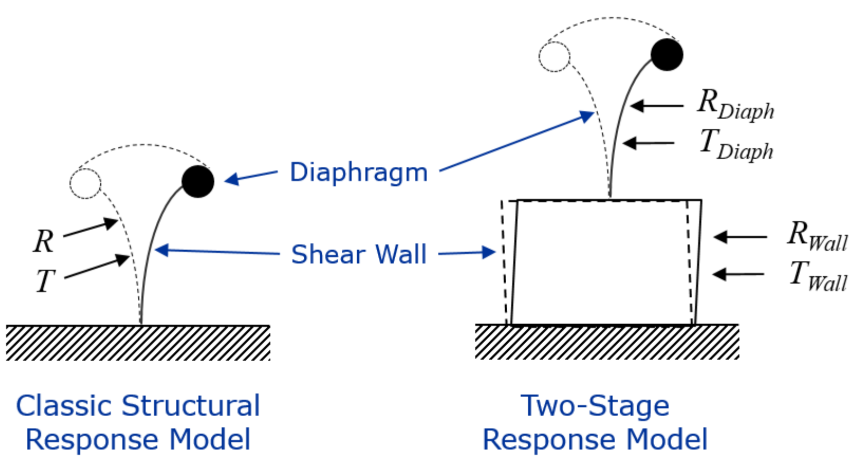

Fig. 1 – Comparison of single degree of freedom vibration models

4.4 Encouraging distributed yielding

The results from the dynamic analyses conducted on current US design practices indicate that large inelastic demands in the diaphragm are concentrated at the extreme edges, despite the current practice of reducing the diaphragm's design shear capacity in conjunction with the shear demand across its length. This is significant because improved seismic performance is achieved when inelastic demand can be better distributed over a larger portion of the diaphragm. In order to better encourage distributed yielding across the diaphragm, the authors investigated strategically weakening the interior and strengthening the end diaphragm regions [15]. Significantly improved margins against collapse were obtained in all the archetypes studied by introducing a diaphragm force mulitiplier of 1.5 for a distance of 10% of the diaphragm span from both side edges (for both principal directions) inconjunction with the introduced R_{dia} = 4.5 [16].

5. Evaluation of proposed alternative procedure in FEMA P1026

Using FEMA P695 methodology, a series of non-linear time history IDAs under the FEMA P695 ground motions set was conducted on an ensemble of the 17 RWFD archetypes with wood structural panel roof diaphragms identical to those evaluated under current US design codes except now designed to the alternate seismic design approach described in FEMA P1026. Using the same methodology as before, the adjusted collapse margin ratio (ACMR) for each archetype was determined from the CMR, period based ductility ratio and the spectral shape factor. For acceptable performance, each individual archetype and its performance group must exhibit satisfactory collapse performance as judged by the ACMR.

Considerable improvement was observed in all the RWFD building individual archetypes and their performance groups designed to the FEMA P1026 procedure compared with current US practice in terms of ACMR [16]. For the performance groups consisting of large archetypes (200-ft x 400-ft, 400-ft x 200-ft, 400-ft x 400-ft, 1-ft = 0.3048m), the ACMRs increased approximately 40% compared to those of the equivalent archetype groups designed to current code provisions. The performance groups for small archetypes (100-ft x 100-ft, 200-ft x 100-ft, 100-ft x 200-ft) was where the current US practice failed to provide acceptable collapse margins; however, under the FEMA P1026 procedure the ACMRs improved 19% for those designed for high seismic risk and 55% for those designed for moderate seismic risk. These improvements resulted in all individual archetypes and performance groups passing the criteria for acceptance under FEMA P695 methodology.

6. Example Building Comparision

In order to illustrate the resulting design differences between a RWFD building engineered using the equivalent lateral force procedure under current US design practice and that under the alternative FEMA P1026 design procedure, a 200-ft x 400-ft (61m x 122m) RWFD building archetype is examined here. As shown in Fig. 2, this building consists of 9.25-inch (0.235m) thick and 33-ft (10m) high precast concrete wall panels, acting as shear walls around the perimeter, supporting a wood structural panel roof diaphragm at an elevation of 30-ft (9.14m). Seismic design base shear forces are based on ASCE 7-10 using mapped spectral accelerations for the site of S_S = 1.5g and S_1 = 0.6g for the 0.2-second and 1-second periods respectfully, resulting in Seismic Design Category D. Additional details of the building design and design forces are found in FEMA P1026.

Under current design practice, this example building has a code estimated period of 0.26s based on the roof height, but using the alternative design procedure, the diaphragm period alone is estimated to be 0.80s in the north/south direction and 0.40s in the east/west direction, based more appropriately on the flexible diaphragm's span length. In larger buildings with wood diaphragms, this increase in period can contribute to a reduction in seismic design forces.

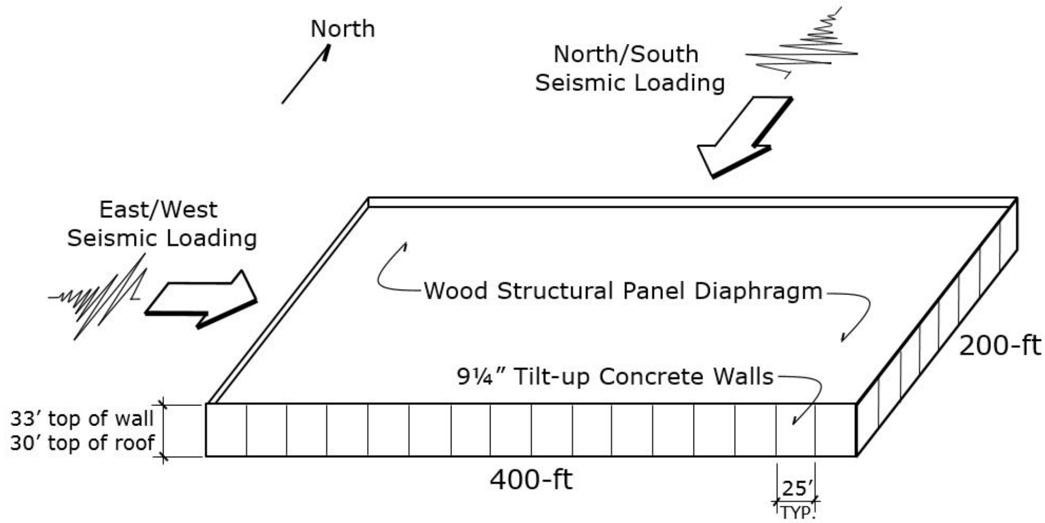

Fig. 2 – RWFD Building Example [1] (1-ft = 0.305m)

Precast concrete buildings such as the one illustrated here are commonly designed as a intermediate precast shear wall SFRS under ASCE 7-10, resulting in a response modification factor of R = 4. Under the alternative design procedure, the diaphragm is designed for an R = 4.5, causing a slight reduction of the seismic design forces; however, this reduction is limited to the interior portions of the diaphragm, because a perimeter region around the diaphragm edge is designed for 1.5 times more seismic shear force to encourage distributed yielding. Fig. 3 illustrates diaphragm design forces and resulting shears under current US practice, with both north/south and east/west directions shown. For comparison, Fig. 4 illustrates the diaphragm design forces and resulting shears under the alternative FEMA P1026 procedure.

With the design loads and resulting diaphragm shears determined, a panelized wood structural panel diaphragm is engineered following standard US practice [7] using allowable design capacities [17] to accommodate the demands. It is common practice in the US to reduce the diaphragm fastening as the shear demand reduces to save installation time and materials, and this is accomplished by designating certain regions or zones of the diaphragm with different fastening requirements. Table 1 provides a list of common diaphragm nailing zones used in the seismically active western US numbered from 1 to 6, with 6 containing the highest allowable stress design (ASD) capacity with the most fasteners.

Using the nailing zones summarized in Table 1, the building's diaphragm is mapped out with these zones to accommodate the shear demand in Fig. 4 by using a 0.7 conversion factor to obtain ASD demands. Fig. 5 and Fig. 6 illustrate the resulting diaphragm fastening for current US practice and for FEMA P1026 respectfully.

Comparing Fig. 5 with Fig. 6, the nailing in the perimeter zone (10% of building dimension) for the FEMA P1026 alternative has increased, but the nailing at the interior areas has decreased. More specifically, both building designs incorporate 2,500 individual wood structural panels; however, Fig. 5's design utilizes a little more than 160 thousand fasteners, while the FEMA P1026 alternative design in Fig. 6 utilizes a little more than 140 thousand fasteners. Overall, in this building example 12% fewer nails are used in the FEMA P1026 design when compared with the design illustrating current US standard practice.

Despite the resulting savings in labor and material costs associated with fewer nails, the building conforming to FEMA P1026 with its strategically placed nailing zones outperforms the current US practice design as indicated by the FEMA P695 collapse margin ratios obtained during the numerical study [16].

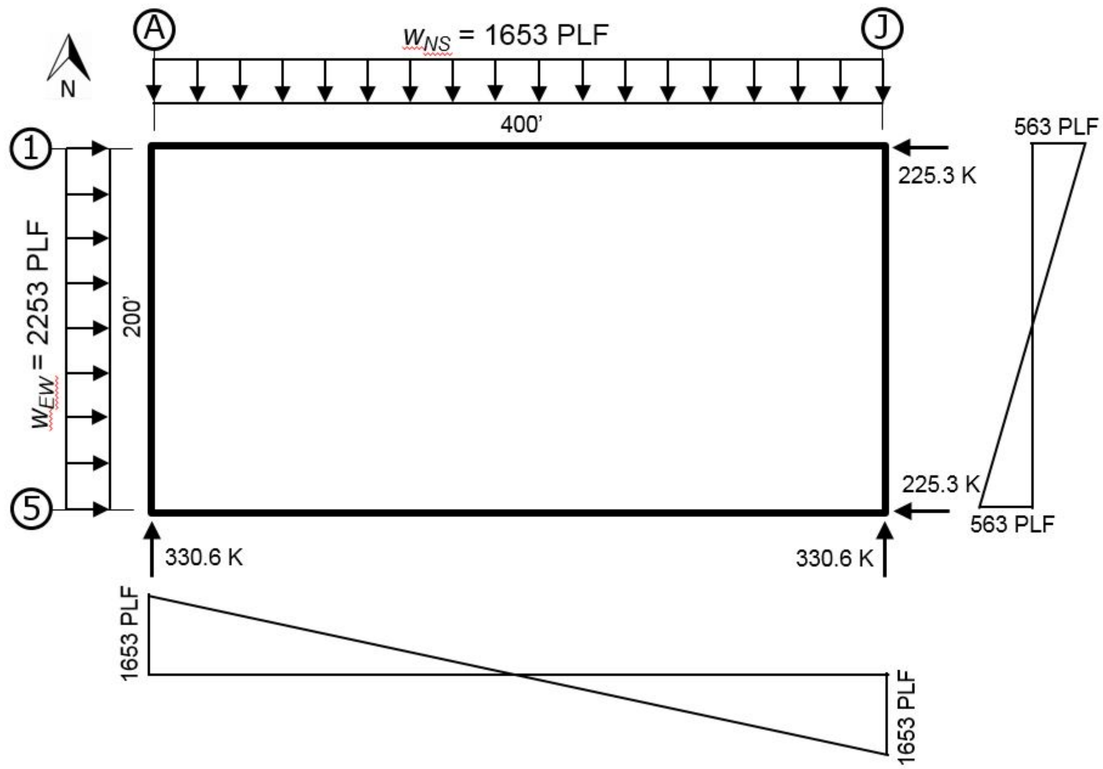

Fig. 3 – Diaphragm Design Loads and Shear Diagrams under current US practice
(1' = 0.305m, 1 K =4.45kN, 1 PLF = 14.6 N/m)

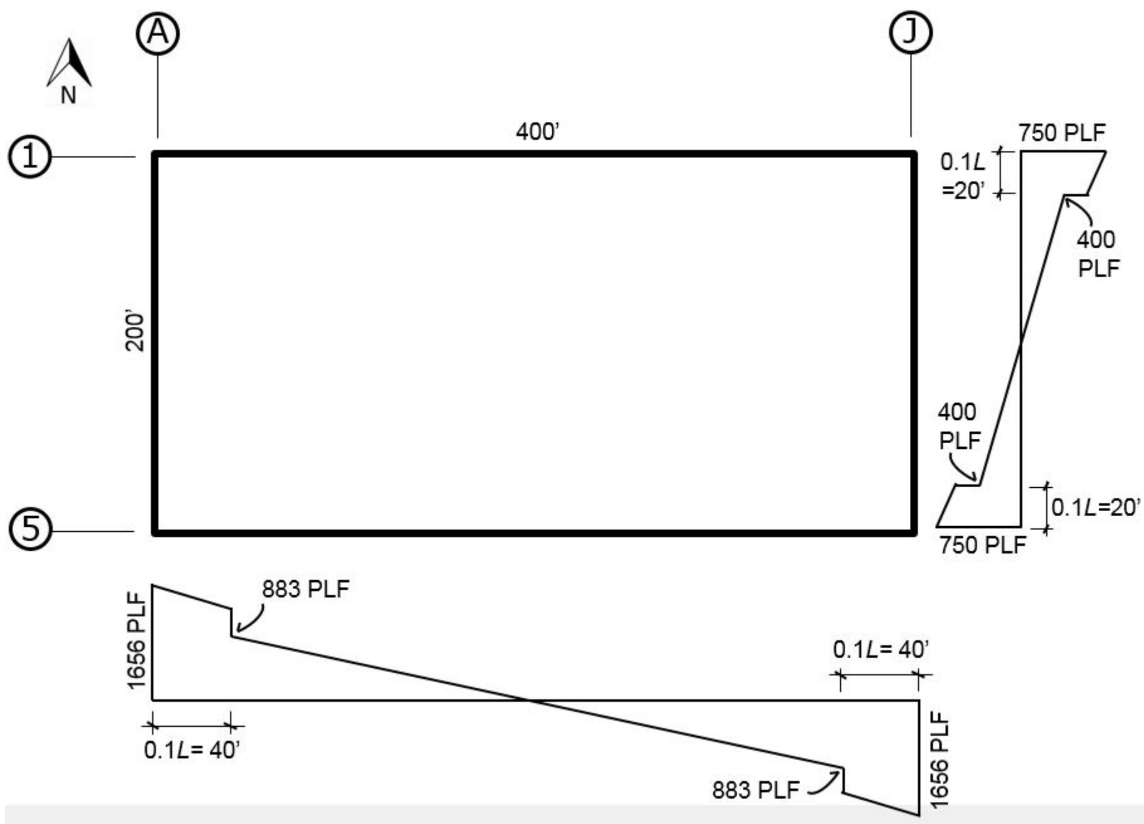

Fig. 4 – Diaphragm Design Loads and Shear Diagrams under FEMA P1026
(1' = 0.305m, 1 K =4.45kN, 1 PLF = 14.6 N/m)

Table 1 – Diaphragm Nailing Zones Commonly Utilized [7, 17].
(1” = 25.4mm, 1 PLF = 14.6 N/m)

15/32” Structural I OSB Sheathing with 10d nails (0.148” diameter x 2” long minimum)					
Zone	Framing Width at Adjoining Edges	Lines of Nails	Nailing per line at Boundary & Continuous Edges	Nailing per line at Other Edges	ASD Allowable Shear (PLF)
1	2x	1	6” o.c.	6” o.c.	320
2	2x	1	4” o.c.	6” o.c.	425
3	2x	1	2½” o.c.	4” o.c.	640
4	3x	1	2” o.c.	3” o.c.	820
5	4x	2	2½” o.c.	4” o.c.	1005
6	4x	2	2½” o.c.	3” o.c.	1290

7. Conclusion

FEMA P1026 provides an alternate procedure to design single-story RWFD buildings with wood structural panel diaphragms within the existing framework of the current 2012 IBC and ASCE 7-10 seismic design provisions. The alternate design procedure is relatively simple to implement and is based on yielding and energy dissipation in the diaphragm rather than yielding, rocking, or sliding of the walls. FEMA P695 collapse studies have demonstrated improved seismic performance of buildings designed using the alternate procedure, beyond the seismic performance anticipated using currently adopted US seismic design provisions.

Current US design practice as well as practice elsewhere in the world, do not appropriately address the unique seismic behavior and performance of new RWFD buildings. There is a disconnect between the building code’s assumed behavior and the actual behavior of these buildings, leaving this class of building vulnerable to earthquake damage. The alternate procedure in FEMA P1026 was developed to fill this gap.

At the time of this writing, the alternate FEMA P1026 design procedure was not adopted by a building code or included in ASCE 7-10. Although strictly following the alternative design procedure may not meet current US building codes, the principles could be applied to a building that meets current code by modifying the procedure to include strengthening above current code requirements in the end zones (amplified shear boundary zones) without including reduction of the design forces in the other areas of the diaphragm.

8. Acknowledgements

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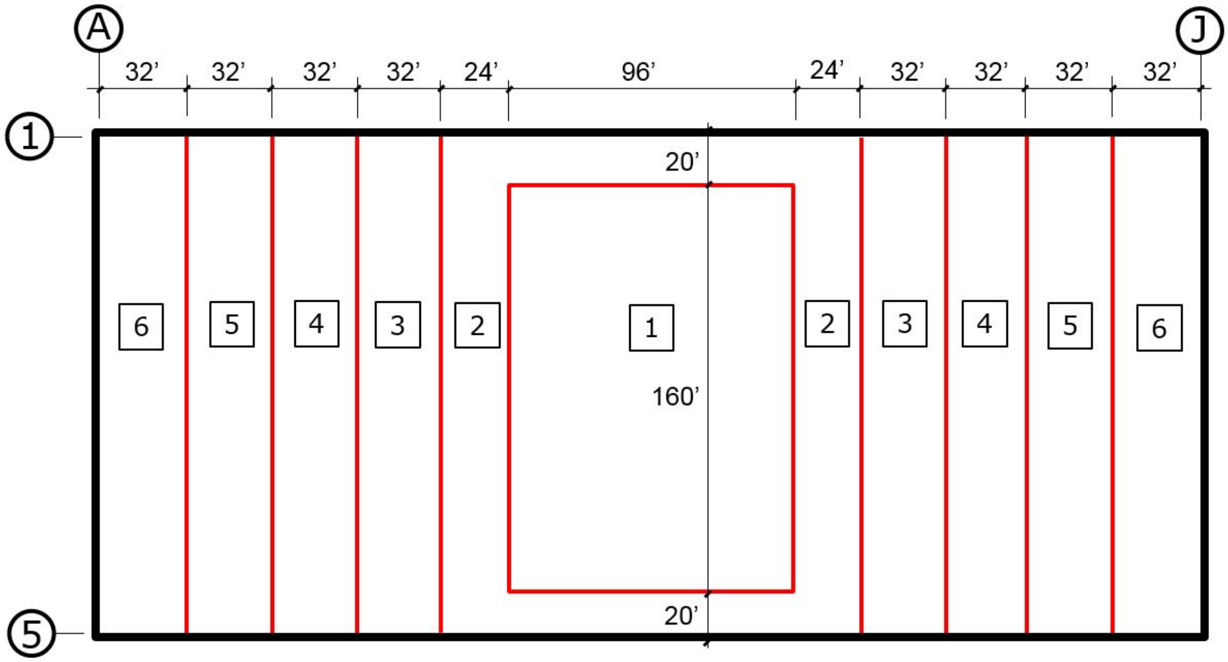

Fig. 5 – Required Diaphragm Nailing under Current US Practice [1]. (1' = 0.305m)

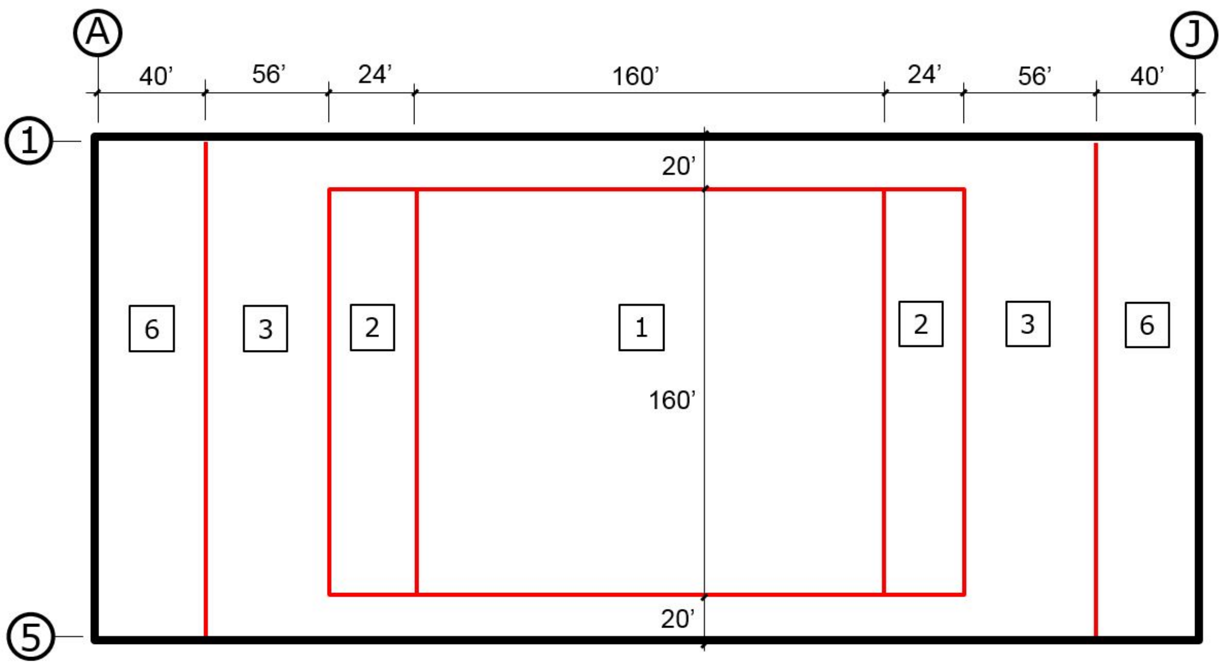

Fig. 6 – Required Diaphragm Nailing under FEMA P1026 [1]. (1' = 0.305m)

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