



New 2020 NEHRP Provisions for Seismic Design of Diaphragms

**Kelly Cobeen, Associate principal
Wiss Janney Elstner Associates, Inc.
Emeryville, California**

Abstract

New provisions for seismic design of diaphragms have been developed for inclusion in the 2020 *NEHRP Provisions*, focusing on wood diaphragms and bare steel deck diaphragms. The new provisions have their basis in consideration of the ductility, displacement capacity and overstrength of diaphragms, a departure from past practice where diaphragm design forces were primarily tied to the R-factor of the vertical elements of the seismic force-resisting system. The resulting diaphragm design methods are believed to provide more rational approaches to diaphragm design, and to provide improved earthquake performance with little or no added cost. Designers are encouraged to consider use when these methods are applicable.

One group of new provisions codifies the rigid wall-flexible diaphragm (RWFD) methodology, published in *FEMA P-1026, Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An alternative Method* (FEMA, 2015a), as well as Part 3 of the 2015 *NEHRP Provisions* (FEMA, 2015b). The RWFD design method incorporates the long-understood concept that the seismic response of RWFD buildings is governed by the response of the diaphragm much more than the vertical elements. The second group of new provisions incorporates bare steel deck diaphragms into the alternative diaphragm design method, developed in the NEHRP 2015 update and found in ASCE 7-16 (ASCE, 2017) Section 12.10.3. The steel deck diaphragm provisions were developed with significant new information from the Steel Diaphragm Innovation Initiative (SDII) Project and the Advancing Seismic Provisions for Steel Diaphragms in Rigid Wall-Flexible Diaphragm Buildings (RWFD) Project.

Proposals moving these provisions through the NEHRP update process were developed through the work of Issue Team 9 (IT9), and significant interaction with the researchers involved in the SDII and RWFD Projects. This paper provides an overview of the new diaphragm seismic design provisions.

Introduction

The new provisions developed in the 2020 NEHRP update process will result in designers being able to select from three different methods when determining seismic design forces for diaphragms and their chords and collectors. The diaphragm seismic design provisions in Sections 12.10.1 and 12.10.2 are the basic design method that has been in ASCE 7 Chapter 12 for a number of years. Section 12.10.3 is an alternate method, first included in ASCE 7-16. Section 12.10.4, is an additional method appropriate for the special case of one-story structures employing flexible diaphragms with rigid vertical elements. For each diaphragm or group of interacting diaphragms, one of these three design methods will need to be selected and implemented. While having three methods may seem a bit overwhelming, both alternative systems have scoping provisions limiting their application by structure type, so it is not likely that all three methods will be applicable to a given structure. Designers are encouraged to consider use when the alternative design methods are applicable.

Section 12.10.3, Alternative Design Provisions for Diaphragms Including Chords and Collectors, provides diaphragm seismic design provisions that specifically recognize and account for the effect of diaphragm ductility and displacement capacity on the diaphragm design forces. This is accomplished with the introduction of a diaphragm design force reduction factor, R_s . This method is currently available in ASCE 7-16 for the diaphragm systems listed in Table 12.10-1. The new NEHRP provisions add steel deck diaphragm systems to this list. Neither the number of stories, nor the building configuration is restricted by the Section 12.10.3 provisions; the types of diaphragm systems for which it can be used is limited, however. Section 12.10.3 is mandatory for precast concrete diaphragms in structures assigned to SDC C, D, E and F, and is optional for precast concrete diaphragms in SDC B and cast-in-place concrete, wood, and bare steel deck diaphragms in structures assigned to all SDCs. The required mandatory use of Section 12.10.3 for precast diaphragm systems in SDC C through F buildings is based on recent research that indicates that improved earthquake performance can thus be attained.

Section 12.10.4, Alternative Diaphragm Design Provisions for One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements, introduces diaphragm seismic design provisions that are permitted to be used for one-story structures combining flexible diaphragms with rigid vertical elements. The seismic design methodology specifically recognizes the dynamic response of these structures as being dominated by dynamic response of and inelastic behavior in the diaphragm. While the most common occurrences of this structure type are the concrete tilt-up and masonry wall big-box structures, the rigid vertical element terminology of this section recognizes a wider range of vertical elements for which this methodology is permitted to be used. This approach is based on numerical studies conducted as part of the development of the FEMA P-1026 guideline document and additional recent steel deck diaphragm research. These studies indicate that improved seismic performance can be obtained for this group of structures through use of this design methodology.

Alternative Diaphragm Design Provisions for Untopped Steel Deck Diaphragms

The ASCE 7 Section 12.10.3 alternative diaphragm design provisions were first developed during the 2015 NEHRP update process. The primary objectives were to better reflect the vertical distribution of diaphragm seismic forces for diaphragm near-elastic behavior, and to use the new diaphragm design force reduction factor, R_s , to adjust the forces based on diaphragm overstrength and displacement capacity. The R_s factor, as a result, creates a design efficiency by permitting use of lower seismic design forces for diaphragm systems identified to have higher ductility and deformation capacity. Conversely, diaphragm systems with less ductility will be required to be designed for higher seismic forces.

ASCE 7-16 provides design parameters for use of this methodology for precast concrete diaphragms, cast-in-place concrete diaphragms, and wood diaphragms. New in the 2020 NEHRP update is the extension of this methodology to bare steel deck diaphragms. New R_s values are proposed for Table 12.10-1, addressing bare (untopped) steel deck diaphragms designed in accordance with new provisions in AISI S400 (AISI, 2019). These include diaphragms meeting newly developed requirements for special seismic detailing, for which an R_s factor of 2.5 has been derived, and all other bare steel deck diaphragms for which an R_s factor of 1 has been derived.

The ductility of bare steel deck diaphragms is largely driven by the performance of the deck profile and its interaction with sidelap and structural connections. It has been found for a specific class of WR roof deck that if the sidelap and structural connections have adequate ductility and deformation capacity, the full bare steel deck diaphragm can similarly develop

productive levels of ductility with sufficient system-level deformation capacity (O'Brien et al 2017, Schafer 2019). These findings formed the basis for prescriptive special seismic detailing requirements that are under development for AISI S400.

The derivation of the bare steel deck diaphragm force reduction factor, R_s , is summarized in Appendix 1 of Schafer (2019). The ductility and deformation capacity of sidelap and structural connections employed in bare steel deck diaphragms is established by evaluation of new cyclic shear testing (NBM 2017, 2018, Schafer 2019). The ductility of bare steel deck diaphragms has preliminarily been established by assembly and evaluation of existing cyclic cantilever diaphragm tests (O'Brien et al. 2017). The impact of the connector and cantilever diaphragm tests on full building performance is assessed in a 3D building model as detailed in Schafer (2019a). The model shows that only bare steel deck diaphragms with connections that have sufficient ductility and deformation capacity provide adequate inelastic diaphragm performance – thus leading to special seismic detailing requirements. For the subset of cyclically tested diaphragms that meet the special seismic detailing requirements, the tested subsystem ductility and system overstrength are established (Schafer 2019). To establish the diaphragm system ductility an additional correction is provided for the reduction in ductility of a roof that experiences varying shear across its width, compared with a cantilever diaphragm test which is under constant shear (O'Brien et al. 2017, Schafer 2019). From the system ductility and overstrength the diaphragm force reduction factor R_s was developed based on μ - R relations using the method documented in ATC-19.

The AISI S400 provisions for special seismic detailing of bare steel deck diaphragms, used in conjunction with the R_s of 2.5, were balloted by and received input from both NEHRP IT9 and the PUC. The final determination of these detailing provisions is now in the hands of the AISI standards committee. The prescriptive detailing provisions address the steel deck panel type, base steel thickness and material requirements. These provisions require that structural connections between the steel deck and supporting members use mechanical connectors. Structural connections and sidelap connections have further limits on pattern, spacing, etc. AISI S400 also provides performance-based criteria to establish that selected detailing associated with a particular bare steel deck diaphragm (new profile, new connector, etc.) meets the same performance objectives as the prescriptive system – and is thus deemed to provide an intended ductile mechanism. Other bare steel deck diaphragms have fasteners and system behavior that is less ductile. As a result, the R_s factor is smaller, resulting in design for near-elastic level forces.

Rigid Wall (Vertical Element) Flexible Diaphragm Provisions

The 2015 guideline document *Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedure (FEMA P-1026)* introduced a new seismic design methodology that specifically recognizes the dynamic response of rigid wall-flexible diaphragm structures as being dominated by dynamic response of and inelastic behavior in the diaphragm.

The development of the FEMA P-1026 guidelines included numerical studies using FEMA P-695 methodology (FEMA, 2009). Studies included archetype buildings with concrete perimeter tilt-up walls, and plan dimensions ranging from 100 feet by 100 feet to 400 feet by 400 feet with various plan aspect ratios. The studies that modeled wood structural panel diaphragms fastened to wood framing or nailers, demonstrated that diaphragms designed to new criteria proposed by the guideline consistently met or exceeded the FEMA P-695 acceptance criteria and, for this building type, provided improved performance relative to diaphragm design in accordance with ASCE 7 Sections 12.10.1 and 12.10.2.

FEMA P-1026 recommendations for seismic design of the RWFD building type include:

- a) Recognizing that the diaphragms often yield and dominate the building behavior while the walls typically remain mostly in the elastic range for in-plane loading,
- b) Recognizing the distinct periods of both the shear wall system and the diaphragm, and using a two-stage equivalent lateral force analysis to capture this distinct behavior,
- c) Proposing the creation of a zone of reduced nailing away from the diaphragm perimeter, where distributed yielding can occur without jeopardizing the diaphragm connection to the vertical element.

The FEMA P-1026 research team also examined bare steel deck roof diaphragms in RWFD buildings, but at that time felt it premature to make recommendations. FEMA P-1026 detailed a series of reservations with regard to the application to steel roof decks. As detailed in Schafer (2019), since the conclusion of the FEMA P-1026 project, work carried out by NBM Technologies and later by Ben Schafer addressed the reservations from the original FEMA P-1026 and demonstrated that properly detailed bare steel deck roof diaphragms could develop sufficient ductility. Based on this work, the analysis used to develop FEMA P-1026 was re-assessed and it was concluded in Schafer (2019) that the proposed design method was valid for steel deck diaphragms, so long as the deck details provided sufficient ductility. As a

result, RWFD provisions have been extended to steel deck diaphragms. Schafer, B.W. (2019). “Research on the Seismic Performance of Rigid Wall Flexible Diaphragm Buildings with Bare Steel Deck Diaphragms” provides background information.

Eligibility for use of Section 12.10.4

A series of limitations must be met in order to use the Section 12.10.4 methodology. These are detailed in new Section 12.10.4.1. The intent of these limitations is to restrict use of the methodology to flexible diaphragm-rigid vertical element structures that are consistent with the FEMA P-1026 numerical study basis. These limitations are:

- When these alternative provisions are used, they are to be used in both orthogonal directions,
- Use is limited to wood structural panel or bare steel deck diaphragms,
- Wood structural panel sheathing is required to be fastened to wood framing members or fastened to wood nailers (e.g. wood nailers attached to steel open-web joists) with nailing as specified in the Special Design Provisions for Wind and Seismic (SDPWS, 2015) diaphragm tables.
- Bare steel deck diaphragms are to be designed in accordance with the recently developed steel deck design and detailing provisions of AISI S400 and provisions of AISI S310.
- Use is prohibited where materials installed over the diaphragm would add significant diaphragm stiffness. This limitation is included because where such materials are used, the diaphragm period and therefore seismic forces would not be appropriately estimated by these provisions, and the diaphragm may no longer qualify as flexible.
- Use is prohibited with horizontal irregularities including torsional and diaphragm discontinuity irregularities, while use with re-entrant corners is permitted.
- Very important is that the diaphragm being designed will need to be broken down into a series of rectangular elements for purposes of diaphragm design. Buildings to which this methodology might be applied are often not completely rectangular in plan, and often combine both long and short diaphragm spans. This provision requires that each section of diaphragm be defined as spanning between boundaries consisting of either vertical elements or collectors, with each span referred to as a diaphragm segment. Figure 1a illustrates diaphragm segments for transverse seismic forces, with each segment supported on all sides by concrete or masonry walls. Figure 1b illustrates the same concept, with a building

plan in which, for transverse design, the diaphragm segments span to a combination of walls and collectors. This limitation would also prohibit application to non-rectangular diaphragm segments such as triangular, trapezoidal or curved configurations; this is because of the more complex response of these configurations, and the difficulty in defining the effective diaphragm span and resulting diaphragm period. See Figure 2.

- Vertical elements of the seismic force-resisting system are limited to systems that are inherently rigid for in-plane forces. A list of such systems is provided in lieu of a numerical criteria. The modifiers ordinary, intermediate, and special are not included for the vertical elements, with the intent that all types are sufficiently rigid and therefore permitted.
- Vertical elements of the seismic force-resisting system are to be designed using the equivalent lateral force procedures of Section 12.8, except when designed in accordance with a new two-stage analysis procedure.

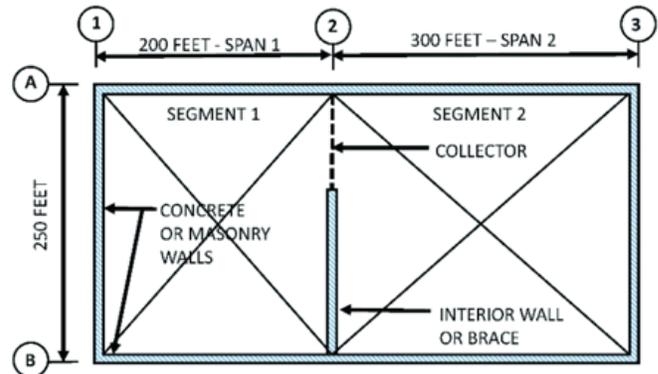


Figure 1b. Structure in plan view, divided into rectangular diaphragm segments for purposes of transverse seismic design. The Line 2 collector serves as a boundary between diaphragm segments for transverse direction loading.

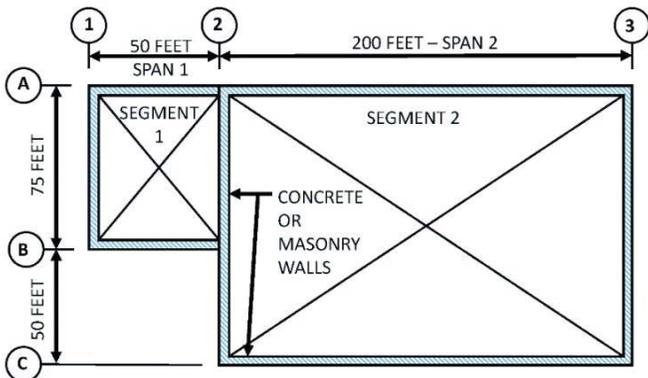


Figure 1a. Structure in plan view, divided into rectangular diaphragm segments for purposes of transverse seismic design. Note that identification of different segments will be required for longitudinal diaphragm forces.



Figure 2. Structure with non-rectangular diaphragm is beyond the scope of these provisions.

Section 12.10.4 Design Forces

New equations are provided, to be used in place of ASCE 7 Equation 12.10-1 for calculation of diaphragm design forces as follow:

$$F_{px} = C_{s\text{-diaph}} * W_{px} \quad (12.10-15)$$

where

W_{px} = the effective seismic weight tributary to the diaphragm,

$$C_{s\text{-diaph}} = \frac{S_{DS}}{R_{diaph}/I_e} \quad (12.10-16a)$$

and need not be greater than:

$$C_{s\text{-diaph}} = \frac{S_{D1}}{T_{diaph} * (R_{diaph}/I_e)} \quad (12.10-16b)$$

where

- S_{DS} = the design spectral response parameter in the short period range as determined from Section 11.4.5 or 11.4.8,
- R_{diaph} = 4.5 for wood structural panel diaphragms,
 = 4.5 for bare steel deck diaphragms that meet the special seismic detailing requirements of AISI S400, and
 = 1.5 for all other bare steel deck diaphragms
- I_e = the Importance Factor determined in accordance with Section 11.5.1.
- T_{diaph} = 0.002 L_{diaph} , for wood structural panel diaphragms, and
 = 0.001 L_{diaph} for profiled steel deck panel diaphragms determined for each rectangular segment of the diaphragm in each orthogonal direction [seconds].

These equations incorporate the newly defined diaphragm approximate period, T_{diaph} . Where T_{diaph} is greater than T_s , this will permit C_s to be defined by the descending velocity-controlled portion of the response acceleration spectrum, thereby reducing diaphragm design forces. This is a distinct deviation from past design practices where seismic forces for diaphragm design were determined exclusively based on the approximate period and response modification factor of the vertical elements of the seismic force-resisting system.

Seismic response modification coefficient, R_{diaph} , is provided for both wood and bare steel deck roof diaphragms. The selected values for wood diaphragms and bare steel deck diaphragms with mechanical fasteners are based on studies reported in FEMA P-1026 (2015) and Koliou et al. (2015a,b). Based on the work of Schafer (2019) bare steel deck diaphragms were separated into two classes: diaphragms meeting special seismic detailing requirements where ductile diaphragm performance can be reliably provided are given an R_{diaph} of 4.5, and other diaphragms where ductility is not required due to design forces representing near-elastic response, which are given an R_{diaph} of 1.5. The special seismic detailing requirements are the same as referenced in ASCE 7 Section 12.10.3, and found in AISI S400.

Amplified Shear Boundary Zone

Diaphragm shear forces for all diaphragms designed using this methodology are modified in an effort to manage the diaphragm's inelastic behavior. This is done by creating an "amplified shear boundary zone." In larger diaphragm segments with spans of 100 feet or more, the amplified shear boundary zone is provided at the supported ends of the diaphragm segment span. Figure 3 illustrates amplified shear

boundary zones for diaphragm spans in both orthogonal directions.

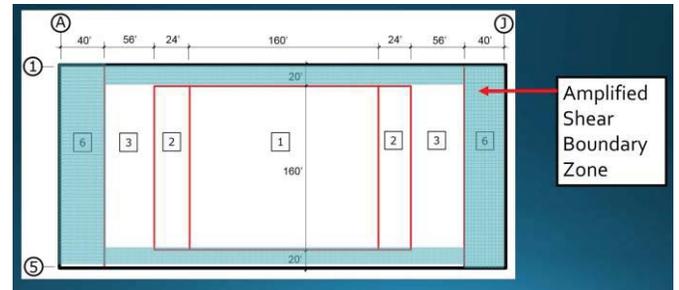


Figure 3. Diaphragm plan view with the amplified shear boundary zone indicated by shading.

The boundary zone is strengthened to reduce the inelastic demand within this zone, and push inelastic behavior to the interior of the diaphragm segment. FEMA P-1026 studies found that the strengthening of the diaphragm segment's ends resulted in broadly distributed inelastic behavior towards the diaphragm segment interior and significantly improved diaphragm performance. This also served to move inelastic demand away from the diaphragm-to-vertical element interface, where it can be most damaging and most greatly affect structural performance.

Small diaphragm segments with spans less than 100 feet have limited width available to distribute inelastic behavior; thus the amplification of diaphragm shears over the full diaphragm segment serves to limit inelastic behavior overall. It is recognized that the different treatment of diaphragms based on being above or below the 100 foot span introduces a step function into the design process; while ideally this step function would not exist, it is necessary based on information currently available, and likely to have limited impact on design as the primary use of the methodology is intended to be diaphragms with spans greater than 100 feet.

When wood diaphragms are designed using the RWF method and amplified shear boundary zones around the perimeter, the resulting nailing at the diaphragm interior is less than would be required when designing to Sections 12.10.1 and 12.10.2. For these wood diaphragms, it is important that this interior zone of reduced nailing be incorporated into construction, as this is the primary intended source of diaphragm inelastic behavior. There is a wide spread belief that putting in more nails is always better, but in this case putting in more nails could result in reduced performance.

Other Seismic Design Parameters

In addition to developing R_{diaph} factors, the FEMA P-1026 and RWF projects developed $\Omega_{0-diaph}$ and $C_{d-diaph}$ parameters for use in seismic design. When using the provisions of Section

12.10.4, collectors and their connections to vertical elements are to be designed for the diaphragm design forces of Section 12.10.4.2.1, and in SDC D through F, amplified by an overstrength factor, $\Omega_{0\text{-diaph}}$, determined as a part of the FEMA P-1026 numerical studies. Section 12.10.4.2.5 provides a diaphragm deflection amplification factor, $C_{d\text{-diaph}}$, intended to be used where the seismic design provisions currently require calculation of deflection. No new uses or checks of deflection are intended to be imposed by Section 12.10.4 provisions. The $C_{d\text{-diaph}}$ factors have been derived from the FEMA P-1026 nonlinear response history analysis (NLRHA) studies in conformance with FEMA P-695 procedures (developed for vertical elements of the seismic force-resisting system) with some modifications.

Additional Considerations

The following provides general discussion of diaphragm deflections calculated in accordance with ASCE 7 Chapter 12 and SDPWS procedures, as well as those predicted by the FEMA P-1026 numerical studies, and their impact on seismic performance. This is provided because very significant diaphragm deflections can occur in RWFD structure diaphragms. The ability to accommodate these deflections and meet applicable ASCE 7 limitations can be challenging, and is deserving of attention during design.

FEMA P-1026 provides design examples of a wood structural panel diaphragm with a 400 foot span and 200 foot width using common engineered design practice and SDPWS. Chapter 3 provides a design example of the diaphragm using Section 12.10.1 diaphragm forces. Chapters 5 and 6 provide a parallel design example using the new provisions of Section 12.10.4. Using Section 12.10.1 diaphragm design forces and a C_d of 4, consistent with an intermediate precast shear wall, the estimated maximum diaphragm deflection is 29 inches using the 3-term equation of SDPWS Section 4.2.2. Using Section 12.10.4 diaphragm design forces, the prescribed C_d of 3.0, and the design assumptions used in the FEMA P-1026 examples, the estimated maximum diaphragm deflection is 19 inches. Depending on calculation assumptions and calculation methods, it is anticipated that design engineers might calculate maximum diaphragm deflection as being anywhere between 10 and 19 inches. The 10 to 19 inches is a relative estimated displacement between the foundation and roof diaphragm at diaphragm mid-span, which will be a maximum imposed drift on the vertical elements of the gravity system. The primary contributions to this roof diaphragm deflection come from the shear deformation of the wood structural panel diaphragm (combined nail slip and panel shear deformation) and flexural deformation from tension and compression of the chord member.

Numerical studies used as the basis for FEMA P-1026 provide data on analytical predictions of average peak diaphragm displacements. Diaphragm drift ratios published in Koliou et al, (2015a, 2015b) are average peak ratios published for the FEMA P-695 ground motion suite, scaled to $S_{DS}=1.0$. The published diaphragm drift ratios correspond to an average peak roof deflection of seven inches for the Chapter 3 example of the 400 foot span and 200 foot wide diaphragm designed for Section 12.10.1 forces. The published diaphragm drift ratios correspond to an average peak roof deflection of ten inches for a structure designed using a method close to but not exactly matching Section 12.10.4 (the design of this similar building model used a period that combined diaphragm and shear wall period, modestly increasing the period, lowering the design forces, and lowering diaphragm stiffness).

The user will notice that the SDPWS engineered design estimate of peak diaphragm deflection of 19 inches (or the range of 10 to 19 inches) is generally larger than the NLRHA analytically predicted deflections of seven and ten inches. A few reasons potentially contribute to this disparity. First, the FEMA P-1026 calculation conservatively computed the diaphragm's flexural deflection based on a single steel angle chord; however, numerous other building elements will engage as inadvertent chord elements including concrete and masonry walls, wall reinforcing and roof structure continuous ties, significantly reducing the flexural contribution to the deflection. Second, the 3-term deflection equation of SDPWS Section 4.2.2 significantly overestimates the diaphragm deflection compared to the more accurate 4-term equation in the SDPWS Commentary. Third, the nail slip contribution of the SDPWS diaphragm deflection equation is conservatively based on considering only the larger "nail spacing at other panel edges"; however, significant amounts of additional stiffness are contributed by the tighter "continuous edge nailing" in the direction of loading. Fourth, interior regions of each nailing zone have significantly more stiffness than assumed by the SDPWS diaphragm deflection procedure due to the stiffness nonlinearity of nail slippage. And lastly, the selection of $C_d = 3.0$ is potentially conservative as well. Finally, it is understood that the NLRHA, while a best available tool, provides approximate results and is most reliable for study of relative or approximate behavior, and not absolute determination of deflection. It is anticipated that actual deflection of diaphragms for most buildings of interest would fall in a range between the SDPWS engineered design and NLRHA values. Diaphragm deflections calculated using SDPWS engineered design methods are anticipated to conservatively estimate deflections.

Deflection of diaphragms is limited by Section 12.12.2, which requires that deflection be limited such that attached gravity load-carrying elements retain structural integrity. There are two primary aspects of structural integrity that should be

checked. The first is the ability of the concrete or masonry walls (or other vertical elements) to maintain support of the prescribed loads through the wall out-of-plane rotation. Where gravity supports (walls, columns) have rotational fixity at their top or bottom, the ability to support gravity loads in the displaced configuration should also be evaluated. Diaphragm deflection causes second order moments in these elements which should be considered in conjunction with axial forces. The second is the ability of the connections within the gravity system to maintain strength as the vertical elements rock and rotate relative to the horizontal diaphragm; detailed discussion follows. Additionally, interior full-height partitions or demising walls and other nonstructural components may suffer from racking or connection failure.

Consideration of typical roof system connections to the vertical elements can provide insight into the ability of gravity load carrying systems to withstand estimated roof diaphragm deflections. This discussion is affected, however, by whether the NLRHA analytically predicted diaphragm deflections or the SDPWS estimated deflections are used. Using the higher predicted mid-diaphragm deflection of 10 inches from the FEMA P-1026 NLRHA numerical studies, and story heights of 20 and 30 feet, this would create a gap of between 1/3 and 1/2 inch between an exterior wall and a twelve inch deep ledger and joist, as seen in Figure 4(a) for a wood-framed roof system. This amount of deformation can reasonably be taken up at several different interfaces within this connection without connection failure being likely. Similarly for wood system girder supports (Figure 4(b)) and interior columns (Figure 4(c)), the connections should be able to withstand this level of deflection. As the diaphragm deflection is increased to approximately 19 inches based on SDPWS calculations, the gaps increase to between 2/3 and 1 inch for the joists, which is approaching but likely not reaching gap levels that could potentially unseat rafters from hangers and cause damage to ledgers that are susceptible to cross-grain tension failure. Higher wall deflections or shorter wall heights would create gaps that could potentially push these connections to failure, and so deserve detailed consideration during design.

The higher mid-diaphragm deflection of 10 inches from the NLRHA numerical studies and roof diaphragm heights of 20 and 30 feet would likely be acceptable for an open-web steel joist connection to an exterior wall, as seen in Figure 5(a). The behavior of this connection is considered reasonably close to pinned. The same would be true of truss girder connections to the exterior walls and interior columns, provided the girder truss connections are close to a pinned condition. Of concern in the steel open-web joist system is when a joist girder bottom chord has insufficient clearance or is axially connected to the wall or column (Figures 5(b) and 5(c)). For a three foot girder depth, the gapping required would be on the order of 1 to 1-1/2 inches at a diaphragm drift of 10 inches. If the diaphragm drift

were to be 19 inches, the needed gap would increase to 2 to 3 inches. If not detailed for this gap, the roof diaphragm deflection could be very damaging to the girder truss and connections. These illustrations serve as a reminder to the designer that this provision must be checked, and may limit detailing choices.

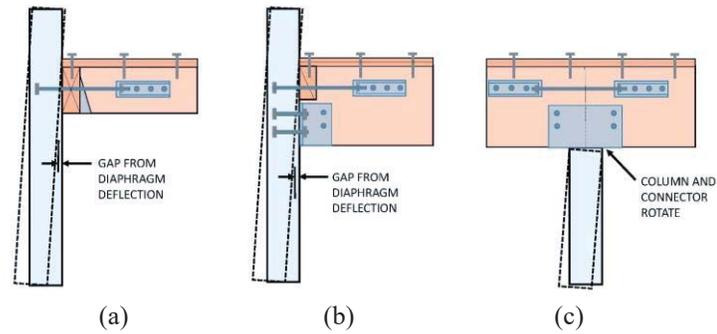


Figure 4. Wood framed, wood sheathed diaphragm connections (a) joist to perimeter concrete or masonry wall, (b) girder connection to perimeter concrete or masonry wall, and (c) girder to interior column.

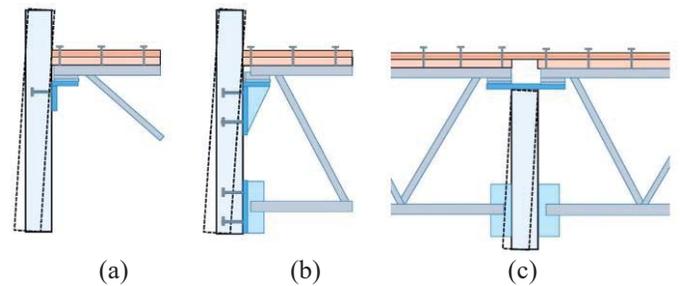


Figure 5. Steel open-web joist framed, wood sheathed diaphragm connections (a) joist to perimeter concrete or masonry wall, (b) joist girder to perimeter concrete or masonry wall, and (c) joist girder to interior column.

In addition to structural integrity considerations, global structural stability is a separate consideration where, the diaphragm deflection's contribution may lead to potential $P\Delta$ instability of the system as a whole. As the roof mass horizontally translates and the gravity system rotates, secondary forces and moments develop, potentially leading to instability. ASCE 7 Section 12.8.7 provides a methodology using a stability coefficient θ to determine whether the secondary effects are significant enough to require consideration; however, this section was developed expressly for buildings where the deformation is associated primarily with the vertical system, not the horizontal diaphragm. Never

the less, the provisions can be adapted by considering P_x as the building weight tributary to the diaphragm (diaphragm weight plus half the rotating wall weight) and Δ as the weighted average diaphragm deflection. This approach is illustrated in FEMA P-1026.

Conclusions and Next Steps

At time of writing, the RWFD provision proposals have completed the balloting process to be included in the 2020 NEHRP Provisions, and have been provided to ASCE 7 to be considered for adoption into ASCE 7-22. The steel deck diaphragm proposals have almost completed the NEHRP ballot process, and will similarly be put forward to be considered for ASCE 7 adoption. Also developed from the FEMA P-1026 studies, but not addressed in this paper are provisions for an optional two-stage seismic analysis of RWFD structures, considering in two separate stages the forces contributed by the mass of the flexible diaphragm and combined with the forces from the rigid vertical elements. This proposal is also being put forward to ASCE 7.

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