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EVALUATION AND REHABILITATION OF LOW-RISE MASONRY BUILDINGS WITH FLEXIBLE FLOOR AND ROOF DIAPHRAGMS

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SUMMARY

This paper outlines the last two phases of a joint research study performed by The University of Texas at Austin and the United States Army Corp of Engineers, Construction Engineering Research Laboratory, Engineer Research and Development Center (CERL). The study involves the seismic evaluation and rehabilitation of low-rise reinforced masonry buildings with flexible diaphragms and coordinates and synthesizes experimental testing, analytical modeling, practical implementation, and real-world application to enhance the predominant seismic evaluation and rehabilitation methodologies for these types of buildings.

INTRODUCTION

In response to Executive Order 12941 [1], the United States government began a large coordinated effort to assess and mitigate the seismic hazards of its existing owned and leased facilities. As part of that effort, the US Army assessed its existing building inventory and preliminarily determined that, in the roughly 4500 seismically vulnerable Army-owned buildings in the continental US, the chief potential seismic deficiency is flexible diaphragms. Furthermore, the 4500 vulnerable buildings comprise mostly low-rise reinforced masonry construction. This study was intended to enhance the accuracy and efficiency of seismic hazard assessment and mitigation for these types of buildings and was realized in four distinct phases of study: behavior; analysis; evaluation; and application. The results and conclusions of each phase, and how they related to those of other phases, are now summarized.

Information summarized in this paper, specifically that in sub-sections Phase 1: Behavior and Phase 2: Analysis, is presented in detail by Cohen [2, 3, 4, 5, 6].

PHASE 1: BEHAVIOR

To characterize the seismic response of these types of buildings, two half-scale low-rise reinforced masonry building specimens with flexible roof diaphragms were constructed based on identified

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prototypical configurations, and subjected to a coordinated seismic testing program on the US Army Tri-axial Earthquake and Shock Simulator (TESS). This testing qualitatively and quantitatively confirmed the generally accepted premise (by the earthquake engineering technical community) that diaphragm flexibility can significantly affect the seismic response of these types of buildings. In contrast to what is usually assumed in design, these tests suggested that these types of buildings do not behave as systems with a single degree of freedom associated with the in-plane response of the shear walls. Rather, they behave at least as two-degree-of freedom systems (with one degree of freedom associated with the in-plane response of the shear walls and another with that of the roof diaphragm), and essentially as single-degree-of-freedom systems, with that degree of freedom associated with the in-plane response of the roof diaphragm.

The two shaking-table specimens had different roof diaphragms; one with a diagonally sheathed lumber diaphragm, the other with a welded metal-deck diaphragm. Following shaking-table testing, the roof diaphragms and top four courses of attached masonry wall were removed from the shaking-table specimens, repaired, and subjected to reversed cyclic quasi-static displacements. These tests characterized the hysteretic behavior of the diaphragms, and related observed seismic behavior with different levels of deformation and damage. Diaphragm deformations discussed in this paper are described in the context of the diaphragm drift ratio (DDR), which characterizes seismic damage in walled structures with flexible horizontal diaphragms.

$$DDR = \frac{\Delta_{diaph}}{L/2} \qquad \textbf{Equation 1}$$

where, Δ_{diaph} is the in-plane deflection of the diaphragm at a given floor level relative to the supporting shear walls at that level, and L is the plan length of the diaphragm.

PHASE 2: ANALYSIS

Based on observations and conclusions from physical testing, a simple tool for the seismic analysis of these types of buildings was developed, tested, and validated. A two-degree-of-freedom (2DOF) analysis tool was developed for the general case and then analytically bounded, through parameter studies, to the particular analysis of low-rise reinforced masonry buildings with flexible diaphragms. One degree of freedom was associated with in-plane response of the transverse shear walls; the other degree of freedom was associated with in-plane response of the roof diaphragm. Parameter studies suggested that the 2DOF tool could be further simplified to a single-degree-of-freedom (SDOF) system, with that degree of freedom associated with the in-plane response of the diaphragm, only. The 2DOF and SDOF tools were validated in the linear elastic and nonlinear ranges using data from shaking-table testing, finite-element modeling, and lumped-parameter modeling.

PHASE 3: SEISMIC EVALUATION

In this phase of study, data and knowledge from Phase 1: Behavior, additional data from other studies, and the analysis tool developed in Phase 2: Analysis are combined and integrated with the predominant existing seismic evaluation methodology, *Handbook for the Seismic Evaluation of Buildings – A Prestandard* FEMA 310 [7], to fill identified gaps in that methodology.

Critical Review and Potential Gaps in Existing Evaluation Methodology

Low-rise reinforced masonry buildings with flexible diaphragms may have many different seismic deficiencies. Seismic evaluation provisions of FEMA 310 designed to identify such deficiencies comprise tiered evaluation criteria of incrementally increasing rigor: the Screening phase (Tier 1); the Evaluation phase (Tier 2); and the Detailed Evaluation phase (Tier 3). The Screening phase uses limited analyses and checklists to quickly identify probable seismic deficiencies. The checklist items are chiefly based on correlations between observed seismic damage and specific building configurations or characteristics. If deficiencies are identified in the Screening phase, the evaluating engineer can choose to perform the Evaluation phase (Tier 2) or can directly recommend rehabilitation. The Evaluation phase (Tier 2) involves more rigorous evaluations on either a deficiency-specific or a building-wide basis. In the former and more-common case, only deficiencies identified by the Screening phase are reevaluated; in the latter, the entire structure is reevaluated. In the deficiency-specific case, each checklist item from the Screening phase corresponds to a complementary procedure in the Evaluation phase. If deficiencies are still identified by the Evaluation phase, the evaluating engineer can choose to perform the final Detailed Evaluation phase (Tier 3) or can directly recommend rehabilitation. The Detailed Evaluation phase basically comprises a rigorous analysis of the deficient structure or its deficient components, according to accepted methodologies for seismic rehabilitation or for new construction, such as *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* FEMA 356 [8], and the International Building Code [9], respectively.

FEMA 310 was revisited to identify and propose refinements for potential gaps in its evaluation methodology. In this process, potential deficiencies were critically compared with the existing evaluation criteria intended to identify them.

Checklists of the Screening phase (Tier 1) do not explicitly require the comparison of diaphragm shear demand and capacity, or of diaphragm deformation demand and capacity, and hence do not sufficiently characterize the performance of flexible diaphragms. In some cases, these potential limit states are checked qualitatively. For example: diaphragm shear forces are implicitly checked by the requirement that straight-sheathed lumber diaphragms have aspect ratios less then or equal to 2:1 (for Life Safety performance objectives); diaphragm deflections are implicitly checked by the requirement that wood diaphragms with spans greater than 24 ft. be constructed of diagonal sheathing or structural paneling (for Life Safety performance objectives); and other checklist items are similar. While these checklist items and others like them are effective for some buildings, they do not categorically identify diaphragm force and deformation limit states.

Procedures of the Evaluation phase (Tier 2) directly address diaphragm capacity, and indirectly address diaphragm deformation capacity (through the use of component-specific force-reduction factors). These procedures are activated, however, only if the diaphragm is first found to be deficient in the Screening phase (Tier 1). It is principally this gap (the disjointedness of the Screening and Evaluation phases for flexible diaphragms) that this study is intended to address.

Development of Proposed Supplementary Methodology

Fundamental to the supplementary methodology is the development of a basic index of probable diaphragm performance, and a method of including that index in the evaluation procedure. To characterize diaphragm performance, test data from previous diaphragm tests, performed by others, were reevaluated in the context of performance-based engineering. Data from previous studies initially designed to identify strength and initial stiffness of lumber diaphragms (Atherton [10], Johnson [11, 12, 13], and Stillinger [14]) and metal-deck diaphragms (Nilson [15], Luttrel [16], and Ellifritt [17]) were reevaluated to correlate deformation, strength, and damage.

Two key parameters were extracted from the test data: DDRs; and the measure of initial diaphragm rigidity, G'. The latter is related to shear rigidity, $A'G$,

$$G'B = A'G, \qquad \textbf{Equation 2}$$

where B is the diaphragm width in the direction of loading, A' is the effective shear area of the diaphragm, and G is the shear modulus of the diaphragm. The complex nature of most flexible diaphragms, whether constructed of lumber or metal deck, precludes the explicit definition of either a diaphragm shear modulus or an effective shear area. For that reason, G' is widely used and represents an effective quantity describing the shear rigidity of the diaphragm per unit width, in the direction of loading. In the case of metal-deck diaphragms, DDRs and stiffnesses were extracted at 40 % of the ultimate capacity of the diaphragm. That percentage is generally accepted as the load level at which metal-deck diaphragms begin to sustain measurable damage and exhibit nonlinearity in their load-displacement responses (Luttrel [16]). Diaphragm studies listed earlier suggested that lumber diaphragms exhibit similar behavior (incipient damage and nonlinearity) at roughly 50 % of their ultimate capacity; stiffnesses and DDRs were therefore extracted at that load level. These two quantities were extracted and compared for the two types of diaphragms.

Figure 1 shows that, for metal-deck diaphragms, there is an inverse relationship between G' and the diaphragm drift ratio at 40 % of the ultimate load. The dotted curve in that figure is,

$$DDR_{40\% Pu} = \frac{2}{G'} \qquad \textbf{Equation 3}$$

where G' is in units of kips per inch and $DDR_{40\%\,Pu}$ is in units of percent. For wood diaphragms, Figure 2 shows a similar inverse relationship,

$$DDR_{50\% Pu} = \frac{1}{G'} \qquad \textbf{Equation 4}$$

Equation 3 and Equation 4 describe an important interrelationship between an intrinsic characteristic of a diaphragm (G') and its seismic performance (DDR at 40 % and 50 % of ultimate capacity). This implies that the level of deformation in a diaphragm at the onset of damage (yielding) is not purely kinematical, but it also depends on its stiffness. In an elastic-plastic steel-plate diaphragm, in contrast, yielding (damage) is purely kinematical, occurring at the same deformation (DDR) regardless of the stiffness of the diaphragm.

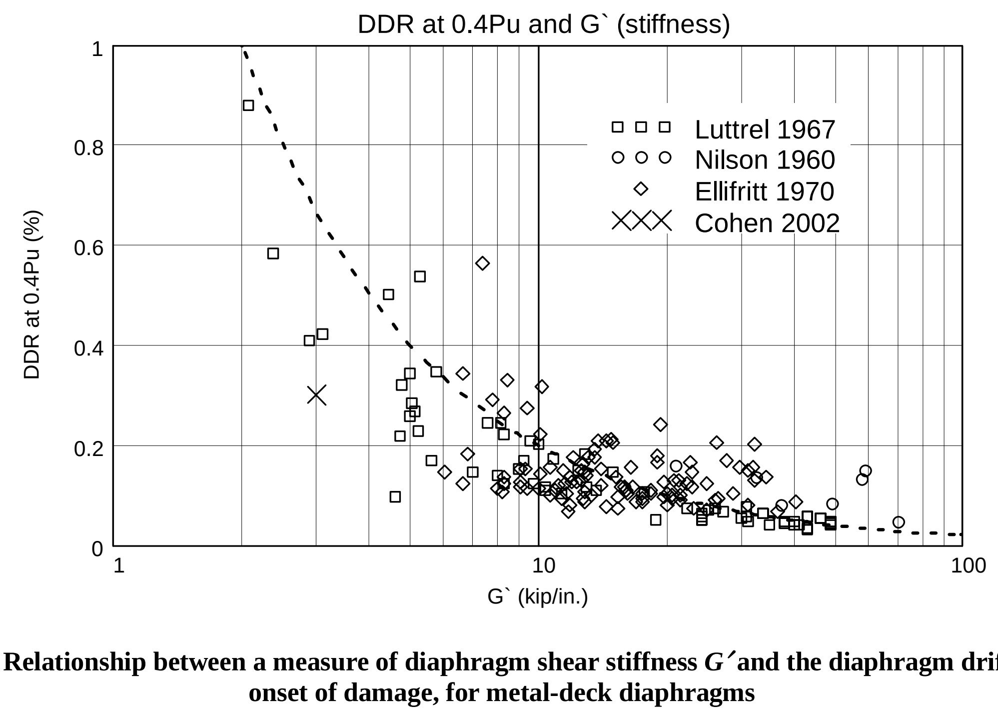


Figure 1 Relationship between a measure of diaphragm shear stiffness G' and the diaphragm drift ratio, at onset of damage, for metal-deck diaphragms

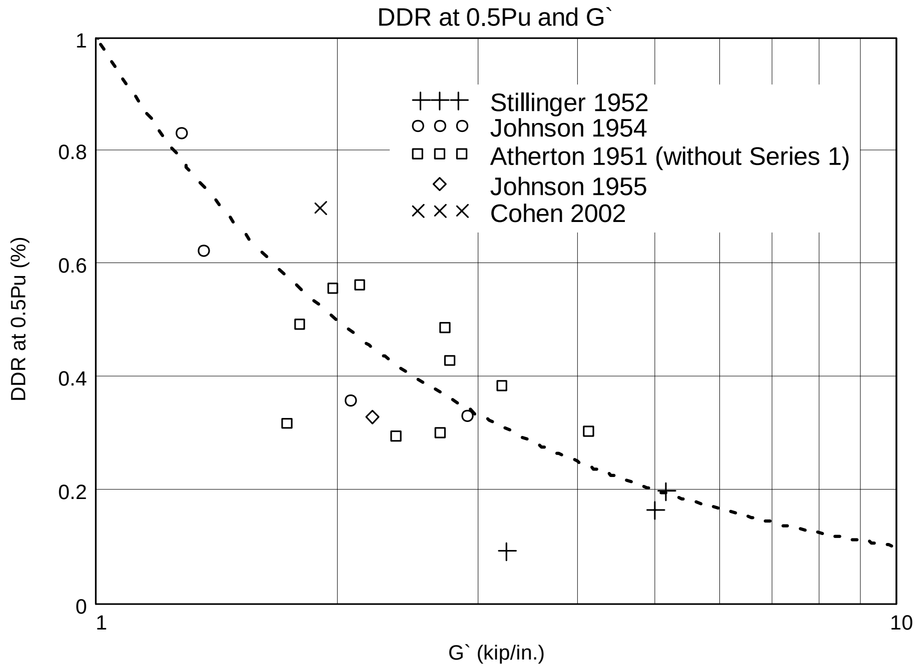


Figure 2 Relationship between a measure of diaphragm shear stiffness *G′* and diaphragm drift ratio, at onset of damage, for lumber sheathed diaphragms

The relationships of Equation 3 and Equation 4 make physical sense as well. The in-plane stiffness of these types of diaphragms depends on complex mechanisms that, for lumber diaphragms, chiefly derive from nailing patterns, nail sizes, and lumber sizes. For metal-deck diaphragms, they chiefly derive from welding patterns, weld sizes, deck thickness, side-lap fastener patterns, and deck profile. These same elements also contribute to diaphragm strength. For instance, the more nails in a lumber diaphragm or welds in a metal-deck diaphragm, the greater its strength.

The FEMA documents define three seismic performance levels: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). A design earthquake would cause little to no damage for IO; some damage but no immediate threat to human life for LS; and large amounts of damage but continued overall structural stability for CP. The relationships of Equation 3 and Equation 4 roughly define boundaries between the first and latter two performance levels (Figure 3); deformation levels at or below those described by the equations are consistent with IO, and levels above them are consistent with LS and CP.

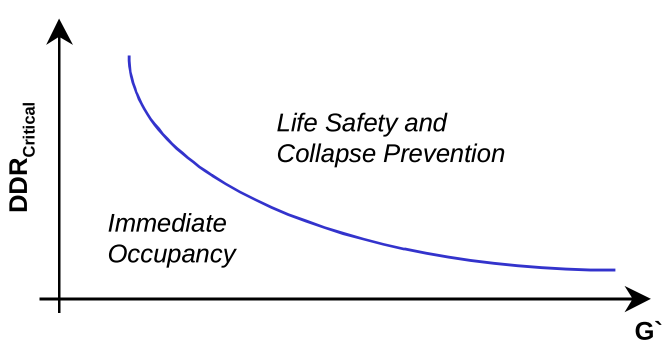


Figure 3 Link between FEMA Performance Levels and proposed methodology

Proposed Supplementary Methodology

Observations made during Phase 1: Behavior, the analysis tool developed in Phase 2: Analysis, and the diaphragm performance index and criterion developed in this phase were combined to form a supplementary seismic evaluation methodology. The supplementary methodology was designed to fill the potential gap in the FEMA 310 methodology and is outlined in Figure 4.

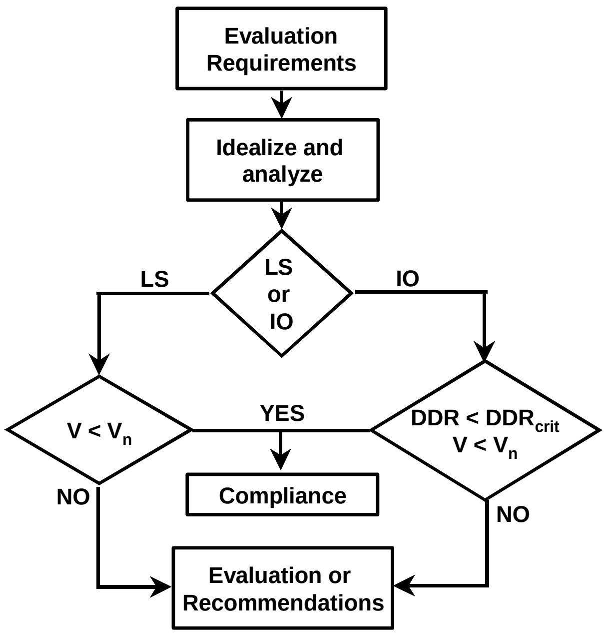


Figure 4 Basic organization of proposed supplementary methodology for FEMA 310

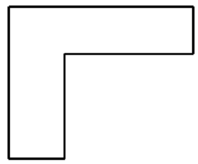
The methodology is presented step-wise using the example building plan configuration of Figure 5a.

1. **Define diaphragm systems.** Buildings with multiple flexible diaphragms should be described as a set of individual diaphragm systems. For example, a building with the plan of Figure 5a could be described as the collection of diaphragm systems in Figure 5b.
2. **Couple mass and assign stiffness to diaphragm degree of freedom.** Using the methods developed in Phase 2: Analysis, appropriate mass and stiffness values should be assigned to each diaphragm system. The mass coupled with each diaphragm system is one-half the total mass of the diaphragm itself, plus one-half the mass of any out-of-plane walls associated with response of the diaphragm. This is illustrated by the darkly shaded areas in Figure 5c. The deformed shapes of the diaphragm systems are approximated as sinusoids (Figure 5d). The in-plane stiffness consistent with this is,

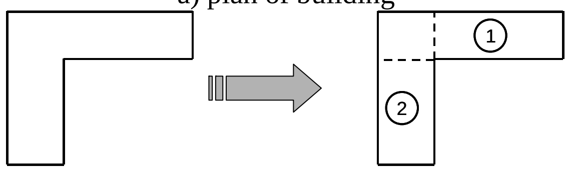
$$k = \frac{BG'}{L}\frac{\pi^2}{2}, \qquad \textbf{Equation 5}$$

   where, B is the diaphragm width and L is the diaphragm length.
3. **Calculate period of each diaphragm.** Treating each as an independent single-degree-of-freedom system, calculate a period for each diaphragm system (Figure 5e).
4. **Calculate response of each diaphragm.** Using appropriate loading criteria (for example, a response spectrum) calculate in-plane forces and diaphragm drift ratios for each diaphragm system (Figure 5e).
5. **Compare calculated responses with capacities.** For each diaphragm system, compare applied loads to known capacities. For Immediate Occupancy performance levels, also compare calculated diaphragm drift ratios to critical values (Equation 3 and Equation 4).

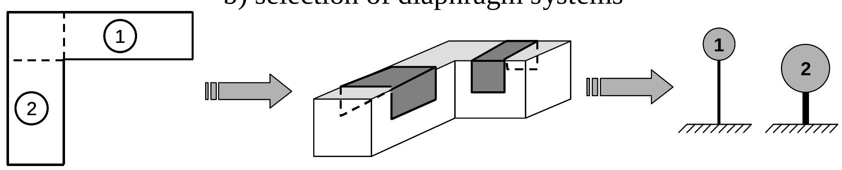
6. **Recommend further evaluation or rehabilitation.** Based on results of Step 5, proceed with evaluation as outlined in FEMA 310.



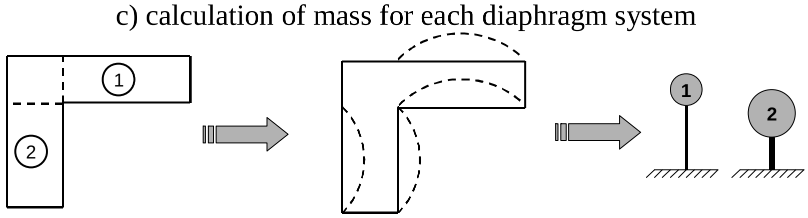
a) plan of building



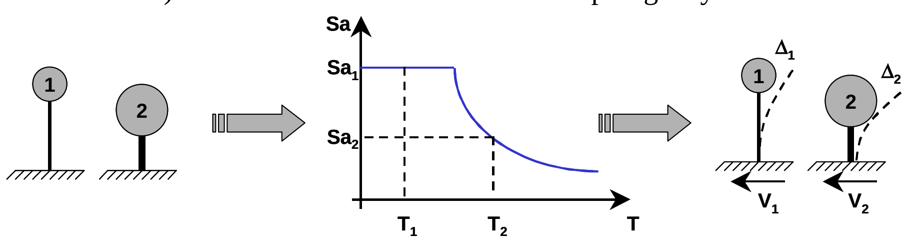
b) selection of diaphragm systems



c) calculation of mass for each diaphragm system



d) calculation of stiffness for each diaphragm system



e) calculation of response for each diaphragm system

Figure 5 Proposed Supplementary Methodology for FEMA 310

PHASE 4: APPLICATION

As the final phase of this study, four existing military-owned low-rise reinforced masonry buildings with flexible diaphragms were evaluated for seismic deficiencies. The buildings were evaluated using two methodologies: the existing methodology of the FEMA 310 document; and the supplementary methodology proposed in this paper. Results of the evaluations were compared with each other, and with the results from existing seismic evaluations of the same buildings performed by URS Greiner Inc. (San Francisco, CA) in the mid-1990s.

Selection of Buildings for Evaluation

In the mid-1990s, the US Army contracted URS Greiner to screen their existing building inventory in Ft. Lewis, Washington, for seismic deficiencies. They performed a facility-wide seismic screening of over 4000 structures. To simplify the evaluation of such a large number of structures, URS Greiner and CERL developed the hierarchal inventory-classification system outlined in Figure 6.

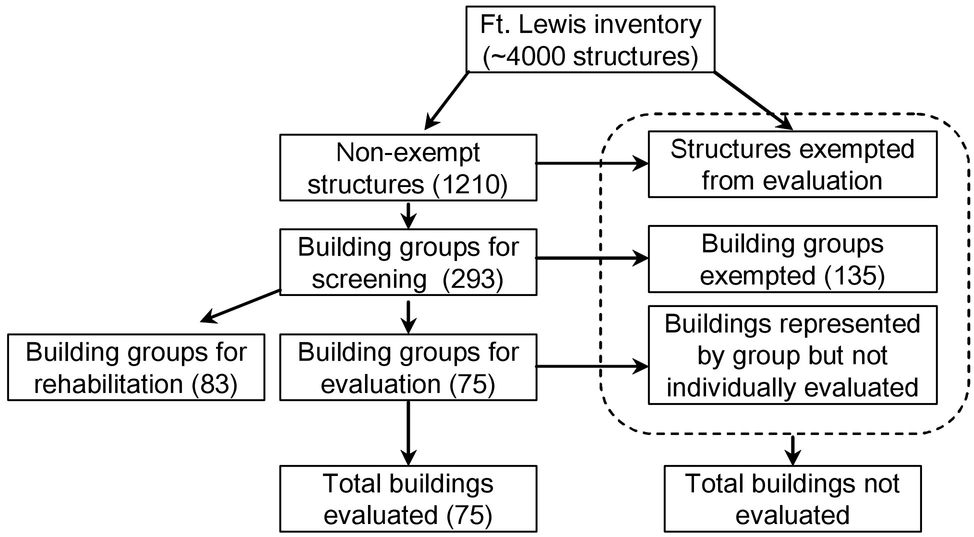


Figure 6 Hierarchy of building classification used by URS Greiner for Ft. Lewis, WA

Ft. Lewis comprises an inventory of over 4000 structures and of those, 1210 were classified as "non-exempt" and considered for seismic evaluation. Based on criteria established by CERL, the remaining inventory of 1210 buildings was divided into 293 "building groups." Each building group comprised a subset of the non-exempt inventory that shared key structural characteristics such as, among others, year of construction, primary structural system, and number of stories. One representative building from each building group was then selected for evaluation, and group-wide dispositions were based on that single evaluation. The number of buildings comprising each group ranged from only one to over 80.

Preliminary screening of the 293 building groups by URS Greiner Inc. determined that 135 building groups were exempted from further evaluation, 83 building groups were classified in need of rehabilitation without further evaluation, and 75 building groups were classified in need of further evaluation before assignment of disposition.

In this study, the entire non-exempt Ft. Lewis building inventory was screened to identify candidate buildings for evaluation. The inventory was specifically screened for the subset of one-story reinforced masonry buildings with flexible diaphragms using several simple criteria. Presented in the form of questions, those are:

1. Is the building one-storied?
2. Was the building built between the years 1950 and 1980?
3. Was the building already evaluated by URS Greiner Inc.?
4. Is the structure a low-rise masonry building with a flexible wood or metal-deck diaphragm (FEMA 310 Type RM1)?

Of the 1210 non-exempt buildings in Ft. Lewis, 186 buildings complied with Criterion 1; 97 buildings complied with Criteria 1 and 2; 17 buildings complied with Criteria 1, 2, and 3; and 7 buildings complied with all the criteria. A CERL selection process had previously identified an additional 3 buildings, for a total of 10 candidate buildings. Those 10 buildings were further compared with a set of six selection criteria to determine their applicability to this study. Presented in the form of questions, those are:

A. Are the diaphragms flexible?
B. Are the plan aspect ratios of diaphragms greater than one?
C. Are the walls constructed of reinforced masonry?
D. Are the general plan and vertical layouts of the building regular?

E. Are structural drawings available?

F. Is the building located near other potential candidate buildings?

The compliance of each building with the criteria set was assessed numerically. Numerical scores of zero, one, or two representing, respectively, increasing levels of compliance, were assigned to each criterion for each building. Then, overall compliance scores were calculated for each building. Table 1 summarizes results of that assessment; buildings are arbitrarily assigned numerical identifications.

Table 1 Numerical assessment of candidate buildings

Building	Criterion						Total score (Sum of A to F)
	A	B	C	D	E	F	
1	1	0	2	0	2	2	8
2	2	1	2	0	2	0	7
3	2	2	2	1	2	2	11
4	1	2	2	1	1	2	9
5	2	2	2	1	2	2	11
6	2	2	2	1	2	2	12
7	2	2	2	2	2	2	12
8	2	2	2	2	2	2	12
9	2	2	2	1	2	2	11

Based on this, the four highest-scoring buildings were selected for possible further evaluation; those were Buildings 3, 6, 8, and 9 (Figure 7). All four buildings have reinforced masonry CMU barrier walls and welded metal-deck roof diaphragms. Although they had the same compliance scores and the two buildings are very similar in construction, use, and configuration, Building 8 was selected rather than Building 7 because Building 8 has a larger diaphragm plan aspect ratio. Similarly, although it had the same compliance score as other buildings (for example, Buildings 9 and 3), Building 5 was not selected because it is two-storied.

Figure 7 Building 3, 6, 8, and 9 (clockwise from upper left)

Application of Existing Seismic Evaluation Methodologies

The four selected buildings were evaluated three times using two methodologies, FEMA 310 and the methodology developed in this study, for a total of 24 evaluations: once using seismicity consistent with Ft. Lewis, WA and a diaphragm stiffnesses consistent with as-built conditions; again using seismicity consistent with Ft. Lewis, WA but with a hypothetically reduced diaphragm stiffnesses (this is discussed next), and finally using seismicity consistent with San Francisco, CA and a diaphragm stiffnesses consistent with as-built conditions. The buildings were assumed to be founded on soil corresponding to Site Class D (stiff soil) and were evaluated at the Life Safety performance level. In addition to the evaluations performed as part of this study, URS Greiner evaluated the four selected buildings using site-specific seismicity consistent with Ft. Lewis, WA and the *Screening and Evaluation Procedures for Existing Military Buildings* (US Army Corps of Engineers 1995), which is based on FEMA 178 [18].

Diaphragms in the four selected buildings have unusually large in-plane stiffnesses compared to other typical metal-deck diaphragms. As an example, these diaphragms as-built (20 gage, 36/7 puddle welding, button-punched @ 18in. o/c; G′ = 60 kip/in.) are more than 10 times stiffer in-plane than those constructed using another typical configuration (for example, 22 gage, 36/3 puddle welding, button-punched @ 18in. o/c; G′ = 5 kip/in.). The evaluations presented here are intended to demonstrate gaps in the existing FEMA 310 methodology, rather than identify specific deficiencies in specific buildings. The four buildings were thus evaluated twice using seismicity consistent with Ft. Lewis, WA: once, using the nominal diaphragm stiffnesses (G′ = 60 kip/in.); and again using hypothetically decreased, but still typical, diaphragm stiffnesses (G′ = 5 kip/in.).

In the four evaluations performed by URS Greiner, only Building 9 was found deficient. This was due to insufficient shear transfer mechanisms between the diaphragm and the supporting masonry walls. Table 2 summarizes results of the 12 evaluations performed as part of this study.

Table 2 Dispositions of selected buildings using FEMA 310 procedures

Building	Screening (Tier 1)		Evaluation (Tier 2)
	Disposition	Deficiency	Disposition
Ft. Lewis, Washington (S_s = 1.2*g*, S_1 = 0.4*g*) **			
8	Compliant	-	-
9	Non-compliant	shear transfer from diaphragm to wall	Compliant
6	Non-compliant	shear transfer from diaphragm to wall	Compliant
3	Non-compliant	shear transfer from diaphragm to wall	Compliant
San Francisco, California (S_s = 2.0*g*, S_1 = 0.9*g*)			
8	Compliant	-	-
9	Non-compliant	shear transfer from diaphragm to wall	Compliant
6	Non-compliant	shear transfer from diaphragm to wall	Non-compliant
3	Non-compliant	shear transfer from diaphragm to wall	Compliant

** Identical results for cases of evaluations using reduced diaphragm stiffness

The goal of these evaluations was to verify suspected gaps in the FEMA 310 methodology, and to provide a comparison to both the URS Greiner methodology and the supplementary methodology proposed in this study. For seismicity consistent with Ft. Lewis, WA and San Francisco, CA, Screening (Tier 1) indicated deficient diaphragm-to-wall shear-transfer mechanisms in three of the four buildings (Buildings 9, 6, and 3). In those cases, the metal-deck diaphragm itself was connected to the shear walls only through the joist-to-wall connections; that condition was considered deficient. Modern construction of metal-deck diaphragms requires that the metal deck itself be continuously connected to all shear walls along the diaphragm perimeter. This is generally accomplished using continuous structural angles anchored along the tops of perimeter shear walls, and intermittently welded or otherwise connected to the metal deck. Further deficiency-specific Evaluation (Tier 2) of the joist-to-wall connections, indicated that, for seismicity consistent with Ft. Lewis, WA, the connections were actually sufficient to transfer the diaphragm shear. For seismicity consistent with San Francisco, CA, Evaluation (Tier 2) indicated that connections in Building 6 were deficient due to insufficient shear capacities of anchor bolts connecting the roof framing to the masonry walls.

Application of Proposed Supplementary Methodology

The four selected buildings were also evaluated for Life Safety using the supplementary methodology proposed by this study. These evaluations emphasized three items not currently addressed by the Screening phase of FEMA 310:

1. accurate calculation of building period;
2. comparison of diaphragm shear force demand and capacity; and
3. in the case of Immediate Occupancy performance, comparison of diaphragm deformation demand and capacity.

Table 3 and Table 4 summarize results of the evaluations. Table 3 shows that fundamental periods calculated by the FEMA 310 Screening Phase (Tier 1) provisions are generally significantly shorter than those calculated by the proposed supplementary methodology. The FEMA 310 Evaluation Phase (Tier 2), however, includes a period expression developed specifically for low-rise buildings with flexible diaphragms (FEMA 310 Equation 4-1, FEMA 356 Equation 3-8). References listed in the beginning of this paper show that equation to be reasonably accurate for flexible diaphragm systems and would calculate periods similar to those calculated by the proposed supplementary methodology.

Table 3 Fundamental periods calculating using FEMA 310 Screening (Tier 1) provisions and proposed supplementary methodology

Building	Fundamental Period, *sec*	
	FEMA	Proposed supplementary methodology
Nominal diaphragm stiffness (20 gage, 36/7 puddle welds, button-punched at 18in. o/c)		
8	0.17	0.13
9	0.12	0.13
6	0.13	0.22
3	0.14	0.14
Reduced diaphragm stiffness (22 gage, 36/3 puddle welds, button-punched at 18in. o/c)		
8	0.17	0.47
9	0.12	0.47
6	0.13	0.77
3	0.14	0.50

Table 4 shows that evaluations using the proposed supplementary methodology found six of the twelve evaluations to be non-compliant with a Life Safety performance level (fourth column). Each pair of diaphragm demand and capacity values listed in the table (second and third columns) represents the response of one of the diaphragm systems used to idealize the building (Step 1 in the Proposed Supplementary Methodology). For instance, the table shows that Building 6 was idealized with three diaphragm systems. The table also shows, that DDR demands were greater than DDR capacities in many cases. This requirement, according to the proposed supplementary methodology, applies to Immediate Occupancy performance levels only, and is hence not considered further.

Table 4 Dispositions of buildings using proposed supplementary methodology

<table>
<tr><th>Building</th><th>Diaphragm Shear, plf (Demand/Capacity)</th><th>DDR, % (Demand/ Capacity)</th><th>Disposition</th></tr>
<tr><td colspan="4">Ft. Lewis, WA
Nominal diaphragm stiffness
(20 gage, 36/7 puddle welds, button-punched at 18in. o/c)</td></tr>
<tr><td>8</td><td>915 / 780</td><td>0.05 / 0.03</td><td>Non-compliant</td></tr>
<tr><td>9</td><td>473 / 780</td><td>0.05 / 0.03</td><td>Compliant</td></tr>
<tr><td>6</td><td>769 / 780
528 / 780
462 / 780</td><td>0.09 / 0.03
0.06 / 0.03
0.05 / 0.03</td><td>Compliant</td></tr>
<tr><td>3</td><td>397 / 780</td><td>0.04 / 0.03</td><td>Compliant</td></tr>
<tr><td colspan="4">Ft. Lewis, WA
Reduced diaphragm stiffness
(22 gage, 36/3 puddle welds, button-punched at 18in. o/c)</td></tr>
<tr><td>8</td><td>915 / 391</td><td>0.66 / 0.40</td><td>Non-compliant</td></tr>
<tr><td>9</td><td>473 / 391</td><td>0.66 / 0.40</td><td>Non-compliant</td></tr>
<tr><td>6</td><td>486 / 391
478 / 391
447 / 391</td><td>0.68 / 0.40
0.67 / 0.40
0.65 / 0.40</td><td>Non-compliant</td></tr>
<tr><td>3</td><td>386 / 391</td><td>0.54 / 0.40</td><td>Compliant</td></tr>
<tr><td colspan="4">San Francisco, CA
Nominal diaphragm stiffness
(20 gage, 36/7 puddle welds, button-punched at 18in. o/c)</td></tr>
<tr><td>8</td><td>1386 / 780</td><td>0.08 / 0.03</td><td>Non-compliant</td></tr>
<tr><td>9</td><td>716 / 780</td><td>0.08 / 0.03</td><td>Compliant</td></tr>
<tr><td>6</td><td>1166 / 780
799 / 780
699 / 780</td><td>0.13 / 0.03
0.09 / 0.03
0.08 / 0.03</td><td>Non-compliant</td></tr>
<tr><td>3</td><td>601 / 780</td><td>0.07 / 0.03</td><td>Compliant</td></tr>
</table>

Significance of Evaluations and Results

Table 5 compares results from all the evaluations and shows that the proposed supplementary methodology found a significantly greater number of buildings to be deficient, at the Life Safety performance level, than either the FEMA 310 or URS Greiner methodologies. In the deficient cases, shown in Table 4, diaphragm shear demands exceeded diaphragm shear capacities. As demonstrated in Phase 1: Behavior (Cohen [2, 3, 4, 5, 6]) and in other studies (Nilson [15], Luttrel [16], and Ellifritt [17]), metal-deck diaphragms exhibit stiffness degradation and sustain significant damage at load levels greater than about 40 % of ultimate capacity, and exhibit instability, and stiffness and strength degradation at load levels greater than ultimate capacity. Responses calculated using the proposed supplementary methodology therefore imply that, during an earthquake with spectral ordinates consistent with those of the appropriate FEMA 310 response spectrum, diaphragms of the deficient buildings would at least sustain significant damage, likely lose significant strength and stiffness, and possibly lose overall diaphragm action.

This conclusion indicates that a significant gap indeed exists in the FEMA 310 Screening Phase (Tier 1) assessment of low-rise reinforced masonry buildings with flexible diaphragms. The supplementary methodology proposed in this paper is intended to fill that gap.

Table 5 Dispositions of selected buildings from evaluations

Building	URS Greiner (FEMA 178)	FEMA 310			Proposed Supplementary Methodology		
		Ft. Lewis	Ft. Lewis reduced stiffness	San Francisco	Ft. Lewis	Ft. Lewis reduced stiffness	San Francisco
8	C	C	C	C	NC	NC	NC
9	NC	C	C	C	C	NC	C
6	C	C	C	NC	C	NC	NC
3	C	C	C	C	C	C	C

C: Compliant / Sufficient
NC: Non-compliant / Deficient

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

The study was completed in four phases: behavior, analysis, evaluation, and application. The former two (behavior and analysis) are summarized in the beginning of this paper and are reported in detail elsewhere (see references).

Summary of Results and Conclusions from Evaluation and Application Phases

Shaking-table and quasi-static testing demonstrated the affect of diaphragm flexibility on building behavior and also the need for an analysis tool to characterize that behavior. To meet this need, simple 2DOF and SDOF idealizations of low-rise buildings with flexible diaphragms were developed, justified, and verified. The analysis tool was enhanced with the reevaluation of data from and then integrated into the predominant existing seismic evaluation methodology, FEMA 310, to improve its assessment of low-rise reinforced masonry buildings with flexible diaphragms. Critical reviews of the existing evaluation document identified potential gaps in that methodology; it did not sufficiently characterize or assess the seismic performance of these types of buildings.

To fill potential gaps in the FEMA 310 evaluation procedures, a supplementary seismic evaluation methodology was developed and integrated into the existing methodology. First, data from several previous flexible-diaphragm testing programs, performed by others, were reevaluated in the context of performance-based engineering. These data were reevaluated for critical levels of deformation and damage, and then related to specific seismic performance levels described in the FEMA documents. The reevaluations demonstrated that simple describable relationships exist between an intrinsic measure of diaphragm stiffness and critical levels of diaphragm deformation (diaphragm drift ratio). These relationships, in combination with the SDOF analysis tool, comprise the proposed supplementary methodology.

To assess and validate the usefulness of the proposed supplementary methodology, four military-owned low-rise reinforced masonry buildings with flexible diaphragms were evaluated for seismic deficiencies. The four buildings were evaluated 28 times using different combinations of three evaluation methodologies, two levels of seismicity, and two hypothetical diaphragm stiffnesses. The three methodologies were: (performed by URS Greiner in 1997) the US Army *Screening and Evaluation*

Procedures for Existing Military Buildings (1995); the current FEMA 310 methodology; and the supplementary methodology proposed as part of this study.

The evaluations substantiated the hypothesis that the existing FEMA 310 methodology, while complete in many ways, does not sufficiently identify potential diaphragm deficiencies in low-rise reinforced masonry buildings with flexible diaphragms. It was shown that out of 16 buildings evaluated using the existing methodologies (URS Greiner and FEMA 310) only 2 were found to be non-compliant/deficient. In contrast, out of 12 buildings evaluated using the proposed supplementary methodology, 6 were found to be non-complaint/deficient. The proposed supplementary methodology was therefore ultimately shown to be needed, effective, and simple.

Synthesis of Study Elements to Meet Study Objectives

Four phases of study (behavior, analysis, evaluation, and application) were synthesized and the basic study objective was realized. The predominant methodology for the seismic evaluation of these types of buildings was critically assessed and consequently enhanced. Auxiliary to this but equally significant, was the development of a consistent overall approach to the characterization of seismic performance of these types of buildings. Data from shaking-table testing (Phase 1: Behavior) were integrated with dynamic analysis (Phase 2: Analysis) to develop a simple analysis tool used to characterize the seismic behavior of these types of buildings. Data from quasi-static testing (Phase 1: Behavior) and from previous testing programs (Phase 3: Evaluation) were integrated with the diaphragm drift ratio concept to develop a simple seismic performance tool (Phase 3: Evaluation) used to relate seismic behavior with seismic performance. Together, the analysis and performance tools form a methodology for the consistent and accurate seismic evaluation of these types of buildings (Phase 3: Evaluation and Phase 4: Application). The methodology uses the same basic set of tools and criteria for modeling, analysis, and evaluation, regardless of the low-rise reinforced masonry building with flexible diaphragm being considered. Use of such a methodology by the structural engineering technical community will emphasize consistency and reliability in the evaluation of these types of buildings.

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