Quantification of Seismic Performance Factors for Buildings Incorporating Three-Dimensional Construction System

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SUMMARY

The paper presents application Of the FEMA P695 Methodology for 3D panel construction system including sixteen archety1Es of different configurations. The FEMA P695 Methodology is intended for design of new structural systems. It provides a rationale method of evaluating the seismic performance factors (SPFs). including the response modification coefficient (R-factor). the system overstrength factor (Qo), and deflection amplification factor (Cd), Of a proposed seismic force-resisting system. The Methodology incorporates certain Steps for the development, analysis, and the probabilistic assessrnent Of collapse risk of a consistent number Of buildings. The Methodology uses nonlinear analysis techniques to characterize nonlinear static and dynamic behavior of a proposed seismic force-resisting system: another advantage of the Methodology is that it explicitly considers uncertainties in ground motion, modeling, design, and test data. It is shown that the Methodology can be directly applied to a series of 3D panel index archetypes, with a significant number of archetypes satisfying the criteria of the Methodology.

Keywords: FEMA P695 Methodology, seismic performancefactors (SPF's), 3D panel construction system

1. INTRODUCTION

The concept Of panel construction includes those buildings where the majority of structural cornponcnts are standardized and produced in plants in a location away from the building, and then transported to the site for assembly. These components are manufactured by mass production industrial methods in order to build a large number of buildings in a short time at low cost. Urban residential buildings of this type are usually one to ten stories high. There are different types of pancl.s such ms Structural Insulated Panels (SIPS) and Oriented Strand Board (OSB), preca.st concrete, and rnetal panels, as shown in Figure 1.1.

a)b)

Figure 1.1. Typical panels; (a) OSB; (b) precast concrete; (c) metal, after Mashal, (2011)

The 3D construction system is a new cost-effective construction system that uses prefabricated 3D panels for construction of residential and commercial buildings. The 3D panels consist of an expanded polystyrene (EPS) core with a thickness ranging from 40 to 100 mm sandwiched between two planeparallel welded wire mesh sheets (cover meshes). Inclined diagonal wires are welded between the cover mesh's line wires and go through the EPS core as shown in figure 1.2. This results in a lightweight, three-dimensional truss system with a high inherent stiffness (EVG, 2005). The system allows the weight of the building to bc reduced by 40 %, while meeting all structural requirements of the building system; fast construction, high shear stiffness and thermal insulation arc the important properties of 3D panel buildings.
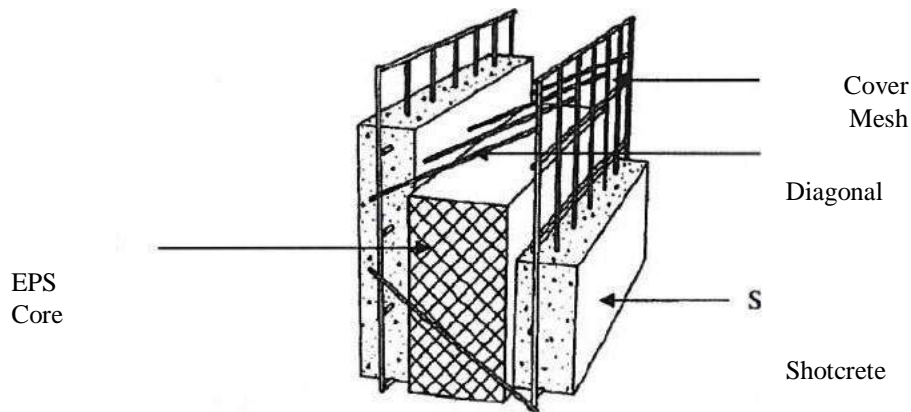

Figure 1.2. Typical 3D panel cross-section, after EVG, (2005)

The 3D panels have been used in construction of low rise buildings in many areas around the world. figurc 1.3 shows the construction methods for 3D panel buildings. This includes erection of 3D walls/slabs and placement of shotcrete.

a)b)c)

Figure 1.3. Construction with 3D panels; (a) erection Of wall panels; (b) slab panel; (c) shotcreting. after Mashal (2011)

This paper covers the application Of the FEMA P695 methodology (FEMA P695, 2005) for 3D panel construction system including sixteen archetypes Of different configurations. More than $5,000$ dynamic analyses including pushover analyses were carried out in order to simulate the actual behavior Of 3D panel system, and further explore its dynamic characteristics. The study calibrates the experimental test data with the analytical results in different steps of the methodology. In summary, the paper provides global seismic performance factors (SPFs), including the response modification coefficient (R-factor), the system overstrength factor and the deflection amplification factor (Cd), of 3D panel construction system for inclusion in model building codes.

2. OVERVIEW OF THE FEMA P695 METHODOLOGY

The FEMA P695 Methodology (FEMA, 2009) provides a rationale method of evaluating the seismic performance factors (SPFs), including the response modification coefficient (R-factor), the system overstrength factor and deflection amplification factor (Cd), of a proposed seismic force-resisting system. Definitions of SPFs arc provided in Figure 2.4, where BEIR is the roof drift corresponding to a design base shear V and ö is the assumed roof drift of the yielded system corresponding to the design earthquake ground motions. The Methodology incorporates certain steps for the development, analysis, and the probabilistic assessment of collapse risk of a consistent number ofbuilding models.
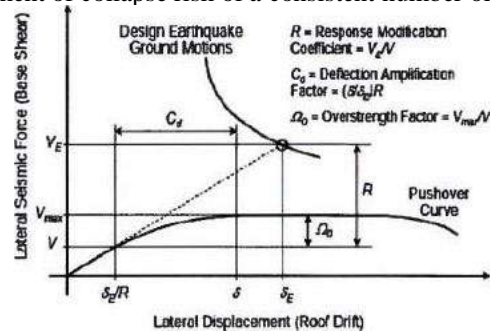

Figure 2.1. Illustration of seismic performance factors (R, and Cd) as defined by the Methodology, after FEMA P695 (2009)

ne FEMA P695 Methodology uses nonlinear analysis techniques to characterize nonlinear static and dynamic behavior of a proposed seismic force-resisting system; another advantage of the Methodology is that it explicitly considers uncertainties in ground motion, modeling, design, and test data. The steps to bc followed in the Methcxlology are shown in Figure 2.2.
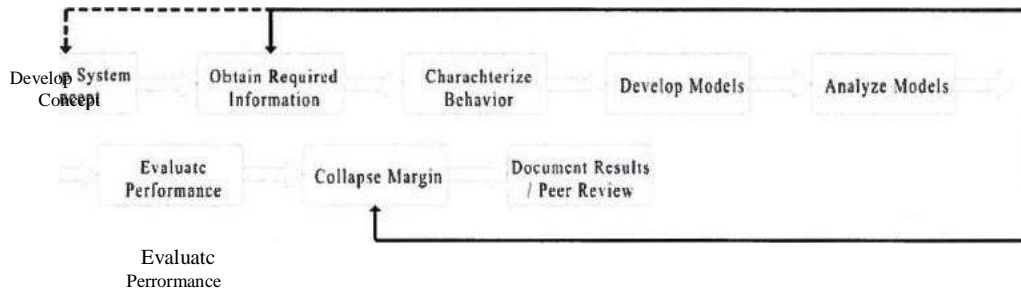
Figure 2.2. Process for quantitatively establishing and documenting seismic performance factors, after FEMA P695 (2009)

2.1. Obtain Required Informaüon

Required system information includes design requirements, experimental data, and system testing.
The development of design requirements are done within the context of the seismic provisions of ASCE/SEI 7-10, (ASCE, 2010) and other applicable standards. The provisions should address the design and detailing Of the proposed seismic force-resisting system, inelastic deformation capacity. component design and detailing, seismic performance factors (R, Cd), drift limits, height limits, and seismic usage restrictions.

Design requirements for 3D panel construction are based on criteria specified in applicable sections of the latest edition of ASCE/SEI 7-10 (ASCE, 2010), ASCE/SEI 41-06 (ASCE, 2007), ACI 318-05 (ACI, 2005), Osterreichisches Normungsinstitut (ÖNORM) series B-4200 and B4600 (Austrian building code), and Deutsches Institut für Normung (DIN) 1045 (German building Code).
There are experimental data of nonlinear static and dynamic testing of the 3D panels, such as tests conducted by Kabir Rezaifar et al. (2008), Kabir et al. (2006), and Rezaifar et al. (2008). Figure 2.3 illustrates the failure nEchanism and the load deflection hysteresis loops for a combined 3D wall panel and steel moment frame in a cyclic loading test respectively.
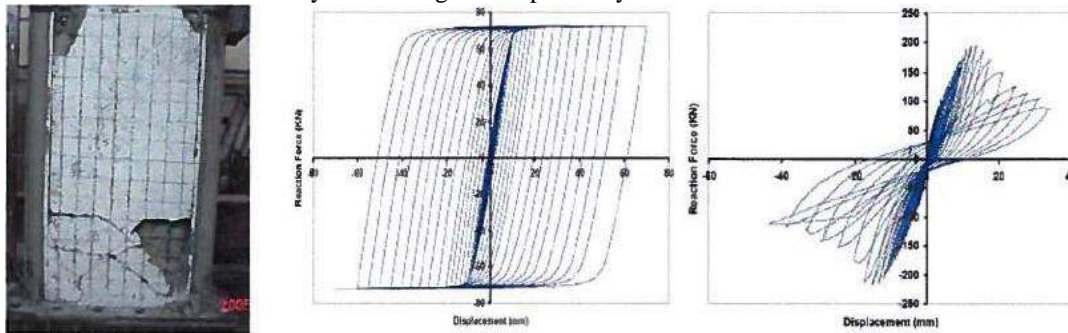
a)b)c)

Figure 2.3. a) Crack pattern and failure mechanism for combined 3D panel slkCimen at the later stages of cyclic loading; b& c) Load-deflection hysteresis energy loops for steel moment frame and combined system respectively after Kabir, (2006)

Completeness and robustness characteristics for the quality of the test data is rated medium for the 3D panel system, since most of the important general testing issues of Section 3.5.2 of the FEMA P695 Methodology are addressed adequately in the testing program. Experimental evidence is sufficient so that all, or nearly all, important behavior aspects at all levels (from 3D panel material to system) are

generally understood, and the results can be used to quantify the parameters which can significantly influence the design requirements and analytical modeling.

2.2. Archetype Development

The behavior of the 3D panel seismic-force-resisting system is investigated through the use of archetypes. According to the FEMA P695 Methodology, an archetype is a prototypical representation of a seismic-force-resisting system. Archetypes are intended to reflect the range of design parameters and system attributes that are used to bridge the gap between collapse performance of a single specific building and the generalized predictions of behavior needed to quantify performance for an entire clmss of buildings.

Up to four 2D archetype performance groups are considered performance can be adequately evaluated using two basic configurations of 3.1m and 5.1m transverse bay width respectively. Table 2.2 shows the properties for each of the archetype designs used in this evaluation. As mentioned earlier all sixteen buildings (archetypes) are designed for seismic design category Drnax. The mapped values of short, and I-second spectral acceleration, site coefficient and design parameters considered for the design of 3D panel archetypes are shown in Table 2.1.

Table 21. suOf desiters for seismic desi cateD

short period	$(SS) = 1.5$	$(SMS) = 1.5$	$(S_{DS}) = 1.0$
I-second period	$= 0.6$	$(SMI) = 0.9$	$(SDI) = 0.6$

2.3. Nonlinear Model Development

'he Methodology requires selection of trial values of the response modification coefficient, R, displacement amplification coefficient, Cd, and overstrength factor, for the preparation of index archetype designs. The 3D panel archetypes are initially classified as reinforced mmsonry shear wall system, this system is covered under bearing wall seismic force-resisting system of Table 12.2.1 of ASCFJSEI 7-10. The trial values for R and are 3.5, and 2.5 respectively. The trial value for the displacement amplification coefficient, Cd is assumed to be equal to R, or 3.5.

Archetype	No. or Stories	Key Archetype Design Parameters						
		sc Desi Criteria						
		Framing (Gravity Loads)	SBC	R			vtw	
					T (sec)	T , (sec)		
Perrnrntnnr«• Group NO, PG-I (Short Period, 3.1 •n Transverse Bay Size)								
ce WallD3.50 0.15540.0655								1.50
		High (Space gearing Wall)		3.50	0.1554	0.06635	o.Z86	1.50
		High (S pace Pearing Wal	D	3.50	0.2618	0 17142	O_28G	50
		High (Space wall)		3.50	0.2618	0.18184	o.Z86	50
Performance Group No, PG-2 (Lang Period. 3.1 m Transverse B" Size)								
		High (Space pear Ing Wall)		3.50	0.411	o. 44024	0.286	1.50
A-IO		High (Space Beanng Wal	D	3.50	0.411	0.51821	0.286	1.50

								Table 2.2. 3D panel building archety1E structural design properties
A-13	6	High (Space earing Wall)	3.50	0.5964	o. 55445	0.286	1.50	
		High (Spare Bearing Wall)	3.50	0.8764	0.8518		-027	
Performance Group NO. PG-3 (Short Period, 5.1 m Bay Sin)								
Pearl Wall) D3.50 0.15540.06928						o.Z86	1.50	
X3		Hlgli (Space Bearing Wal l)	3.50	0.1554	0.07018	0.286	1.50	
		H (Space Bea'Ing	3.50	0.2618	0.18095	0.286	50	
		High (Space Pear ing Wall)	3.50	0.2618	0.19195		50	
Performanr• Group No. PG-a (l ang Period. 5.1 m Transverse Ray Sin)								
	4	High (Space Pearing Wall)	3.50	0.441	0.46357		1 50	
A-12		High (Spare gearing Wall)	3.50	0.411	0.54572	0.286	1.50	
A,,15 A-19	10	(Spare Rearing Wall) i all	3.50 3.50	0.5964 0.8764	0.57815 0.88458	0.286 0.440	1.50 1.027	

To the extent possible, the 3D panel index archetype models are directly simulating all significant deterioration modes that contribute to collapse behavior. The general purpose structural analysis program SAP 2000 (Computers and Structures, Inc., 2011) is used to build the nonlinear models of the 3D panel archetypes. In SAP2()00, a frame element is modeled as a line elenEnt having linearly elastic properties and nonlinear force-displacement characteristics of individual frame elements are modeled as hinges represented by a series of straight linc segments. A generalized force-displacement characteristic of a non-degrading ' frame element (or hinge properties) in SAP2000 is shown in Figure 2.4.
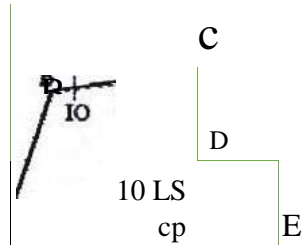

Figure 2.4. Force-displacement characteristic Of a frame element in SAP 2000, after Habibullah and Pyle, (1998)

The points labeled as 10, LS and CP in Figure 2.4 are used to define the acceptance criteria for the assigned hinge, they correspond to the Immediate Occupancy, Life Safety and Collapse Prevention respectively.

In order to simulate the collapse Of thc 3D panel archetypes in the subsequent sections, it is important to limit the story drifts for the simulated collapse modes of the archetypes. It is more likely for the 3D panel structures to behave ms cantilever shear wall system, in order to comply with ASCE/SEI 7-10 drift requirements, and with the fact that all archetypes in this paper are assigned an occupancy category Il; we will limit the maximum allowable drift to 1% for the 3D panel archetypes. Therefore, any drift higher than 1% will be counted as the collapse of the system. The 1% allowable drift will be utilized for the collapse assessment of the archetypes, after carrying the incremental dynamic analyses in later sections.

The Shear strength of the concrete is considered as a non-simulated collapse mode for one-story archetypes. The consideration of non-simulated mode is necessary to establish reasonable median

collapse intensities for one-story 3D panel archetypes. It is also assumed that there is no other nonsimulated collapse mode, rather than the one mentioned above.

2.4. Nonlinear Analysis

The nonlinear static (pushover) and dynamic (time-history) are performed for thc archetypes-The ground motion record set utilized for dynamic analyses of the archetypes in this paper are the 22 FarField record set of the FEMA P695 Methodology. The Methodology requires use of the Far-Field records sct for collapse evaluation for index archetypes designed for Seismic Design Category (SDC) D . The Far-Field record set includes component pairs of 22 horizontal ground motions (44 ground motions in total). The record set docs not include the vertical component of ground motion. Figure 2.5(b) shows the 44 individual acceleration response spectra (i.e., 22 records, 2 components each) for the Far-Field record set, the median response spectrum, and spectra representing onc standard deviation and two-standard deviations above the median. Figure 2.5(a) pmvides the median spectrum of the Far-Field record set anchored to maximum and minimum MCE response spectra of SDC B, C and D, at a period for I-second.
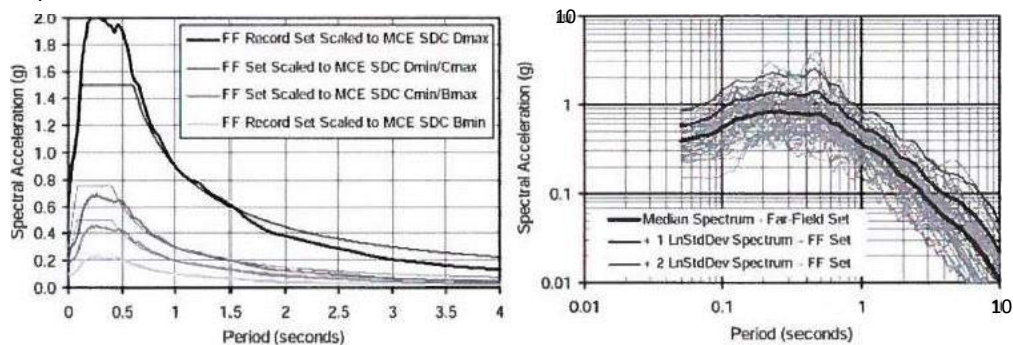

a)b)

Figure 2.5. (a) Median spectrum Of the Far-field record set anchored to different MCE resrx)nse spectra; (b) Far-Field record set response spectra median spectrum of the Far-Field record set anchored to different MCE response spectra, after FEMA P695, (2009)

The nonlinear static analyses are performed according to Section 3.3.3 of ASCE'SEI 41-06 as shown in Figure 2.6.

Nonlinear dynamic (response history) analyses are required under the gravity load and aforementioned 22 pairs of ground motion. The nonlinear dynamic analyses are necessary to establish the median collapse capacity, Scr, and collapse margin ratio (CMR) for the index archetype models. The median collapse capacity is obtained using incremental dynamic analysis (Vamvatsikos and Cornell, 2002) for each archetype model. While performing incremental dynamic analysis (IDA), the ground motions are scaled to increasing intensities until the structure reaches a collapse point. Ground motion intensity, ST, is defined based on the spectral intensity of the Far-Field record set, measured at the fundamental period of the structure.
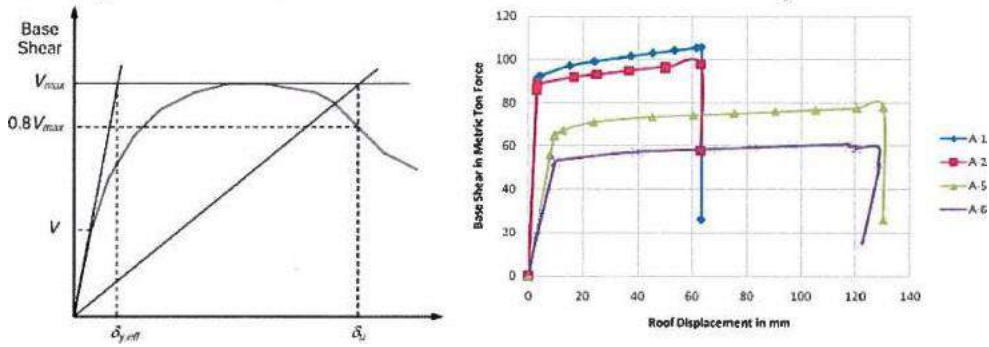
a)b)

Figure 2.6. (a) Idealized nonlinear static pushover curve; (b) the nonlinear static (pushover) curves for performance group I from FEMA P695, (2009)

Figure 2.7 (a) illustrates a sample IDA response plot of spectral acceleration versus maximum story drift ratio for archetype No. 5. In Figure 2.7 (a) each point corresponds to the results Of one nonlinear dynamic analysis of an index archetype model subjected to one ground motion record that is scaled to one intensity level. Using the collapse data from the IDA curves, a collapse fragility curve can be defined through a cumulative distribution function (CDF), which relates the ground motion intensity to the probability Of collapse (Ibarra et al., 2002). Figure 2.7(b) shows the collapse fragility curve.
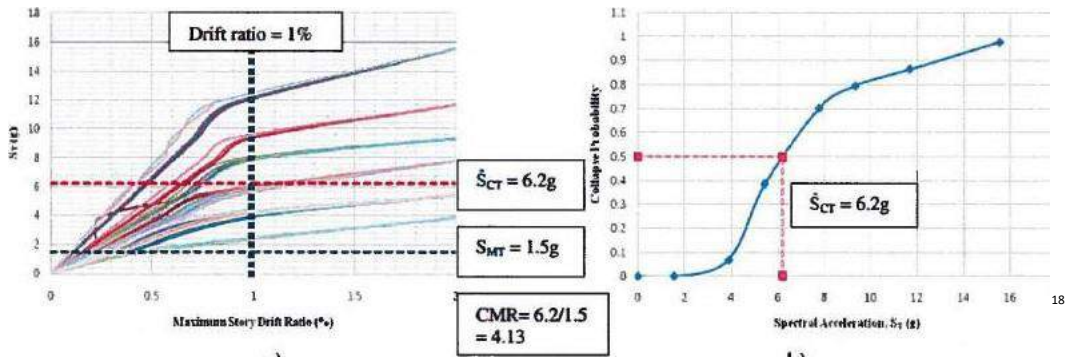
a)b)

Figure 2.7. (a) IDA curve for Archetype NO. 5: (b) Fragility curve, after Mmshal. (2011)

More than 5000 nonlinear dynamic analyses were performed in the SAP 2000 software (CSI, 2005) for all sixteen index archetypes, as discussed above, All 3D panel archetypes are considered as collapsed if the inter-story drift ratio exceeds I %, (or equals 30 mm lateral displacement). This failure criterion (simulated collapse mode) is considered for all archetypes in this paper except archetype I through 4. For archetype 1 through 4, the shear strength in the concrete is considered as a nonsimulated collapse mode. It was observed that in order to reach 1% drift, much greater ground motion intensity is needed for these archetypes, which was unrealistic. Therefore, consideration of a nonsimulated mode was necessary.

The Methodology requires identification of the median collapse intensity, Scr, which can be obtained by scaling all the Far-Field record set to the MCE intensity, SMT, and then increasing the intensity until just over one-half (22 ground motions) of the records cause collapse. The collapse margin ratio can be calculated from Eqn. 2.1.

CMR = Scri(2.1)

2.5. Performance Evaluation

This step utilizes the results of nonlinear static and dynamic analyses of Section 2.3 to evaluate the performance for the 3D panel seismic force-resisting system. The process for performance evaluation includes assessment Of the acceptability of a trial value of the response modification coefficient, R, determining values Of the system overstrength factor, Q), and finally the deflection amplification factor, Cd.

According to the FEMA P695 Methodology, the Adjusted Collapse Margin Ratio (ACMR) for each index archetype, i, can be calculated from Eqn. 2.2.

$$ACMR_i = SSF_i \times CMR_i \quad (2.2)$$

In Eqn. 2.2, SSF denotes the spectral shape factor for each index archetype. It is showed that the collapse margin ratio can be significantly influenced by the frequency content (spectral shape) of the ground motion record set, (Baker and Cornell, 2006). Spectral shape factors, SSF, for index archetypes can be calculated in term of their respective fundamental periods, T, and the period-based ductility, PT. There are many sources of uncertainty which can significantly influencc the collapse capacity of index archetype. These sources are record-to record (β_{RTR}), design requirements (h), test data and modeling (β_{MDL}) uncertainties. Appropriate values and equations for each type of uncertainty and then the total uncertainty (β_{TOT}) are given in FEMA P695 Report (2009).

After calculating the total system uncertainty for each index archetype, the acceptable values for adjusted collapse margin ratio are established based on Table 7.3 of FEMA P695, (2009). $ACMR_{10\%}$ and $ACMR_{20\%}$, based on total system collapse uncertainty and values of acceptable collapse probability, taken as 10% and 20% respectively.

In order to evaluate the response modification coefficient, R, for the index archetypes, the trial value of the response modification coefficient, R = 3.5, must be found acceptable for all performance groups. The governing performance group for the response modification factor, R, is the one With the smallest average value of ACMR. According to the FEMA P695, the acceptable performance is achieved when, for each performance group, adjusted collapse margin ratios, ACMR, for each index archetype meet the following two criteria:

$$ACMR_i \geq ACMR_{10\%} \quad (2.3)$$

$$ACMR_i \geq ACMR_{20\%} \quad (2.4)$$

Table 2.3 presents the summary for the final acceptancc of the trial value of R = 3.5. After applying the above criteria, it was concluded that performance groups I and 3 passes the Methodology criteria. while performance groups 2 and 4 fail. Hence, the results from performance groups I and 3 are used to evaluate the seismic performance factors for 3D panel system.

As mentioned before, the governing performance group for the response modification factor, R, is the one with the smallest average value of ACMR, in this case performance group 3.Therefore, the trial value Of response modification factor, R=3.5, meets the collapse performance objective of the FEMA P695 Methodology, and can be used in designs of one and two story 3D panel buildings. Alternative lower values of R could be used to make the performance groups 2 and 4 pass the criteria of the Methodology.

The average value of archetype overstrength, Q, which is obtained from nonlinear static (pushover) analyses of index archetypes, can be calculated for each performance group. The value of the system overstrength factor, m, for use in design of 3D panel system should not be taken as less than the largest average value of calculated archetype overstrength, Q, from any performance group, it should be noted that only performance group 1 and 3 are considered for the calculation of the overstrength factor, Q,

for the 3D panel system. The overstrength factor for the 3D panel system is bmsed on the largest averagc value of overstrength, Q, for both performance groups I and 3. Table 2.4 summarizes the calculation for the overstrength factor, Q, for the 3D panel system.

Table 2.3. Summ of the final acce tance of the trial res nse modification coefficient, R

PG. No	Average value of ACMR	Average value of	ACMR$_{10\%}$ & ACMR$_{20\%}$	Governing PG. for final value Of R
PG-I	3.77	0.55	2.02 ACMRI	No
PG-2	1.23	0.56	1.59 (ACMR	No
PG-3	2.88	0.56	2.05 ACMR	Yes
PG-4	0.92	0.56	1.6 (ACMRm)	No

Table 2.4. Summ of the overstren th factor, Q, for the 3Del s m

PG. No	Average value Of Q	Trial value of Q	Governing PG for final value of Q
PG-I	10.0	2.5	Yes
PG-3	8.2	2.5	No

According to the FEMA P695 Methodology, the system ovemt_rength factor, Q), need not exceed 1.5 times the response modification coefficient, R. A practical limit on the value of Q) is about 3.0, consistent with the largest value Of this factor specified in Table 12.2.1 of ASCE/SEI 7-10 for all current approved seismic force-resisting systenus.

Limiting system overstrength to Q) = 3.0, as specified in ASCE/SEI 7-10, was considered necessary for practical design considerations. After specifying an overstrength factor of 3.0 for the 3D panel system compared to the overstrcngth factor obtained from the PG-I (Q = 10.0), there can be risk of potential problems for the seismic design of collector elements in 3D panel buildings. For instance, in ca.se of large earthquakes, the 3D walls have very high strength and rigidity, while the collector elements in the building such as 3D slabs behave more like weak links, which can fail before transferring higher loads to the 3D walls. Thus, an overstrcngth factor of 3.0 for design of 3D panel elements cannot be conservative for large earthquakes.

From the FEMA P695 Methodology, the deflection amplification factor, Cd, can bc obtained using Eqn. 2.5.

$$C_d = R / B_I \quad (2.5)$$

where in Eqn. 2.5,
BI = numerical coefficient ms set forth in Table 18.6. I of ASCE/SEI 7-10 for
effective damping, ßl, and period, T.
β_l = component of effective damping of the structure due to the inherent dissipation of energy by elements of the structure, at or just below the effective yield displacement of the seismic forceresisting system, Section 18.6.2.1 of ASCE/SEI 7-10.

An inherent damping of 5 percent of critical is assumed for the 3D panel buildings, this analogy is also consistent with the FEMA P695 Methodology. Referring to Table 18.6-1 of ASCEJSEI 7-10, a corresponding value of the damping coefficient, Bl =1.0 is obtained. Thus, the value Of Cd is equal to the value of R for the 3D panel buildings.

CONCLUSIONS

The 3D panel system is a new construction system that could be a good alternative for the traditional masonry and wood construction. Previous experience and application of the 3D panel system in different regions that were subjected to large earthquakes show that the 3D panel buildings are strong enough to resist large earthquakes. The calculation for flexural, shear and compression forces for 3D panel sections can be done in almost the same way as calculation for the reinforced concrete structural elements. Walls made of 3D panels can be designed as a cantilevered shear wall system with the failure mechanism mainly caused by the formation of cracks near the openings (such as doors or windows). Low-rise 3D panel buildings have great reserve strength (large values of overstrength factor, Q) for severe earthquakes. The FEMA P695 Methodology can be applied on a series of 3D panel index archetypes. It was observed that the 3D panel archetypes with 4 stories and more do not pass the FEMA P695 acceptance criteria for the collapse assessment. The final recommended values for the SPFs for one and two-story 3D panel buildings are as follows: R = 3.5, Cd = 3.5, '20 = 3.0, Using a lower R-value could make four story prototypes pass the Methodology.

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