Response Modification Factor
for Lightweight Steel Panel-Modular Structures

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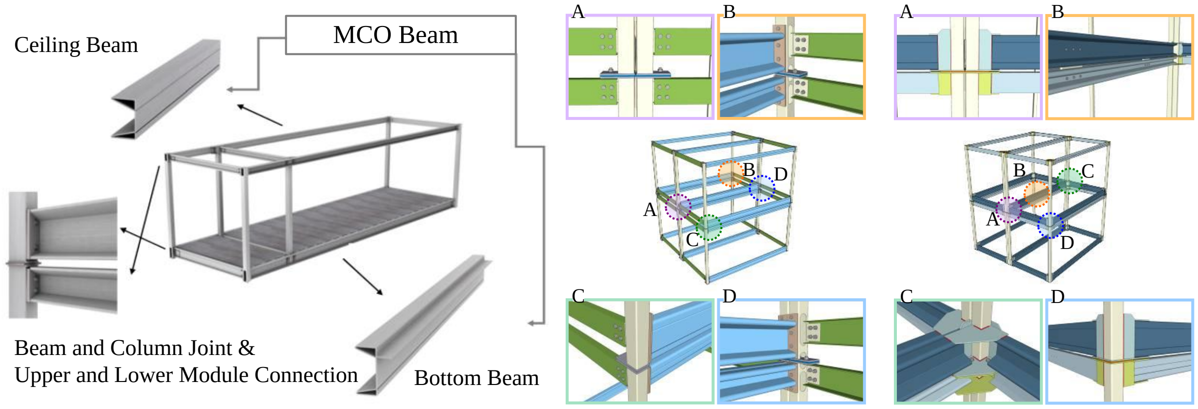
SUMMARY:
This study proposes how to estimate a response modification factor for a lightweight steel panel-modular system which has not been clarified in current building codes. As a component of the response modification factor, an over-strength factor and a ductility factor were drawn from the nonlinear static analysis curves of systems modelled on the basis of performance tests. The final response modification factor is then computed by modifying the previous response modification factor with a MDOF (Multi-Degree-Of-Freedom) base shear modification factor considering the MDOF dynamic behaviours. The results of computations of the structures designed with a dual- frame system, ranging from two-story to five-story structures, produce a value of 4 estimated as the final response modification factor for a seismic design. A value of 5 is considered as the upper limit of the number of stories.

Keywords: Response Modification Factor, Lightweight Steel Panel, Modular Structure, Dynamic Analysis

1. INTRODUCTION

In recent years, the efficiency of modular structures has been appraised in terms of the ability to reduce the construction time of the building sites. This is true for projects such as schools, army barracks, and refugee camps. The modular structure applied to this study was composed of MCO beams (Modular Construction Optimized Beams) manufactured by a roll-forming method in order to improve the workability and reduce the cost. From a full-scale test of modular bottom slabs with MCO beams, it was proved that premature local buckling and torsional deformation due to their use of thin plates and their sectional asymmetry could be prevented. For the joints of each MCO beam-column, a bracket connection with rectangular plates welded at the sides of the beam and column was considered to improve the poor capacity of the semi-rigid connections of the end-plate with a minimum number of bolts.



(a) MCO beams (b) End-plate connection (c) Bracket connection

Figure 1.1. Modular frames with MCO beams and types of connection

Although the joints of beam-column and upper-lower modules using MCO beams are welded, the modular structures of frames have insufficient lateral resistance and are expected to have side-sway failure mechanism due to their strong beam-weak columns.

To overcome these systematic vulnerabilities, a LSP (Lightweight Steel Panel) -modular system was developed based on the previous research as shown in Fig. 1.2. LSPs are building wall components composed of 2mm thick outer steel plates and 0.7mm thick inter-corrugated steel plates, relying on soldering connections. The behaviour of lightweight steel panels as a flexural link wall causes plastic hinges on the upper and lower part of the lightweight steel panels before the yielding of the modular columns. Therefore, the LSP-modular system shows stable behaviour until the columns of the modular frame yield, which enhances the strength and stiffness of the entire frame system.

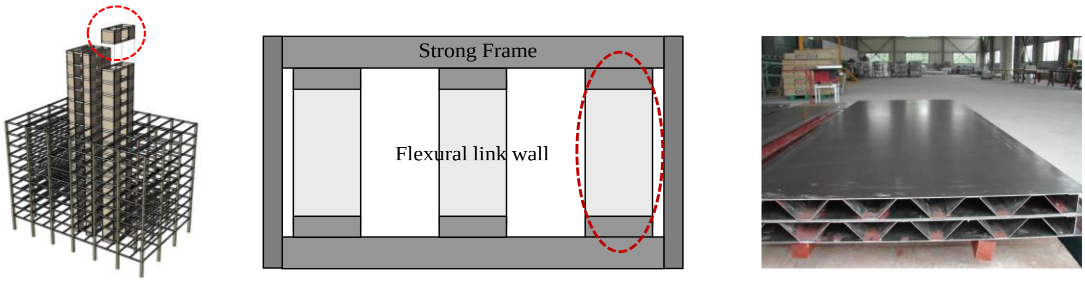


Figure 1.2. Concept of LSP-modular System

To utilize this developed LSP-modular system as one of earthquake resistant systems, a response modification factor for seismic designs is required. Thus far, the response modification factors in the Korean Building Code - Structural (KBC 2009) have been regulated according to the structural type of system. Also, for a new or unspecified system, the code recommends the use of the response modification factor of an existing system which displays similar structural behaviour. However, this recommendation has an uncertain theoretical validation because it is based simply on a comparative analysis of the energy dissipation capacity as obtained from an experimental observation and data.

Therefore, the primary purpose of this study is to show how to determine the response modification factor of a newly developed LSP-modular system through the fundamental seismic characteristics from theoretical and analytical approaches. First, the components of the response modification factor are drawn from nonlinear static analyses performed on the basis of the structural test results. MDOF (Multi-Degree of Freedom) dynamic response modification is then determined through the concept of the MDOF base shear modification. In particular, for a single laminated LSP-modular system without outer moment resisting frames the number of possible laminated stories is limited to five.

2. ESTIMATION OF THE STATIC RESPONSE MODIFICATION FACTOR

2.1. Components of the Response Modification Factor

A response modification factor as a design factor to account for economy, seismic risk, and nonlinear behaviour in seismic designs was proposed for the first time in the paper ATC 3-06 issued by the Applied Technology Council in 1978. Later, the response modification factor considering over-strength, ductility and damping characteristics was proposed by researchers of the University of California-Berkeley. Also, the response modification factor, composed of the over-strength, ductility and redundancy factors, was suggested in ATC-19 and ATC-34. More recently, a means of calculating the response modification factor using the CMR (Collapse Margin Ratio) was proposed in ATC-63. In this study, a static response modification factor R_{static} is estimated by determining the product of the over-strength factor R_O and the ductility factor R_μ as in the following equation:

$$R_{static} = R_O \times R_\mu \qquad (1)$$

2.1.1. Over-strength factor

To asses the over-strength factor for a LSP-modular system, the three components discussed below, regulated in the IBC (International Building Code), are applied in this research. Design over-strength arises from the process of section design, including such factors as the minimum reinforcement ratio and the limit of deflection to control the lateral displacement. Strain hardening and plastic behavior after system yielding lead to material over-strength and system over-strength, respectively.

$$R_D = \frac{F_1}{F_E / R} \tag{2}$$

$$R_M = \frac{F_2}{F_1} \tag{3}$$

$$R_S = \frac{F_3}{F_2} \tag{4}$$

$$R_O = R_D \times R_M \times R_S \tag{5}$$

Here, the term F_E / R is the design load and F_1 is the first yield base shear. F_2 is the actual force which arises at first yielding and F_3 is the ultimate load of systems.

2.1.2. Ductility factor

The ductility ratio μ can be determined by the relationship between the maximum displacement Δ_u and yield displacement Δ_y as drawn from nonlinear static analyses.

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{6}$$

Here, the yield displacement is obtained by controlling the inner areas of nonlinear static analyses curves equal to those of bilinear curves according to FEMA 356. Also, although the strength and stiffness of entire system deteriorate after roof drift of 4%, the ultimate displacement is estimated at a roof drift of 2% to obtain a conservative response modification factor allowing for laminated systems with strong beam-weak columns. Finally, the ductility factor can be calculated from the relationship between the ductility ratio and ductility factor according to the period, as suggested by Newmark and Hall in ATC-19.

$$\begin{aligned} &T < 0.03 &&: \; R_\mu = 1.0 \\ &0.12 \le T \le 0.5 &&: \; R_\mu = \sqrt{2\mu - 1} \\ &T \ge 1.0 &&: \; R_\mu = \mu \end{aligned} \tag{7}$$

(a) Components of the over-strength factor (b) Test setup and results for a LSP-modular system

Figure 2.1. Components of over-strength factor

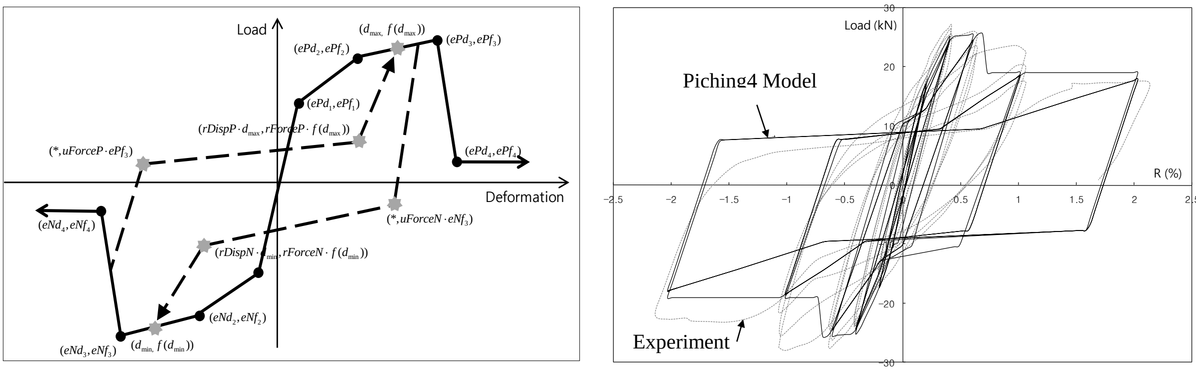
2.2. Nonlinear Static Analysis

2.2.1. Applied numerical model

In this study, numerical analyses have been performed using OpenSees, in which built-in subroutines were developed to account for special hysteretic behavior. Fig. 2.2 (b) shows the results of the flexural performance test for a LSP with a width of 400mm subjected to cyclic loading. In the tests, specific yield lines were shown due to the local buckling of the external thin plates, which resulted in a connection failure and detachment from the internal corrugated steel plates. Along these yield lines, all specimens experienced strength degradation and a pinching effect due to cyclic loading. A nonlinear analysis considering the pinching effect, stiffness and strength degradation can be performed using the Pinching4 model of the program. Fig 2.2 (a) shows the general concept of load-deformation in the pinching4 model. The elements behave on a predefined skeleton curve and the pinching effect is represented by the variables *rDisp*P, *rForce*P, and *uForce*P. The analysis functions provided in OpenSees are as follows:

$$\begin{aligned} k_i &= k_0 \cdot (1-\delta k_i) \\ \delta k_i &= (gK1 \cdot (\tilde{d}_{\max})^{gK3} + gK2) \leq gKLim \\ \tilde{d}_{\max} &= \max\left[\frac{d_{\max i}}{def_{\max}}, \frac{d_{\min i}}{def_{\min}}\right] \end{aligned} \tag{7}$$

Here, def_{max} is defined as a function of the displacement history, while def_{max} and def_{min} are defined as positive and negative deformations, respectively, at failure. Based on the experimental results, the values 0.1, 0.5, 0.1 and 0.9 were used for the variables *gK1*, *gK2*, *gK3*, and *gKLim*, respectively. Also, the values of *rDisp*P, *rForce*P, and *uForce*P were 0.3, 0.5, and 0.4, respectively. The obtained load-displacement relationship curves, buckling phenomena, the peak resistances in push and pull directions, as well as the pinching effect were quite similar to those determined experimentally as shown in Fig. 2.2 (b). Therefore, the proposed nonlinear cyclic analysis scheme is able to adequately predict the behaviour of LSPs.



(a) Definition of the Pinching4 material model (b) Comparison of the test results with the pinching4 model

Figure 2.2. Applied model for numerical analyses

2.2.2. Design of a prototype structure

To evaluate an appropriate response modification factor and the number of stories of the LSP-modular system as an independent structure without outer main frames, mid-low rise LSP-modular systems with a story height of 4m and a span length of 6 m were designed from a two- to five- story system. The modular frames were designed so that they have a wider cross-section area of the beams compared to that of the columns in order to take into account the characteristic that the beams of modular frames should be superposed within the multi-story frame. In addition, because the structural type of these frames was not specified in the design code, they were designed so that the modular frames resist 25% of the design lateral load and so that the LSPs resist the rest of the design lateral load. Live loads were assumed equal to 28 kN/m for both typical floors and the roof. Dead loads, comprising the self-weight of the steel structures, were evaluated as 40 kN/m for typical floors and the roof. For the earthquake design process, an equivalent static analysis was used. The initial response modification factor R=4.5, the sub-soil class S_b, the area factor A=0.11, and the importance factor I_E=1.2 were assumed. The member profiles of the analyzed frame are shown in Table 2.1.

Table 2.1. Member profiles of the analyzed frames

Frame Type	story	Column	Beam	The number of LSP
two-story	2	175×175×11/7.5	200×200×8/12	2
	1	175×175×11/7.5	200×200×8/12	3
three-story	3	175×175×11/7.5	200×200×8/12	1
	2	175×175×11/7.5	200×200×8/12	3
	1	208×202×10/16	244×252×11/11	2
four-story	4	175×175×11/7.5	200×200×8/12	2
	3	175×175×11/7.5	200×200×8/12	4
	2	208×202×10/16	244×252×11/11	2
	1	208×202×10/16	244×252×11/11	6
five-story	5	175×175×11/7.5	200×200×8/12	2
	4	175×175×11/7.5	200×200×8/12	6
	3	208×202×10/16	244×252×11/11	2
	2	208×202×10/16	244×252×11/11	4
	1	244×252×11/11	248×249×8/13	8

2.3. Estimation of the Static Response Modification Factor

Table 2.2 and Table 2.3 show the over-strength factors and ductility factors obtained from the curves of the nonlinear static analyses. Finally, the static response modification factor can be estimated as 6.79, 7.15, 7.49 and 7.85 for two-, three-, four- and five- story frames, respectively.

Table 2.2. Over-strength factors of the analyzed frames

Frame Type	F_E / R (kN)	F_1 (kN)	F_2 (kN)	F_3 (kN)	R_D	R_M	R_S	R_O
two-story	62.33	83.96	83.96	162.50	1.35	1.00	1.95	2.63
three-story	69.33	86.57	86.57	189.82	1.25	1.00	2.19	2.74
four-story	78.22	92.64	92.64	232.05	1.18	1.00	2.50	2.95
five-story	85.92	94.80	94.80	270.52	1.10	1.00	2.85	3.14

Table 2.3. Ductility factors of the analyzed frames

Frame Type	Δ_y (mm)	Δ_u (mm)	μ	R_μ
two-story	60	160	2.67	2.58
three-story	92	240	2.61	2.61
four-story	126	320	2.54	2.54
five-story	160	400	2.50	2.50

3. ESTIMATION OF THE DYNAMIC RESPONSE MODIFICATION FACTOR

3.1. Dynamic Effects of the Response Modification Factor

In order to apply the obtained static response modification factors directly to real MDOF structures which are governed by several translational modes, the validity of the factors must be scrutinized through comparisons with the response modification factors considering dynamic behaviour. In this study, the research of Nassar et al. was applied to compute the response modification factors while allowing for dynamic behaviour. Nassar et al. used a story ductility ratio μ_{story} defined as the maximum dynamic inter-story displacement $\delta_{dynamic}$ normalized by the inter-story static yield displacement δ_y drawn by the ratio of the SDOF base shear $V_{by,SDOF}(\mu_{allow})$ and the story stiffness k_{story}. The story ductility ratio μ_{story} becomes a basic concept of the MDOF base shear modification factor M_{MDOF} which is derived by comparing the strength demand of inelastic MDOF systems with that of corresponding SDOF systems. Fig. 3.1 indicates that there is a difference in A between the MDOF ductility demand and the SDOF allowable ductility if the MDOF base shear $V_{by,MDOF}(\mu_{demand})$ is equal to the modified SDOF base shear $V_{by,SDOF}(\mu_{allow})$. The value of M_{MDOF} can be determined by the following quantificational procedure regarding this difference.

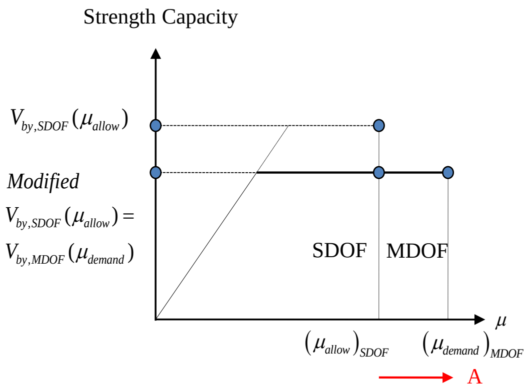


Figure 3.1. Comparison of the MDOF ductility demand with the SDOF allowable ductility

3.2. Estimation of the Story Ductility Ratio

The LSP-modular model was designed according to the KBC 2009 equivalent static lateral load pattern, i.e., the amounts of elastic member stiffness in each story were tuned so that, under the KBC 2009 equivalent static load pattern, the inter-story drift in every story is identical, resulting in a straight-line deflected shape. Structures of 2, 5, 10 and 20 stories with periods ranging from T=0.35, 0.69, 1.16 and 1.95 sec were studied, in which the modular systems contained 3 LSPs per story. Moreover, as shown in Fig 3.2, the inelastic strength of a corresponding SDOF system $V_{by,SDOF}(\mu_{allow})$ can be obtained from the SDOF constant ductility response spectrum drawn by the adopted ground motion for each analyzed structure. In this study, the SDOF constant ductility response spectra with a damping ratio of ζ=5% and allowable ductility μ_{allow} =1, 2, 4 and 8 were drawn for the 10 earthquake ground motions shown in Table 3.1. The SDOF inelastic strength $V_{by,SDOF}(\mu_{allow})$ for each allowable ductility value can be determined from this constant ductility spectrum by selecting the average values of design accelerations corresponding to each natural period. Finally, the MDOF dynamic inter-story displacement $\delta_{dynamic}$ can be provided by conducting dynamic analyses of the earthquake ground motions using the OpenSees nonlinear analysis program. The story ductility demand μ_{story} is then computed for each MDOF system.

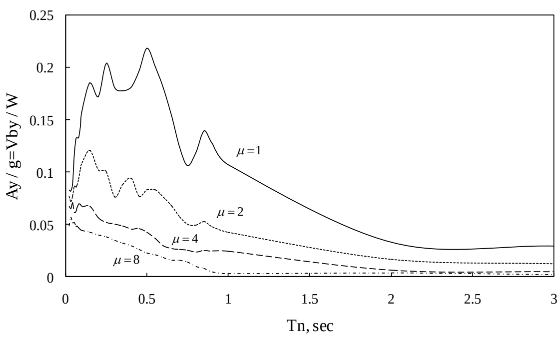


Figure 3.2. Constant ductility response spectrum for El Centro ground motion

Table 3.1. Earthquake ground motions with 10% probability of excess in 50 years

Earthquake Records	Magnitude	Scale factor	PGA (g)
Imperial Valley, 1940, El Centro	6.9	2.01	0.4613
Imperial Valley, 1979, Array #05	6.5	1.01	0.3939
Imperial Valley, 1979, Array #06	6.5	0.84	0.3017
Landers, 1992, Barstow	7.3	3.20	0.4214
Landers, 1992, Yermo	7.3	2.17	0.5201
Loma Prieta, 1989, Gilroy	7.0	1.79	0.6658
Northridge, 1994, Newhall	6.7	1.03	0.6785
Northridge, 1994, Rinaldi RS	6.7	0.79	0.5340
Northridge, 1994, Sylmar	6.7	0.99	0.5698
North Palm Springs, 1986	6.0	2.97	1.0198

As shown in Fig. 3.3, the systems designed according to a higher allowable ductility μ_{story} for the first story has a tendency to increase, which signifies that the story ductility of the first story should not exceed the allowable ductility when assessing the MDOF base shear modification factor.

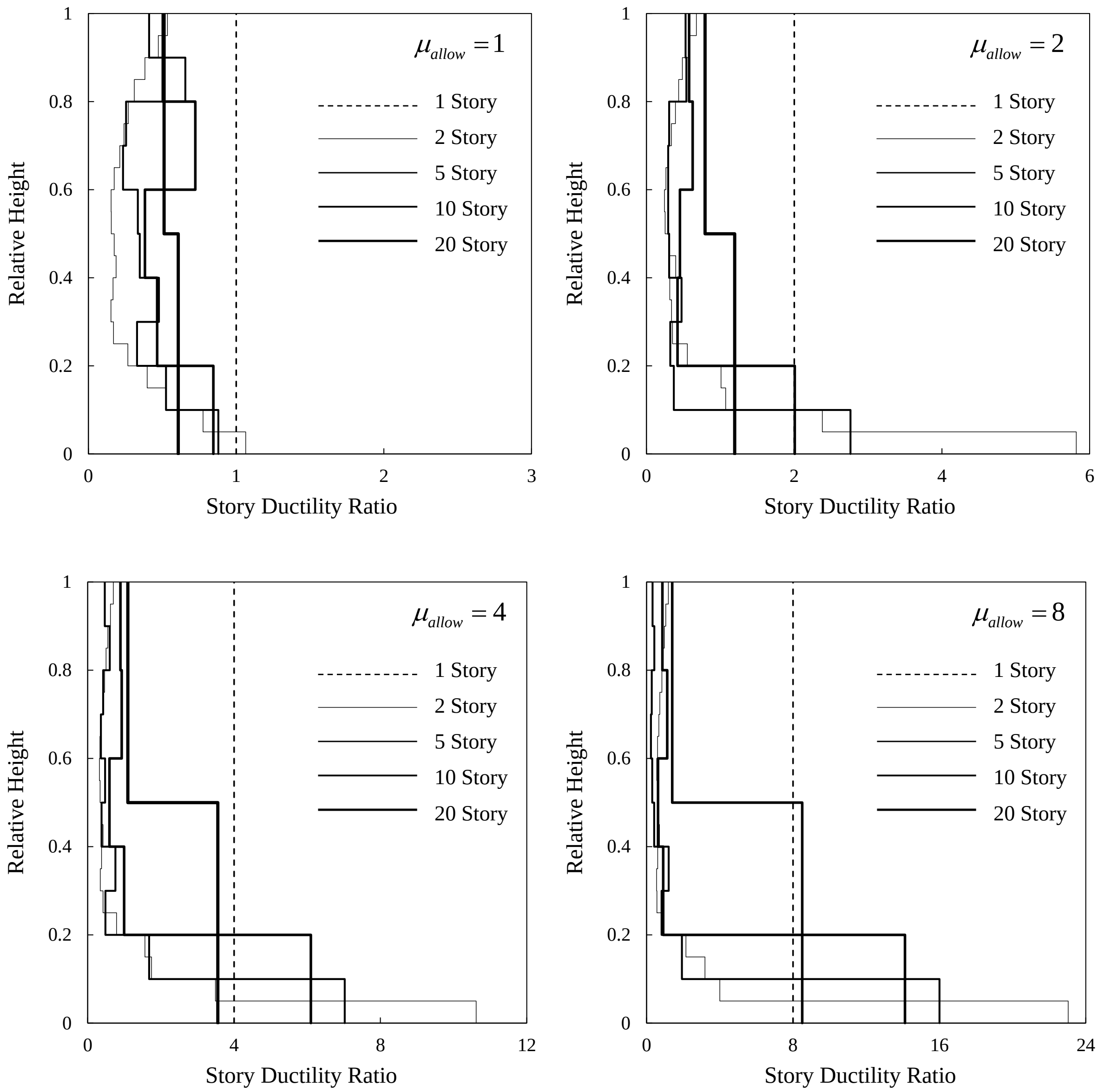


Figure 3.3. Variations of the story ductility ratio and relative height for each allowable ductility

3.3. Estimation of the MDOF Modification Factor

To limit the first story ductility demand lower than the allowable ductility, which is generally the highest value relative to the other stories, the MDOF base shear modification factor is determined as follows:

1) Computation of the SDOF normalized yield strength, $\bar{V}_{by,SDOF}$ for μ_{allow}=1, 2, 4 and 8

$$\bar{V}_{by,SDOF} = V_{by,SDOF}\left(\mu_{allow}\right) / V_{by,SDOF}\left(\mu_{allow} = 1\right) \tag{8}$$

The horizontal axis of the solid lines in Fig. 3.4 shows matching levels of ductility demand, as the SDOF and MDOF normalized yield strengths are equal. Therefore, the MDOF normalized yield strength $\bar{V}_{by,MDOF}$ for the ductility demands of μ_{demand} =1, 2, 4 and 8 can be estimated through a linear interpolation of each graph.

2) Computation of the MDOF base shear strength $(V_{by}/W)_{MDOF}$ for μ_{allow} =1, 2, 4 and 8

$$\left(V_{by}/W\right)_{MDOF} = \bar{V}_{by,MDOF} \times \left(V_{by}/W\right)_{SDOF}\left(\mu_{allow} = 1\right) \tag{9}$$

Fig. 3.5 shows $(V_{by}/W)_{SDOF}$ and the calculated $(V_{by}/W)_{MDOF}$ at the same time. Therefore, the ratio between $(V_{by}/W)_{MDOF}$ and $(V_{by}/W)_{SDOF}$ on the graphs implies the MDOF base shear modification factor.

3) Computation of the MDOF base shear modification factor M_{MDOF}

$$M_{MDOF} = \left(V_{by}/W\right)_{MDOF} / \left(V_{by}/W\right)_{SDOF} \tag{10}$$

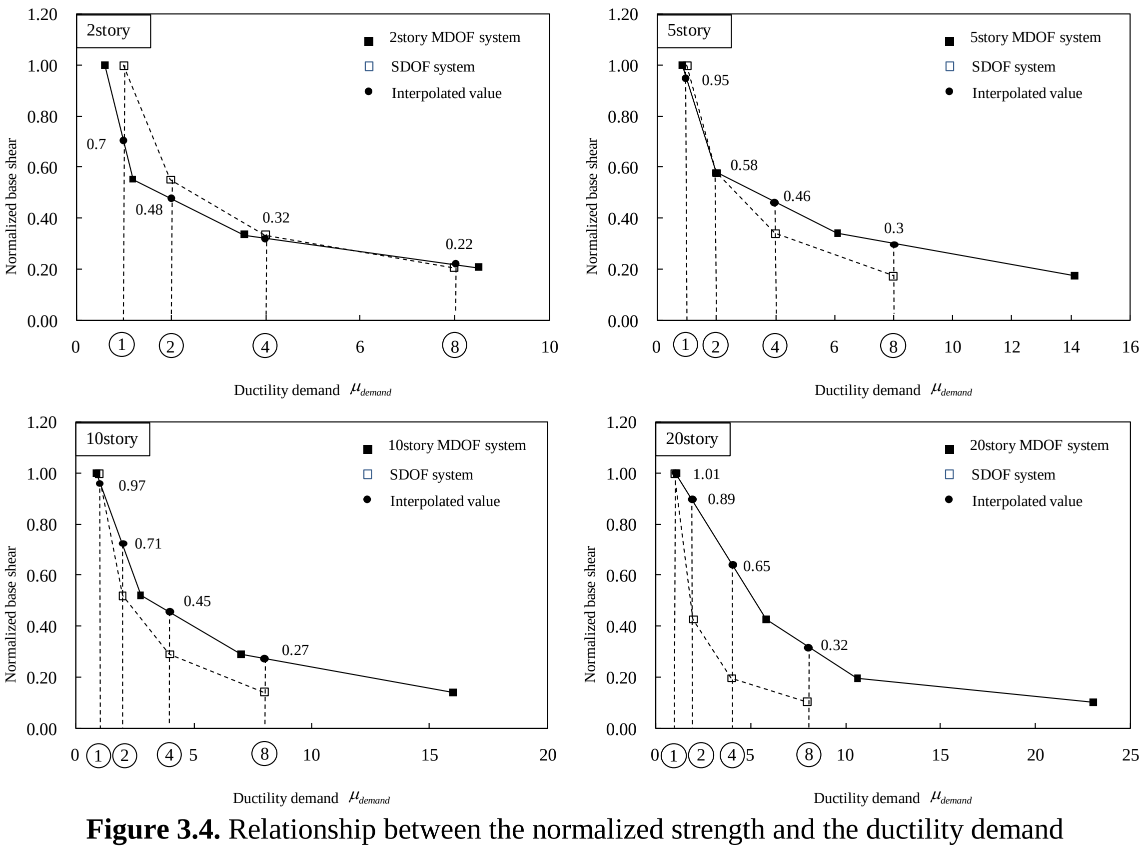


Figure 3.4. Relationship between the normalized strength and the ductility demand

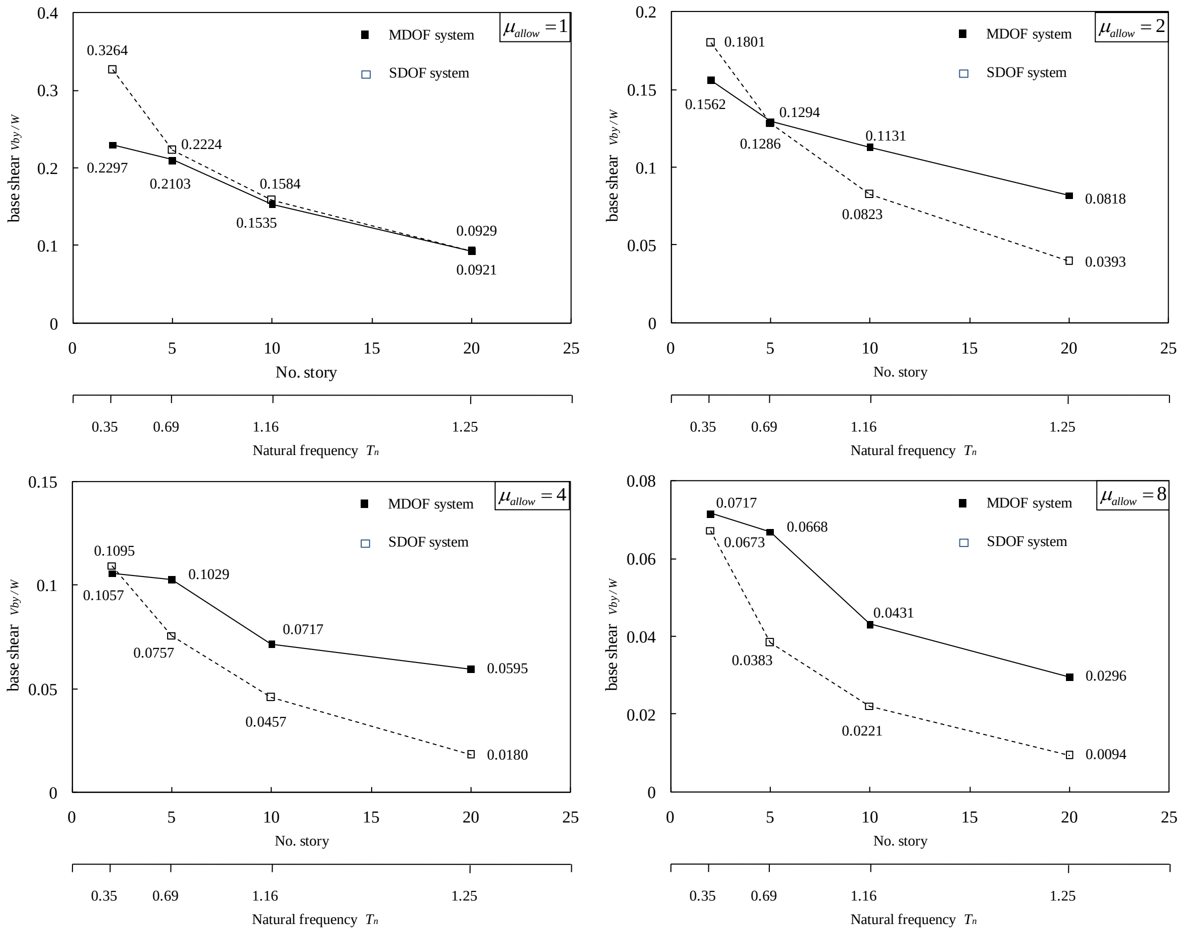


Figure 3.5. Base shears of MDOF and corresponding SDOF systems

3.4. Estimation of the Dynamic Response Modification Factor

Fig. 3.7 shows the R-μ relationship of the MDOF systems and their corresponding SDOF systems for different applied periods. In this figure, the ratio of the response modification factors at points 1 and 3 is the reverse of the ratio of the strength capacities of the SDOF and MDOF systems with the same allowable ductility, that is, the MDOF base shear modification factor.

$$R_{dynamic} = R_{static} / M_{MDOF} \tag{11}$$

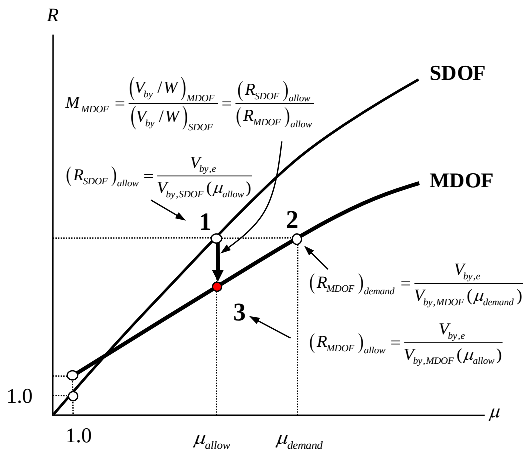


Figure 3.7. R-μ relationship between the SDOF and MDOF systems

The final dynamic response modification factors calculated by Eqn. 11 are presented in Table 3.2, in which M_{MDOF} can be obtained from a linear interpolation of the graphs in Fig. 3.5 for each natural period of the systems under consideration.

Table 3.2. MDOF dynamic response modification factors

Frame Type	T (sec)	R_{static}	M_{MDOF}	$R_{dynamic}$
two-story	0.86	6.79	1.24	5.48
three-story	1.15	7.15	1.42	5.04
four-story	1.32	7.49	1.69	4.43
five-story	1.59	7.85	1.95	4.03

4. CONCLUSION

Based on the proposed response modification factors considering the MDOF dynamic effects, the following conclusions are made:

The values of static response modification factors obtained from the curves of nonlinear static analyses for LSP-modular structures in low to moderate areas of seismicity are in the range of 6 to 8. Although these values are relatively high, presenting similar values of dual systems with special moment frames, they need to be modified considering the pinching effects of each LSP which significantly influence their dynamic behaviour.

According to the proposed procedure, the dynamic response modification factors for low-mid LSP-modular structures are estimated to the range from 5.48 to 4.03, showing a decreasing trend as the number of stories increases. Therefore, it is reasonable to determine a value of 4 as the final response modification factor of LSP-modular structures after limiting the number of stories to five in practice.

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