



PROBABILISTIC ASSESSMENT OF SEISMIC DRIFT DEMANDS FOR FRAME STRUCTURES IN NEAR-FAULT REGIONS

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PROBABILISTIC ASSESSMENT OF SEISMIC DRIFT DEMANDS FOR FRAME STRUCTURES IN NEAR-FAULT REGIONS

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Key words: probabilistic estimation, inter-story drift ratio, near-fault earthquake, spectral displacement, modification factors.

ABSTRACT

The purpose of this paper is to provide engineers with a rapid procedure to assess the maximum inter-story drift for generic moment-resisting frame (GMRF) structures in near-fault regions. In this procedure, the maximum inter-story drift demand of a structure can be evaluated by starting from the median value of elastic spectral displacement at the first period of the structure, and then amplifying it with some modification factors. Statistical parameters (medians and dispersions) of the distributions of these factors are obtained by performing linear, and nonlinear, dynamic simulations on code-compliant steel GMRF structures, to near-fault ground motions during Taiwan's Chi-Chi earthquake in 1999. Statistical equations that use the obtained medians and dispersions are provided for engineers to evaluate the maximum inter-story drift.

Through the study on the simulation data of medians and dispersions of these factors, it is found that the dispersion of α_u — which has been neglected in the previous study by Gupta and Krawinkler — has a great effect on determining the maximum inter-story drift demand of structures. In addition, care must be taken when buildings being designed are less than 2 km from the fault, since the medians $\hat{\alpha}_\theta$ of structures in this region are relatively larger than those of the other regions ($2 \text{ km} < r < 12 \text{ km}$). All statistics obtained in this study are steady and reliable, regardless of the number of stories. Therefore, this rapid method for evaluating seismic demands should be valuable and helpful for engineers in doing the preliminary design.

I. INTRODUCTION

Ground motion records from recent earthquakes show that, near-fault ground motions are different from far-field ground motions, in that they often contain strong coherent dynamic long-period pulses and permanent ground displacements [5, 6, 13, 20, 21]. Characteristics of near-fault ground motions warrant special consideration due to their severe and impulsive effects on structures. These characteristics are unique compared to far-field ground motions, upon which nearly all seismic design criteria are based.

In recent years, many researchers have focused their research on performance evaluation methods [1, 2, 4, 7, 10]. The advantage of these methods is that the seismic demands — roof displacements, story drift ratios, and element ductility ratios — can be estimated, and thus controlled within the structural limits for different intensity levels of earthquakes. In these performance-based design methods, seismic demands can be estimated by the nonlinear response history analysis (NRHA) of structures to recorded earthquake accelerations, or by nonlinear static analysis — the pushover analysis. However, earthquakes are stochastic and their characteristics differ from one to another. Therefore, the NRHA must be completed using data from more representative earthquakes before structures can be qualified for safety. The NRHA is a reliable method, but tedious and time-consuming. The pushover method is simple and direct. However, this method also has shortcomings. Estimates made using this method are often biased when higher mode effects are pronounced [11, 17].

In the design phase, the estimation of deformation demands for performance assessment can be extremely time-consuming because of the iterative procedures required to properly size the structural members. To help engineers in the preliminary design stage, Gupta & Krawinkler [7, 9, 16] have proposed a procedure to estimate the roof drift, and the maximum inter-story drift, for GMRF structures. The procedure uses the median spectral displacement at the first period of the structure, with some modification factors. However, it does not account for the dispersion of spectral displacements generated by different earthquakes, hence, the procedure they proposed can only predict the median roof and inter-story drifts, which can easily be exceeded; with a probability of 50%. Therefore,

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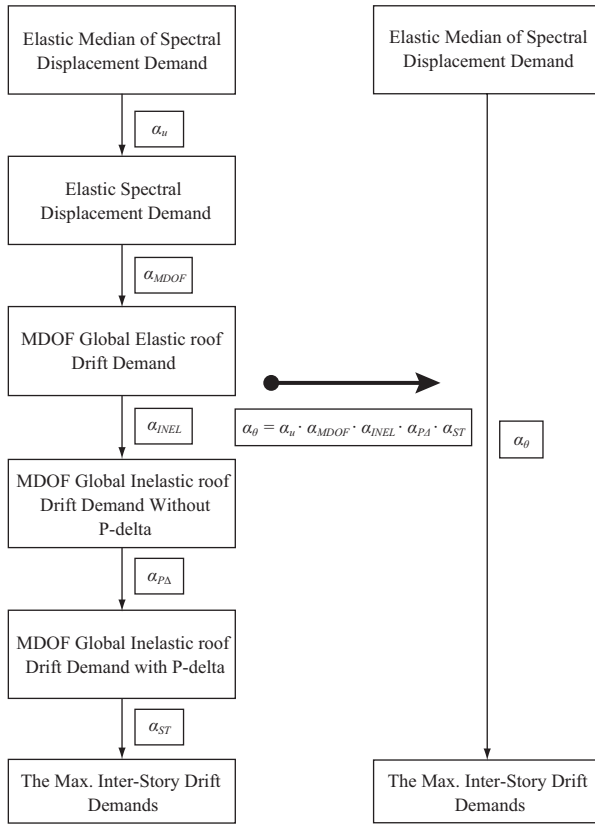


Fig. 1. Process for estimating seismic demands.

improving on Gupta & Krawinkler’s procedure, this study will consider the dispersion of spectral displacement as an important factor in the evaluation of the maximum seismic demands (Fig. 1). Based on probability theory [3], statistical equations will be given in an attempt to provide structural designers with a rapid procedure for evaluating the maximum inter-story drift for both analysis and design.

II. MODIFICATION FACTORS

The following modification factors are used in this study:

1. Spectral displacement modification factor α_u represents the ratio of u_i to the median of all u_i , where u_i is the elastic spectral displacement of a single-degree-of freedom structure to the i th earthquake.
2. Elastic roof displacement modification factor α_{MDOF} is the ratio of the elastic roof displacement demand of a MDOF structure to the spectral displacement u_i at the first period of the structure, neglecting p -delta effects.
3. Inelastic modification factor α_{INEL} relates the elastic roof drift demand to the inelastic roof drift demand, neglecting p -delta effects.
4. P -delta modification factor $\alpha_{P\Delta}$ is the ratio of the inelastic roof displacement, with considering p -delta effects, to inelastic roof displacement without considering p -delta effects.

Table 1. The fundamental periods and participation factors of buildings at different distances, from Chelungpu Fault.

Classified distance condition	Building Height (stories)	1st-mode period T_1 (sec)	Participation factor of the 1 st mode Γ_1
Model 1 $r \leq 2$ km	5	1.16	1.35
	10	2.10	1.39
	20	3.82	1.44
Model 2 2 km < $r \leq 5$ km	5	1.18	1.35
	10	2.10	1.39
	20	3.88	1.44
Model 3 5 km < $r \leq 8$ km	5	1.19	1.36
	10	2.16	1.38
	20	3.89	1.44
Model 4 8 km < $r \leq 12$ km	5	1.19	1.36
	10	2.17	1.38
	20	3.89	1.44

Table 2. Structural models and earthquakes near Chelungpu-fault.

Site	$r \leq 2$ km	2 km < $r \leq 5$ km	5 km < $r \leq 8$ km	8 km < $r \leq 12$ km
Model	Model 1	Model 2	Model 3	Model 4
Earthquakes	TCU 052	TCU 054	TCU 053	TCU 072
	TCU 067	TCU 065	TCU 071	TCU 089
	TCU 068	TCU 082	TCU 075	TCU 120
	TCU 102	TCU 129	TCU 076	TCU 122

r : distance from Chelungpu fault

5. Story drift modification factor α_{ST} relates the maximum inter-story drift demand to the roof drift demand of a GMRF structure, considering p -delta effects.
6. Product modification factor α_θ is the product of the previously defined five modification factors.

$$\alpha_\theta = \alpha_u \cdot \alpha_{MDOF} \cdot \alpha_{INEL} \cdot \alpha_{P\Delta} \cdot \alpha_{ST} \tag{1}$$

The medians of α_u , α_{MDOF} , α_{INEL} , $\alpha_{P\Delta}$, α_{ST} and α_θ are denoted by $\hat{\alpha}_u$, $\hat{\alpha}_{MDOF}$, $\hat{\alpha}_{INEL}$, $\hat{\alpha}_{P\Delta}$, $\hat{\alpha}_{ST}$ and $\hat{\alpha}_\theta$, and their dispersions are represented by δ_u , δ_{MDOF} , δ_{INEL} , $\delta_{P\Delta}$, δ_{ST} and δ_θ .

III. STRUCTURAL SYSTEMS

Four models (Table 2) of strong-column/weak-beam moment-resisting frames of the same height were designed in compliance with Taiwan’s steel structural design specifications [14, 15]. The frames had 5, 10, or 20 stories. Optimal member sections (WF section) of structures were selected automatically with the aid of SAP2000 software [19]. The structures considered are single bay, 7 m by 7 m in the horizontal plan, 4 m high for the first story, and 3.2 m for the other

stories. The mass, considered from dead load and 1/4 of live load, is 1784 kgf at the roof level and 2354 kgf for the other stories. Table 1 shows the fundamental periods and the first modal participation factors of the designed frames. In nonlinear analyses, the program DRAIN-2D+ [18] was used to perform dynamic and static analyses. Plastic hinges, which had 2 percent strain hardening in bilinear models of moment-rotation relationships, were assigned at column bases and beam-ends. The form of the Rayleigh damping matrix is considered to be $C = \alpha M + \beta K$. The α and β factors are computed to satisfy a damping ratio of 5 percent for both the first and second modes.

IV. EARTHQUAKE DATA

Sixteen Chi-Chi earthquake data sets near-Chelungpu-fault (Table 2) were used in this study. Nonlinear inelastic simulations of the structures' responses have been conducted for both the life safety performance level (0.33 g) and the collapse prevention level (0.4 g). The ground accelerations were adjusted by near-fault factors N_A, N_V (Table 3). Figure 2 shows the elastic spectral acceleration/PGA diagrams for PGA = 0.33 g. The median curve in the figure was obtained using (2),

$$\hat{x} = \exp \left[\frac{\sum_{i=1}^n \ln x_i}{n} \right] \quad (2)$$

Where x_i is the individual sample value, \hat{x} is the median value of the sample, and n is the size of the sample ($n = 16$ in this study, 4 earthquakes for each of 4 models of structure).

V. ANALYSIS PROCEDURE

The analysis procedure can be briefly described as follows:

- Design code-compliant structural models for different distances from the Chelungpu fault.
- Determine the elastic spectral displacement u_i of the SDOF system subjected to an individual earthquake. The u_i of the collapse prevention level can be scaled up from the u_i of the life safety performance level by a factor of 0.4 g/0.33 g.
- Determine the median of u_i , denoted by \hat{u} .
- Conduct the elastic response history analyses for the buildings to each of the Chi-Chi earthquakes specified in Table 2; find the maximum roof displacements in each case using DRAIN2D+.
- Conduct non-linear response history analyses for the buildings for the same earthquake records, and find the maximum roof displacements and story drift angles in each case (without the P-delta effect).
- Perform the same analyses as in step (e) considering the P-delta effect. In other words, consider the geometric stiffness in these analyses.

Table 3. Near-Fault factors N_A, N_V for Chelungpu fault.

(a) Adjustment factors for design force

N_A	$r \leq 2$ km	$2 \text{ km} < r \leq 5$ km	$5 \text{ km} < r \leq 8$ km	$8 \text{ km} < r \leq 12$ km
	1.23	1.16	1.07	1.03
N_V	$r \leq 2$ km	$2 \text{ km} < r \leq 5$ km	$5 \text{ km} < r \leq 8$ km	$8 \text{ km} < r \leq 12$ km
	1.36	1.32	1.22	1.10

(b) The maximum adjustment factors for design force

N_A	$r \leq 2$ km	$2 \text{ km} < r \leq 5$ km	$5 \text{ km} < r \leq 8$ km	$8 \text{ km} < r \leq 12$ km
	1.25	1.20	1.10	1.03
N_V	$r \leq 2$ km	$2 \text{ km} < r \leq 5$ km	$5 \text{ km} < r \leq 8$ km	$8 \text{ km} < r \leq 12$ km
	1.50	1.45	1.30	1.15

r : distance from Chelungpu fault

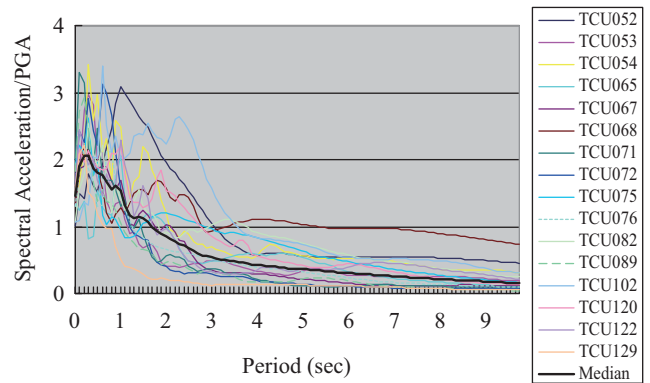


Fig. 2. The normalized pseudo-acceleration response spectra of near-Chelungpu fault, PGA = 0.33 g.

- Calculate a set of modification factors $\alpha_u, \alpha_{MDOF}, \alpha_{INEL}, \alpha_{P\Delta}, \alpha_{ST}$ for each structure, relevant to each individual earthquake record. Also evaluate their medians and dispersions using (2) and (3) respectively.

$$\delta = \left[\frac{\sum_{i=1}^n (\ln x_i - \ln \hat{x})^2}{n-1} \right]^{1/2} \quad (3)$$

Where δ = the dispersion of a sample
 x_i = the individual sample value
 \hat{x} = the median value of a sample
 n = the sample size ($n = 16$, same as that in (2))

- Calculate the median of α_θ using (4).

$$\hat{\alpha}_\theta = \hat{\alpha}_u \cdot \hat{\alpha}_{MDOF} \cdot \hat{\alpha}_{INEL} \cdot \hat{\alpha}_{P\Delta} \cdot \hat{\alpha}_{ST} \quad (4)$$

The derivation of (4) is shown in Appendix A.

- Calculate the dispersion of α_θ directly by (3) or indirectly by (5).

$$\delta_\theta^2 = \delta_u^2 + \delta_{MDOF}^2 + \delta_{INEL}^2 + \delta_{P\Delta}^2 + \delta_{ST}^2 + 2(\delta_{u,MDOF} + \delta_{u,INEL} + \delta_{u,P\Delta} + \delta_{u,ST} + \delta_{MDOF,INEL} + \delta_{MDOF,P\Delta} + \delta_{MDOF,ST} + \delta_{INEL,P\Delta} + \delta_{INEL,ST} + \delta_{P\Delta,ST}) \quad (5)$$

Where $\delta_u^2, \delta_{MDOF}^2, \delta_{INEL}^2, \delta_{P\Delta}^2, \delta_{ST}^2$ are variances and $\delta_{u,MDOF}, \delta_{u,INEL}, \delta_{u,P\Delta}, \delta_{u,ST}, \delta_{MDOF,INEL}, \delta_{MDOF,P\Delta}, \delta_{MDOF,ST}, \delta_{INEL,P\Delta}, \delta_{INEL,ST}$, and $\delta_{P\Delta,ST}$ are co-variances of the modification factors. The derivation of (5) is given in Appendix A.

- (j) Calculate the maximum inter-story drift ratio using (6), (7), or (8), corresponding to a confidence level of 50%, 84.14%, or 97.73%.

$$\theta_{\max,50\%} = \hat{\alpha}_\theta \frac{\hat{u}}{H} \quad (6)$$

$$\theta_{\max,84.14\%} = \exp(\ln \hat{\alpha}_\theta + \delta_\theta) \frac{\hat{u}}{H} \quad (7)$$

$$\theta_{\max,97.73\%} = \exp(\ln \hat{\alpha}_\theta + 2\delta_\theta) \frac{\hat{u}}{H} \quad (8)$$

Where \hat{u} is the median of elastic spectral displacement and H is the height of the structure. Depending on the importance of the structure, one may choose to use (6), (7), or (8).

VI. DISCUSSION OF RESULTS

The calculated median and dispersion values of $\alpha_u, \alpha_{MDOF}, \alpha_{INEL}, \alpha_{P\Delta}, \alpha_{ST}$, and α_θ are shown in Tables 4-5 and 7-10. The following observations can be made from the results.

- (a) As shown in Table 4, the medians $\hat{\alpha}_u$ are approximately 1.0 for both life safety performance level and collapse prevention level, regardless of structural height. This result is reasonable, since the evaluation is starting from the median value of elastic spectral displacement at the first period of the structure. Due to the great randomness of near fault earthquakes, the dispersions of α_u (between 0.43 and 0.65) are relatively larger than the dispersions of other modification factors. Hence, δ_θ in (5) is dominated by δ_u and, in turn, δ_u is very influential in determining the maximum inter-story drift ratio by (6), (7), and (8).
- (b) The median values, $\hat{\alpha}_{MDOF}$, (Table 5) of 5 and 10-story buildings are approximately equal to Γ_1 — the participation factor of the first mode, while the values of 20-story building are greater than Γ_1 , implying that, only for taller buildings, the higher modes have more effect on $\hat{\alpha}_{MDOF}$. Table 6 shows the ratio of $\hat{\alpha}_{MDOF}$ to the participation factor Γ_1 .

Table 4. Median and dispersion of α_u .

Seismic levels	Life safety performance level			Collapse prevention level		
	5F	10F	20F	5F	10F	20F
Story No	5F	10F	20F	5F	10F	20F
Median	1.01	0.98	1.00	1.03	1.03	1.02
Dispersion	0.44	0.65	0.64	0.43	0.65	0.64

Table 5. Median and dispersion of α_{MDOF} .

Seismic levels	Life safety performance level			Collapse prevention level		
	5F	10F	20F	5F	10F	20F
Story No	5F	10F	20F	5F	10F	20F
Median	1.35	1.34	1.68	1.35	1.34	1.68
Dispersion	0.14	0.36	0.29	0.14	0.36	0.29

Table 6. Ratio of the modification factor α_{MDOF} to participation factor Γ_1 .

Seismic levels	Life safety performance level			Collapse prevention level		
	5F	10F	20F	5F	10F	20F
Story No.	5F	10F	20F	5F	10F	20F
Model						
Model 1 $r \leq 2$ km	0.97	1.02	1.14	0.97	1.02	1.14
Model 2 $2 \text{ km} < r \leq 5$ km	0.98	1.03	1.02	0.98	1.03	1.02
Model 3 $5 \text{ km} < r \leq 8$ km	1.08	0.97	1.14	1.08	0.97	1.14
Model 4 $8 \text{ km} < r \leq 12$ km	0.97	0.86	1.38	0.97	0.87	1.38

Table 7. Median and dispersion of α_{INEL} .

Seismic levels	Life safety performance level			Collapse prevention level		
	5F	10F	20F	5F	10F	20F
Story No	5F	10F	20F	5F	10F	20F
Median	0.97	0.93	0.88	0.93	0.91	0.87
Dispersion	0.18	0.16	0.17	0.23	0.19	0.16

Table 8. Median and dispersion of $\alpha_{P\Delta}$.

Seismic levels	Life safety performance level			Collapse prevention level		
	5F	10F	20F	5F	10F	20F
Story No	5F	10F	20F	5F	10F	20F
Median	1.00	1.02	1.06	1.01	1.03	1.11
Dispersion	0.04	0.04	0.15	0.05	0.10	0.19

- (c) The statistics of the inelastic modification factor, $\hat{\alpha}_{INEL}$, are given in Table 7. The medians are clearly less than 1.0 (0.87~0.97), having no conspicuous correlation with earthquake intensity and structural height. This phenome-

Table 9. Median and dispersion of α_{ST} .

Seismic levels	Life safety performance level			Collapse prevention level			
	Story No	5F	10F	20F	5F	10F	20F
Median		1.35	1.52	1.81	1.40	1.61	1.99
Dispersion		0.09	0.12	0.17	0.10	0.13	0.17

Table 10. Median and dispersion of α_θ .

Seismic levels	Life safety performance level			Collapse prevention level			
	Story No	5F	10F	20F	5F	10F	20F
Median		1.77	1.91	2.81	1.84	2.07	3.26
Dispersion		0.50	0.59	0.50	0.55	0.71	0.68

- non can be attributed to the hysteretic dissipation of energy. The dispersions are also satisfactorily less than 0.23.
- (d) The p -delta factor, $\hat{\alpha}_{p\Delta}$, which is defined as the ratio of roof displacements with and without p -delta effects, is presented in Table 8. P -delta always reduces the effective stiffness and thus increases the roof displacements. Detailed study of p -delta effects in reference [8] concludes that p -delta is a relatively benign phenomenon, unless the ground motion drives the structure into the range of negative post-yield stiffness. This conclusion is also supported by the results of this study, as shown in Table 8—all medians are between 1.0 and 1.11, and dispersions are not greater than 0.19.
- (e) The statistical measurements of story drift modification factor $\hat{\alpha}_{ST}$, which relate the maximum inter-story drift demand over the height of the structure to the roof drift demand, and which consider the p -delta effect, are given in Table 9. The medians range from 1.35 to 1.99, depending on the number of stories and the earthquake intensity. It can be seen from Table 9 that, taller buildings increase the effects on both median and dispersion. In general, the earthquake intensity has a weakly increasing effect on the median of this factor. The medians $\hat{\alpha}_{ST}$ is in such a narrow range because the structural members are optimally selected by computer program i.e. SAP2000. Member sizes gradually change along the height of the structure.
- (f) The medians and dispersions of structures for all models pertaining to the same height are given in Table 10. The medians ($\hat{\alpha}_\theta$) are in a narrow range between 1.77 and 3.26, varying increasingly with the number of stories, but indifferent to earthquake intensity. The dispersions δ_θ are greater than 0.5. Table 11 is given as an example to show the components of δ_θ^2 Eq. (5), in which δ_u^2 is the largest contributor among all components. Qualitative comparison of medians and dispersions for each of the four models is given in Figs. 3 and 4. It can be seen that,

Table 11. The co-variances of logarithms of α_u , α_{MDOF} , α_{INEL} , α_{PA} & α_{ST} in δ_θ^2 .

5-story buildings	Chelungpu fault	
	Life safety level	Collapse prevention level
Co-variances		
δ_u^2	189.47E-03	186.75E-03
δ_{MDOF}^2	20.82E-03	20.51E-03
δ_{INEL}^2	32.62E-03	51.96E-03
δ_{PA}^2	1.66E-03	2.11E-03
δ_{ST}^2	8.81E-03	10.61E-03
$\delta_{u, MDOF}$	-22.1E-03	-22.03E-03
$\delta_{u, INEL}$	-24.9E-03	-23.33E-03
$\delta_{u, PA}$	-2.00E-03	5.44E-03
$\delta_{u, ST}$	20.5E-03	22.23E-03
$\delta_{MDOF, INEL}$	2.09E-03	-0.86E-03
$\delta_{MDOF, PA}$	0.87E-03	0.22E-03
$\delta_{MDOF, ST}$	-2.59E-03	-1.63E-03
$\delta_{INEL, PA}$	6.14E-03	4.51E-03
$\delta_{INEL, ST}$	6.97E-03	14.32E-03
$\delta_{PA, ST}$	1.64E-03	2.99E-03

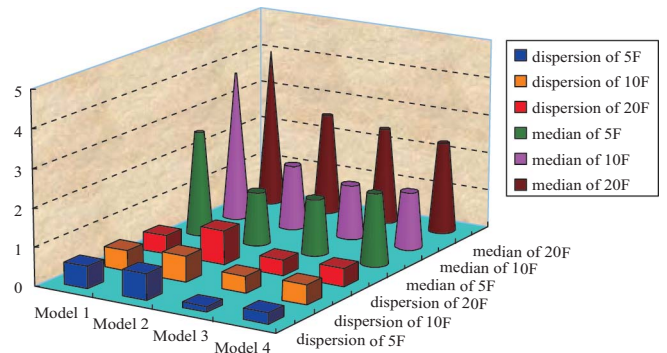


Fig. 3. Statistics of the modification factor α_θ for buildings different distance from the fault (life safety performance level)

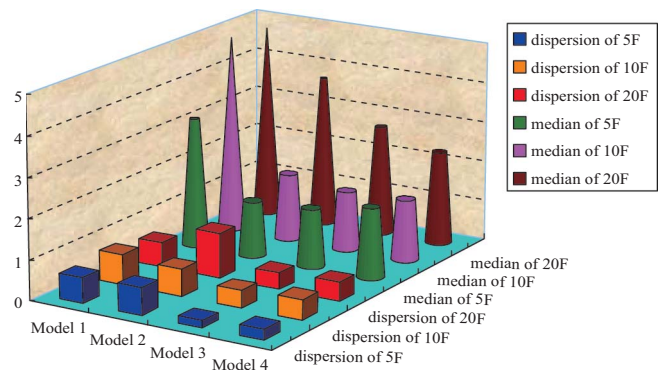


Fig. 4. Statistics of the modification factor α_θ for buildings different distance from the fault (collapse prevention level).

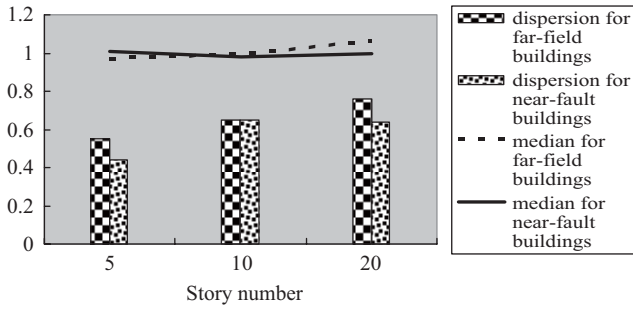


Fig. 5. Comparison of the modification factor α_u between near-fault and far-field buildings (life safety performance level).

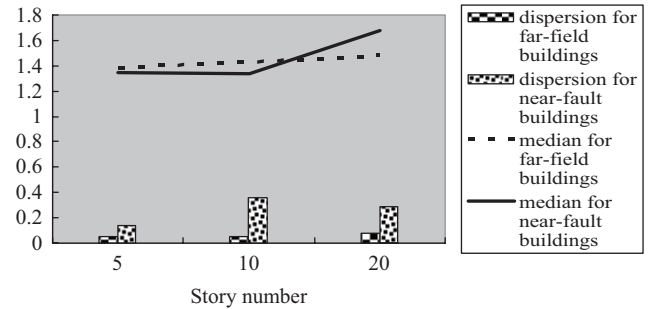


Fig. 7. Comparison of the modification factor α_{MDOF} between near-fault and far-field buildings (life safety performance level).

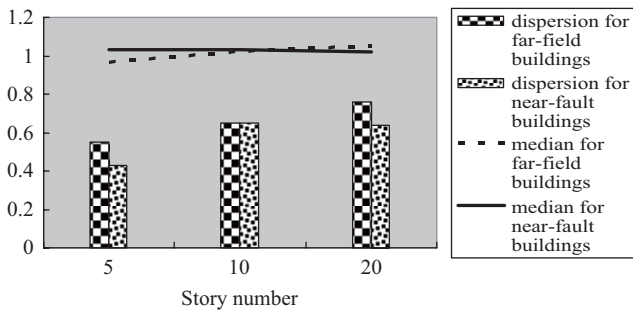


Fig. 6. Comparison of the modification factor α_u between near-fault and far-field buildings (collapse prevention level).

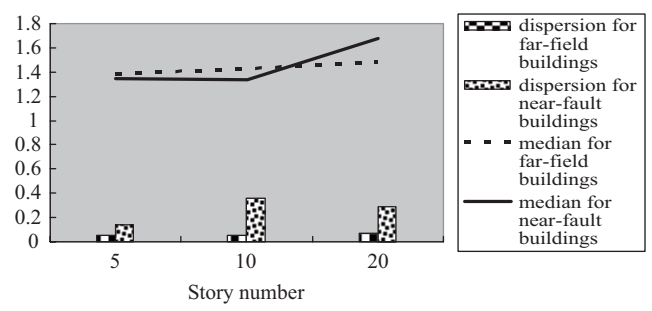


Fig. 8. Comparison of the modification factor α_{MDOF} between near-fault and far-field buildings (collapse prevention level).

despite of the performance level, the medians of model 1 ($0 < r < 2$ km) are much greater than those of the other models. This phenomenon reflects that the displacement and impulse are relatively large in this region and care must be taken for structures being designed in this region.

VII. COMPARISON OF MODIFICATION FACTORS FOR STEEL GMRF STRUCTURES IN NEAR-FALT AND FAR-FIELD REGIONS

The medians and dispersions of modification factors obtained in this study are compared with those in Reference [12] by the authors, for both life safety and collapse prevention levels. Some important results are given as follows:

- (a) As shown in Figs. 5 and 6, the medians of $\hat{\alpha}_u$ are matching well for structures in both regions. However, the dispersions of near-fault buildings are slightly smaller than, those of buildings in the far-field regions.
- (b) The comparison of $\hat{\alpha}_{MDOF}$ are shown in Figs. 7 and 8. The medians of near-fault region are less than that of far-field region for 5 and 10-story buildings, but greater than that of far-field region for 20-story buildings. The dispersions are, in general, greater for near-fault buildings in comparison with that of far-field buildings.
- (c) The medians of $\hat{\alpha}_{INEL}$ (Figs. 9 and 10) are slightly increasing with the story height for the far-field buildings, but are slightly decreasing with the story height for the

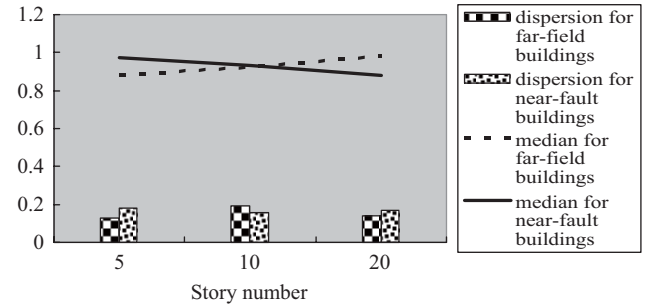


Fig. 9. Comparison of the modification factor α_{INEL} between near-fault and far-field buildings (life safety performance level).

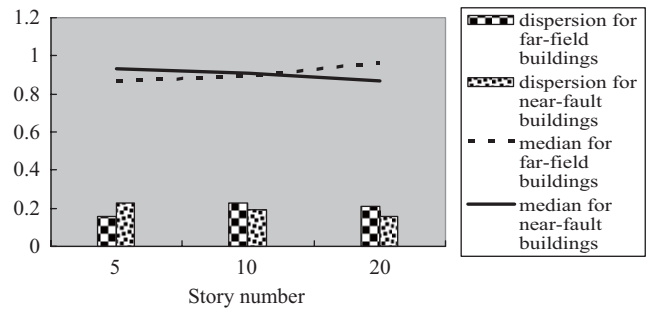


Fig. 10. Comparison of the modification factor α_{INEL} between near-fault and far-field buildings (collapse prevention level).

near-fault buildings. The dispersions are small for buildings in both regions.

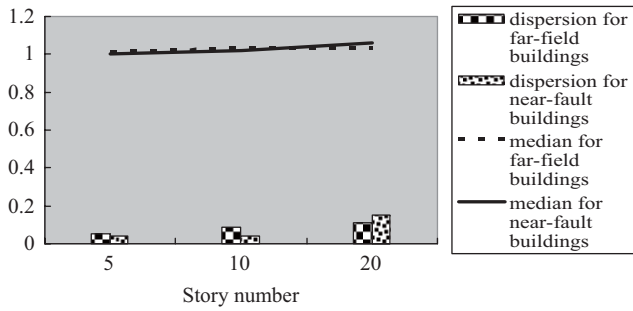


Fig. 11. Comparison of the modification factor α_{pA} between near-fault and far-field buildings (life safety performance level).

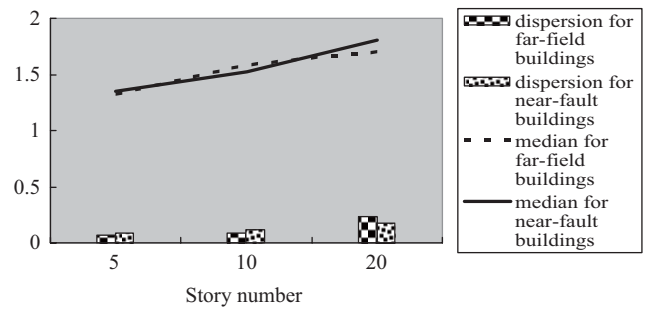


Fig. 13. Comparison of the modification factor α_{ST} between near-fault and far-field buildings (life safety performance level).

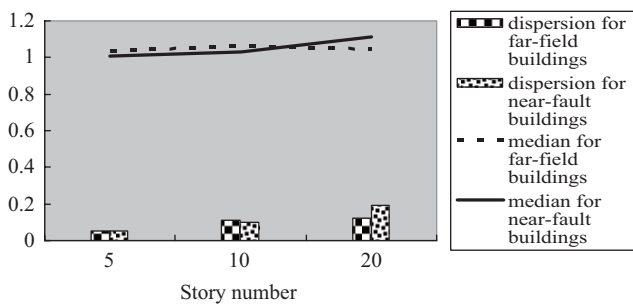


Fig. 12. Comparison of the modification factor α_{pA} between near-fault and far-field buildings (collapse prevention level).

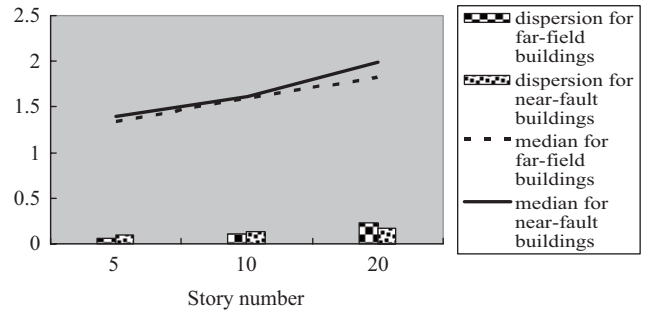


Fig. 14. Comparison of the modification factor α_{ST} between near-fault and far-field buildings (collapse prevention level).

- (d) The medians of $\hat{\alpha}_{pA}$ (Figs. 11 and 12) are matching well for buildings in both regions. The dispersions are also small for buildings in both regions.
- (e) The medians of $\hat{\alpha}_{ST}$ (Figs. 13 and 14) have a tendency of increasing with the story height for buildings in both regions and are not much different from each other. The dispersions are also small for buildings in both regions.
- (f) For both regions, the medians of $\hat{\alpha}_\theta$ (Figs. 15 and 16) are increasing with the story height and in a narrow range between 1.6 and 3.3. The dispersions are less than 0.78.

VIII. APPLICATION

Referring to the analysis results presented in Table 10, engineers can rapidly evaluate the maximum inter-story drift ratio of an existing structure using (6), (7), and (8), for any desired confidence probability. Application of this evaluation can also be extended to the design of a new GMRF structure. In order to meet the design requirements; the structural height, number of stories, generic dead and live loads, and the performance levels of earthquake force for a specified earthquake region and soil condition, the following story-drift-ratio-based design procedure is suggested:

1. Design Procedure for a GMRF Structure

- (1) Design a new GMRF structure to meet the design requirements, using software such as SAP 2000, which allows selection of optimal structural elements.

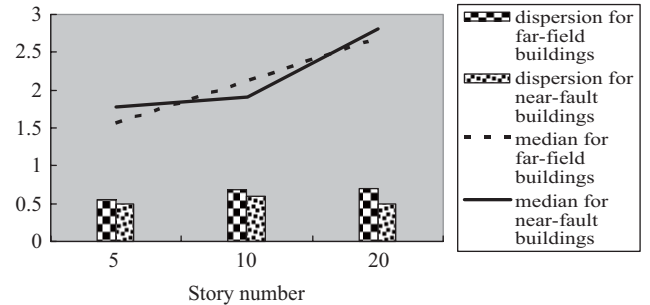


Fig. 15. Comparison of the modification factor α_θ between near-fault and far-field buildings (life safety performance level).

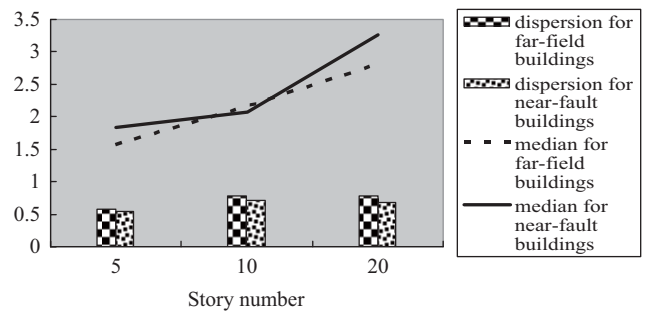


Fig. 16. Comparison of the modification factor α_θ between near-fault and far-field buildings (collapse prevention level).

- (2) Calculate the fundamental frequency, ω_1 of the newly designed GMRF structure.

- (3) Determine the median of normalized pseudo-acceleration A from the acceleration response spectra shown in Fig. 2 and adjust it for specified seismic performance levels.
- (4) Obtain the median of elastic spectral displacement \hat{u} using

$$\hat{u} = \frac{A}{\omega_1^2} \quad (9)$$

- (5) Find the median and the dispersion of α_θ from Table 10, (by interpolation if necessary).
- (6) Calculate the maximum inter-story drift ratios using (6), (7), or (8) for all specified performance levels, and compare them with the limits specified by FEMA 273. If any one of them exceeds its corresponding limit, then repetition of steps 1 to 6 is required until the maximum inter-story drift ratios for all specified performance levels fall within their limits. Since the medians of model 1 ($0 < r < 2$ km) are much greater than those shown in Table 10, one is suggested to use (8) when a structure to be designed is less than 2 km from the fault and if data of $\hat{\alpha}_\theta$ and δ_θ in Table 10 are used.

2. Design Example

A 20 story structure, between two to five kilometers from Chelungpu Fault, 4 m high for the first story and 3.2 m for the others, is to be designed. The live load is 300 kg/m² for all floor levels, and the dead load is 300 kg/m² for the roof and 400 kg/m² for other floor levels. The structure must satisfy the inter-story drift ratio limits at two hazard levels — 0.025 for the life safety level and 0.05 for the collapse prevention level. Consider confidence probability levels of 50% and 84.14%.

Solution: Equations (6) and (7) can be used to estimate the maximum inter-story drift ratio. The natural period of the preliminarily designed structure is 3.53 sec ($\omega_1 = 1.78$ rad/sec), the height of the building is 64.8 m.

Life safety level ($\ddot{u}_{g0} = 0.33$ g)

From Fig. 2, $A = 0.484 \ddot{u}_{g0}$, $\hat{u} = \frac{A}{\omega_1^2} = 0.49$ m.

From Table 10, $\hat{\alpha}_\theta = 2.81$, $\delta_\theta = 0.50$, and by (6) and (7)

$$\theta_{\max,50\%} = \hat{\alpha}_\theta \cdot \frac{\hat{u}}{H} = 0.021 \leq 0.025 \Rightarrow \text{OK}$$

$$\theta_{\max,84.14\%} = \exp(\ln \hat{\alpha}_\theta + \delta_\theta) \frac{\hat{u}}{H} = 0.035 > 0.025$$

It is seen that $\theta_{\max,84.14\%}$ exceeds the limit of 0.025; redesign is needed. By increasing the base shear of the structure and by iterating steps 1-6 in the design procedure, a new GMRF structure with a fundamental natural period of 2.42 sec ($\omega_1 = 2.60$ rad/sec) is designed. The member sizes are shown in Appendix B.

From Fig. 2, $A = 0.74 \ddot{u}_{g0}$, $\hat{u} = \frac{A}{\omega_1^2} = 0.35$ m.

$$\theta_{\max,50\%} = \hat{\alpha}_\theta \cdot \frac{\hat{u}}{H} = 0.015 \leq 0.025 \Rightarrow \text{OK}$$

$$\theta_{\max,84.14\%} = \exp(\ln \hat{\alpha}_\theta + \delta_\theta) \frac{\hat{u}}{H} = 0.025 \leq 0.025 \Rightarrow \text{OK}$$

Collapse prevention level ($\ddot{u}_{g0} = 0.4$ g)

$$\hat{u} = 0.35 \text{ m} \left(\frac{0.4 \text{ g}}{0.33 \text{ g}} \right) = 0.42 \text{ m}, \hat{\alpha}_\theta = 3.26, \delta_\theta = 0.68$$

(from Table 10)

$$\text{Similarly } \theta_{\max,50\%} = \hat{\alpha}_\theta \cdot \frac{\hat{u}}{H} = 0.021 \leq 0.05 \Rightarrow \text{OK}$$

$$\theta_{\max,84.14\%} = \exp(\ln \hat{\alpha}_\theta + \delta_\theta) \frac{\hat{u}}{H} = 0.042 \leq 0.05 \Rightarrow \text{OK}$$

IX. FINDINGS, CONCLUSIONS, AND LIMITATIONS

This study has conducted research on the modification factors α_u , α_{MDOF} , α_{INEL} , $\alpha_{P\Delta}$, α_{ST} and α_θ by performing simulations on Taiwan code-compliant steel structures of different heights. Sixteen Chi-Chi earthquake data sets near Chelungpu fault were used for nonlinear time-history analyses. The medians and dispersions of all these modification factors were evaluated. Equations have been established to calculate the maximum inter-story drift demands of steel GMRF structures for any confidence level. The analysis results have shown that all statistical data are steady and reliable, regardless of the number of stories. Therefore, this rapid method for evaluating seismic demands should be acceptable and reliable for both analysis and design, and would be especially valuable during the preliminary design stage.

Among the modification factors, the dispersion of α_u is very influential in determining the inter-story drift demand. If the dispersion of α_u is excluded, then the dispersion of α_θ will be around 0.3 or less, which means that the lognormal distribution of α_θ will have a normal shape. Unfortunately, in most cases in this study, the dispersions of α_θ are greater than 0.5. Therefore, the distribution of α_θ is lognormal in shape. The comparison of medians and dispersions of modification factors were made for buildings in the near-fault and far-field regions. It is found that the differences of them are small between the two regions. However, care must be taken when buildings to be designed are less than 2 km from the fault, since the medians $\hat{\alpha}_\theta$ of structures in this region are relatively larger than those of the other regions ($2 \text{ km} < r < 12 \text{ km}$).

The derived equations can be applied to the evaluation of

the maximum inter-story drift of any structure for any desired confidence level, depending on the importance of the structure. However, use of the statistical data obtained in this study is limited to steel GMRF structures where members are optimally chosen (using software such as SAP2000), but not for other special structures, like strong-beam/weak-column systems, or systems with weak stories.

X. FURTHER STUDY WORK

- (a) Similar study on 3-bay steel moment-resisting frames has been conducted by the authors. The results, which will be published in a separate paper, have shown that the medians $\hat{\alpha}_\theta$ and dispersions δ_θ are, in general, slightly less than those obtained in this study. Therefore, using statistic values of $\hat{\alpha}_\theta$ and δ_θ , obtained in this study to estimate the maximum inter-story drifts of multi-bay frame structures may obtain conservative values.
- (b) Usually, the structural members are rolled steel sections, the dispersion of member size is believed to be small, and therefore, as compared to the modification factors in this study, its effect on the maximum inter-story drift ratio is negligible. However, if structural members are not rolled steel sections, then the member size should be considered as an additional factor in this rapid estimation method.

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APPENDIX A.: Derivation of Eqs. (4) and (5)

Rewrite (1) as

$$\alpha_\theta = \alpha_u \cdot \alpha_{MDOF} \cdot \alpha_{INEL} \cdot \alpha_{P\Delta} \cdot \alpha_{ST} \quad (A1)$$

Since α_u , α_{MDOF} , α_{INEL} , $\alpha_{P\Delta}$ and α_{ST} are assumed log-normal in this study, their natural logs are jointly normal. For simplicity, let $\ln \alpha_\theta = \alpha_0$, $\ln \alpha_u = \alpha_1$, $\ln \alpha_{MDOF} = \alpha_2$, $\ln \alpha_{INEL} = \alpha_3$, $\ln \alpha_{P\Delta} = \alpha_4$ and $\ln \alpha_{ST} = \alpha_5$. Denote the means of the natural logs by μ_0 , μ_1 , μ_2 , μ_3 , μ_4 and μ_5 , respectively. Taking the natural log on both sides of (A1), we obtain

$$\alpha_0 = \sum_{k=1}^5 \alpha_k \quad (A2)$$

rewrite (A2) as

$$\alpha_0 = \sum_{k=1}^5 \mu_k + \sum_{k=1}^5 (\alpha_k - \mu_k) \quad (A3)$$

Taking the expectation on both sides of (A3) the mean and variance of α_0 are obtained as follows:

$$\mu_0 = \mu_1 + \mu_2 + \mu_3 + \mu_4 + \mu_5 \quad (A4)$$

$$\sigma_{\alpha_0}^2 = \sum_{k=1}^5 \sum_{l=1}^5 E(\alpha_k - \mu_k)(\alpha_l - \mu_l) \quad (A5)$$

The exponential value of (A4) can be written as:

$$e^{\mu_0} = e^{\mu_1} \cdot e^{\mu_2} \cdot e^{\mu_3} \cdot e^{\mu_4} \cdot e^{\mu_5} \quad (A6)$$

Equation (A6) can also be expressed by (A7):

$$\hat{\alpha}_\theta = \hat{\alpha}_u \cdot \hat{\alpha}_{MDOF} \cdot \hat{\alpha}_{INEL} \cdot \hat{\alpha}_{P\Delta} \cdot \hat{\alpha}_{ST} \quad (A7)$$

where the modification factors with hats denote medians. In (A5) the expectation term can be calculated numerically, i.e.:

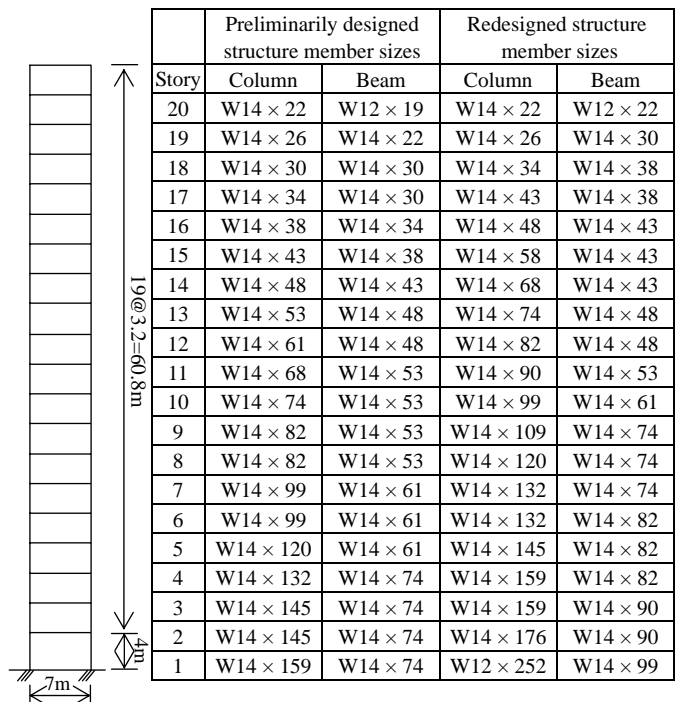
$$E[(\alpha_k - \mu_k)(\alpha_l - \mu_l)] = \frac{1}{n-1} \sum_{i=1}^n (\alpha_k - \mu_k)_i (\alpha_l - \mu_l)_i \quad (A8)$$

where n is the number of earthquakes considered for a structure. For example, for k = 2 and l = 3, Eq. (A8) can be written as:

$$\delta_{MDOF \cdot INEL} = \frac{1}{n-1} \sum_{i=1}^n (\ln \alpha_{MDOF} - \ln \hat{\alpha}_{MDOF})_i \cdot (\ln \alpha_{INEL} - \ln \hat{\alpha}_{INEL})_i \quad (A9)$$

where $\delta_{MDOF \cdot INEL}$ is the covariance of the natural logs of the modification factors α_{MDOF} and α_{INEL} . Expanding (A5), we arrive at (5) of analysis procedure (i).

APPENDIX B.: Member details and geometry for GMRF structure



	Preliminarily designed structure member sizes		Redesigned structure member sizes	
Story	Column	Beam	Column	Beam
20	W14 × 22	W12 × 19	W14 × 22	W12 × 22
19	W14 × 26	W14 × 22	W14 × 26	W14 × 30
18	W14 × 30	W14 × 30	W14 × 34	W14 × 38
17	W14 × 34	W14 × 30	W14 × 43	W14 × 38
16	W14 × 38	W14 × 34	W14 × 48	W14 × 43
15	W14 × 43	W14 × 38	W14 × 58	W14 × 43
14	W14 × 48	W14 × 43	W14 × 68	W14 × 43
13	W14 × 53	W14 × 48	W14 × 74	W14 × 48
12	W14 × 61	W14 × 48	W14 × 82	W14 × 48
11	W14 × 68	W14 × 53	W14 × 90	W14 × 53
10	W14 × 74	W14 × 53	W14 × 99	W14 × 61
9	W14 × 82	W14 × 53	W14 × 109	W14 × 74
8	W14 × 82	W14 × 53	W14 × 120	W14 × 74
7	W14 × 99	W14 × 61	W14 × 132	W14 × 74
6	W14 × 99	W14 × 61	W14 × 132	W14 × 82
5	W14 × 120	W14 × 61	W14 × 145	W14 × 82
4	W14 × 132	W14 × 74	W14 × 159	W14 × 82
3	W14 × 145	W14 × 74	W14 × 159	W14 × 90
2	W14 × 145	W14 × 74	W14 × 176	W14 × 90
1	W14 × 159	W14 × 74	W12 × 252	W14 × 99

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