Study on Response Modification Factor for Stiffened Steel Shear Wall Systems

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SUMMARY:
Design recommendations for steel plate shear wall (SPSW) systems have recently been introduced into seismic provisions for steel buildings. Response modification factor (R) for SPSW systems presented in design codes is based on professional experience and engineering judgment.
In this paper, overstrength, ductility and response modification factor of stiffened steel plate shear walls are evaluated. Buildings with various stories, thicknesses and spans are considered. Static pushover analysis and nonlinear incremental dynamic analysis have been performed utilizing sophisticated Finite Element Analysis (FEA). The effects of some parameters influencing response modification factor, including the height of the building and the thickness and span of SPSWs, are investigated. In this article seismic response modification factor for each of SPSW systems has been determined separately, and tentative value of 10 has been suggested for allowable stress design method.

keywords: Steel plate shear wall, Response modification factor, Stiffener, Nonlinear analysis

1. INTRODUCTION

Lateral load resisting systems are one of the most essential parts of the structures. In this regards, the interest of using steel plate shear wall (SPSW) systems have been grown globally in past decades, as a system that enjoys various advantages including primarily, substantial ductility, high initial stiffness, fast pace of construction, and the reduction in seismic mass [Astaneh-Asl, 2001]

The reserved strength (overstrength) and capacity to dissipate energy (ductility) in structures have usually been considered by seismic load reduction within design codes. These capabilities are taken into account in structural design through a force reduction or response modification factor. This factor shows ratio of maximum seismic force exerted on a structure during specific seismic hazard level if it has to be remained elastic resisting design seismic force. Thus, actual seismic forces are reduced by R factor to achieve design forces. The main flaw in codes procedure is that they use linear methods but rely on nonlinear behavior. The response modification factors were first proposed in ATC3-06 [ATC, 1978]. In ATC-19 [ATC, 1995a] and ATC-34 [ATC, 1995b], the R factors were calculated as the product of three factors: overstrength factor, ductility factor, and redundancy factor.

The main aim of this study is to investigate the relationship of story number, steel plate wall thickness and span with response modification factor in diverse numerical models of structures which benefit stiffened steel plate shear wall as their lateral load-resisting system.

1.1. Steel Shear Wall

During the last three decades, interest has grown globally in the application of steel plate shear walls as building lateral load resistance system. In recent years, steel plate shear walls have been used in number of buildings as part of the lateral force resisting system, mainly in Japan and North America. Steel plate walls have also become recognized as a practical seismic resisting system for buildings located in highly seismic regions. The 23-story U.S. Courthouse building in Seattle is a prime example of the application of this system in the United States. A six-story building in Saint Georges, Quebec is a representative example of recent application of this technology in Canada. One of the most significant buildings constructed with this system in a highly seismic area is a 35-story high-rise building in Kobe, Japan. This structure was constructed in 1988 and performed very well during the 1995 Kobe earthquake [Astaneh-Asl, 2001].

The theory that governs the design of a ductile steel plate wall structure is drawn from that of plate girders, although it has to be taken into account that the relative high bending strength and stiffness of the beams and columns are expected to have a significant effect on the overall behavior of a building incorporating this type of system.

Some of the advantages of using a DSPW system compared with traditional ductile reinforced concrete shear walls are as follows. First, this system has relatively high initial stiffness, thus very effective in limiting the drift. Second, it is very ductile and has relatively large energy dissipation capability. Moreover, compared to reinforced concrete shear walls, the steel shear wall is much lighter that can result in less weight to be carried by the columns and foundations as well as less seismic load due to reduced mass of the structure. From architectural point of view, smaller thickness of steel plate shear walls in comparison with reinforced concrete shear walls lead to smaller occupied space for this system [Astaneh-Asl, 2001].

The steel shear walls can be divided into two main groups, stiffened and unstiffened ones. The idea of using stiffeners comes from the fact that they can prevent the steel plate from elastic buckling. However, heavy stiffed steel shear walls, which were designed before 1990, was costly and this factor limited their applications. But, researchers have found that thin plate steel shear walls, could be considered as a reliable lateral load resisting system in seismic areas, because, the buckling of them would not stop their functionality, and also, it can lead to considerable lateral resisting capacity as a result of tension field effects. On the other hand, the elastic buckling of the plate, reduces its capacity to dissipate energy in later loading cycles, so the hysteresis diagram will be narrowed gradually (Pinching). To solve this problem, the use of stiffeners is studied again keeping in mind that that steel shear wall can buckle plastically and retaining energy dissipation ability.

1.2. Response Modification Factor

The elastic analysis of structures under earthquake can lead to base shear force and stress which are noticeably bigger than structural real response. Owing to structural high level of earthquake energy dissipation capacity in inelastic deformation domain, structural design force was reduced in order to meet economical benefits. This strategy lead to two important concepts in seismic design methodology : over strength and response modification factor. Over strength in structures is related to the fact that the maximum lateral strength of a structure generally exceeds its design strength. Response modification factor includes inelastic performance of the structure and indicates over strength and ductility of structure in inelastic stage [Asgarian & Shokrgozar, 2009; ATC, 1995a].

As shown in Fig. 1.1., usually real nonlinear behavior is idealized by a bilinear elasto perfectly plastic relationship. The yield base shear coefficient of structure is shown by C_y and the yield displacement is Δ_y. In this figure , C_{eu} corresponds to the elastic response strength of the structure. The maximum base shear ratio in an elasto perfectly behavior is Δ_y [Uang, 1991]. The ratio of maximum base shear coefficient considering elastic behavior C_{eu} to maximum base shear coefficient in elasto perfectly behavior C_y is called force reduction factor (Eqn. 1.1.):

$$R_\mu = \frac{C_{eu}}{C_y} \qquad (1.1)$$

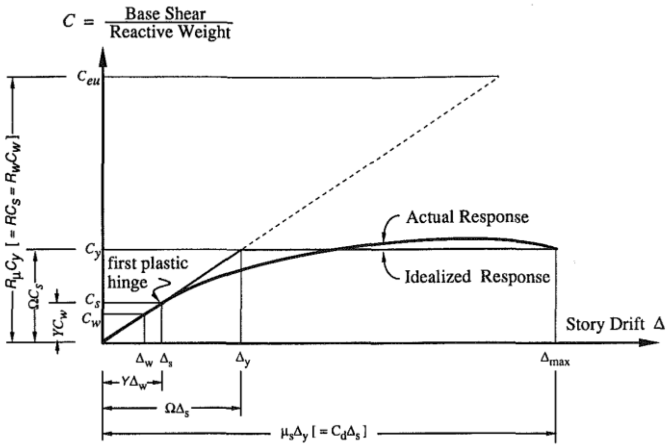

Figure 1.1. General structural response [Uang, 1991]

The overstrength factor is defined as the ratio of maximum base shear coefficient in actual behavior C_y to first significant yield strength in structure C_s (Eqn. 1.2.):

$$\Omega = \frac{C_y}{C_s} \tag{1.2}$$

The concept of overstrength, redundancy and ductility, which are used to scale down the earthquake forces need to be clearly defined and expressed in quantifiable terms.

To design for allowable stress method, the design codes decrease design loads from C_s to C_w. This decrease is done by allowable stress factor which is defined in Eqn. 1.3. :

$$Y = \frac{C_S}{C_W} \tag{1.3}$$

The range of this factor is about 1.4 to 1.5. In this paper allowable stress factor Y was considered as 1.4 [ATC, 1995a; Uang, 1991].

In the other word, the response modification factor accounts shows the difference in the level of stresses considered in the design of structure. It is generally expressed in the following form.

$$R = \frac{C_{eu}}{C_S} = \frac{C_{eu}}{C_Y}\frac{C_Y}{C_S} = R_\mu\,\Omega \tag{1.4}$$

$$R_W = \frac{C_{eu}}{C_W} = \frac{C_{eu}}{C_Y}\frac{C_Y}{C_S}\frac{C_S}{C_W} = R_\mu\,\Omega\,Y \tag{1.5}$$

Eqn. 1.4. shows the seismic response modification factor in ultimate strength design method. Also, Eqn. 1.5. indicates seismic response modification factor in allowable stress design method.

Structural ductility, μ, is defined in terms of maximum structural drift (Δ_{max}) and the displacement corresponding to the idealised yield strength (Δ_y), as given in Eqn. 1.6.

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{1.6}$$

Many investigators have discussed the two main components of R factor presented in Eqn. 1.4., in particular, the ductility dependent component, R_μ, has received considerable attention. Reviews of these discussions can be seen in works by Uang [1991], Kappos [1999], Elnashai and Mwafy [2002]. Ductility reduction factor R_μ is a function of both characteristics of the structure including ductility, damping and fundamental period of vibration (T), and the characteristics of earthquake ground motion [Maheri and Akbari, 2003].

Newmark and Hall [1982] arrived at a set of equations expressing R_μ in terms of the above characteristics. They concluded that for T > 0.5 s, R_μ is effectively equal to the ductility factor of the structure, μ. Later works by others including those of Krawinkler and Nassar, and Miranda and Bertero showed the T-dependence of R_μ for T higher than 0.5 s. They also showed the influence of underlying soil type on the values of R_μ. Krawinkler and Nassar [1991] presented a relation for R_μ as Eqn. 1.7.

$$R_\mu = [c(\mu - 1) + 1]^{1/c} \tag{1.7}$$

That c achieved from Eqn. 1.8.

$$c(T, \alpha) = \frac{T^a}{1+T^a} + \frac{b}{T} \tag{1.8}$$

In Eqn. 1.8., α is the post-yield stiffness given as a percentage of the initial stiffness of the system. If α=0, a and b are equal to 1.0 and 0.42, respectively.

Miranda and Bertero [1994] presented below equations using 124 ground motions recorded on a wide range of soil conditions, and assumed five percent of critical damping. Their equation for the ductility factor is given as Eqn. 1.9.

$$R_\mu = \frac{\mu - 1}{\Phi} + 1 \tag{1.9}$$

φ in this equation for alluvium sites is shown in Eqn. 1.10.

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} e^{-2(\ln(T) - 0.2)^2} \tag{1.10}$$

2. STUDIED MODEL STRUCTURES

In this study, the response modification factor (R), is investigated on 2, 4, 6 and 10 story structures, as the same structures in [Topkaya and Kurban, 2009], which have steel shear walls with 3 and 6 millimeters thickness in 3, 4.5 and 6 meters spans. The general placement of stiffeners on steel shear walls is presented in Fig. 2.1.

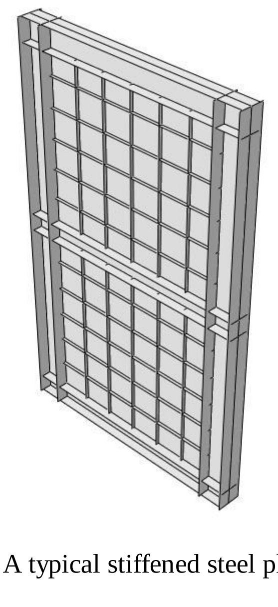

Figure 2.1. A typical stiffened steel plate shear wall

The properties of exploited stiffeners are indicated in Table 3.1 which are designed considering AISC-design guide 20 [Rafaelli and Bruneau, 2006] and AISC 360 [AISC, 2005b].

Table 3.1. Properties And Behaviour Parameters Of Models

No. of stories	Plate thickness (mm)	Plate width (m)	Stiffener thickness (mm)	Stiffener width (cm)	Stiffener spacing (cm)	μ	T	Ω	$R_{\mu 1}$	$R_{\mu 2}$	R
2	3	3	4	3	30	5.51	0.26	1.74	3.35	3.83	9.34
4	3	3	4	3	30	4.06	0.51	1.95	3.70	3.85	10.52
6	3	3	4	3	30	3.04	0.77	2.00	3.07	3.53	9.90
10	3	3	4	3	30	2.29	1.28	1.97	2.36	2.68	7.39
2	3	4.5	4	3	30	7.40	0.26	1.50	3.99	4.48	9.42
4	3	4.5	4	3	30	5.29	0.51	1.66	4.68	4.84	11.24
6	3	4.5	4	3	30	4.05	0.77	1.68	4.10	4.69	11.03
10	3	4.5	4	3	30	3.10	1.28	1.67	3.27	3.71	8.67
2	3	6	4	3	30	8.49	0.26	1.36	4.31	4.57	8.69
4	3	6	4	3	30	6.48	0.51	1.51	5.59	5.64	11.92
6	3	6	4	3	30	5.01	0.77	1.53	5.09	5.73	12.27
10	3	6	4	3	30	3.95	1.28	1.52	4.26	4.76	10.12
2	6	3	10	5	50	7.19	0.26	1.70	3.92	4.44	10.56
4	6	3	10	5	50	4.56	0.51	1.93	4.10	4.27	11.52
6	6	3	10	5	50	3.43	0.77	1.96	3.47	3.99	10.96
10	6	3	10	5	50	2.48	1.28	1.93	2.57	2.92	7.89
2	6	4.5	10	5	50	7.11	0.26	1.52	3.89	4.42	9.40
4	6	4.5	10	5	50	5.33	0.51	1.71	4.71	4.87	11.65
6	6	4.5	10	5	50	4.06	0.77	1.70	4.11	4.71	11.21
10	6	4.5	10	5	50	3.37	1.28	1.70	3.58	4.04	9.61
2	6	6	10	5	50	6.80	0.26	1.47	3.79	4.33	8.92
4	6	6	10	5	50	5.48	0.51	1.62	4.82	4.98	11.28
6	6	6	10	5	50	4.51	0.77	1.62	4.58	5.21	11.81
10	6	6	10	5	50	3.73	1.28	1.63	3.99	4.48	10.23

2.1. Structural Modeling

Finite element modeling has been selected for investigating the behaviour of studied structures. All the meshing procedure was done by GID. For simulating the behavior of steel plate shear walls, shell element is used. Different mesh sizes are tested to get to stable results. The results show that placing 16 elements per each panel between stiffeners lead to appropriate outcome.

To calculate the response modification factors, the pushover curve of structures are required. To evaluate the structural response curve, two different methods of analysis are generally used. These include, the inelastic pushover (static) analysis and the inelastic dynamic analysis. Owing to the simplicity of the former method compared to the latter, the inelastic pushover analysis is more widely used. In a study, Mwafy and Elnashai [2001] examined the applicability and limitations of the two methods on 12 reinforced concrete buildings with various characteristics. They concluded that the inelastic pushover analysis is more suitable for the short period and regular structures, but for long period (high rise) and special buildings, the inelastic dynamic analysis is preferable as it is better suited to account for the effects of higher modes. In line with the above conclusion, Incremental static inelastic analysis is applied to all models which were under triangular distribution loading to determine mentioned curves. Furthermore, the results of the six-story SPSW were compared with those obtained from dynamic inelastic response time-history analysis with incrementally increasing peak ground acceleration to verify the applicability of the static procedure.

Inelastic pushover analysis of the multi-storey systems under investigation was carried out at horizontal load steps with Static Riks analysis. A constant gravity load equal to total dead load plus 20% live load was also applied to each story. Pushover and idealized curves are shown in Fig. 2.2. for a six story model.

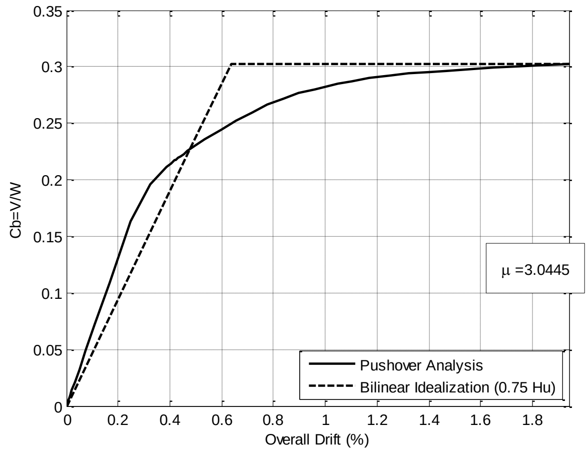

Figure 2.2. Pushover and idealized curve for six story model

3. RESULTS AND DISCUSSION

A number of performance parameters may govern the capacity of a structure. In order to carry out an inelastic pushover analysis, one or a number of these parameters should be considered for determination of the displacement limit state (Δ_{max}). For the type of regular, ductile buildings considered in this study, the global drift (maximum roof displacement) or the interstorey drift are commonly used for failure criteria. The interstorey drift is the collapse parameter that controls the response of buildings designed to modern seismic codes. For evaluation of R parameters in the present study, the ultimate capacity of each model was assumed to have been reached when the interstory drift reached 3% of story height. The R factor parameters for each system were extracted from the respective pushover response curve. The ductility dependent component, R_μ was calculated using Eqn. 1.9. and overstrength component, Ω, was determined from Eqn. 1.2. The fundamental period of vibration T was also determined by Eqn. 3.1. which proposed by Grondin. et.al [Anjan et al. 2011].

$$T = 0.04h_n \tag{3.1}$$

where, h_n is the building height.

Selective fundamental periods of vibration, T and the behavior factor parameters, μ, R_μ, Ω and R, evaluated from the pushover curves are listed in Table 3.1.

3.1. Nonlinear Static Analysis of Model Structures

3.1.1. Over strength factor

The capacity envelopes obtained from pushover analysis were utilized to evaluate overstrength factors. To find out the yield point, the original curve idealized by Paulay and Priestley method [ATC, 1995a]. The overstrength factors of all the analysis model structures are presented in Table 3.1., also, the Ω factors are plotted in Fig. 3.1.

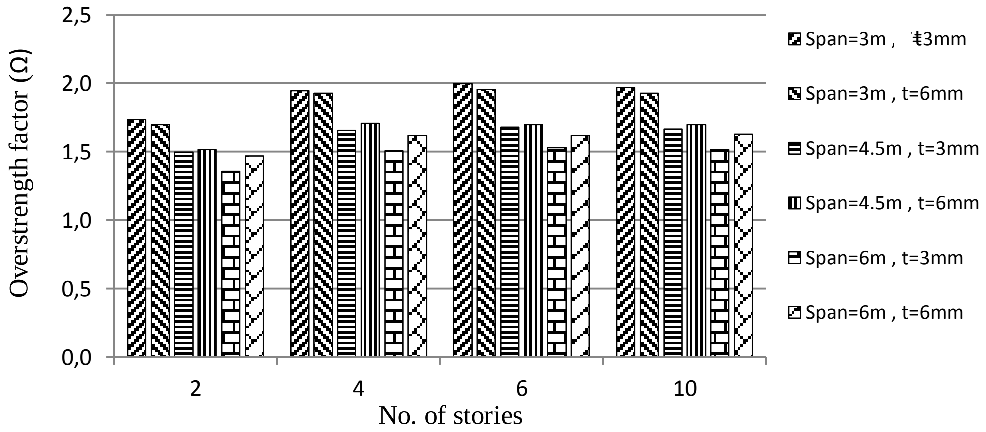

Figure 3.1. Overstrength factor of structures

It can be observed that the overstrength factors of models increase as the span length decreases. However those of SPSWs are not much affected by the change in number of stories and plate thickness.

3.1.2. Ductility factor

As mentioned previously, the ductility factor R_μ was obtained using the system ductility factor μ by the procedure proposed by Krawinkler and Nassar ($R_{\mu 1}$) and Miranda and Bertero ($R_{\mu 2}$). Fig. 3.2. and 3.3. show the system ductility factor R_μ of SPSWs. In all cases the factors computed by Miranda and Bertero turned out to be larger than those by Krawinkler and Nassar. It can be observed that the ductility factors increase as the span length increases.

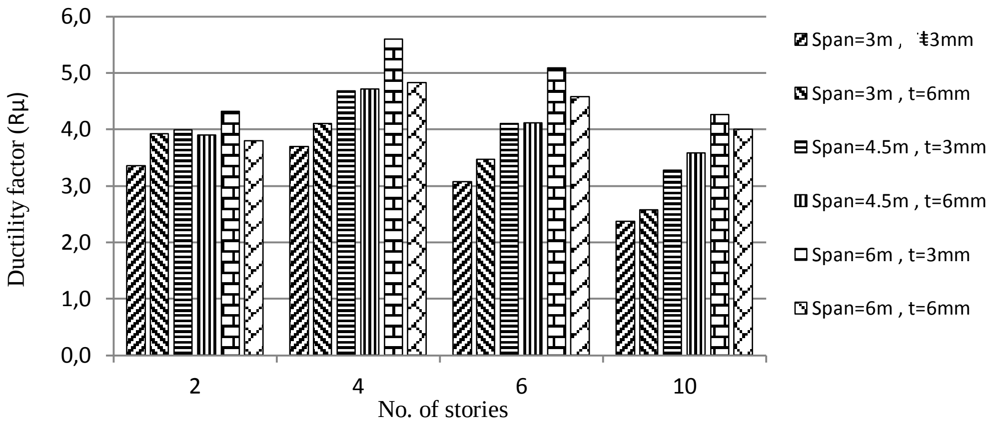

Figure 3.2. Ductility factor (by Krawinkler and Nassar [ATC, 1995a])

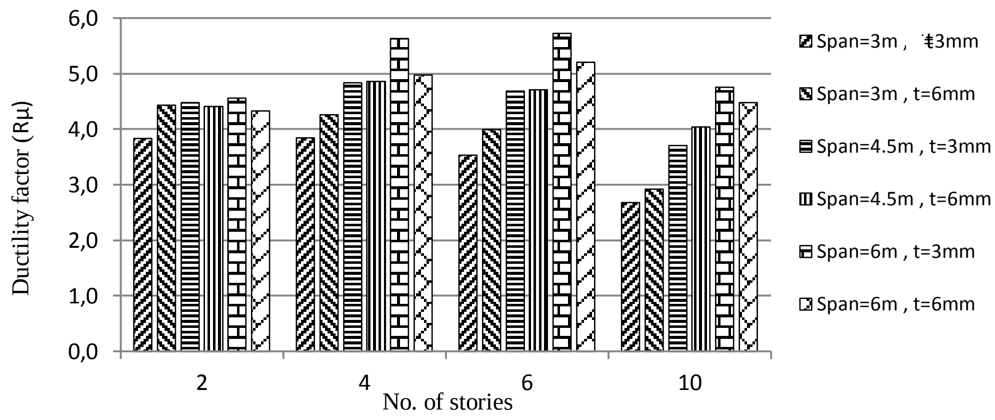

Figure 3.3. Ductility factor (by Miranda and Bertero [1994])

3.1.3. Response modification factor

The response modification factors, presented in Fig. 3.4., are computed by multiplying the allowable stress factor, overstrength factor and the ductility factor obtained in the previous sections.

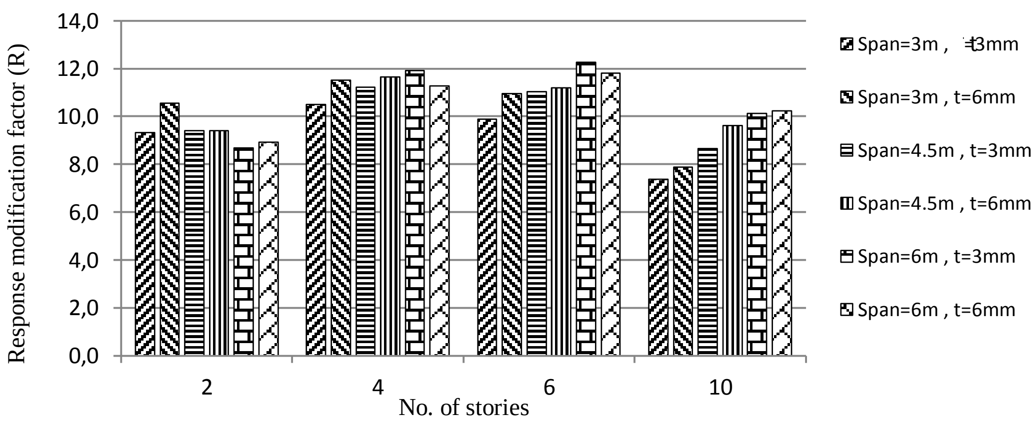

Figure 3.4. Response modification factor (by Miranda and Bertero [1994])

It can be observed that the response modification factors and number of stories, don't have any clear relation, while the R factors have range between 7.89 to 12.27. Also, the R factors appear to be comparatively little affected by spans and thicknesses.

3.2. Comparison with Incremental Dynamic Analysis Results

A series of incremental dynamic analyses were performed until all the predefined limit states were exceeded in order to verify the results of nonlinear static analyses. Five records were selected for dynamic analyses. Inelastic time history analyses were carried out using the 6 story model with 3 m span length, and the dynamic pushover envelopes were obtained by plotting the point corresponding to the maximum base shear and the maximum top story displacement computed for each scaled record. The intensities of the time history records were varied by multiplying appropriate scaling factors. The dynamic pushover envelopes were compared with the static pushover curve in Fig. 3.5., which shows that the dynamic envelopes form upper bound, specifically for displacement larger than the yield point.

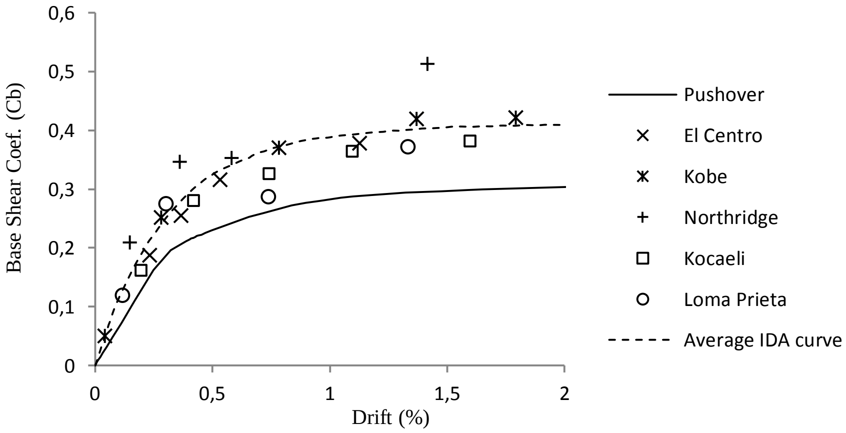

Figure 3.5. Static and dynamic pushover curves of the six-story SPSW

To obtain behavior factors, the five dynamic pushover envelopes are averaged and the average curve has been fitted into a bi-linear curve. The overstrength factor obtained in this way is 2.53 which is 26% larger than the factor obtained from the static pushover curve. Also, the ductility factor computed 4.44, which is larger than 3.53 obtained from the static pushover curve. Consequently the response modification factor results in 15.7, which is clearly 1.5 times larger than the corresponding value obtained from static pushover analysis, i.e approx. 10.

4. CONCLUSIONS

The overstrength, ductility, and the response modification factors of the 24 stiffened steel plate shear walls with various stories, plate thicknesses and span lengths were evaluated by performing pushover analyses. Some of the results were compared with those from nonlinear incremental dynamic analyses.

The overstrength factors of SPSWs increase as the span length decreases, but those are not much affected by the change in number of stories and plate thicknesses. Ω factors turned out to be 1.7–2 for 3 m span, 1.5–1.71 for 4.5 m span, and 1.36–1.63 for 6 m span.

The ductility factors that computed by Miranda and Bertero turned out to be larger than those by Krawinkler and Nassar. This factors increase as the span length increases. The ductility factors were obtained as 2.36–5.59 (Krawinkler and Nassar procedure) and 2.68–5.73 (Miranda and Bertero) for SPSWs.

The response modification factors were in the range of 7.89 to 12.27. Thus, this factor for stiffened steel plate shear wall are suggested as 7 and 10 for two designing methods, i.e. ultimate limit state method and allowable stress method, respectively.

The maximum base shear envelopes obtained by incremental dynamic analyses generally formed an upper bound to the static pushover curve. The response modification factors obtained from this procedures is clearly bigger than the value obtained from static pushover analysis.

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