COMPARISON OF PERFORMANCE OF THIN STEEL SHEAR WALLS AND CONCENTRIC BRACES BY CAPACITY SPECTRUM METHOD

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ABSTRACT :

In order to protect existing structure against probable damages in coming earthquakes, and also, in order to expand buildings or to change their applications, studying present conditions of structures and making them resistant against earthquake (if required) are necessary. Steel shear wall was considered as a lateral load carrying member in 1907s. Steel shear wall was used in buildings such as Nippon steel building in Tokyo and Sylmar building in Lose Angeles for the first time. Many experimental and theoretical studying has been done on steel shear walls. But, only in Canada building code (CAS, 1994) a discrete section is devoted to steel shear walls. Among methods of designing structures against earthquake, designing based on performance levels has been taken into consideration in building codes such as ATC40, FEMA273, and FEMA274 because of considering inelastic behavior of structures. Designing based on performance levels is used in the improvement code of Iran, too. Capacity spectrum method and nonlinear static method (NSP) is one of the proposed methods by the improvement code of Iran for making existing weak structures resistant against earthquake. In this paper, earthquake-resistance of weak steel moment frame structures with steel shear walls, cross and chevron braces, are compared with each other using performance levels and capacity spectrum method. SAP and ANSYS soft wares will be used for modeling and analysis.

KEYWORDS: steel shear wall, performance levels, capacity spectrum method

برسد به دست دکتر معلم نژاد

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چکیده:

به منظور پیشگیری از خسارت احتمالی بر ساختمانهای موجود در زلزله های آتی و نیز برای توسعه ساختمانها ویا تغییر کاربری,بررسی وضعیت کنونی و در صورت نیاز مقاوم سازی آنها الزامی است.دیوار برشی فولادی(SSW) به عنوان عضو باربر جانبی در دهه 1970 مورد توجه قرار گرفت .ساختمانهای نیپون استیل در توکیو و سیلمار در لس آنجلس جز اولین ساختمانهایی هستند که در آنها از دیوار برشی فولادی استفاده شده است. علیرغم بررسی های آزمایشگاهی و تئوری روی دیوار برشی فولادی تنها در آیین نامه کانادا (CAS,1994) به طور صریح یک بخش به این عنصر باربر اختصاص داده شده است.اخیرا از میان روشهای طراحی ساختمانها در برابر زلزله روش "طراحی بر اساس سطوح عملکرد"به دلیل در نظر گرفتن رفتار غیر ارتجاعی سازه ها در دستورالعملهای ATC40,FEMA274,FEMA273 مورد توجه قرار گرفته است.دستورالعمل بهسازی ایران نیز از طراحی بر اساس سطوح عملکرد استفاده کرده است.یکی از روشهای پیشنهادی دستورالعمل بهسازی برای مقاوم سازی سازه های ضعیف موجود در برابر زلزله روش طیف ظرفیت و به کار گیری روش استاتیکی غیر خطی(NSP)می باشد.در این مقاله مقاوم سازی سازه های فولادی قاب خمشی ضعیف در برابر بار زلزله به وسیله دیوار برشی فولادی یا بادبندهای ضربدری و هشتی(CHEVRON)با استفاده از سطوح عملکرد و به روش طیف ظرفیت مقایسه می شوند.برای مدل کردن و آنالیز از نرم افزارهای ANSYSوSAP2000 استفاده خواهد شد.

واژه های کلیدی:دیوار برشی فولادی،طراحی بر اساس سطوح عملکرد،روش طیف ظرفیت.

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1. INTRODUCTION

Stability and resistance of structures due to earthquake important and essential matter in Iran as it is located on Alp-Himalaya earthquake belt. Steel moment frames, are used normally as its ability to over come the architectural problems. Although these structures have a suitable ductility, they don't have enough stiffness which results increasing lateral displacements. for controlling "lateral displacements", we have to increasing the beams and columns sections which is not economic. as important the lateral displacement control retrofitting of structures with steel moment frames are requested by their owners.

Concentric Braces are usually used for controlling the lateral displacement of moment frame steel structures, but using of these members decrease the ductility of structures. There for thin steel shear walls have been used recently to retrofitting of the weak structures against lateral displacement. In a

Comparison with the other methods of retrofitting such as concentric and eccentric bracing, thin steel shear walls are not used in Iran as the limited knowledge about the performance and design methods. In this article, the behavior of 5 intermediates steel moment frames which were retrofitted by thin steel shear wall, X shaped and chevron brace are studied and compared with initial structure. The steel frames have different height and bays. The behavior of structures are studied using performance levels, Capacity Spectrum Method, and their behavior factor of structures have been evaluated.

2. PERFORMANCE LEVEL DESIGH METHOD[1]

"Strength Design Method", which is based on the parameter of strength, are not suitable to represent the behavior of structures in many cases. in this method three main performance levels is defined for structures. So because of this, in recent years, the issue of "performance level design" is mattered instead of "strength design". This modern design method, describes 3 original performance levels for structure members. (Figure 1)[2]. In this method, the performance point is determined by attention to the expected performance level. For determining the performance level, various methods are mentioned in various codes. The most important of them are "Capacity Spectrum" and "Displacement Factor" method. "Capacity Spectrum" method which is used in this article is represented by ATC American council in magazine No.40 [3]. In this method, the intersection point of Capacity Spectrum of structure and decreased seismic demand spectrum, is introduced as Performance Point.

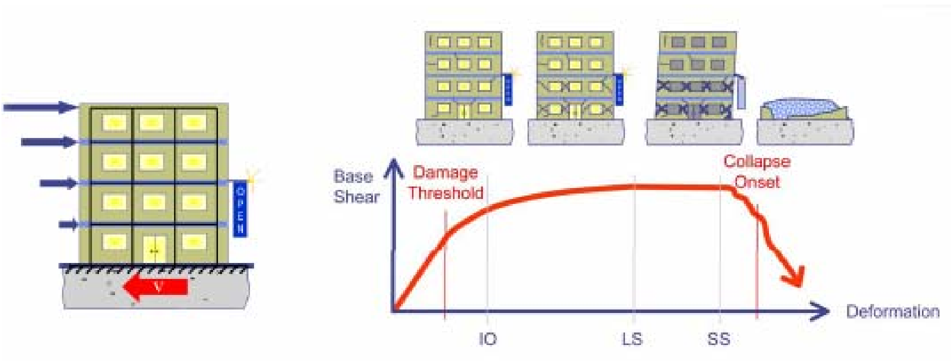

Fig1. Performance Level of Structure

For achieving the capacity spectrum which is really load-displacement curve of structure. A series of step by step analysis is done. In each step, the model of structure is modified so that decrease in strength cause by formation of plastic hinge is considered. This process will continue until the structure approaches to predefined restrictions. Demand Spectrum, in fact, is the design spectrum in code which its value is decreased in nonlinear part-because of incensement in the affective damping of structure. According to this matter, ATC-40 magazine introduces a reduce factor for the fields of constant acceleration and velocity spectrum, related to the amount of structure approach to nonlinear field, as following:

$$SR_A = \frac{3.21 - .68Ln(\beta_{eff})}{2.12} \quad (1)$$

$$(2) \quad SR_V = \frac{2.31 - .41Ln(\beta_{eff})}{1.65}$$

$$(3) \quad \beta_{eff} = \beta_0 + 5$$

β_{eff} =effective damping, β_0 = the viscose damping resulted from Hysterias behavior of structure

It's necessary to mention, in this method, the demand spectrum and capacity spectrum must be converted in form of S_a against S_d Chart (ADRS). By attention to this matter that the amount of effective damping of the points on capacity spectrum is different, so reduction factor of demand spectrum will be different for various points. So because of this, achievement of performance point of structure is a repetitive process with try and fault.

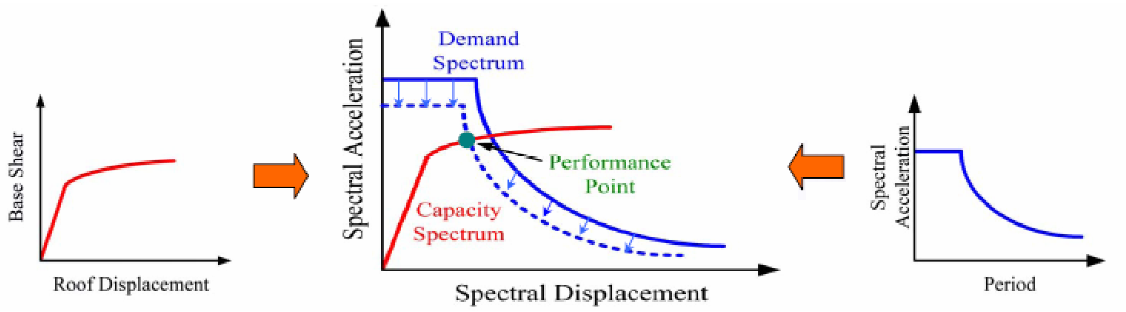

Fig2. Process of determining performance point

This method, when used for determining seismic parameters of structure, then the performance point is clear and in fact is the failure point of structures-considering safety band. In ACT-40, the maximum allowable lateral displacement for satisfying health safety-similar to what is named "intermediate important buildings" in 2800 Iranian Code- is 0.02 of structure height.

3. THIN STEEL SHEAR WALL

Steel plate shear walls are a kind of innovated resisting systems against wind and earthquake lateral loads as what is shown in figure (3). This system includes a series of separate panels which each one is inscribed in two beams and columns and a steel sheet is connected to these circumscribed elements. This configuration, will make the "steel shear wall" similar to a cantilever plate girder which columns are as its flanges and beams are as its vertical stiffeners and plate is as its web. [4] (Fig (4))

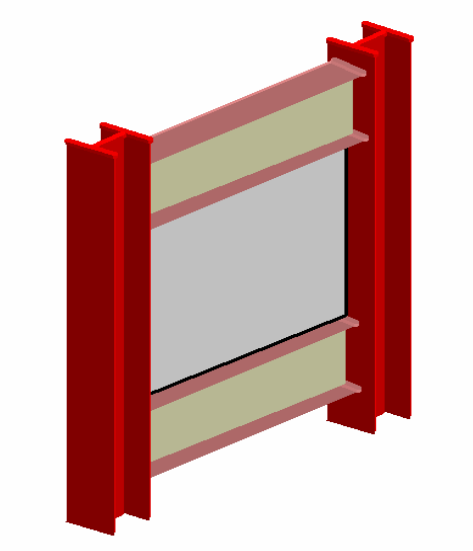

Fig.3-steel shear wall and circumscribed moment frame.

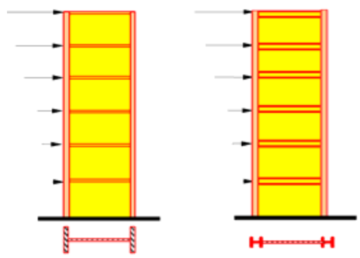

Fig.4-steel shear wall (Right) and plate girder (Left)

The basic idea of use of thin steel shear wall is usage of diagonal tension field which is formatted after buckling of steel plate. This post buckling strength of thin steel plate, results in damping of a lot amount of earthquake applied energy. Certainly, enough and well strengthening of plate in borders is the necessary condition of formation of this tension field which in this situation post buckling strength will be some times of buckling strength.[5]

4. THE INTRODUCTION OF UNDER STUDY FRAMES

In this article as shown in Figures No. (5),(6),(7) , 5 intermediate moment resistant frames are considered which don’t satisfy displacement necessaries of Iranian 2800 code. These frames have been retrofitted by X-shaped or chevron braces or thin steel shear wall. The equal element between these completing systems is "weight". On the other hand, X-shaped and chevron braces and steel shear wall have equal weights.

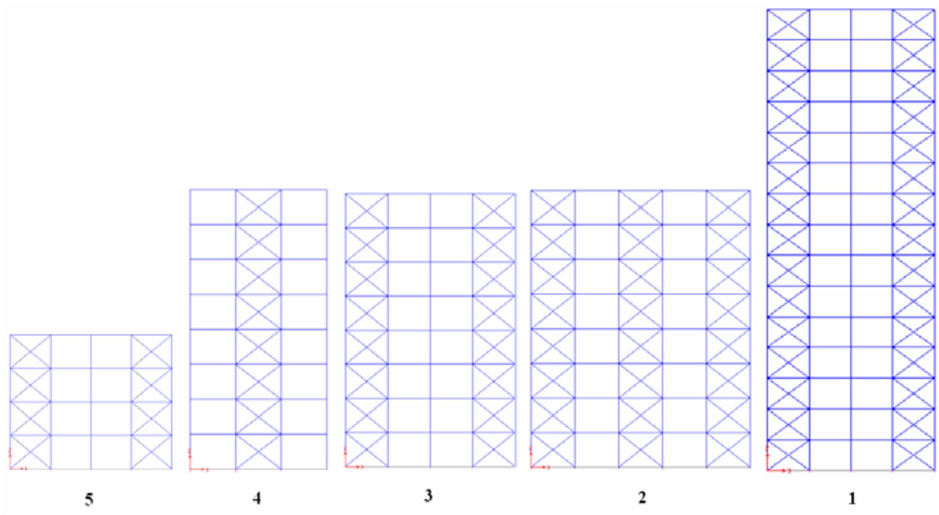

Fig.5-frames retrofitted by X-shaped brace.

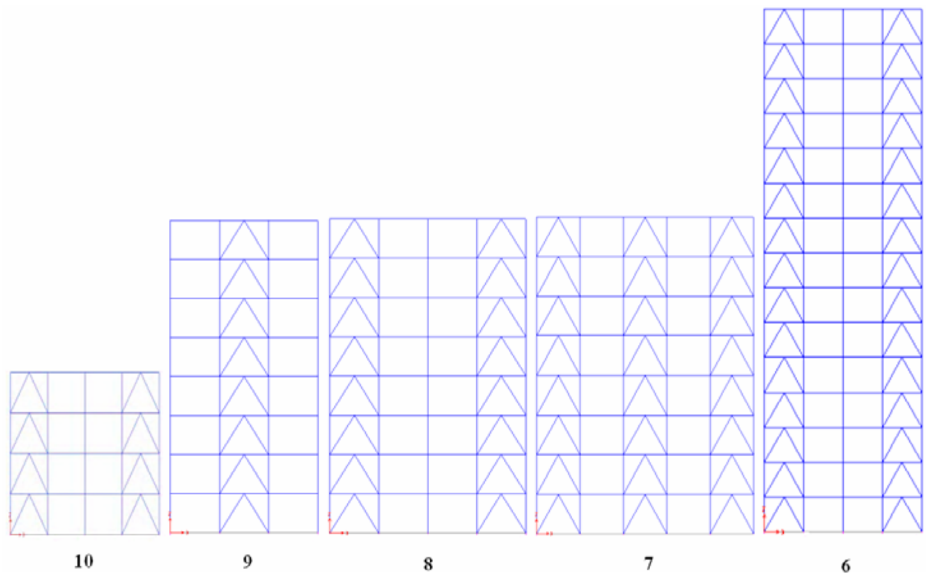

Fig.6-Frames retrofitted by chevron brace.

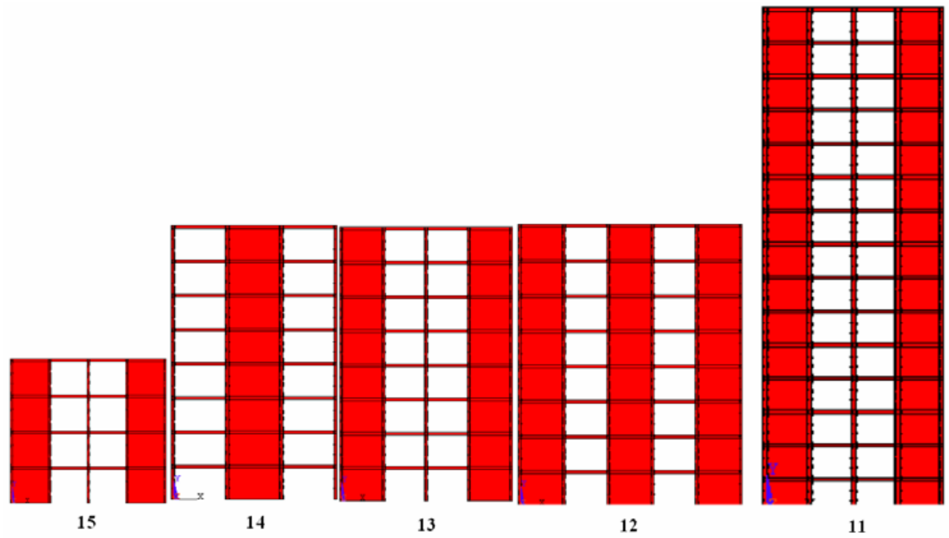

Fig.7-Frames retrofitted by thin steel shear wall.

These frames belong to the buildings witch have occupational application and have 4 meter equal bays and 3 meter story height. The position of building is relatively high hazard region and has type II soil. The considered roof type is joist and block. Dead and live loads of stories is calculated 600 and 200 kg/m^2 respectively.
IPB sections are used for columns and IPE sections for beams and double channel sections are used for braces. All columns and braces of a story are of the same type and also all of 3 stories are the same in height (Except 4-story frames which are of the same type in each 2 stories).

1. DETERMINING OF THE PERFORMANCE POINT OF FRAMES

Frames retrofitted by thin steel shear wall have been modeled and analyzed by ANSYS 5.4 finite element software and Frames retrofitted by concentric braces have been modeled and analyzed by SAP2000 Software and the base shear curve against roof displacement is achieved.
And then performance point is calculated by the use of Microsoft office excel. As a sample the performance point-which is the intersection point of capacity spectrum and demand spectrum-is shown in Figure No. (8) For frames No. 3, 8, 13.

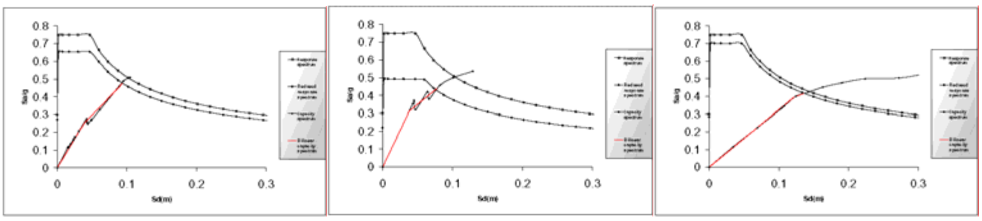

Fig.8- performance point of frame 13 (Right hand). Frame 8 (middle). Frame 3 (left hand).

The Displacement and base shear force of performance point for under-study frames is represented in Table No.(1).

Table 1. Displacement and base shear of performance point for studied frames. (Displacements are in cm and base shear is in Ton).

Frame No.	Roof Displacement	Base shear	Frame No.	Roof Displacement	Base shear	Frame No.	Roof Displacement	Base shear
1	-	-	6	-	-	11	37.04	115.81
2	-	-	7	-	-	12	17.54	128.36
3	13.17	124.4	8	10.1	100.9	13	19.11	97.47
4	19.4	85.25	9	13.3	72.6	14	22.13	61.48
5	2.62	69.1	10	2.18	78.5	15	8.9	57.05

The gaps of table demonstrate no intersection of capacity spectrum and demand spectrum. Obviously, "frames (1), (2), (6), (7)" aren't desirable retrofitting systems. All of the numbers related to lateral displacement don't satisfy the condition "0.25H" which is related to ATC-40 for occupation buildings.

2. BEHAVIOR FACTOR OF THIN STEEL SHEAR WALL IN MOMENT FRAME

The nearest analyze type to actual behaviour of structures encountering earthquake is nonlinear dynamic analysis. But because it's time-consuming, Designers usually try to use simpler analysis methods. One of these methods is linear static analysis while considering a suitable behaviour factor. Behaviour factor equalizes the nonlinear behaviour of structure. There are two main methods to determine behaviour factor.

1- Uang ductility factor method.[6]
2- Freeman capacity spectrum.

The first method is used in this article.

4.1. UANG DUCTILITYFACTOR METHOD

In figure (9), the general behavior of a structure, is shown in the form of base shear against lateral displacement curve graph. In this graph, the curve is described by the actual behavior and bilinear ideal behavior. The vertical and horizontal axes in this graph are respectively stating the base shear and general displacement or story drifts of structure.

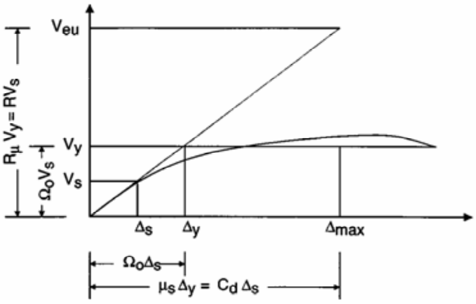

Figure (9). The actual and ideal capacity curve.

These equations are used to determine behavior factor:

$$R = R_\mu \times \Omega_0 \times \gamma \quad (4)$$

$$(5)\qquad \Omega_0 = \frac{V_y}{V_S}$$

$$(6)\quad \mu_s = \frac{\Delta_{\max}}{\Delta_y}$$

In equation No.4 R_μ is called the reduction factor resulted from ductility and is calculated through μ_s which is called ductility factor. γ (Allowable tension factor) is recommended 1.40 in most of codes. And Ω_0 is called the reduction factor resulting from excessive strength. For achieving R_μ through Miranda method [7] and according to equation No.7 that is recommended for sedimentary soils.

$$(7)\quad R_\mu = \frac{\mu_s - 1}{\varphi} + 1 \le \mu_s$$

$$(8)\quad \varphi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp[-2(\ln T - 0.2)^2]$$

In above equations because of considering T, which is the fundamental period of structure, the more exact answers are expectable.
By attention to capacity curve and above equations, behavior factor for dual intermediate moment frame system is calculated and inserted into table (2).

Table 2. the behavior factor of dual intermediate moment frame and thin steel shear wall.

Frame No.	11	12	13	14	15
Behavior Factor	8.88	8.29	9.53	10.52	6.53

5.CONCLUSIONS

1. By attention to performance point, use of thin steel shear wall for retrofitting of intermediate and tall buildings is more suitable than concentric braces.
2. Retrofitting by the use of X-shaped brace looks more suitable than chevron brace because chevron brace increases the probability of formation of plastic hinge in column and in following increases the probability of happening brittle failure.
3. The achieved behavior factor for dual intermediate moment frame system and steel shear wall for short buildings (6.5) is less than behavior factor of intermediate moment frame system and X-shaped brace but this factor for tall and intermediate buildings is more than behavior factor of dual intermediate moment frame and X-shaped brace.

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