2003 Pacific Conference on Earthquake Engineering

The relationship between overstrength and members ductility of RC moment resisting frames

M. Mahmoudi

Department of Civil Engineering, Sari Technical College, Sari, Iran.

ABSTRACT: Overstrength is the strength in excess of seismic code requirements. It depends on different factors which the most important of them is members ductility. This paper investigates the relationship between overstrength and members ductility of various R.C. moment resisting frames having one, two, three, four, five, six, eight, ten and fifteen stories with three spans. Assuming they are located in high seismicity regions, they were loaded on the basis of the Iranian seismic code and designed according to ACI-318 code. This investigation was carried out using DRAIN-2DX computer program by applying the monolithically increasing horizontal loading (pushover analysis). The results indicate that the overstrength depends on members ductility considerably and its amount is not equal for structures having low, medium and high ductility.

1 INTRODUCTION

According to the seismic codes the buildings are allowed to use overstrength against strong earthquakes. Overstrength is the strength in excess of seismic code requirements resists while plastic hinges formed frequently in relevant structural members. Overstrength helps structures to stand against sever earthquakes safely. Overstrength also reduces the elastic strength demand. This object is performed using force reduction factor by several codes of practice.

The overstrength depends on the different factors. The main sources of overstrength may be of the following (Fischinger & Fajfar 1991).

- Redundancy of the structure and redistribution of forces (stresses) between structural members.
- Reinforcement steel strength greater than the specified yield strength.
- Additional strength of concrete at a given age of the structure being higher than specified.
- Strain rate effect at inelastic state.
- Code’s recommendations such as limitation of lateral displacement, minimum reinforcement and members size.
- Presence of non-structural elements.
- Ignoring the three dimensional effect on two dimensional analysis.
- The ductility of structure, which is the most important one.

Only the ductile structures exhibit overstrength. The ductility of structures relate to their components ductility. This paper investigates the relationship between overstrength and members ductility of R.C. moment resisting frames having various stories.

The inelastic non-linear analysis was carried out using DRAIN-2DX computer program by applying the monolithically increasing horizontal loading, called pushover analysis, to all frames.

2 STRUCTURAL MODEL

In this investigation nine, two dimensional R.C. moment resisting frames were selected. These three-span frames comprise one, two, three, four, five, six, eight, ten and fifteen stories. Story height and span length throughout the structure for all frames were assumed to be 3 and 4 meter respectively. Same cross section for beams and columns in each story was assumed. The structures were loaded on the basis of the Iranian seismic code (standard no. 2800) in high seismicity regions. Their analysis was carried out using SAP90 computer program and they were designed according to ACI 318-92.

f_c' And f_y for all frames were assumed 240 kg/cm^2 and 2400 kg/cm^2 respectively. The members dimensions and percentage of tensile reinforcement (the area of longitudinal flexural tensile steel over the gross section area) is provided in Table 1 and 2 (Tasnimi and Mahmoudi 1998).

Table 1. Members dimensions

Member Symbol	Dimension (cm)	Member Symbol	Dimensions (cm)
B1	20 × 30	C1	30 × 30
B2	25 × 30	C2	35 × 35
B3	25 × 35	C3	40 × 40
B4	25 × 45	C4	45 ×45
B5	25 × 50	C5	50 × 50
B6	30 × 50	C6	55 × 55
B7	35 × 50	C7	60 × 60
		C8	70 × 70

3 PUSHOVER ANALYSIS

Pushover analysis is an inelastic non-linear analysis method carried out by applying the monolithically increasing horizontal loading. It has been stated that the pushover analysis is simple and provides reasonable accurate results compare to non-linear dynamic time-history analysis. It provides that the structure oscillates predominantly in its first mode. All of two dimensional frames were subjected to pushover analysis using DRAIN-2DX computer program. The invert triangular distribution load along the height of the building was used recommended by Iranian building code of practice for seismic resistant design (IBCS-2800).

Design base acceleration equal to 0.35g was selected for the structures locate in high seismicity regions. Calculated dynamic characteristics such as fundamental periods (T), response coefficient (B, obtained from response spectra), base shear coefficient (C), effective weight (W_{eff}) and base shear (V) for all frames tested in the analysis are listed in Table 3.

The increment of each loading step was selected, one to two percent of total load (base shear) throughout the analysis (elastic and inelastic range). Inelastic range starts at the stage of first plastic hinge formation and ends when the mechanism is formed.

Two parameters were determined during pushover analysis (in each step), imposed total force (base shear) and rotation of members at both ends, which will be explained more later.

4 DETERMINATION OF OVERSTRENGTH

As mentioned earlier, the overstrength is the strength in excess of seismic code requirements. So it is revealed in elastic and inelastic steps but in this paper the overstrength related to inelastic behaviour will be described. Overstrength usually is defined using overstrength factor obtained by dividing strength level appeared in each step of inelastic range to strength level existed on first plastic hinge.

Table 2. Members specification

Frames	Position of members at relevant stories	Beams		Columns	
		symbol	steel	symbol	steel
One story	1st	B1	0.024	C1	0.02
Two stories	1st	B3	0.015	C2	0.013
	2nd	B3	0.015	C1	0.0127
Three stories	1st and 2nd	B3	0.02	C3	0.02
	3rd	B2	0.023	C2	0.014
Four stories	1st and 2nd	B4	0.02	C4	0.017
	3rd and 4th	B3	0.014	C2	0.014
Five stories	1st and 2nd	B5	0.017	C4	0.017
	3rd and 4th	B4	0.017	C3	0.017
	5th	B2	0.02	C2	0.026
Six stories	1st and 2nd	B5	0.02	C5	0.013
	3rd and 4th	B5	0.017	C4	0.011
	5th and 6th	B3	0.02	C3	0.014
Eight stories	1st and 2nd	B6	0.02	C6	0.013
	3rd and 4th	B6	0.021	C5	0.01
	5th and 6th	B5	0.019	C5	0.01
	7th and 8th	B3	0.022	C3	0.017
Ten stories	1st and 2nd	B7	0.02	C7	0.015
	3rd and 4th	B6	0.022	C6	0.011
	5th and 6th	B6	0.021	C5	0.014
	7th and 8th	B5	0.019	C5	0.015
	9th and 10th	B3	0.022	C3	0.019
Fifteen stories	1st and 3rd	B7	0.023	C8	0.011
	4th and 6th	B7	0.024	C7	0.01
	7th and 9th	B7	0.022	C6	0.014
	10th and 12th	B6	0.021	C5	0.011
	13th and 15th	B6	0.018	C4	0.017

To obtain the overstrength factors (R_s) for each step of loading, it should be divided the strength resisted by frames (or imposed base shear force, V_i) by base shear force at first hinge formation point (V_s), as shown in Equation 1 and Figure 1.

$$R_s = \frac{V_i}{V_s} \tag{1}$$

Table 3. Dynamic characteristic based on IBCS-2800 recommendation

Frames	T (s)	B	C	W (ton)	V (ton)
One story	0.16	2.0	0.14	29.66	4.12
Two stories	0.268	2.0	0.14	59.33	8.29
Three stories	0.36	2.0	0.14	88.99	12.45
Four stories	0.45	2.0	0.14	118.66	16.60
Five stories	0.53	2.0	0.14	148.32	20.76
Six stories	0.61	2.0	0.14	177.99	24.91
Eight stories	0.76	1.89	0.13	237.32	30.85
Ten stories	0.9	1.69	0.118	296.65	35.00
Fifteen stories	1.22	1.36	0.097	444.98	43.15

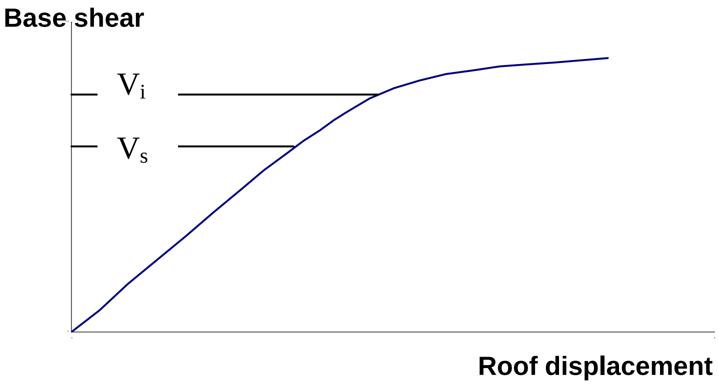

Figure 1. Global response of structure

5 MEMBERS DUCTILITY

It is evident that the structure will be ductile when their members are ductile. So it is important the recognition of members ductility (local ductility) and its relationship to overstrength.

There are two ways to identify members ductility: rotational ductility factor and curvature ductility factor.

5.1 Curvature ductility factor

Curvature ductility factor could be explained by Figure 2. In this figure the moment-curvature response of members is shown by bilinear ideal model. Horizontal axis represents the curvature of the end of members and vertical axis represents the moment at the end of members. Curvature ductility factor (m_f) is obtained using Equation 2.

$$m_f = \frac{f_u}{f_y} \tag{2}$$

Where f_u is the maximum curvature and f_y is the yield curvature determined by Equation 3.

$$f_y = \frac{M_y}{EI} \tag{3}$$

M_y is the yield moment, E is the modulus elasticity and I is the moment of inertia of cracked section.

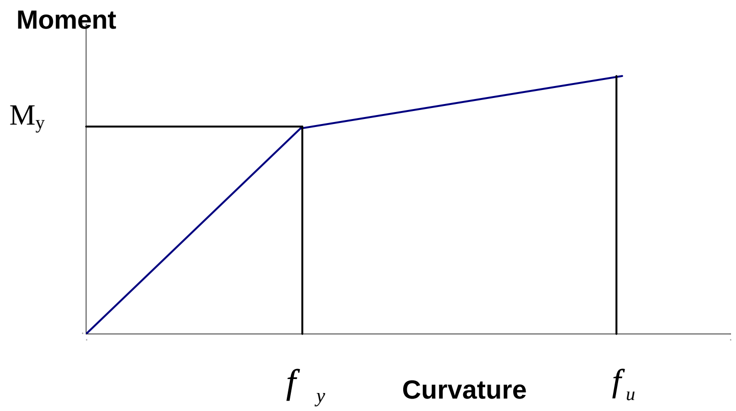

Figure 2. Moment-Curvature curve

5.2 **Rotational ductility factor**

Rotational ductility factor (m_q) could be obtained using Equation 4.

$$m_q = \frac{q_u}{q_y} \tag{4}$$

Where q_u is the maximum rotation at the end of member and q_y is yield rotation found Equation 5 (Plumier 1988).

$$q_y = \frac{M_y}{6EI/L} \tag{5}$$

L is the length of member.

There is relation between rotational ductility and curvature ductility (Mahmoudi 1999) represented by Equation 6.

$$m_q = \frac{6L_p}{L}(m_f - 1) \tag{6}$$

Where L is the length of member and L_p is the plastic hinge length defined as follow (Wakabayashi 1986).

$$L_p = 0.5d + 0.05z \tag{7}$$

d is the effective depth of section (the distance between area center of tensile steel and farthest point of compressive concrete) and z is the distance between plastic section and turning point. (the later is usually the half of total length of member).

Using the Equation 6 it is possible to obtain rotational ductility in terms of curvature ductility. So there is no problem to select rotational ductility factor as member ductility.

6 OVERSTRENGTH AND MEMBERS DUCTILITY RELATION

To identify the relationship between overstrength and members ductility, it is necessary to calculate overstrength factor and ductility factor simultaneously in each step.

Base shear relevant to formation of first plastic hinge (V_s) for all frames are as Table 4.

Table 4. Base shear relevant to formation of first plastic hinge

Frames	Vs (ton)	Frames	Vs (ton)
1 story	11.5	6 stories	36.47
2 stories	12.1	8 stories	44.94
3 stories	17.62	10 stories	50.88
4 stories	24.55	15 stories	62.61
5 stories	30.45		

Using Equation 5, yield rotation (q_y) were calculated for all members (beams and columns) that the plastic hinge were formed at the end of them.

Using equations the overstrength factors and rotational ductility factors were obtained for all members subjected to yield plastic hinge in each step of loading in all frames separately.

Members ductility factors for said members were plotted against calculated overstrength factors for all frames separately. Using the curves the critical members were selected and the relationship between overstrength factors and the most critical beams and columns ductility factors were prepared and illustrated in Figures 3-11. For example Figure 3 shows the said relationship in which vertical axis represents the members ductility factors for the most critical beams and columns selected among the beams and columns having plastic hinge(s), and horizontal axis represents the overstrength factors related to one story frame. Using this figure, it is possible to define ductility demand factors in terms of overstrength factor, or to specify overstrength factor in terms of ductility capacity factors

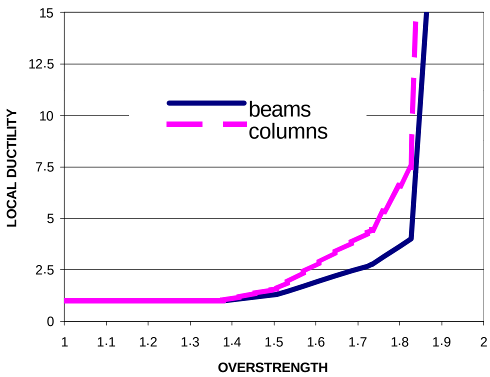

Figure 3. Overstrength and local ductility relationship for one story frame

The results extracted from figures 3-11 are as follow:

- The ductility in excess of 7.5 doesn't affect on overstrength factor seriously.
- The effect of columns ductility factors on overstrength factor is higher than beams ductility factors effect.
- The higher buildings have the lower overstrength.
- The role of ductility factor on determination of overstrength factor for high buildings is lesser than short buildings.
- The overstrength depends on members ductility strongly. The relationship between them is introduced in Table 5 for beams and columns separately.

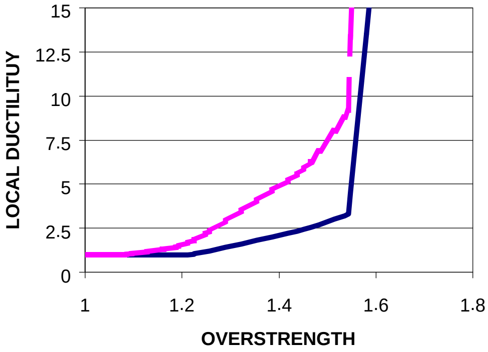

Figure 4. Relationship for two stories frame

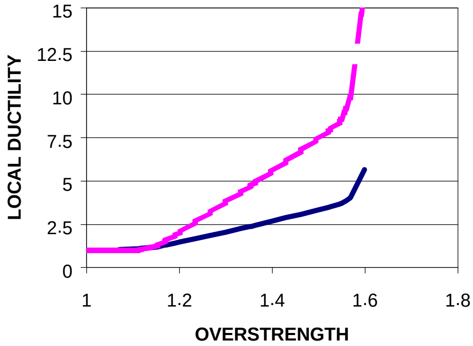

Figure 5. Relationship for three stories frame

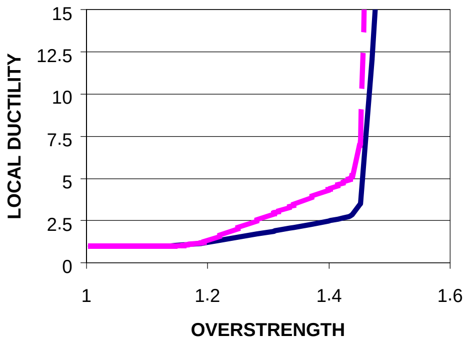

Figure 6. Relationship for four stories frame

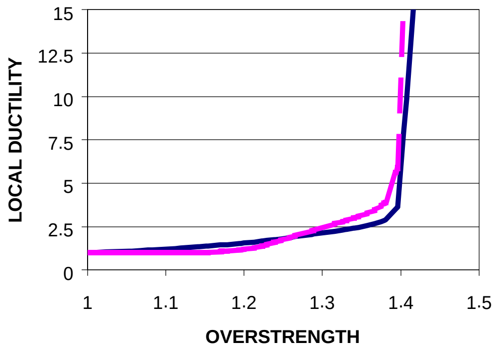

Figure 7. Relationship for five stories frame

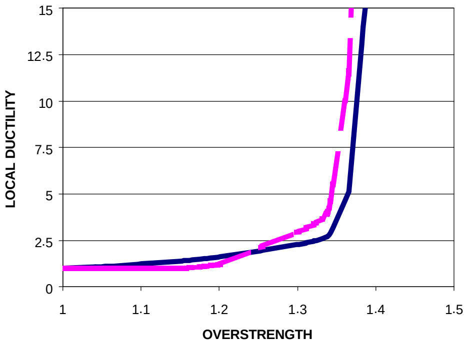

Figure 8. Relationship for six stories frame

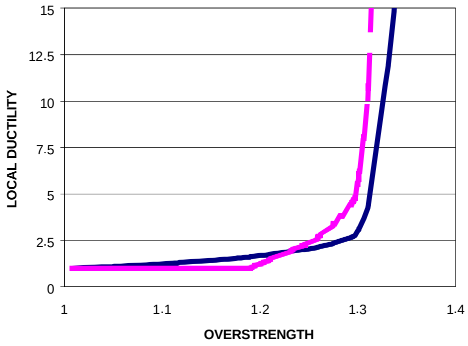

Figure 9. Relationship for eight stories frame

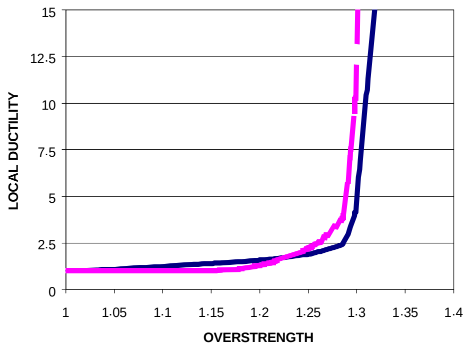

Figure 10. Relationship for ten stories frame

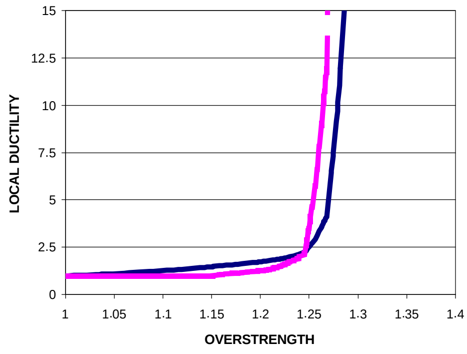

Figure 11. Relationship for fifteen stories frame

Table 5. Dependence of overstrength on beams and columns ductility factors

Members ductility capacity		One story	Two stories	Three stories	Four stories	Five stories	Six stories	Eight stories	Ten stories	Fifteen stories
2.5	Beams	1.69	1.47	1.36	1.40	1.35	1.32	1.28	1.29	1.25
	Columns	1.56	1.27	1.23	1.28	1.30	1.28	1.25	1.26	1.25
5	Beams	1.83	1.55	1.58	1.44	1.4	1.37	1.31	1.30	1.27
	Columns	1.75	1.44	1.37	1.43	1.39	1.34	1.30	1.29	1.25
7.5	Beams	1.84	1.57		1.46	1.41	1.38	1.32	1.31	1.28
	Columns	1.83	1.49	1.5	1.45	1.4	1.35	1.31	1.29	1.26

7 CONCLUSION

The simple inelastic non-linear static analysis of R.C. moment resisting frames provide good estimate of relationship between overstrength factor and members ductility factor.

The results indicate the overstrength depends on members ductility strongly and it is not equal for the low, medium and high ductility structures. The higher buildings have the lower overstrength. The dependence of overstrength on columns ductility is more than beams ductility.

REFERENCES:

Fischinger, M. & Fajfar, P. 1991. On the response modification factors for reinforced concrete buildings, *Proceeding of fourth U.S. national conference on earthquake eng., 20-24 May 1990. Vol. 21.*

Mahmoudi, M. 1999. Prediction of global ductility based on local ductility (Persian), *Proceeding of sixth conference on earthquake engineering, Rasht, Iran, May 1999, P. 122-134*

Plumier, A. & Boushaba, B. 1988. Relation entre la ductilite lelocale et le facteur de comportement sismique de structures en acier (French), *construction me tallique, 1988, No. 2, P. 59-70.*

Tasnimi, A.A. & Mahmoudi, M. 1998. Prediction of overstrength of reinforced concrete moment resisting frames, using non-linear inelastic analysis, *Proceedings of the eleventh european conference on earthquake engineering,Paris, 1998.* Balkema: Rotterdam.

Wakabayashi, M. 1921. Design of earthquake-resisting buildings.