Dynamic inelastic behavior of chevron braced steel frames

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ABSTRACT: A parametric study of the dynamic inelastic behavior of chevron braced steel frames reveals some important phenomenae. The effects of brace slenderness, beam stiffness and frame participation on several response quantities are investigated. Design guidelines are presented based on these analytical results.

INTRODUCTION

Concentrically braced frames, chevron or K braced frames in particular, have shown a poor performance in recent earthquakes. Their behavior is complex and still not as well understood as that of other structural systems. Several studies (Nilforoushan 1973, Singh 1977, Jain 1978, Inoue 1978, Sakamoto 1978, Fujiwara 1980) have looked at different aspects of braced frames behavior and have arrived at conflicting conclusions. Poor performance and lack of definitive design guidelines have prompted many to propose banning K-braced frames entirely or to stipulate the use of large design lateral loads (Aoyama 1981, SEAONC 1982, SEAONC 1985, Walpole 1985).

As part of an ongoing research program at Berkeley on the behavior of K braced steel frames subjected to earthquake loading, a systematic investigation of the parameters controlling their response is currently underway. In the first phase, force redistributions under quasi-static, monotonically increasing lateral load are studied. In the second phase, the sensitivity of response to dynamic loading is investigated, considering various design parameters. These results provide a qualitative understanding of the influence of the governing design parameters and serve as a basis by which to judge the performance of the optimization algorithm used in the third phase. In the third and last phase an optimization approach is being used in an attempt to improve the response of chevron braced frames.

This paper describes the results of the dynamic analysis phase of the research. It consists of three main sections. The first section summarizes the findings of the first phase of the research program. The second section describes the investigation plan used in the dynamic analyses and the third section summarizes the findings, identifying design guidelines as appropriate.

BEHAVIOR UNDER QUASI-STATIC LOADING

Nonlinear quasi-static analyses of K braced frames indicate that the main parameters controlling force redistribution were the brace slenderness and the beam relative stiffness (Khatib 1987). When a K braced bay is subjected to a lateral load the braces resist most of the load; one in tension and the other in compression. After the compression brace buckles a vertical unbalance force results on the beam at the brace junction. The subsequent resistance characteristics of the structure depend mainly on brace slenderness and beam stiffness (figure 1).

Brace slenderness was found to influence the post-buckling stiffness of the brace. Braces with small slenderness ratios (0-80) have a mildly negative post-buckling stiffness. Their high buckling stress means that only a relatively small cross-sectional area is needed to achieve a given buckling load, implying a smaller elastic stiffness compared to more slender braces. The frame participation in resisting the load is accordingly more significant. Moreover, the high buckling stress reduces the maximum unbalance load that can be applied on the beam.

Braces with relatively high slenderness ratios (130-200) have a long range of elastic buckling where the brace axial stiffness is theoretically zero. This is followed by a rapid deterioration of load capacity. Since their buckling stress is very low, they must have large cross-sectional areas to achieve the desired buckling load. Their elastic stiffness is therefore very high compared to that of the frame. Large unbalance loads could be imposed on the beam.

Braces with intermediate slenderness ratios (80-130) are characterized by a rapidly deteriorating post buckling resistance; they have the most negative post-buckling stiffness. The unbalance load that they can apply on the beam grows rapidly with increasing deformation.

Equally important are the stiffness and strength of the beam to which the braces are connected. A relatively flexible beam does not allow the tension brace to develop its full resistance; the story force deformation curve is similar to that of the buckled brace. This is particularly detrimental in case of intermediate slenderness braces. A stiff beam allows the tension brace to develop greater load. If the beam is sufficiently strong then the tension brace will yield and the story force deformation curve will be trilinear. The required beam stiffness is proportional to the area of the braces and therefore increases with the square of brace slenderness ratio. In practice, for usual frame geometries, most braces are of intermediate slenderness and most beams are too flexible to develop the full strength of the tension brace (even considering composite action with a floor slab).

DYNAMIC RESPONSE ANALYSES

The second phase of the research was aimed at investigating in a realistic setting the effect of brace slenderness and beam stiffness on dynamic response. In addition, the effects of frame participation and structural configuration were also studied. The investigation of structural configurations covered the comparison of isolated braced bents versus dual systems as well as variants of the K braced frame and frames with braced bays allowed to uplift. A ductile moment resisting frame and an isolated X braced frame were used for reference. Due to space limitations, this discussion will be limited to the effects of brace slenderness, beam stiffness and frame participation on frame response. The simulations are confined to severe ground motions only, because K braced frames show an excellent response in the elastic range (corresponding to moderate excitations). In all parameter variations presented the braces design buckling loads are kept constant. This is necessary to keep within the spirit of limit states design. The lower limit on buckling loads of braces is determined by the need to have an elastic response in the service and damageability limit states [Reference]. It also corresponds to the typical design situation where equivalent lateral loads are specified.

STRUCTURAL MODELS

Each of the frames analysed consists of a six story tall by three bay wide steel frame. The central bay is K-braced (figure 2). Uniformly distributed dead loads of 110 psf and live loads of 80 psf are assumed in the design of the frames. The framing plan is such that the gravity loads on the braced frames are applied at nodal points. Floor diaphragms are considered as rigid in plane and flexible out of plane. In designing the beams of the braced bays, the support provided by the braces at midspan was neglected. A weak-beam strong column philosophy was applied in selecting member proportions. A36 steel was used for all structural steel members. The inertial mass for the horizontal degree of freedom at each story consists of half the story mass (there are two symmetric braced frames in each direction). For the vertical degrees of freedom, inertial mass is computed based on tributary area and dead load weight.

The first frame is designed as an isolated braced frame and will be refered to as NDBF. In the NDBF the braced bay resists all lateral loads alone. Other framing resists gravity loads only. Another frame, refered to as DUAL, is designed as a dual system. In this frame, the braced bay resists the total lateral load, but the frame is also designed to resist a fraction (25%) of that load. The last frame, named SBBF, is similar to NDBF except for the beams of the braced bay. These beams are ten times as stiff and strong as the corresponding ones in the NDBF. Given the geometry of the braced bays and the loading assigned to them all braces have a slenderness ratio between 92 and 110 (intermediate slenderness).

Subsequently, the NDBF and DUAL frames were refitted with braces of same buckling load but of half the original slenderness. Since no existing steel section could exactly satisfy these requirements, the section properties had to be computed; they do not correspond to any real steel section, but they fit between existing steel sections in the AISC tables. The frames containing them are refered to STO-NDBF and STO-DUAL.

EARTHQUAKE LOADING

The design static lateral loads were computed according to UBC Zone 4 requirements with K=1 for the NDBF and K=0.8 for the DUAL frame, but for the dynamic analysis each frame was subjected to the "worst ten seconds" of six earthquakes selected for their damaging potential. The damaging potential used in selecting the chosen earthquakes was measured in terms of two criteria. The first criterion is the earthquake power spectral intensity in the frequency range corresponding to the frames first fundamental frequencies. This criterion measures the damaging effect of the earthquake to elastic structures that are likely to enter in resonance. The second criterion is the severeness of acceleration pulses as measured by maximum ground velocity changes. This component is particularly punishing for structures that yield or otherwise exhibit inelastic behavior. It tends to exacerbate the ductility demand in these structures (Bertero 1978, Bertero 1981).

The "worst ten seconds" from each chosen earthquake were obtained by selecting the ten seconds window where there is a maximum increment in root mean square acceleration. The ten second window extracted from the Elcentro 1940 NS record was scaled to a peak ground acceleration of 0.5g and the remaining excitations were then scaled to the same root mean square acceleration. The overall mean of the peak ground acceleration for the excitations used is still about 0.5g (table 1). The Tstart entry in table 1 indicates the beginning of the 10 s window in the original record.

RESPONSE QUANTITIES

Since the inelastic response is highly nonlinear and time history dependant, the response quantities considered are measured by their mean and their coefficient of variation (COV). The COV is the ratio of the standard deviation estimate to the mean value. Mean values are used to show trends and COV should reflect the sensitivity of the response quantity measured (Ang and Tang 1984). In judging the structure's performance from a reliability point of view one would compute the reliability index as the difference between the safe allowable value and the mean demand, divided by the square root of the sum of variances of the safe allowable value and the mean demand. It is then desirable to have as small a standard deviation estimate as possible to increase the reliability index and hence the reliability of the structure (Madsen 1986).

Individual response quantities considered here are:

- The total energy dissipation demand and its distribution over the individual stories. The total energy dissipation demand is a measure of the overall amount of damage inflicted to the structure. The distribution over the individual stories is a measure of design effectiveness: presumably the more uniform the distribution, the better the design.
- The maximum story shears. Maximum story shears in conjunction with story ductility may be used heuristically to decide how much one should increase or decrease story strength to achieve a more uniform ductility demand over the structure. Furthermore, an investigation of story shears at first brace buckling indicates the desirable story strength distribution to achieve a simultaneous or successive brace buckling and hence a more uniform energy dissipation demand.
- The maximum story drift. Maximum story drift is another measure of damage incurred in the structure. It should be kept below a certain limit (eg. 1/75 or 1/50 for the strength limit state) to avoid geometric instability (Mahin 1981).
- Maximum column compression. The maximum column compression load was found to be a good indicator of the frame participation in the response. Beam moments were not considered because the frames were proportioned according to weak-beam-strong-column design philosophy. Beam yielding is desirable and not as critical as column yielding for overall structural integrity.

ANALYSIS STRATEGY

The structural analysis program ANSR was used for the numerical simulations. The braces were modeled using Ikeda's physical theory brace model (Ikeda and Mahin 1984).

After several preliminary trials the following analysis strategy was adopted as the best compromise between stability, accuracy, and economy:

- Newmark's average acceleration scheme, for its unconditional stability.
- An integration time step of 0.001 s to capture higher modes effects.
- Constant stiffness iteration at each time step to insure convergence despite softening force deformation characteristics.
- A Rayleigh damping of 1% in the first two modes. The resulting damping in the highest significant mode did not exceed 7%.
- Floors were assumed to be rigid in plane and flexible out of plane.
- All vertical degrees of freedom were retained in anticipation of the effect of vertical vibration modes.

DYNAMIC EFFECTS

A very interesting and disturbing effect is the interaction of vertical vibration with story strength. The sudden buckling of the compression brace imposes an impact loading on the attached beam. The beam then starts to oscillate vertically. On its downward displacement it increases the compressive deformation in the buckled brace and reduces its resistance. It also reduces the elongation in the tension brace thus reducing its tensile load. The net result is a sharp reduction in the story strength. On its upward displacement, the beam tends to reduce the compressive deformation in the buckled brace. However, if the lateral story deformations continue to increase, this effect is compensated for. The buckled brace keeps compressing or elongates slightly; in both cases its resistance keeps decreasing. The tension brace elongates sharply, and since it is still elastic its tension also increases sharply. The net effect is a momentary but noticeable increase in story strength. All this is superimposed on the slower but larger hysteretic cycling of the story resistance (figure 3). This phenomenon was equally observed with stocky and regular slenderness braces. It was less noticeable in the case of strong stiff beams (SBBF).

Initially it was thought that there was some error in the analysis. But after plotting the time histories of vertical displacements at the braces junction and of the sum of vertical forces across the story, this doubt was put to rest. It was noticed that the vertical displacement and vertical force were constant until brace buckling. Both curves then showed intense high frequency oscillations that decayed slowly. The vertical displacement showed a permanent deformation and the sum of vertical forces returned to its previous static equilibrium value. It was also noticed that for the top story the oscillations of the sum of vertical forces had negative excursions, meaning an overall tension component! This was found to coincide with the elastic nonlinear stiffening force-deformation curves of that story (figure 4).

Despite all changes in stiffness in going from regular to stocky braces or from NDBF to DUAL systems, the values of the modal periods did not change considerably. The first mode period remained about 0.36 s, the second mode period hovered around 0.12 s, and the vertical beam vibrations remained around 0.012 s.

elastic. The net result is an increased story strength at brace buckling.

The increase in COV with increasing frame participation, increasing beam stiffness, and decreasing brace slenderness reflects the variability of the frame resistance mechanism. This mechanism depends to a large extent on the joints rotations at column ends. If the joints at the ends of a column are rotating in the same direction while story drift occurs, column resistance is increased. If they are rotating in opposite directions, column resistance is decreased. On the other hand in a NDBF maximum story shear is very simply linked to the buckling strength of the braces.

The story shears distribution at first brace buckling in the NDBF is remarkably constant. This distribution is compatible with a quarter sine curve distribution of the static lateral design loads instead of the linear distribution recommended by UBC. This agrees with the observed tendency of damage to concentrate in the lower stories and of brace buckling to initiate in the lower stories (first or second). It also indicates the relative importance in this case of the second mode of vibration. It seems that for the same base shear the UBC lateral loads distribution makes braces in the upper stories stronger than necessary.

ENERGY DISSIPATION DEMAND

The energy dissipation demand was concentrated mostly in the braces. The columns practically remained elastic and the total energy dissipation demand in the beams was negligible compared to that in the braces. Since the energy dissipation capacity of the braces is much less than that of the beams, the discussion of energy dissipation demand is limited to that of the braces. From Tables 2, 3, 4, 5, and 6 it can be noticed that inelastic deformations are concentrated in a few soft stories near the bottom of the building. However, as the frame participation increases the maximum energy dissipation demand increases and tends to move upwards (from the first story to the second). Similarly, as the braced bay beam stiffness and strength are increased the maximum energy dissipation demand increases and moves upward in the structure (to the third story). The COV of energy dissipation demand decreases with increasing frame participation, increasing beam stiffness, and decreasing brace slenderness. This trend is more acute when stocky braces are used (Tables 5 and 6).

It can be inferred that as frame participation is increased by increasing column sizes, frame stiffness and strength increase. The increase is most noticeable in the first story where the base fixity allows the columns to develop their full strength and stiffness. The increase is less noticeable in other stories where the development of strength and stiffness depends on joint fixity and the restraint provided by the beams. The first story being now relatively stronger, damage tends to concentrate in the second story. The aforementioned trend is even more noticeable when stocky braces, with smaller relative stiffness are used. In the SBBF all braced bay beams are stronger, therefore the third story, where column sizes are reduced, becomes the weakest story. This is confirmed by the percentage of energy dissipation there (Table 7). There is more inelastic activity in the columns of the SBBF than in those of the NDBF.

MAXIMUM STORY SHEAR

DUAL systems have smaller maximum story shears by design; this is reflected in Tables 2 and 3. As frame participation increases the COV of maximum story shear increases in the stories where inelastic activity occurs. There is no definite increase in maximum story shear as beam stiffness is increased. This is probably because the maximum story shear is determined to a large extent by braces strength. In DUAL systems there is no definite increase in maximum story shear in going from regular braces to stocky braces. On the other hand, using stocky braces in the NDBF causes a definite increase in maximum story shears. This can be explained as follows: The stocky braces have a smaller stiffness, they must deform more to reach their buckling load which is kept constant. The increased deformation means an increased resistance in the frame which is still

MAXIMUM STORY DRIFT

Maximum story drifts are larger for DUAL systems than for NDBF because DUAL systems are designed for smaller lateral loads. Story drifts are reduced almost to the half when stiff beams are used. Comparing NDBF to STO-NDBF and DUAL to STO-DUAL shows that the increase in mean drift as brace slenderness is reduced is due to the accompagnying reduction in brace stiffness. The reduction in COV of story drift as frame participation increases may be due to the reduction in the difference between the post buckling stiffness and the prebuckling stiffness.

INTERIOR COLUMN COMPRESSION

The maximum column compression has four components:

- A gravity load component that is static.
- A vertical inertial component due to the excitation of vertical oscillation modes. This component is particularly important in the braced bays.
- A quasi-static component due to the vertical unbalance load applied on the beam of the K-braced bay, upon brace buckling.
- A component due to the overturning moment on the structure caused by the inertial lateral loads.

For interior columns all four components are significant. For exterior columns, the first and the last components are the most significant.

Maximum column compressions for the DUAL systems are very close to those of the NDBF even though maximum story shears are clearly smaller. Maximum column compressions for STO-DUAL systems are noticeably smaller than those for STO-NDBF. Interior column compression increases with decreasing brace slenderness and increasing beam stiffness. This means that as brace slenderness is reduced the reduction in unbalanced force on the beam does not compensate for the increase in frame participation, unless the beam is very stiff. The COV of interior column compression increases with increasing frame participation, increasing beam stiffness and increasing brace slenderness.

EXTERIOR COLUMN COMPRESSION

Exterior column compressions are definitely larger for DUAL systems than for the NDBF and their COV is larger. Exterior column compressions increase when stocky braces are used instead of regular ones. These observations are compatible with an increased frame action in resisting overturning moments.

SUMMARY

The major findings of this investigation can be summarized as follows:
- The impact loading on the beam creates a detrimental coupling between vertical and lateral vibrations and reduces the benefit expected from stocky braces.
- The energy dissipation demand concentrates in the braces.
- Energy dissipation demands tends to concentrate in stories where changes in section sizes occur.
- Energy dissipation demand is quite sensitive to members rotational restraints.
- Maximum story shear increases with decreasing brace slenderness.
- The distribution of story shears at first brace buckling is stable and different from the design maximum story shears distribution.
- Maximum story drift increases with decreasing brace slenderness.
- Maximum interior column compression does not necessarily decrease with decreasing brace slenderness; it increases with increasing stiffness of braced bay beams.
- Maximum exterior column compression increases with increasing frame participation.
- COV of all response quantities are larger in inelastic stories than in elastic stories.

Conversely, if a DUAL system is adopted instead of a stronger NDBF system, then one can expect the following:
- An increase in the energy dissipation demand in the braces and the frame.
- An increase in maximum story drift.
- An increase in the COV of maximum story shear.
- An increase in exterior columns compression.
- A decrease in maximum story shear.
- A decrease in the COV of energy dissipation demand in the braces.
- A decrease in interior columns compression.

And for the same brace buckling load, using small slenderness braces instead of regular slenderness braces will cause:
- An increase in the energy dissipation demand in the braces and in the structure.
- An increase in the COV of both interior and exterior columns compression.
- An increase in the COV of maximum story shear.
- A decrease in the COV of energy dissipation in the braces.
- A decrease in the COV of maximum drift.

Finally, increasing the stiffness of the beam in the braced bay will cause:
- An increase in maximum energy dissipation demand.
- An increase in interior columns compression.
- A decrease in story drift.

CONCLUSION

There are three major phenomenae at work in chevron braced frames. The vertical vibration of the braced bay beam is detrimental to the development of story resistance. The moment resisting frame resistance is very sensitive to nodal rotation restraints. By keeping the buckling load of braces fixed, one introduces a coupling between brace slenderness and brace stiffness.

The results of the dynamic analyses agree in general with those of the quasi static analyses. They both show the indesirability of intermediate slenderness braces; either small slenderness or large slenderness braces should be used. It is more difficult to achieve a stiff beam effect with large slenderness braces, hence one should try to use stocky braces. The instantaneous story tangent stiffness varies considerably during the earthquake; this condemns all the schemes aiming at controlling the load distribution by manipulating the initial story stiffnesses.

Damage tends to concentrate in the stories where there are sudden changes in stiffness. This is unavoidable in steel structures where sections come in discrete sizes. It can be considerably reduced in larger structures by scheduling section changes over several stories. This was not possible in this case due to the limited number of stories.

One limitation of the results obtained is that they only show trends; they are not sufficient by themselves to decide if the structures simulated would be excessively damaged. To this end one needs to collect enough data about the energy dissipation capacity of braces or similar damage criteria.

The results show that the inelastic dynamic response of K braced frames is indeed complex. For any given response quantity there are several parameters that affect it in opposing directions. For example, reducing brace slenderness reduces maximum column compression only if the braced bay beam is very stiff. Reducing brace slenderness improves the shape of hysteresis loops of braces thus reducing their displacement ductility demand. However it also reduces their energy dissipation capacity because of the reduced section size. Reducing brace slenderness increases the COV of energy dissipation demand because of the increased frame participation.

Similarly, by increasing frame participation, the post-buckling stiffness is improved as well as the energy dissipation capacity. However, it also increases the COV of energy dissipation demand and the weight of the structure. That is why the problem may best be handled by an optimization approach that implicitly takes care of all these parameters and their interactions.

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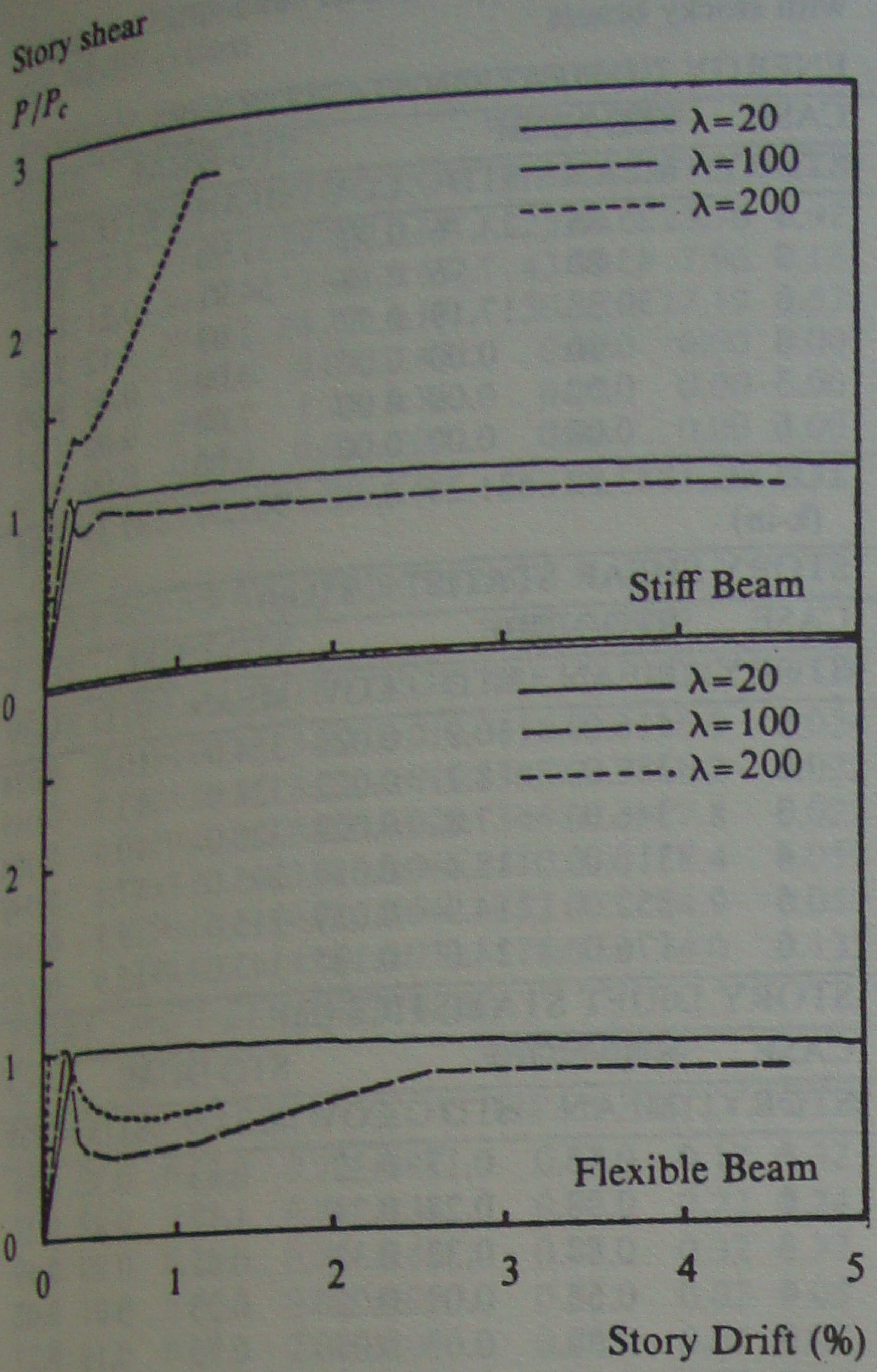

Figure 1. Quasi-static story force-deformation curves for various beam stiffnesses and brace slendernesses.

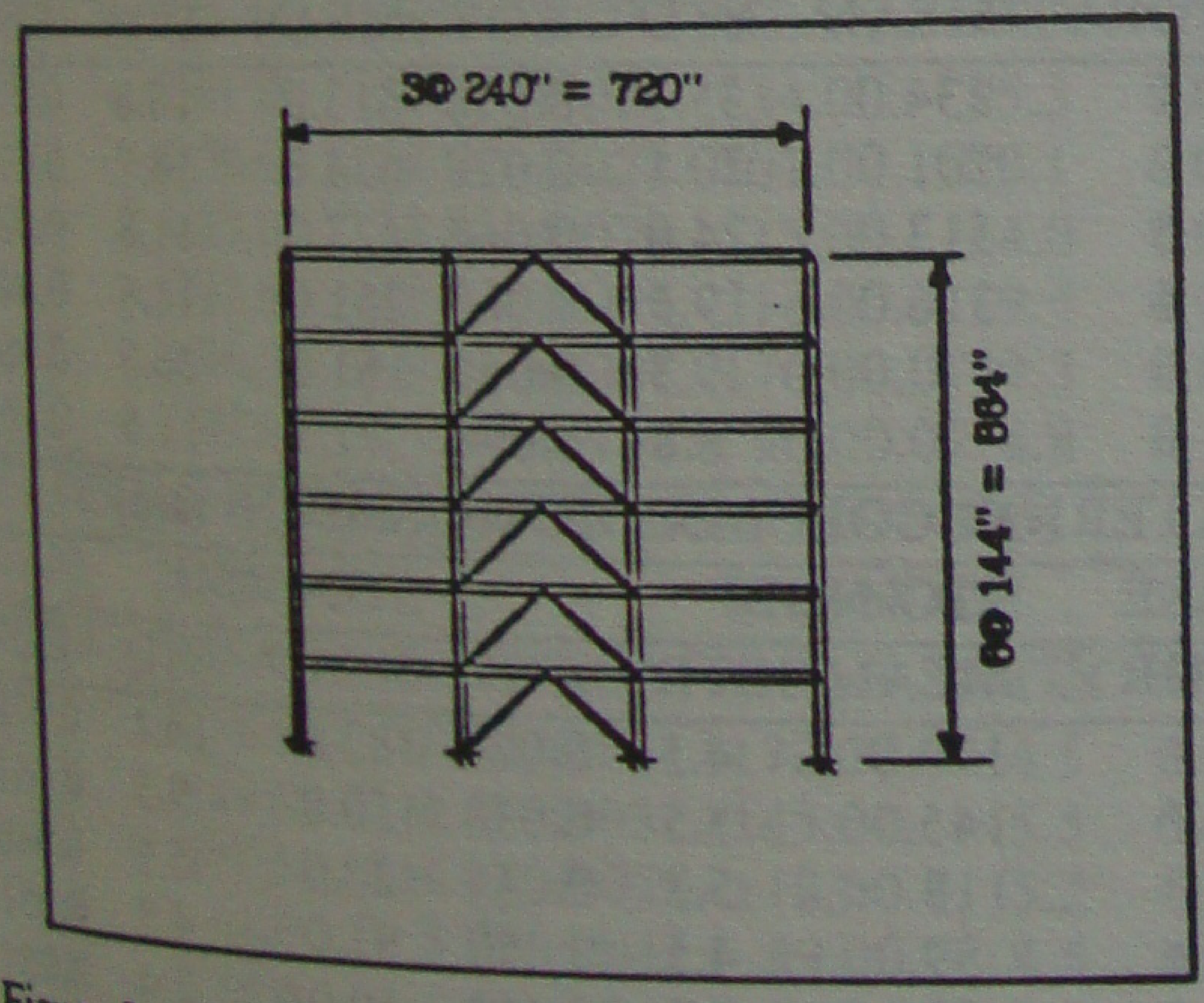

Figure 2. Typical Braced Frame Elevation.

Table 1. Earthquake records used

EARTHQUAKE RECORDS USED

EARTHQUAKE	LABEL	Tstart (s)	Max Acc (g)
ELCENTRO 1940 NS	EC40N	0.92	0.50
HELENA 1935 EW	HE35EW	0.24	0.68
OLYMPIA 1949 N86W	OL4986	9.84	0.56
TAFT 1952 S69E	TA5269	3.48	0.44
PACOIMA 1971 N34E	PA7134	0.96	0.62
PARKFIELD N65E	PAN65E	1.66	0.59

Table 2. Comparison of NDBF and DUAL systems with regular braces

ENERGY DISSIPATION STATISTICS (%)

CASE	NDBF			DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	47.79	29.93	0.63	35.58	13.10	0.37
2	30.27	17.44	0.57	48.02	12.66	0.27
3	20.93	24.22	1.15	2.93	7.18	2.44
4	0.03	0.07	2.45	0.00	0.00	0.00
5	0.99	1.74	1.76	13.46	11.19	0.83
6	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL (k-in)	633.73	361.27	0.57	703.43	337.54	0.48

STORY SHEARS STATISTICS (kip)

CASE	NDBF			DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	394.0	5.29	0.012	353.0	8.3	0.024
2	367.0	3.72	0.010	332.0	3.2	0.010
3	329.0	6.49	0.020	311.0	19.3	0.049
4	303.0	13.91	0.046	272.0	15.7	0.049
5	250.0	10.21	0.042	212.0	13.0	0.049
6	196.0	14.30	0.073	168.0	23.3	0.146

STORY DRIFTS STATISTICS (in)

CASE	NDBF			DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	0.73	0.22	0.42	0.88	0.46	0.51
2	0.83	0.29	0.34	1.03	0.37	0.34
3	0.72	0.34	0.46	0.52	0.24	0.46
4	0.50	0.02	0.05	0.46	0.02	0.05
5	0.50	0.02	0.05	0.84	0.22	0.27
6	0.48	0.05	0.10	0.46	0.02	0.05

INTERIOR COLUMNS COMPRESSION (kip)

CASE	NDBF			DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	881.0	31.0	0.035	809.0	37.7	0.047
2	658.0	18.0	0.028	627.0	29.2	0.046
3	487.0	38.0	0.078	429.0	18.7	0.044
4	322.0	11.0	0.035	286.0	7.2	0.025
5	186.0	9.0	0.047	185.0	10.9	0.059
6	87.0	17.0	0.191	80.0	10.4	0.130

EXTERIOR COLUMNS COMPRESSION (kip)

CASE	NDBF			DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	170.0	1.9	0.011	176.0	5.3	0.029
2	144.0	4.5	0.031	149.0	4.3	0.029
3	117.0	4.4	0.037	123.0	6.0	0.049
4	88.0	3.7	0.042	94.0	4.5	0.047
5	59.0	2.0	0.033	62.0	2.7	0.044
6	28.0	1.0	0.038	30.0	1.4	0.046

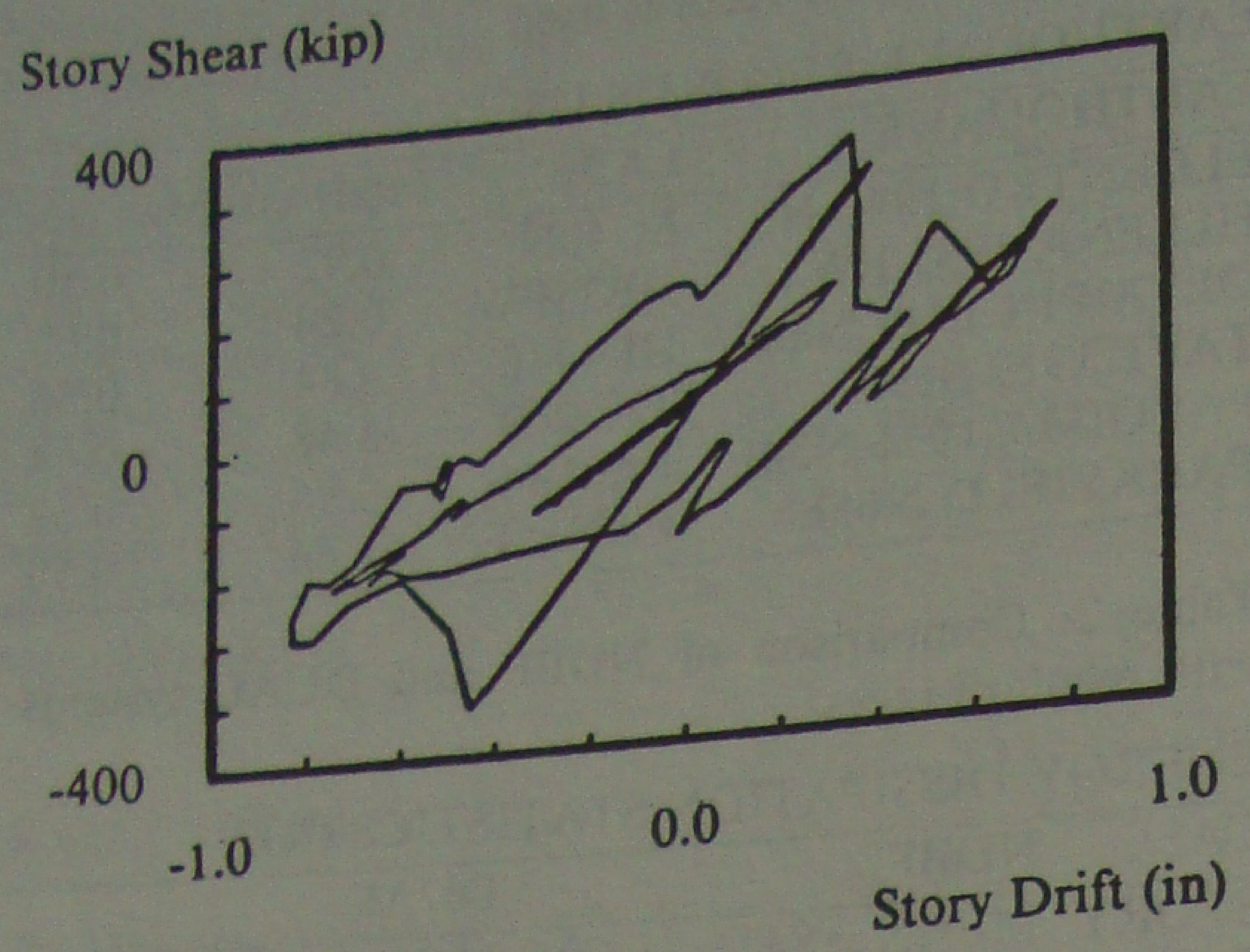

Figure 3. Third story force deformation curve for NDBF and PAN65E.

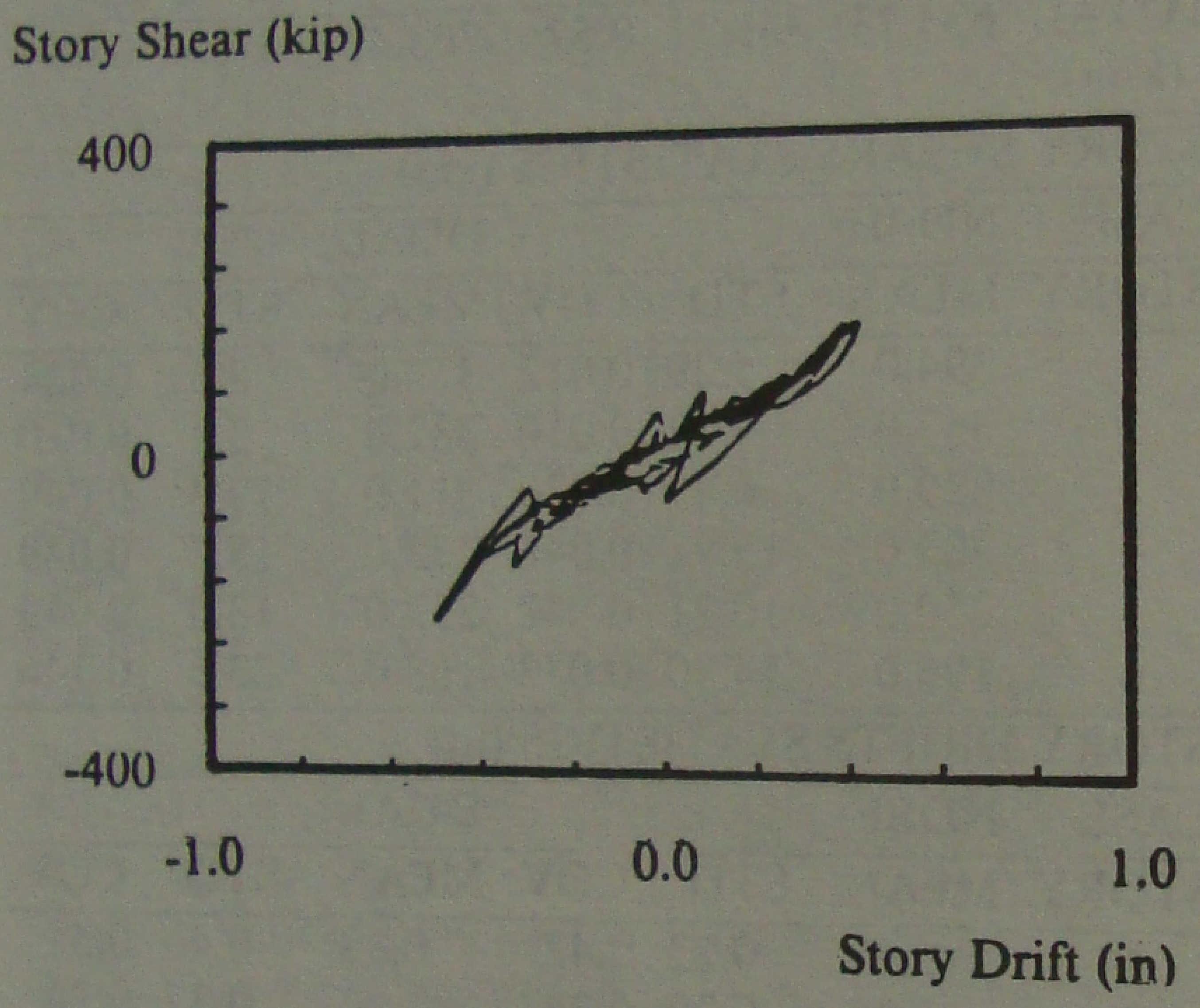

Figure 4. Sixth story force deformation curve for NDBF and OL4986.

Table 3. Comparison of NDBF and DUAL systems with stocky braces

ENERGY DISSIPATION STATISTICS (%)

CASE	STO-NDBF			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	25.85	23.76	0.92	35.90	4.53	0.12
2	43.80	7.96	0.18	54.91	10.21	0.19
3	30.33	17.19	0.57	2.09	5.12	2.44
4	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	7.09	9.60	1.35
6	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL (k-in)	743.82	431.24	0.58	901.71	589.39	0.65

STORY SHEAR STATISTICS (kip)

CASE	STO-NDBF			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	416.0	10.9	0.024	354.0	10.0	0.024
2	388.0	8.2	0.022	328.0	13.7	0.049
3	346.0	7.8	0.022	320.0	10.5	0.024
4	310.0	18.4	0.059	295.0	17.1	0.049
5	252.0	14.9	0.059	215.0	9.8	0.049
6	176.0	24.0	0.135	142.0	15.9	0.122

STORY DRIFT STATISTICS (in)

CASE	STO-NDBF			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	0.53	0.17	0.32	0.83	0.32	0.39
2	0.93	0.22	0.24	1.12	0.42	0.37
3	0.87	0.32	0.37	0.61	0.29	0.49
4	0.55	0.02	0.05	0.53	0.01	0.02
5	0.59	0.05	0.07	0.75	0.19	0.27
6	0.50	0.05	0.01	0.49	0.02	0.05

INTERIOR COLUMNS COMPRESSION (kip)

CASE	STO-NDBF			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	934.0	35.5	0.038	813.0	18.8	0.023
2	701.0	20.1	0.029	634.0	18.7	0.029
3	513.0	24.0	0.047	427.0	11.8	0.027
4	316.0	19.5	0.062	261.0	11.6	0.044
5	170.0	9.3	0.055	161.0	26.5	0.164
6	90.0	8.8	0.098	71.1	11.5	0.161

EXTERIOR COLUMNS COMPRESSION (kip)

CASE	STO-NDBF			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	172.0	4.5	0.026	182.0	14.1	0.078
2	145.0	5.5	0.038	150.0	9.5	0.063
3	118.0	5.2	0.044	120.0	5.5	0.046
4	89.0	4.5	0.050	91.0	4.7	0.051
5	59.0	2.9	0.049	59.0	3.4	0.057
6	28.0	1.4	0.049	28.0	1.9	0.070

Table 4. Comparison of regular and stocky braces in the NDBF system

ENERGY DISSIPATION STATISTICS (%)

CASE	NDBF			STO-NDBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	47.79	29.93	0.63	25.85	23.76	0.92
2	30.27	17.44	0.57	43.80	7.96	0.18
3	20.93	24.22	1.15	30.33	17.19	0.57
4	0.03	0.07	2.45	0.00	0.00	0.00
5	0.99	1.74	1.76	0.00	0.00	0.00
6	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL (k-in)	633.82	361.27	0.57	743.82	431.24	0.58

STORY SHEAR STATISTICS (kip)

CASE	NDBF			STO-NDBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	394.0	5.29	0.012	416.0	10.9	0.024
2	367.0	3.72	0.010	388.0	8.2	0.022
3	329.0	6.49	0.020	346.0	7.8	0.022
4	303.0	13.91	0.046	310.0	18.4	0.059
5	250.0	10.21	0.042	252.0	14.9	0.059
6	196.0	14.30	0.073	176.0	24.0	0.135

STORY DRIFT STATISTICS (in)

CASE	NDBF			STO-NDBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	0.73	0.22	0.42	0.53	0.17	0.32
2	0.83	0.29	0.34	0.93	0.22	0.24
3	0.72	0.34	0.46	0.87	0.32	0.37
4	0.50	0.02	0.05	0.55	0.02	0.05
5	0.50	0.02	0.05	0.59	0.05	0.07
6	0.48	0.05	0.10	0.50	0.05	0.10

INTERIOR COLUMN COMPRESSION (kip)

CASE	NDBF			STO-NDBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	881.0	31.0	0.035	934.0	35.5	0.038
2	658.0	18.0	0.028	701.0	20.1	0.029
3	487.0	38.0	0.078	513.0	24.0	0.047
4	322.0	11.0	0.035	316.0	19.5	0.062
5	186.0	9.0	0.047	170.0	9.3	0.055
6	87.0	17.0	0.191	90.0	8.8	0.100

EXTERIOR COLUMN COMPRESSION (kip)

CASE	NDBF			STO-NDBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	170.0	1.86	0.011	172.0	4.5	0.026
2	144.0	4.49	0.031	145.0	5.5	0.038
3	117.0	4.35	0.037	118.0	5.2	0.044
4	88.0	3.72	0.042	89.0	4.5	0.050
5	59.0	1.97	0.033	59.0	2.9	0.049
6	28.0	1.05	0.038	28.0	1.4	0.049

Table 5. Comparison of regular and stocky braces in the DUAL system

ENERGY DISSIPATION STATISTICS (%)

CASE	DUAL			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	35.58	13.10	0.37	35.90	4.53	0.12
2	48.02	12.86	0.27	54.91	10.21	0.19
3	2.93	7.18	2.44	2.09	5.12	2.44
4	0.00	0.00	0.00	0.00	0.00	0.00
5	13.46	11.19	0.83	7.09	9.60	1.35
6	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL (k-in)	703.43	337.54	0.48	901.71	589.39	0.65

STORY SHEAR STATISTICS (kip)

CASE	DUAL			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	353.0	8.3	0.024	354.0	10.0	0.024
2	332.0	3.2	0.010	328.0	13.7	0.049
3	311.0	19.3	0.049	320.0	10.5	0.024
4	272.0	15.7	0.049	295.0	17.1	0.049
5	212.0	13.0	0.049	215.0	9.8	0.049
6	168.0	23.3	0.146	142.0	15.9	0.122

STORY DRIFT STATISTICS (kip)

CASE	DUAL			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	0.88	0.46	0.51	0.83	0.32	0.39
2	1.03	0.37	0.34	1.12	0.42	0.37
3	0.52	0.24	0.46	0.61	0.29	0.49
4	0.46	0.02	0.05	0.53	0.01	0.02
5	0.84	0.22	0.27	0.75	0.19	0.27
6	0.46	0.02	0.05	0.49	0.02	0.05

INTERIOR COLUMN COMPRESSION (kip)

CASE	DUAL			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	809.0	37.7	0.047	813.0	18.8	0.023
2	627.0	29.2	0.046	634.0	18.7	0.029
3	429.0	18.7	0.044	427.0	11.8	0.027
4	286.0	7.2	0.025	261.0	11.6	0.044
5	185.0	10.9	0.059	161.0	26.5	0.164
6	80.0	10.4	0.130	71.0	11.5	0.161

EXTERIOR COLUMN COMPRESSION (kip)

CASE	DUAL			STO-DUAL		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	176.0	5.2	0.029	182.0	14.1	0.078
2	149.0	4.3	0.029	150.0	9.5	0.063
3	123.0	6.0	0.049	120.0	5.5	0.046
4	94.0	4.5	0.047	91.0	4.7	0.051
5	62.0	2.7	0.044	59.0	3.4	0.057
6	30.0	1.4	0.046	28.0	1.9	0.070

Table 6. Comparison of regular and strong beams in the NDBF system

ENERGY DISSIPATION STATISTICS (%)

CASE	NDBF			SBBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	47.79	29.93	0.63	20.22	3.42	0.17
2	30.27	17.44	0.57	14.97	13.62	0.91
3	20.93	24.22	1.15	51.97	18.12	0.35
4	0.03	0.07	2.45	6.25	8.13	1.30
5	0.99	1.74	1.76	6.55	4.57	0.70
6	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL (k-in)	633.73	361.27	0.57	587.04	276.07	0.47

STORY SHEARS STATISTICS (kip)

CASE	NDBF			SBBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	394.0	5.29	0.012	397.0	28.79	0.072
2	367.0	3.72	0.010	374.0	19.17	0.051
3	329.0	6.49	0.020	336.0	10.56	0.031
4	303.0	13.91	0.046	310.0	25.53	0.082
5	250.0	10.21	0.042	248.0	23.81	0.096
6	196.0	14.30	0.073	169.0	39.60	0.234

STORY DRIFTS STATISTICS (in)

CASE	NDBF			SBBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	0.73	0.22	0.42	0.43	0.17	0.39
2	0.83	0.29	0.34	0.45	0.14	0.31
3	0.72	0.34	0.46	0.81	0.30	0.37
4	0.50	0.02	0.05	0.59	0.14	0.24
5	0.50	0.02	0.05	0.63	0.18	0.28
6	0.48	0.05	0.10	0.49	0.06	0.13

INTERIOR COLUMN COMPRESSION (kip)

CASE	NDBF			SBBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	881.0	31.0	0.035	931.0	44.1	0.047
2	658.0	18.0	0.028	687.0	31.7	0.046
3	487.0	38.0	0.078	535.0	75.4	0.141
4	322.0	11.0	0.035	309.0	43.6	0.141
5	186.0	9.0	0.047	177.0	55.6	0.314
6	87.0	17.0	0.191	86.0	42.7	0.495

EXTERIOR COLUMN COMPRESSION (kip)

CASE	NDBF			SBBF		
STORY	MEAN	STD	COV	MEAN	STD	COV
1	170.0	1.86	0.011	170.0	10.71	0.063
2	144.0	4.49	0.031	147.0	10.37	0.070
3	117.0	4.35	0.037	120.0	9.55	0.079
4	88.0	3.72	0.042	92.0	7.96	0.087
5	59.0	1.97	0.033	61.0	5.20	0.086
6	28.0	1.05	0.038	29.0	2.68	0.092