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STRUCTURAL DAMAGE OF STEEL FRAME MODELS COMPILED IN EARTHQUAKE RESPONSE TEST DATABASE, *SCARLET*

by

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1. INTRODUCTION

Recently, earthquake response simulations based on actual structural behaviors have been performed by many researchers through shake table tests and on-line pseudo-dynamic tests. Consequently, many records of inelastic response and failure process are being collected. For whatever special purpose each simulation was intended, systematic processing over a set of test runs is expected to provide another fruitful perspective about earthquake resistant design.

More than forty test-runs on various types of single-story planer steel frame models have been collected by the authors' research group, and these data are compiled in the database, *SCARLET*, which stands for 'System for Computer-Aided Research on Limit states using Earthquake response Test data.' This paper outlines this database, and discusses about the classification of structural damage with the emphasis on the deterioration of lateral resistance.

2. CONTENTS OF SCARLET

Forty-two single-degree-of-freedom(SDOF) response tests in the database are carried out on planar single-story frames subjected to unidirectional earthquake motions. Excitations are the scaled N-S component recorded at El Centro in 1940 except for the test code 01. Tested structural models are as follows:

(1) A portal frame composed of rigid beam and rectangular cross-section columns cut from hot-rolled mild steel plate[1]; the scaled E-W component recorded at Hachinohe Harbor in 1968 is used as the excitation; test code 01.
(2) Braced frames composed of rectangular cross-section columns and braces cut from hot-rolled mild steel plates[2]; test code 02 through 09.
(3) Portal frames composed of rigid columns with pinned feet and hot-rolled mild steel H-shaped beams[3]; test code 10 through 12.
(4) Portal frames composed of rigid beams and H-shaped columns welded from low-yield-ratio high-strength steel plates[4]; test code 13 through 24.
(5) Portal frames composed of rigid beams and square-box columns welded from low-yield-ratio high-strength steel plates[5]; test code 25 through 34.
(6) Cantilever H-shaped columns welded from low-yield-ratio high-strength steel or mild steel plates[6]; test code 35 through 42.

The following data can be referred to and called on the display by the operator who makes an access to the system:

(a) Time histories of input ground motions
(b) Time histories of response displacements
(c) Time histories of restoring forces
(d) Test parameters, such as fictitious mass, fictitious damping, vertical loads, peak response values, applied axial loads, and so on.

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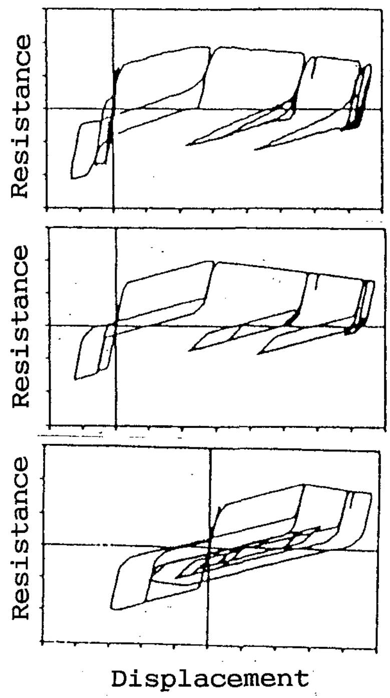
Table 1 SDOF Inelastic Responses Compiled in *SCARLET*

Test Code	Specimen Length L(cm)	Elastic Stiffness Ke(ton/cm)	Yield Strength Qy(ton)	Axial Load N(ton)	Energy Absorption E(ton cm)	Yield Displacement $\delta_y=Q_y/K_e$(cm)	Peak Displacement δ_{max}(cm)	Negative Slope/Elastic Stiffness $-\beta$	Response Ductility δ_{max}/δ_y	g-factor	Linear-elastic Response Shear Qe	Damage Classification
1	47.8	0.717	0.875	1.93	11.2515	1.220	3.947	-0.02717	3.235	0.939	3.377601	B
2	50.0	48.7	3.6	0	13.9538	0.074	0.955	0	12.905	1.0	26.80063	A
3	50.0	49.1	5.5	0	46.9908	0.112	2.562	0	22.875	1.0	66.09539	A
4	50.0	51.5	2.4	0	8.74507	0.047	0.774	0	16.468	1.0	20.79611	A
5	50.0	51.5	4.8	0	29.6733	0.093	2.215	0	23.817	1.0	51.99066	A
6	50.0	51.5	4.8	0	14.6425	0.093	0.833	0	8.957	1.0	32.3999	A
7	50.0	48.5	6	0	51.5571	0.124	3.268	0	26.355	1.0	61.38978	A
8	50.0	53.3	2.8	0	10.9519	0.053	0.774	0	14.604	1.0	32.00846	A
9	50.0	52.4	4.2	0	39.1443	0.080	3.412	0	42.650	1.0	77.98415	A
10	145.0	7.7	10.21	0	222.6353	1.326	4.695	-0.46132	3.541	0.978	56.50453	B
11	145.0	7.7	10.2	0	130.775	1.325	2.716	0	2.050	1.0	53.92888	A
12	145.0	7.7	10.21	0	418.638	1.326	9.287	-0.32234	7.004	0.891	80.89342	C
13	100.0	2.97	5	0	66.7046	1.684	3.57056	0	2.120	1.0	7.16848	A
14	100.0	2.48	2.5	48	13.6016	1.008	3.36182	-0.14354	3.335	0.880	2.833151	C
15	100.0	2.25	2.5	48	23.9764	1.111	4.02466	-0.15515	3.623	0.896	3.790908	C
16	130.0	3.7	7	0	88.7885	1.892	3.66944	0	1.939	1.0	9.298582	A
17	130.0	3.13	3.7	64	25.8372	1.182	3.31786	-0.14540	2.807	0.932	4.470104	B
18	130.0	3.34	3.7	64	42.48	1.108	7.05322	-0.53558	6.366	0.346	5.913248	C
19	160.0	3.5	9	0	121.106	2.571	5.05005	0	1.964	1.0	10.54398	A
20	160.0	3.2	5	80	44.682	1.563	2.98096	0	1.907	1.0	5.939994	A
21	160.0	3.16	5	80	56.1094	1.582	8.12623	-0.92707	5.137	0.0	7.849964	D
22	200.0	3.35	10	0	65.873	2.985	4.38354	0	1.469	1.000	11.58338	A
23	200.0	3.35	10	0	204.524	2.985	6.16333	0	2.065	1.0	12.0998	A
24	200.0	3.15	6.1	98	26.9104	1.937	6.60644	-1.41915	3.411	0.0	6.462603	D
25	100.0	3.84	6	0	83.8049	1.563	3.61816	0	2.315	1.0	9.540082	A
26	100.0	3.37	3.5	52	19.3194	1.039	1.95923	0	1.886	1.0	4.089697	A
27	100.0	3.43	3.5	52	30.6384	1.020	6.58448	-0.83727	6.455	0.0	5.484339	D
28	140.0	4.89	8	0	155.831	1.636	4.49341	0	2.747	1.0	14.31407	A
29	140.0	4.5	6.5	80	40.6005	1.444	2.38404	-1.45336	1.651	0.953	6.875038	B
30	140.0	4.5	6.5	80	40.7366	1.444	5.43823	-2.29064	3.766	0.0	8.031589	D
31	140.0	4.5	6.5	80	34.0374	1.444	5.48218	-2.30686	3.797	0.0	9.123878	D
32	200.0	4.17	12	0	240.888	2.878	6.32447	-2.05270	2.198	0.716	16.97085	C
33	200.0	3.92	9.35	54	75.3632	2.385	5.20752	-2.54962	2.183	0.429	9.513661	C
34	200.0	3.92	9.35	54	83.9745	2.385	8.18115	-1.76464	3.430	0.0	9.850731	D
35	46.5	0.321	0.55	3.67	5.80348	1.713	4.13086	-0.02603	2.411	0.942	1.846862	B
36	46.5	0.321	0.55	3.67	9.65499	1.713	11.858	-0.06704	6.922	0.0	2.773747	D
37	46.5	0.325	0.68	4.28	7.47234	2.092	4.14551	-0.05474	1.982	0.850	2.069387	C
38	46.5	0.325	0.68	4.28	7.73212	2.092	11.7957	-0.08291	5.638	0.0	3.10756	D
39	46.5	0.327	0.38	1.98	2.5339	1.162	2.50488	-0.01607	2.156	0.961	1.072855	B
40	46.5	0.327	0.38	1.98	4.14216	1.162	2.97366	-0.02237	2.559	0.892	1.602268	C
41	46.5	0.314	0.42	2.62	3.49965	1.338	6.1084	-0.03399	4.565	0.763	1.37964	C
42	46.5	0.314	0.42	2.62	7.14145	1.338	3.82324	-0.03879	2.857	0.835	2.066094	C

2. MODELING ERROR STUDY USING *SCARLET*

A pair of the restoring force and the response displacement compiled in the database gives a sample of hysteresis curve. On the other hand, a number of ordinary cyclic loading tests were carried out on steel members and frames in the past. Of course, the hysteresis curves compiled in *SCARLET* can be used to build or check a mathematical model as done with ordinary cyclic test data. The hysteresis curves in *SCARLET*, however, are particularly advantageous, because the restoring force and the displacement always satisfy the equation of motion under a certain earthquake.

Consider a test hysteresis curve of a braced frame as shown in Fig. 1(a). To simulate this curve, a certain hysteresis model is chosen, and the behavior of the model under the same displacement history is found to be as shown in Fig. 1(b). Most of engineers feel satisfaction with this model, because it seems to express basic features of the test hysteresis curve. Such a checking could be done even if the test hysteresis were the result from an ordinary cyclic loading test. However, when the test hysteresis is a *SCARLET* hysteresis, a further checking can be done. The response hysteresis simulated by the model under the same ground motion is shown in Fig. 1(c). Some of engineers do not think this result is good. Thus, a good model in a static sense is not always a good model in the prediction of dynamic inelastic response. *SCARLET* hysteresis data can be used to check the validity of a mathematical model when used in the dynamic response analysis.



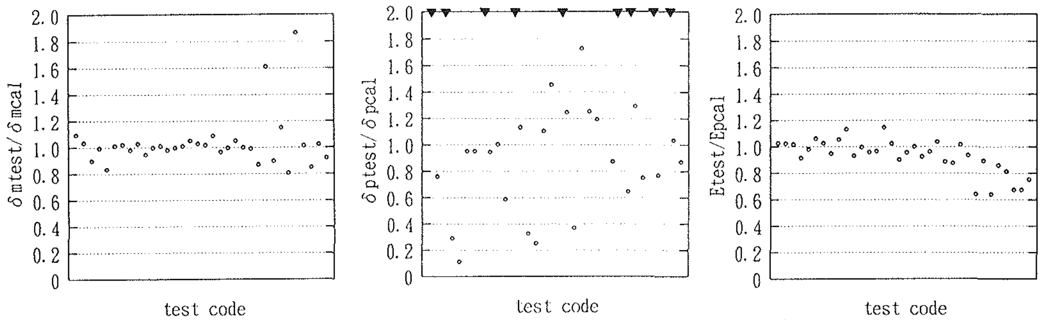
(a) Test Hysteresis Curve

(b) Model Behavior under Identical Displacement History

(c) Model Behavior when used in Response Analysis

Fig.1 Modeling Error Study using *SCARLET*

Bilinear hysteresis is sometimes used as the simplified model that exhibits the basic hysteresis behavior of a steel moment frame. The SDOF responses of unbraced frames in *SCARLET* are compared with those predicted by bilinear hysteresis, and shown in Figs. 2(a) through 2(c). As for the values of the initial stiffness and the yield resistance in the bilinear hysteresis, those read from the test curve are assigned in the analysis. By adjusting the value of the second slope of the bilinear hysteresis in a bit-by-bit manner, the response prediction error of the peak displacement is suppressed as much as possible. The model parameters determined in such a way are also used as they are for the prediction of other response quantities. It can be seen that larger errors are induced in the prediction of the permanent sets after earthquakes as shown in Fig. 2(b), while the prediction errors of the energy absorption are suppressed even smaller as shown in Fig. 2(c).



(a)Peak Displacement (b)Permanent Set after Earthquake (c)Energy Absorption

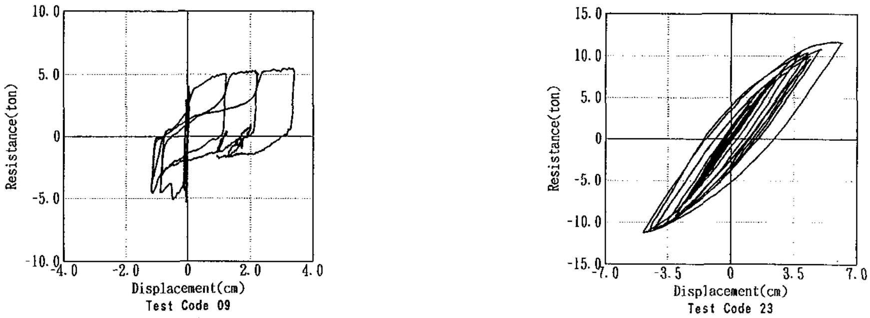
Fig.2 Response Prediction Errors of Bilinear Hysteresis Model

3. CLASSIFICATION OF DAMAGE

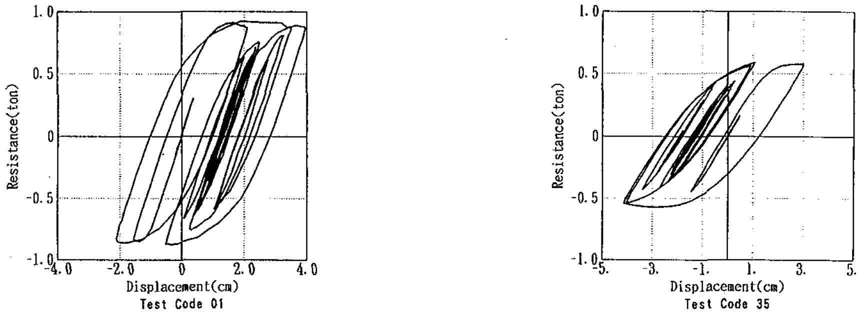
Magnitude or degree of structural damage after a severe earthquake may be assessed and evaluated from various factors such as magnitude of deformation or permanent set, occurrence and growth of local buckling or fracture, occurrence of overall structural instability, and so on. In this paper, the discussion is limited to the factor that is directly measured from the hysteresis loops, and the g–factor is chosen as the damage indicator and defined as the ratio of the remaining lateral resistance after the earthquake to the maximum lateral resistance during the earthquake. According to the value of the g–factor, the test runs are classified into the following four degrees of structural damage:

A: '*No Deterioration*' for $g = 1.0$ and inelastic response
B: '*Early Deterioration*' for $1.0 > g >= 0.9$
C: '*Considerable Deterioration*' for $0.9 > g > 0.0$
D: '*Complete Collapse*' for $g = 0.0$

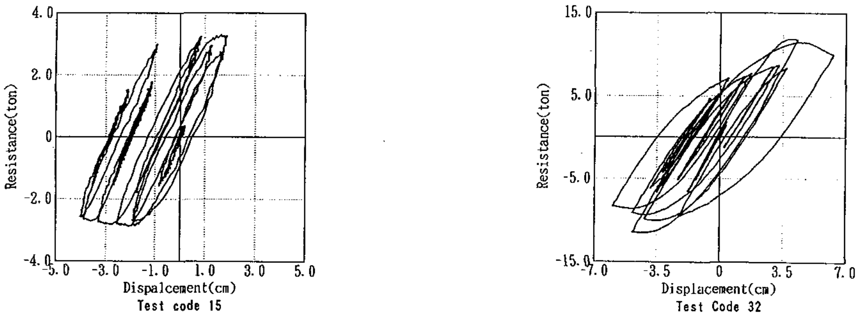
Examples of hysteresis loops classified into the four classes are shown in Figs. 3(a) through 3(d). Most of the hysteresis curves classified into 'C: *Considerable Deterioration*' and 'D: *Complete Collapse*' have plastic deformations that are accumulated to one direction, and then the curves are very similar to the monotonic test curves. Therefore, if a monotonic skeleton curve is given, the g–factor can be determined from the peak displacement.



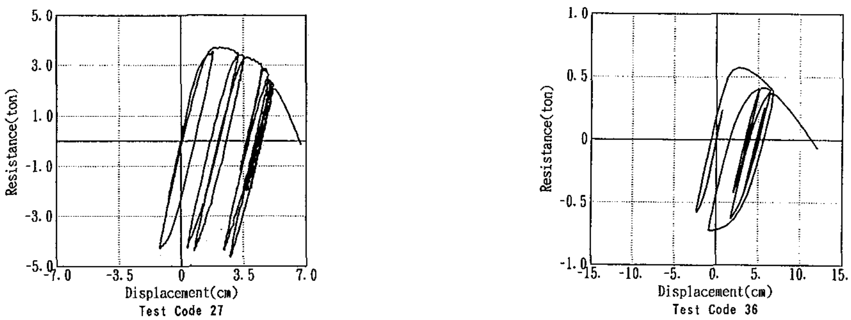
(a) No Deterioration



(b) Early Deterioration

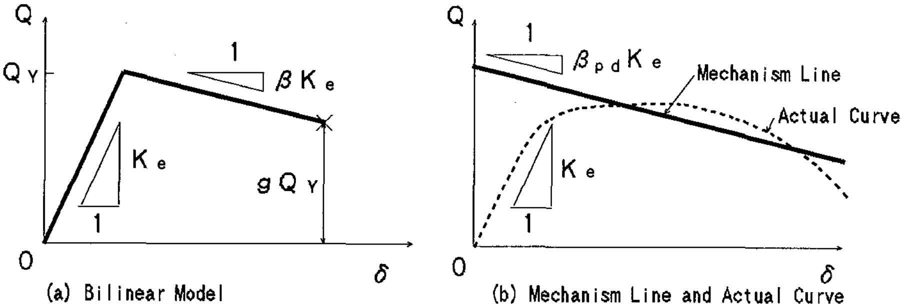


(c) Considerable Deterioration



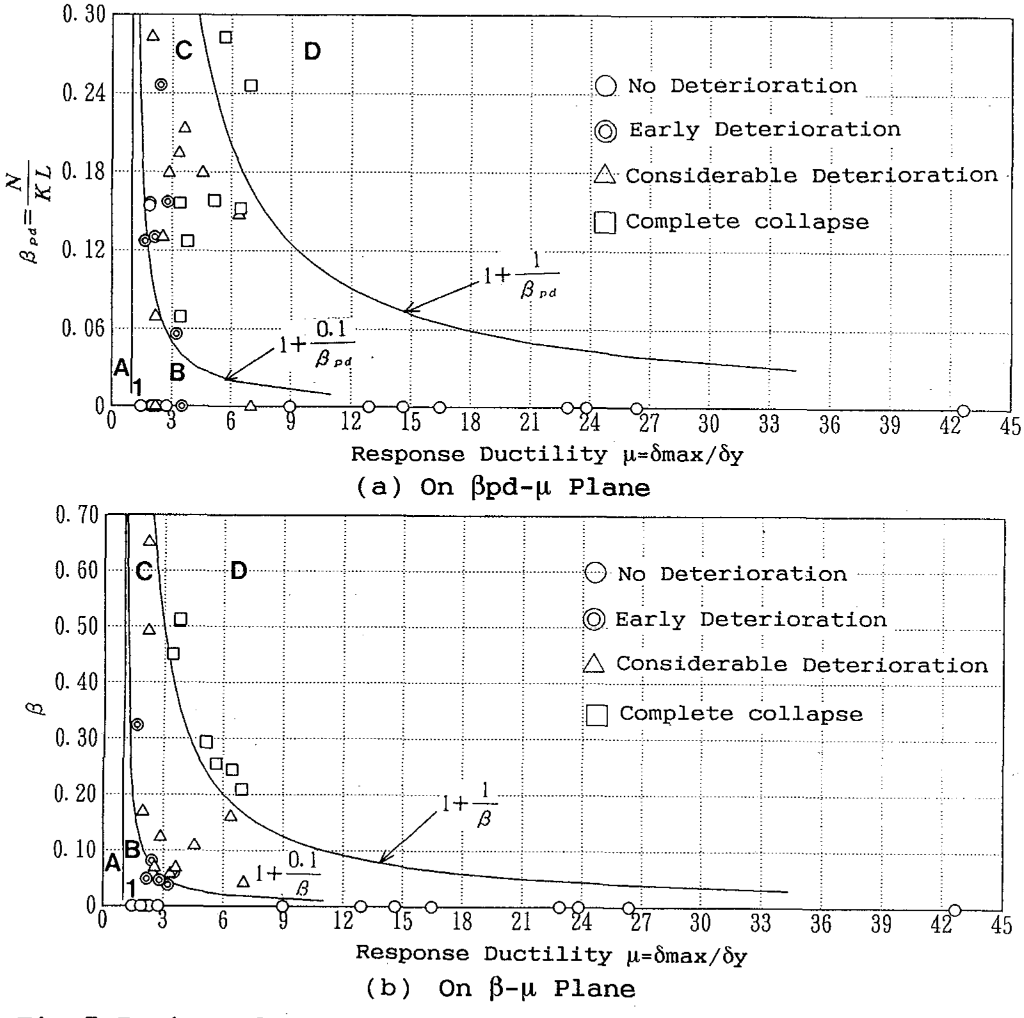
(d) Complete collapse

Fig.3 Classification of Damage



(a) Bilinear Model (b) Mechanism Line and Actual Curve

Fig. 4 Simplification of Monotonic Skeleton Curve



(a) On βpd-μ Plane

(b) On β-μ Plane

Fig.5 Zoning of Damage Classification or βpd-μ or β-μ Plane

When the monotonic skeleton curve is simplified as a bilinear curve as shown in Fig. 4(a), the g–factor and the response ductility factor, μ, are related with:

$$\mu = 1 + (1 - g)/ \beta \qquad (1)$$

where β: the ratio of the second slope in the bilinear curve to the initial slope

Eq. (1) can generate boundaries between two of the damage degrees in β–μ coordinate plane. First, the slope of the first–order mechanism line denoted by β_{pd} is used for the value of β in Eq. (1), and the results are shown in Fig. 5(a). The damage zones predicted do not well agree with the actual classification, that is, some of the cases classified into 'D: *Complete Collapse*' or 'B: *Early Deterioration*' are placed in the 'C: *Considerable Deterioration*' zone predicted. The reason of this is that the slope of the first–order mechanism line does not always represent the negative slope observed in actual hysteresis loops, which is affected by strain–hardening and local buckling as well. Then, the average slopes between the peak and the remaining resistance points are read from the actual hysteresis loops, and used instead of β_{pd}. The results are shown in Fig. 5(b). The boundaries of damage degrees predicted again match with the actual damage classifications. From this discussion, it is suggested that a careful estimation of the negative slope in the plastic range of skeleton curve is needed to assess the possible damage degree of a certain steel frame to severe earthquakes.

4. CORRELATION WITH LINEAR–ELASTIC RESPONSE

As described in Section 2, the peak displacement SDOF responses of steel moment frame would be approximately predicted even by use of a simple hysteresis rule such as bilinear, if the model parameters could be adjusted to the optimal values. Instead of carrying out numerical response analysis, the peak displacement of inelastic system is sometimes predicted from the linear–elastic response as proposed in the past literature. In this section, the hypothesis that identical amounts of strain energy are exerted into inelastic and linear–elastic systems[7] is adopted to predict the damage degrees in the term of g–factor.

From the hypothesis of identical strain energy shown in Fig. 6, we obtain:

$$0.5\,(Q_e^2/K_e) = 0.5\,(Q_Y^2/K_e)\,\{\, 1 + (1+g)(1-g)/\beta \,\} \qquad (2)$$

where Q_e: linear–elastic response shear force
Q_Y: maximum resistance
K_e: elastic stiffness

then $$(Q_Y/Q_e) = \sqrt{\beta/(\beta+1-g^2)} \qquad (3)$$

The boundaries of the damage degrees predicted by Eq. (3) are plotted on (Q_Y/Q_e)–β coordinate plane as shown in Fig. 7. These boundaries have a tendency to overestimate the actual damage degree especially for the β values smaller than 0.2. However, when the maximum resistance is less than the half of linear–elastic response, the boundaries of damage degrees are placed very close to each other, regardless of predicted or actual. This suggests that the deterioration of lateral resistance immediately after yielding shall be prevented for frames with insufficient resistance.

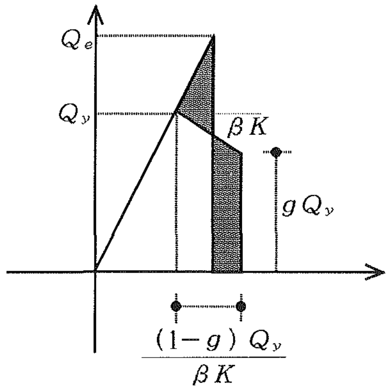


Fig.6 Hypothesis of Identical Strain Energy

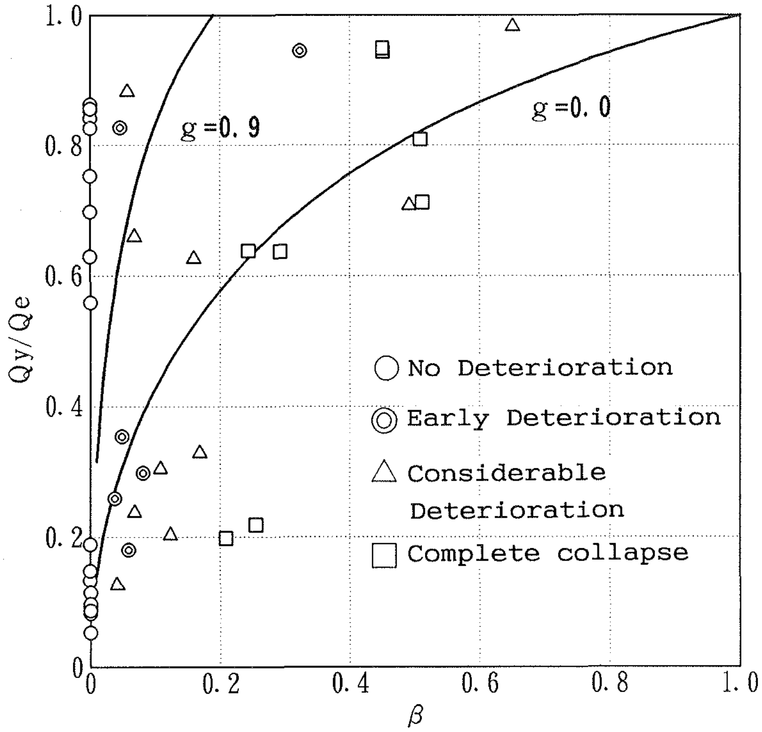


Fig.7 Prediction of Damage Degree from Linear-elastic Response

5. CONCLUDING REMARKS

A database of earthquake response tests on steel frame models, *SCARLET*, is outlined, and the damage degrees observed in forty–two SDOF tests on steel frames are classified with respect to the amount of the resistance deterioration into four degrees: '*No, Early, Considerable Deterioration,*' and '*Complete Collapse.*'

(1) The hypothesis of the monotonic–like response can be used to relate the damage degrees with the negative slope of the skeleton curve and the maximum deformation. To evaluate the negative slope for such a purpose, not only P–Δ effect but also strain hardening and local buckling shall be carefully considered.

(2) The hypothesis of the identical strain energy with linear–elastic response may be used to predict the damage degree approximately. However, the boundaries of 'B: *Early Deterioration*' through 'D: *Complete Collapse*' are very close to each other for the frame that has smaller resistance than the half of its linear–elastic response, and then the resistance deterioration immediately after yielding shall be prevented for such a frame.

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REFERENCES

[1] Takanashi, K. and Ohi, K.: "A Correlation Study in Pseudo–Dynamic On–line Tests vs. Shaking Table Tests," 1984 Annual Technical Session and Meeting of Structural Stability Research Council, April 1984.
[2] Ohi, K. and Takanashi, K.: "Hysteresis Loops Observed in Earthquake Response Tests on Steel Frame Models," Bull. of ERS, No. 20, March 1987.
[3] Ohi, K. and Takanashi, K.: "An Improvement of On–line Computer Test Control Method," 9WCEE, Tokyo–Kyoto, August 1988.
[4] Takanashi, K., Ohi, K., Meng, L. and Chen, Y: "Collapse Simulation of Steel Frames with Local Buckling," 10WCEE, Madrid, July 1992.
[5] Meng, L., Ohi, K. and Takanashi, K.: "A Simplified Model of Steel Structural Members with Strength Deterioration Used for Earthquake Response Analysis," Jour. of Struct. Constr. Engng. AIJ, No. 437, July 1992. (In Japanese)
[6] Kuwamura, H., Suzuki, T., Ohi, K. and Meng, L.: "Pseudo–Dynamic Failure Test of Small–Scale Columns of Different Steel Grades," Jour. of Struct. Engng., Vol. 35B, March 1989. (In Japanese)
[7] Veletsos, A. and Newmark, N.M.: "Effect of Inelastic Behavior on Response of Simple System to Earthquake Motions," 2WCEE, 1960.