A COMPARATIVE STUDY OF SEISMIC DESIGN CRITERIA FOR DUCTILE STRUCTURES

by

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SYNOPSIS

Design criterion developed for the design of ductile structures must be carefully evaluated prior to its application. Analytical techniques and material testing have made such an evaluation meaningful. A procedure for evaluating the effectiveness of design criterion for ductile structures is presented and used in evaluating five design criteria. The evaluation is made on a ten story ductile concrete frame and a ten story ductile steel frame. Member ductility demand is compared with cyclic load test results.

INTRODUCTION

During the past five years the building codes used in the United States of America have dramatically increased the lateral force design requirements. Many proposals (as yet not adopted) have been made which would make the potential increase of 100% from the 1973 Uniform Building Code to its 1976 predecessor seem quite nominal. A careful and thorough evaluation of proposed lateral force increases must be made if economy as well as safety is to be a consideration.

Procedures for the evaluation of ductile designs can be developed from the information currently available to the engineering community. Sophisticated analytical procedures are available to analyze the response of both elastic and inelastic frame structures. These analytical procedures have been used to predict actual performance as measured by seismographs. Even though all engineers may as yet not have confidence in the ability of these analytical techniques to predict the performance of a structure the ability to predict performance with enough precision to evaluate design techniques is certainly reasonable. Extensive testing of beams, columns, sub-assemblages, small frames and other building components has been conducted over the past years. These tests report the performance of the members when subjected to a variety of cyclic loads which far exceed the elastic capacity of the member. A careful matching of analytically predicted distortions with appropriate physical tests should produce a fair evaluation of the lateral force design criterion.

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The development of a new national lateral force design criterion commonly referred to as ATC III by the Applied Technology Council (research arm of the Structural Engineers of California) prompted the development of this comparative study. For a selected site in the Los Angeles basin, a series of design earthquakes were developed:

(1) A damage level earthquake with a 50 percent probability of occurrence during the seventy (70) year life expectancy of the structure.

(2) A collapse level earthquake with a 10 percent probability of occurrence. These design earthquakes were developed in form of smoothed elastic response spectra as well as time histories of ground motion.

Five lateral force design procedures were used to design bracing systems for a ten story concrete structure and a ten story steel frame structure:

(1) The 1973 Uniform Building Code, (UBC 73).

(2) The 1976 Uniform Building Code, (UBC 76).

(3) ATC III procedure 2, (ATC III-2), a multi-degree of freedom response spectra approach similar to the Uniform Building Code approach.

(4) ATC III procedure 3, (ATC III-3), a single degree of freedom response spectral approach.

(5) An inelastic spectral design technique as suggested by Newmark & Hall (1) (N.H.) with 10 percent damping and a system ductility of factor of 1.5.

The various bracing systems were designed in accordance with the aforementioned criteria and then analyzed to determine their response when subjected to both the damage and collapse level earthquakes. All frames were designed so as to preclude the occurrence of a plastic hinge in the columns and thus the components of story drift include elastic column deformation, elastic beam deformation and inelastic beam deformation. The results of these studies are presented in the form of indicated ductility demand (Tables 3 and 4).

CONCRETE FRAME

The ten story concrete frame was proportioned so that the concrete sections would be of adequate size to develop yield moments required by the ATC III-2 criterion. Two column sizes were used. The size of the frame beam is the same at each level. Columns and beams were reinforced in accordance with each criterion. Ductility demands were then computed at the second and ninth level. These ductility demands were then compared to a hysteresis loop representative of the behavior of a ductile concrete beam. (2)

A comparison of concrete test results and associated analytical data developed using techniques consistent with those used in the frame analysis was made. The calculated yield load (24^k) is associated with a cracked section deflection of .36 inches, (uncracked section deflection is .18 inches) as opposed to the actual measured deflection of .62 inches. If the calculated yield deflection is used in defining the ductility factor the ductility factor which is attainable through at least five cycles of load is 7.8.

The deformations reported on in Table 1 are based upon uncracked concrete sections. The ductility factors developed use the uncracked section momemt of inertia for determining the system response as well as the elastic deformation of the various members. This procedure is conservative as the deformations associated with a reduction in system stiffness produce lower ductility demands since the member yield deflection increases at a more rapid rate than the story drift.

The ductility demands presented in Table 3 indicate that the 1973 UBC criterion could be improved upon. An increase from the UBC 76 criterion to the ATC III-2 criterion is probably not necessary or necessarily advisable. Both the UBC 76 and the ATC III-2 criterion can be satisfied within the member sizes selected, however, the percentage of reinforcing must be increased by 25 to 30 percent in order to develop the yield moments required by the ATC III-2 criterion. This higher quantity of reinforcing will reduce the ductile capacity of the members and thereby reduce the apparent increased factor of safety to somewhere around 50 percent even though the design base shear (and story accelerations) have been doubled.

STEEL FRAMES

Structural steel frames were designed in accordance with the various criteria. Yield moments used were those associated with the plastic moment capacity of the members. It was assumed that each frame supported only lateral loads so as to avoid the masking effect of dead and live load moments. Thus the rigidities of the various frames are quite a bit less than might be encountered in a steel frame structure.

The ductility demands for the various frame beams (see Fig. 2) seems to be well within the capability of steel sub-assemblages. The nomalized hysteresis loop is representative of the performance of a rigidly attached steel beam subjected to cyclic loads (3). The yield load and deflection used in developing the nomalized hysteresis loop were those associated with the plastic moment capacity of the number tested. The increased safety associated with designing for higher force levels is quite nominal. The increase in cost associated with this increase is not quite as nominal. If the weight of the UBC 73 frame is used as a base the percentage increase required to satisfy the UBC 76 criterion is 50 percent while a 100 percent increase is required to satisfy the Newmark-

Hall design. If we assume that steel costs $.40 per pound, this amounts to increases of $.60 per square foot and $1 per square foot respectively (for bracing both axes of the building).

CONCLUDING REMARKS

Obviously a thorough comparative study of ductile design criteria is beyond the scope of unfunded research.

The need for a carefully correlation of analytical procedures and material tests is however essential to the development of a realistic design criterion for ductile structures. Hopefully research efforts will be directed towards obtaining a correlation between system ductility factors and member ductility demands so that reasonable designs which do not require numerous expensive inelastic time history analytical studies can be accomplished.

REFERENCES

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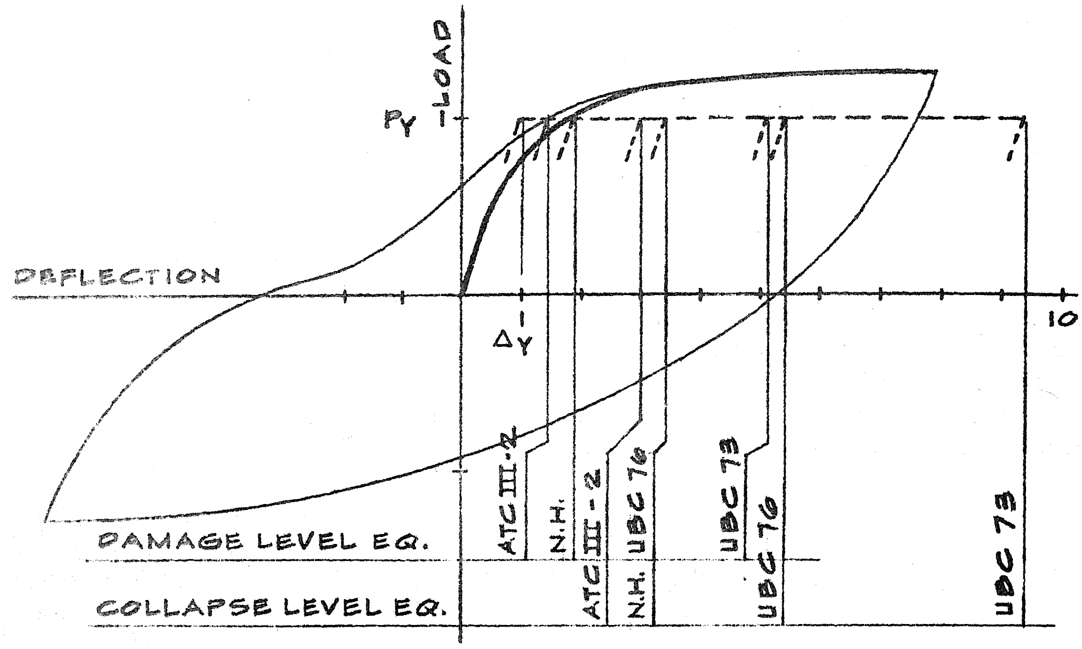

FIG. 1 NORMALIZED LOAD/DEFLECTION DIAGRAM
CONCRETE FRAME BEAM - LEVEL 2

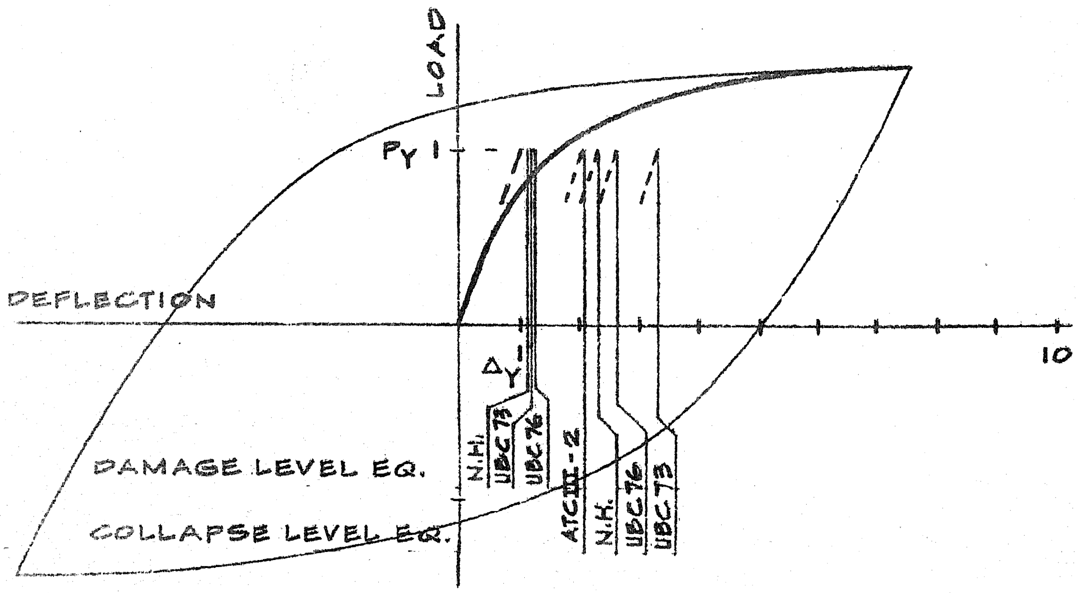

FIG. 2 NORMALIZED LOAD/DEFLECTION DIAGRAM
STEEL FRAME BEAM - LEVEL 9

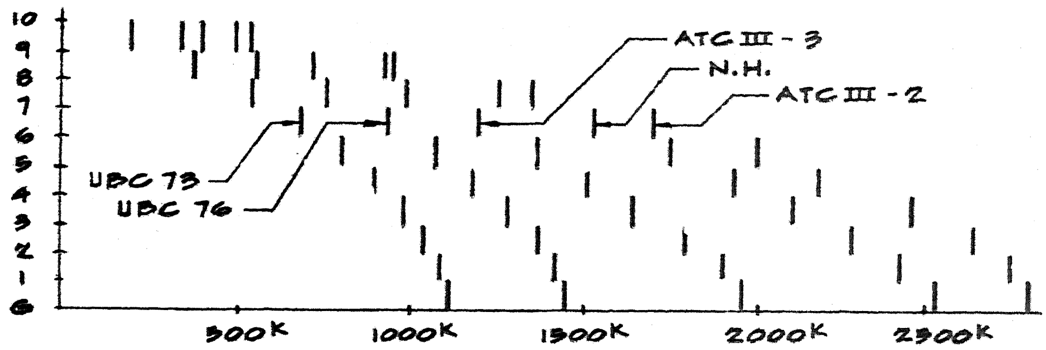

TABLE 1 ULTIMATE STORY SHEARS - 10 STORY CONC. STRUCTURE

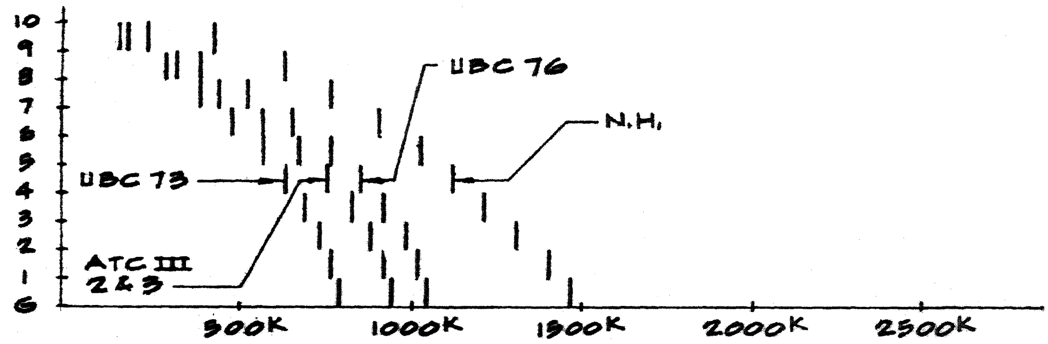

TABLE 2 ULTIMATE STORY SHEARS - 10 STORY STEEL STRUCTURE

TABLE 3 DUCTILITY DEMAND (μ) - CONCRETE FRAME BEAMS

DESIGN CRITERIA	2ND LEVEL ELASTIC DISPLACEMENTS			DAMAGE LEVEL EQ.				COLLAPSE LEVEL EQ.			
				2ND LVL		9TH LVL		2ND LVL		9TH LVL	
	COL.	BM.	TOTAL	Δ_T	μ	Δ_T	μ	Δ_T	μ	Δ_T	μ
UBC 73	.044	.106	.150	.61	5.35	.25	7.6	1.04	9.5	.39	12.3
UBC 76	.065	.160	.225	.61	3.40	.25	4.8	1.04	5.5	.39	7.9
ATCIII - 2	.130	.300	.430	.61	1.60	.25	2.7	1.04	3.0	.39	4.6
ATCIII - 3	.090	.210	.300	.61	2.48	.25	3.7	1.04	4.5	.39	6.1
N. H.	.110	.270	.380	.61	1.85	.25	2.6	1.04	3.4	.39	4.5

TABLE 4 DUCTILITY DEMAND (μ) - STEEL FRAME BEAMS

DESIGN CRITERIA	2ND LEVEL ELASTIC DISPLACEMENTS			DAMAGE LEVEL EQ.				COLLAPSE LEVEL EQ.			
				2ND LVL		9TH LVL		2ND LVL		9TH LVL	
	COL.	BM.	TOTAL	Δ_T	μ	Δ_T	μ	Δ_T	μ	Δ_T	μ
UBC 73	.28	.65	.93	1.00	1.11	1.29	1.14	1.62	2.06	3.16	3.30
UBC 76	.23	.50	.73	.61	—	1.32	1.23	1.27	2.08	2.36	2.65
ATCIII-2	.17	.49	.66	.53	—	1.00	—	1.01	1.71	1.80	2.05
N.H.	.13	.49	.62	.73	1.22	.90	1.12	1.27	2.32	1.61	2.32