ON INSTRUMENTAL VERSUS EFFECTIVE ACCELERATION, AND DESIGN COEFFICIENTS

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INTRODUCTION

There is much discussion, and even confusion, about the differences between peak acceleration as obtained from instrumental records of ground motion, the peak acceleration used to construct response spectra for dynamic analysis, and the base shear coefficients specified in building codes. Although these three values should not be compared directly, they often are. Peak measured accelerations have been recorded up to one gravity, and slightly beyond in a case or two. Dynamic amplification at typical building periods and damping ratios leads to spectral accelerations perhaps two or more times the zero-period acceleration. Code base shear coefficients, after adjustments have been made for zone, framing, soil, period, importance, etc., are now generally in the range of 0.05 to 0.15 in severe seismic areas; for a few special-risk buildings, the coefficients are greater. Many consider these differences anomalous. Some would like either to design for peak instrumental acceleration and the corresponding spectra or to be given scientific reasons why they should not do so. This is not a simple matter, albeit a very important one. The purposes of this paper are to attempt to clarify the situation, identify the most significant parameters and considerations inherent in the problem, and -- to the extent feasible -- reconcile or revise the great variations between recorded peak accelerations, dynamic design peak accelerations, spectral response accelerations, and code base shear coefficients. Acceleration, rather than velocity or displacement, will be used for convenience in comparisons.

SOME BASIC TERMS AND RELATIONSHIPS

Before proceeding, it is desirable to define certain terms and relationships basic to treatment of the subject.

Peak Instrument Acceleration, A_q, is defined here as the peak acceleration recorded during the entire earthquake motion by a reliable strong-motion instrument situated in the free field or what may be considered a free field because it is not significantly affected by soil-structure interaction or by topographic conditions. This peak is usually taken as the peak absolute value in an entire time history.

Peak Effective Acceleration, a', has had no precise definition, but common usage somehow implies that it is fully effective in causing structures to respond whereas any acceleration with a greater value is not effective at all. Such is not the case. There generally is no sharp dividing line between absence of response and full response. Therefore, the following definition will be used: "effective acceleration is that acceleration which exists, or would exist, at the zero-period or infinite-

frequency part of a response spectrum used in analysis or design." This value, which is independent of damping, is sometimes also called the zero-period acceleration, or the anchor-point acceleration. If we let $a' = cA_q$, then c is an adjustment factor or modifier for effectiveness, with a maximum value of unity.

Design Base Shear Coefficient, termed B, here, is in most codes the product of many factors that depend upon such matters as seismic zone, period, materials, framing methods, ductility, redundancy, soil conditions, allowable stresses, importance of the structure, etc. For example, in the 1976 Uniform Building Code, the base shear V is determined by V = (ZKCSI)W, which here can be termed simply V = BW. Thus, B is here defined (regardless of code) as that factor which is multiplied by the total seismic weight, W, to obtain design base shear V for use with yield-level allowable stresses.

Dynamic Base Shear Coefficient, C_b. B cannot be compared directly with a' because of dynamic phenomena. In the first place, a' is by definition the zero-period acceleration at one end of the response spectrum. Structures have fundamental periods, and other natural periods, that place them not at the zero-period part of the spectrum but at some other period or periods that may have more or less acceleration than that at zero period. Taking the fundamental response value as S_a for the appropriate damping, and spectral base shear, V_s, for the fundamental mode of a lumped-mass system (Ref. 1):

$$V_s = \gamma S_a g \left(\sum_{j=n}^{1} m_j \phi_j \right) \tag{1}$$

where:

- g = gravity in appropriate units
- m_j = mass of the jth story
- ϕ_j = fundamental mode deformation of story j relative to the nth or top story, dimensionless
- S_a = spectral acceleration for fundamental mode and appropriate damping (gravity units)
- γ = the participation factor for the fundamental mode $= (\sum_j m_j \phi_j)/(\sum_j m_j \phi_j^2)$

As shown in Ref. 1, if the story masses m_j are considered equal (this is often a good approximation), W = the total weight of the building, and C_b = a dynamic base shear coefficient = V_s/W, then:

$$\frac{C_b}{S_a} = \alpha = \frac{\left(\sum_{j=n}^{1} \phi_j \right)^2}{n \left(\sum_{j=n}^{1} \phi_j^2 \right)} = \frac{V_s}{WS_a} \tag{2}$$

Thus, the dynamic base shear as a function of S_a depends upon the mode shape and the number of stories, n. If we assume certain idealized but representative fundamental mode shapes (Refs. 1 and 2), α and C_b are readily determined with the aid of Table I.

TABLE I: α VALUES FOR FUNDAMENTAL MODES OF MODELS WITH UNIFORM STORY HEIGHTS, UNIFORM MASSES, AND FIXED BASES

		α for n stories, n =			
Model	Mode shape	5	10	20	30
(a)	Straight line	0.82	0.79	0.77	0.76
(b)	Bilinear, $\phi_1 = \frac{2}{n+1}$	0.89	0.84	0.80	0.78
(c)	Bilinear, $\phi_1 = \frac{4}{n+3}$	0.95	0.90	0.85	0.82
(d)	Soft first story, $\phi_1 = 1.0$	1.00	1.00	1.00	1.00
(e)	Shear building	0.92	0.88	0.86	0.85
(f)	Vertical cantilever	0.62	0.59	0.57	0.56

Source: Refs. 1 and 2

Note that α in the table is always 1.00 or less but, except for the vertical cantilever building, (f), the lower limit is about 0.75.* This rather limited variation below unity by no means explains the difference between spectral base shears V_s and code base shears V, although it is one of the many considerations. With Equations 1 and 2 and the above definitions it can be shown that $V/V_s = B/C_b = B/\alpha S_a$. With a typical value of B at 0.10, and a low value of αS_a at 1.0g, the code-determined base shear V is only one-tenth of the dynamic base shear V_s. Obviously, there are factors other than dynamic phenomena involved in this problem; otherwise buildings would not have survived as most have. These factors will be explored.

A BRIEF REVIEW OF THE HISTORICAL RECORD

The first strong-motion records of a damaging earthquake were taken in 1933. The Long Beach earthquake had a magnitude of 6.3, and the moving fault was close to the damaged areas. The peak horizontal acceleration, A_q, recorded in a basement below the surface was about 0.2g. Subsequently, motion was recorded at less densely populated areas; at El Centro, for example, in 1934 and again in 1940. The latter record had a maximum horizontal component Aq of 0.33g. In 1952 the Kern County earthquake occurred, producing considerable damage, but no local motion was recorded except at Taft some 42 km from the moving fault, where for the main shock (M = 7.7) a peak value of 0.18g was recorded. For all of these events, the damage was blamed mostly on poor design and construction, of which there was indeed considerable evidence from an earthquake-resistance

*The addition of higher mode values may increase the base shears from those obtained with the α values in Table I.

point of view. Lack of damage was generally associated with the safety factor between design level and ultimate strength. Little attention was paid to the (often massive) noncalculated or nonstructural elements, ductility and redundancy, or to the recorded accelerations, which gave portent of even greater values to come.

In 1958, Blume (Ref. 3) published the results of extensive studies in which both old, traditional office buildings and contemporary buildings were subjected to spectral response diagrams from several of the then-existing strong-motion records. It was found that buildings were subject dynamically to many times the forces implied or specified by the seismic codes if the buildings were to remain elastic; that most buildings, especially the old, traditional ones with heavy nonstructural walls, were much stronger than they were given credit for in the codes; and that these two factors generally reconciled the historical record. However, it was also shown that modern buildings were neither as rigid nor as resistant as their predecessors unless designed to be ductile and to absorb energy in the inelastic range without overall failure or collapse (Ref. 4). The real power of the earthquake was now recognized as relative to building values, even before large earthquakes had yet occurred in densely populated areas or any acceleration A_q greater than 0.33g had yet been recorded. In the same work, it was emphasized that peak acceleration is a very poor index of damage potential except for very rigid structures. Instead, the keys were spectral acceleration (or spectral velocity, or spectral displacement, as might be preferred or indicated), the energy-absorbing capacity of the structure, and redundancy. Acceleration will be considered in this paper because of the way codes and spectral diagrams are developed and used and because it provides the best means of communication among those in the various disciplines concerned with the earthquake problem.

Subsequent earthquakes in California and elsewhere in the world have shown that peak ground motion can indeed be strong, even with rather small magnitudes if the moving fault is close and the focus shallow. However, recent recording of some large peak instrumental accelerations should not now cause us to be so surprised or to abandon all reason and forego all judgment. We must consider the overall picture and study all the evidence. In this we need not limit ourselves to cases with recorded time histories. For example, we know that a magnitude 8 to 8.3 earthquake occurred in 1906 with the moving fault only about 17 km from 52 major downtown San Francisco buildings; the Huachipato Steel Plant in Chile suffered violent shaking in 1960; the ESSO Refinery at Managua, Nicaragua, had a recorded motion up to 0.39g; strong accelerations have been recorded at Sendai in 1978, in Mexico in 1978 and 1979, close to underground nuclear explosions, and elsewhere without catastrophic or even severe damage. These and other examples will be evaluated to reconcile theory with the historical record and to account for the gaps between instrumental and effective acceleration and design shear coefficients. It is not sufficient to vaguely ascribe the results to safety factors, dynamic phenomena, ductility, soil-structure interaction, or nonstructural elements. Although these play important roles, we must be more specific if we are to avoid future disasters. In approaching the problem of the gaps, there are four basic avenues to be considered and to be reconciled with each other: observation of what has happened and what has not happened in actual

earthquakes; theory and analysis; testing and experiments; and engineering judgment.

OBSERVATIONS

If the shaking from the San Francisco earthquake of 1906 (M = 8.25), with the moving San Andreas fault about 17 km from many major buildings, were estimated according to many current procedures, the peak instrumental acceleration would no doubt exceed gravity (Ref. 5). The spectral accelerations, if they are based on A_q, have acceptable damping ratios, and are at the natural periods of the then-major buildings, would be 2g or more. However, of the 52 major buildings in San Francisco (none specifically designed for earthquake resistance except as a by-product of their wind design), all but 7 were repaired and put back into use (Ref. 5). Most are still in use today. Of those that were not repaired, four were destroyed by fire and at least one was very poorly constructed. A few of the surviving buildings still in use include the 19-story Central Tower, the Fairmont Hotel, the old portion of the St. Francis Hotel, the Post Office at Seventh and Mission streets, the Ferry Building, the Monadnock Building, the Emporium building, and the Flood Building. The original Palace Hotel had only rather minor earthquake damage, but it burned. Fort Point, only a few miles from the fault, had very minor damage. None of these buildings would be able to meet current code requirements. If one were to estimate average base shear coefficients to cause yielding, the first stories would have to be given major consideration because of their height and the major openings in many of their walls. An average base shear coefficient of 0.04 to cause a yield-level stress would be a generous estimate for these buildings.

The ESSO refinery complex at Managua, Nicaragua, was subjected to a 1972 earthquake that killed 10,000 persons. The magnitude was 6.25, but the moving fault was only about 5 km away from the refinery and the focus only about 3 km deep. The accelerations were recorded at the refinery: the peaks were 0.39g EW, 0.34g NS, and 0.33g vertical. The plant structures and vessels had various design levels ranging up to a maximum base shear coefficient of 0.20 and averaging about 0.10 to 0.13 under various editions of Uniform Building Code criteria. There were all sorts of vertical vessels, pumps, heat exchangers, pipes, buildings, tanks, foundations, and instruments. There was only minor damage. The plant was shut down manually for inspection and then started up in less than 24 hours (Ref. 5).

The Huachipato Steel Plant, near Concepcion, Chile, was subjected to a 7.5M earthquake on May 21, 1960, that caused about 0.4% damage but no collapses. The plant was shut down for six days and was then back on normal operations (Ref. 5). The design was static rather than dynamic, with various coefficients believed to have been from 0.10 to 0.30. However, not only certain dynamic phenomena but also buckling phenomena were not fully considered in design because of the then-available knowledge. Much of the damage can be attributed to those factors. A generous equivalent design coefficient, B, would be 0.18 at yield stress. There was no record of A_q, but an extensive study (Ref. 7) led to the most likely S_a diagram. The most probable S_a value at the period and damping of the most critical structure is 1.2g.

Acapulco, Mexico, on October 6, 1974, was subjected to very sharp ground motion with A_q 0.53g. The earthquake epicenter was close, with an original report (Ref. 8) of 12 km and a later report (Ref. 9) of 35 km. The site is on rock. "The damage was light. In general it consisted of small cracks in some construction and partial collapse of fences that were particularly weak and built of brittle materials." There were few injuries, and "the effects on the population were minimal and promptly forgotten" (Ref. 8).

Sendai, Japan, was subjected to severe ground shaking on June 12, 1978 (Ref. 6). The peak ground motion over the area ranged from about 0.25g to 0.4g. The exposure was great, with a metropolitan area population of 1,250,000 and buildings up to 20 stories high. The damage was slight, if any, to buildings designed to recent seismic standards, but there were some failures of older structures. It will be assumed for this study that the average base shear design coefficient, B, was 0.2 and that the average damage to buildings of such design level was 1% of replacement cost.

The Salonica, Greece, June 20, 1978, earthquake had a recorded motion of 0.13g. The building design criteria and details were not those of modern USA codes. In a densely populated city of over 700,000 persons, one building collapsed and killed its 40 occupants. An average damage ratio of 3% is estimated.

Other disasters have occurred worldwide, some bringing almost total destruction. They are excluded here only because of lack of data on recorded ground motion or reliable information that could be used to make estimates of the motion, or because the design and construction practices were so far below those normally required for seismic resistance as to make the results obvious -- in such cases, severe motion leads to severe or total damage. Their omission does not lead to bias in the observations.

COMPARISON OF OBSERVATIONS

The above observations and others were compared and related to accelerations. The first step was to (provisionally) assume that the spectral acceleration is based on instrumental acceleration, A_q. Where the spectral acceleration at the average fundamental period of the buildings involved was not known, it was estimated by multiplying A_q by an appropriate dynamic amplification factor, DAF, for that period and damping. DAF factors ranged from 1.5 to 2.0. If S_a was available it was used directly. Then an average α-value was selected from Table I. This led to αS_a and then to C_b, the average dynamic base shear coefficient, or the demand on an assumed elastic structure. The average design base shear, B, for assumed yield-stress levels was obtained or estimated. Then K or B/C_b was plotted against D, the average damage cost factor, in %; D is defined as the damage repair cost divided by the current replacement cost.

Figure A shows the results of this effort. The triangular points represent precode conditions or what might be considered pre-modern seismic code, the circles are for cases in which some modern code was employed, and the crosses are for small residential buildings. The lines are drawn "by eye" and are not based on any regression analysis or other formal

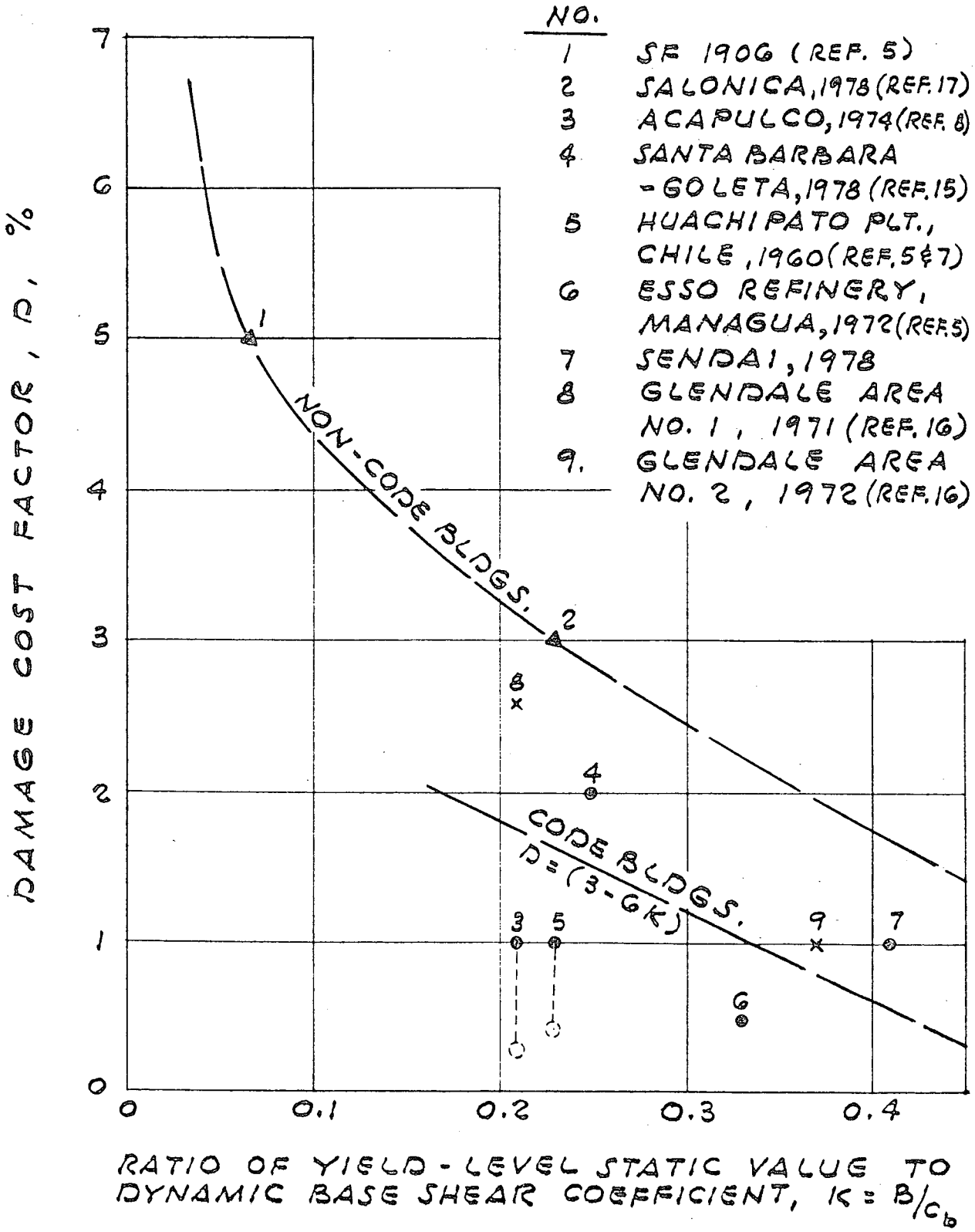

FIG. A - DAMAGE VS. RELATIVE DESIGN LEVEL

procedure not justified by the data. Points 3 and 5 are arbitrarily plotted at D = 1% although reported values were about 0.25% and 0.4%, respectively. Point 4 is also expected to fall well below the 2% plotted. The lower line would apply solely to the data range of from K = 0.2 to K = 0.4, which seems to include all cases but one. The upper line was based partly upon another point (off the plot) and is, of course, merely an approximation. This work represents a tremendous amount of empirical evidence combined with theoretical response factors and judgment based upon long study and experience. Although specific points may have some error, the overall evidence is compelling.

The interpretations of Figure A, which represents mean conditions (no attempt has been made to cover variations including extremes from the mean empirical data), are: (1) only about 0.5% to 2% of damage is associated with K factors as low as 0.2 to 0.4 where some reasonably adequate code design applies, or where the buildings are small wooden residential structures; (2) K's of 0.2 to 0.4 mean that, according to conventional code design procedures and elastic stresses to yield, the dynamic base shear coefficient C_b, based on instrumental peak acceleration, is 2-1/2 to 5 times the yield-point design value. When allowance is made for possible errors in design or construction or for faulty materials, these values could easily double; (3) where modern codes have not been used, the K value will be less, or the damage level for a given K value will increase; (4) in view of this and the great reserve of resistance in the inelastic range of ductile structures (which has not yet been included), the A_q is considered a poor index of demand; (5) if a line were extended to K = 0.5, there would be no damage indicated even though the allowable yield point value would be only half of C_b; if errors were discounted, the allowable yield point value could be only one-fourth of the dynamic base shear coefficient with little or no damage. It can be shown that

$$K = \frac{xBc\sqrt{2\mu-1}}{(DAF)a'\alpha} \tag{3}$$

for which all terms have been defined previously except for x, which is the ratio of the real capacity at yield to the conventional (code) capacity at yield, and μ, which is the first-story ductility factor. In Figure A, x and μ have been taken as unity and hence are not shown. It is known that there are many factors which generally lead to greater values or capacity than the codes allow. These have been outlined in other papers (Refs. 3, 5, 10) and includes the fact that material strengths provided exceed the minimum values specified and used in design; the conservative manner in which allowable stresses are determined from laboratory tests; safety factors; etc. Here they are all lumped into x. The inelastic range, with its energy-absorption capacity, is conservatively lumped into $\sqrt{2\mu-1}$. If the value of K' is 1 (as in Figure B) or greater, there is no building damage at all, nor are there any demands in the inelastic range. A K'-value of 1.00 can be obtained with any value selected from Figure B. For example, if the x factor is 2 (that value or more can be readily demonstrated, Refs. 5 and 10), allowable ductility is 1.5, and A_q is 0.8g, what are c and a'? Figure B provides c = 0.35, so a' = (0.35)(0.8g) = 0.28g. K' is defined here as $xc\sqrt{2\mu-1}$.

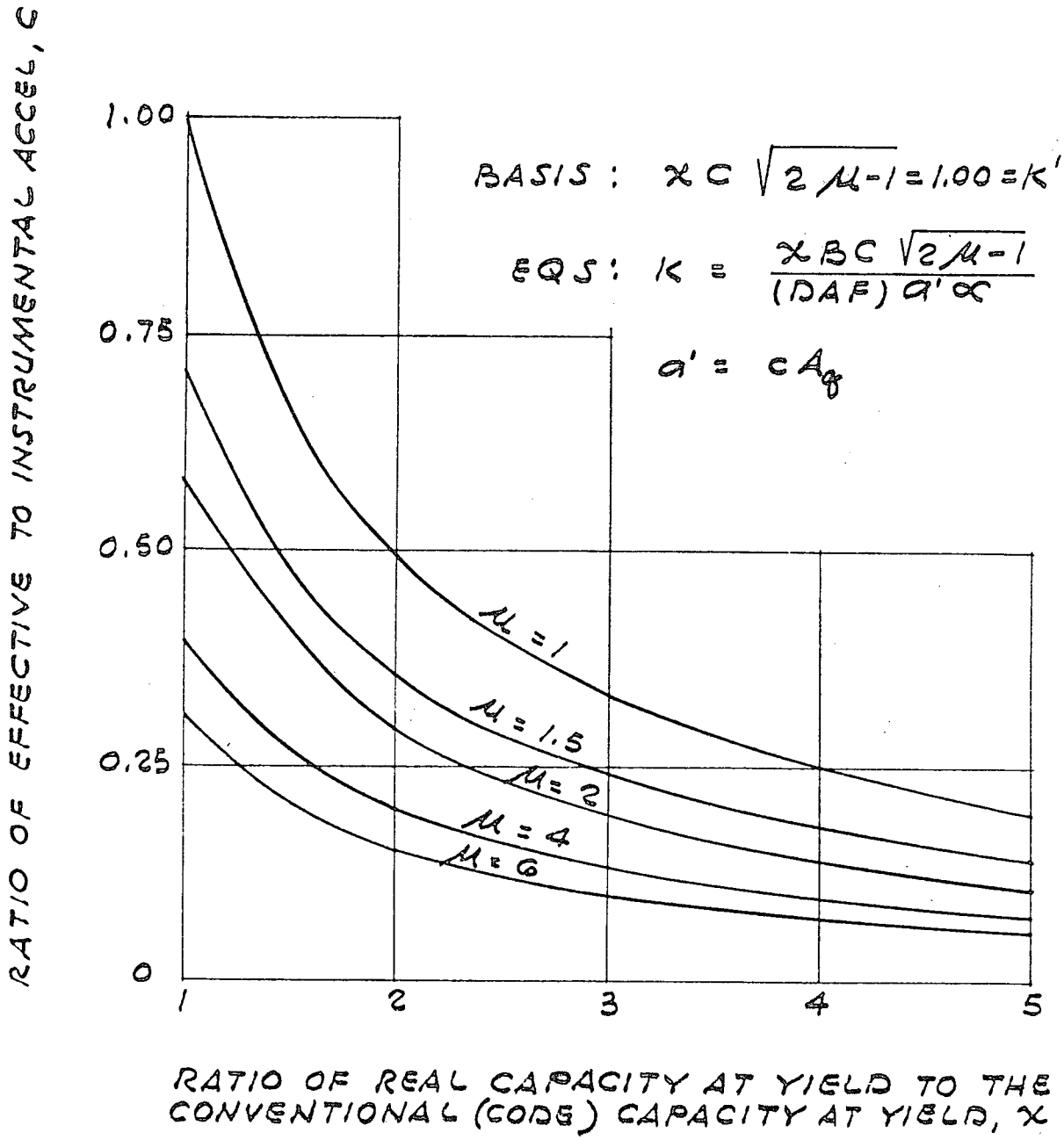

FIG. B - VALUES χ, C AND μ FOR $K' = 1.0$

As another example, consider a well-designed and well-constructed tall building for which A_q is 1.1g, x = 2.5, μ = 4, α = 0.70, and DAF is 1.2. What is the design basis? From Figure B, c = 0.15, so a' = (0.15)(1.1g) = 0.165g, and B for K' = 1.0 is 0.14 at yield-level stress. There are now possible adjustments: (a) to code-allowable rather than yield-allowable stresses; (b) to greater safety factors, K; and (c) to lesser K' factors (as K in Figure A, where a small percentage of damage may be acceptable). Assuming code-allowable stresses at 70% of yield, the alternatives shown in Table II are obtained.

TABLE II: DATA FOR A HYPOTHETICAL TALL BUILDING

Dynamic safety factor, K'	Coefficient B at 1-1/3 times normal code stresses	Estimated damage (%)
1.5	0.15	none
1.0	0.098	none
0.8	0.078	minor
0.4	0.039	0.5 to 1
0.2	0.020	1 to 2

The above values must be considered in proper perspective. The prior assumptions as to x, μ, α, and DAF are most important as are the assumptions that higher modes are not (here) contributing significantly to base shear, that the first story is the critical one, that the increase in design shear does not shift the period and thus the spectral acceleration to greater values, that drift or P-Δ effects are not critical, and that there is excellent quality control throughout. If and as these caveats apply, there is reason why tall buildings designed for coefficients that are a fraction of instrumental acceleration can survive. Obviously, for severe earthquake exposure, the margins are thin, and individual analysis is strongly recommended.

Table III is a useful compilation of data for various combinations of K', μ, x and c, where K' represents the coefficients as shown.

TABLE III: COEFFICIENT c FOR VARIOUS CONDITIONS

Basis: $K' = xc\sqrt{2\mu-1}$; $a' = cA_q$; for K' = 1, see Figure B

K'	x = 1	x = 2	x = 3	x = 4
		μ = 1		
0.2	0.20	0.10	0.067	0.050
0.3	0.30	0.15	0.10	0.075
0.4	0.40	0.20	0.13	0.10
0.5	0.50	0.25	0.17	0.13
		μ = 1.5		
0.2	0.14	0.071	0.047	0.035
0.3	0.21	0.11	0.071	0.053
0.4	0.28	0.14	0.094	0.070
0.5	0.35	0.18	0.12	0.088

K'	x = 1	x = 2	x = 3	x = 4
		μ = 3		
0.2	0.089	0.045	0.030	0.022
0.3	0.13	0.067	0.045	0.034
0.4	0.18	0.090	0.060	0.045
0.5	0.22	0.11	0.075	0.056
		μ = 5		
0.2	0.067	0.33	0.022	0.017
0.3	0.10	0.050	0.033	0.025
0.4	0.13	0.067	0.044	0.033
0.5	0.17	0.083	0.056	0.042

It is clear from this study of observed events that conventional buildings need excellent quality control and much ductility and (preferably) also redundancy to withstand very high spectral accelerations at their period values. It is also apparent that so-called high-risk installations are designed with very conservative accelerations and procedures. It is obvious that in all cases studied there was no need to design to peak instrumental accelerations or even to a major percentage of those values.

THEORETICAL CONSIDERATIONS

It is well recognized that the response spectrum, when suited to a site and smoothed, is a good criterion for structural response. The peak ground acceleration taken alone, whether instrumental or effective, is a very poor criterion. An extensive study was made (Ref. 11) in which 20 earthquake time histories were modified by clipping or by augmenting the acceleration peaks prior to making response spectra for them. The clipping was done in an absolute manner -- that is, both positive and negative peaks were clipped or shaved to various levels compared with the greatest peak. Such clipping planes were applied throughout the entire record. The same was true of the augmentation, in which peaks were artificially extended. The 20 records were for peak A_q values ranging from 0.19g to 1.17g, for epicentral distances of 0.1 to 42 km, and for magnitudes from 4.7 to 7.7. In every case it was found that severe clipping ratios caused rather minor effects on peak spectral accelerations. The same was true for peak augmentations. The only exception, of course, is at very high frequencies, where the spectral curves approached the peak ground motion. Other investigators have found similar results (Refs. 13, 14).

The inference is clear: peak accelerations constitute a poor or conservative index of response. The effective energy is contained largely in the body of the time history, and relatively little is available in a few strong pulses or spikes or even in many narrow pulses. The number of peaks clipped seemed to have no correlation with the effect on the spectral diagrams. In general, 30% peak clipping (using only 70% of the record, in an absolute sense, closest to the zero line) produced only about 5% reduction in 5%-damped peak spectral response. Figure C shows the results by period band for the Taft 1952, S69E record, and Figure D shows the results for El Centro 1940, NS by peak period.

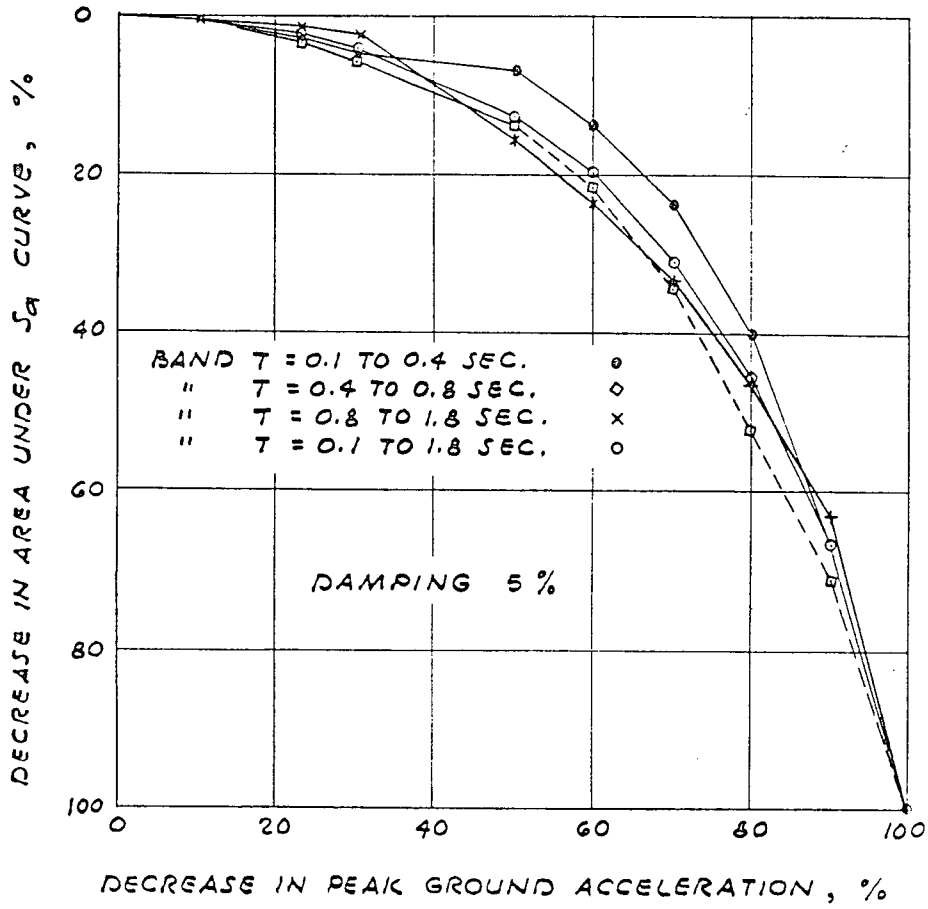

FIG. C - EFFECT OF CLIPPING ON SPECTRAL AREAS FOR TAFT 1952, S69E RECORD

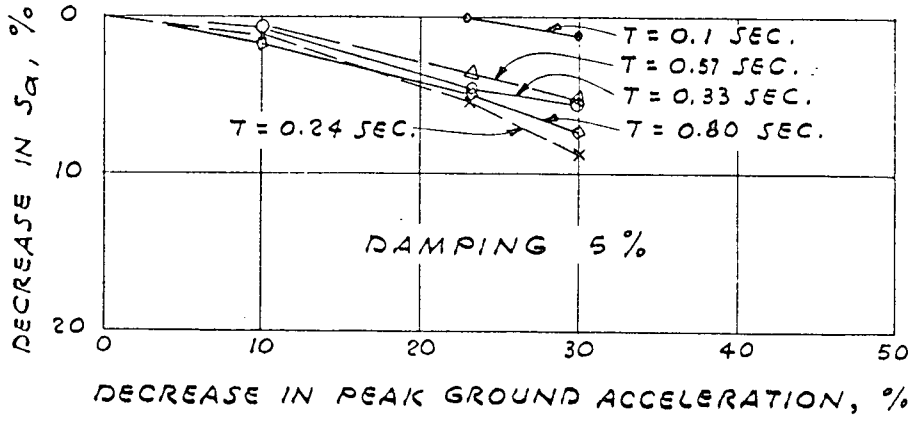

FIG. D - EFFECT OF CLIPPING ON SPECTRAL PEAKS, EL CENTRO 1940, NS RECORD

Is the effective-acceleration concept applicable only close to the energy source or also at some distance? At great distances, the motion may be essentially harmonic and of longer period; for example, at a soft site some distance from a major earthquake. A study was made as to whether or not the effects of peak clipping changed with epicentral distance. Only one clipping coefficient, 0.70, was used and one damping ratio, 5%. The percentage decrease in spectral area (from that with no clipping) was plotted against epicentral distance by period band. Distances ranged from 0.1 to 42 km. The results show a slight average tendency for there to be less change in the mean spectral response under clipping with distance than close in; this is the opposite of the expectation for the distance range studied (to 42 km). However, the coefficient of variation is greater close in than farther out. No doubt much of this is due to complexities among epicentral, hypocentral, and normal fault distances. All period bands show the same tendencies, but the 0.8-sec to 1.8-sec band has lower mean values and contains several points at which 30% reduction in PGA has no effect on S_a whatsoever. In general, 30% clipping results in only 5% spectral reduction on the average, with slightly less reduction at distances between 15 and 42 km than at those within 15 km. Rock and alluvial stations were included and show no difference. In general, the correlation with distance is very poor, showing that the peak clipping phenomenon is not just a close-in parameter.

The clipping factor accounts for some of the c-coefficient effect. There are other theoretical factors too numerous to list here (Refs. 5, 10).

TESTING AND EXPERIMENTS

Testing data have for years been used in the most conservative manner for design by taking the extreme low values to create design equations. Controlled testing shows clearly that strength, ductility, energy absorption, and damping are -- on the average -- much greater than they are given credit for in the codes. Furthermore, the minimum values specified are almost always greatly exceeded in practice (Refs. 5, 12). Many factors contribute to coefficient x, which can be shown to be 2, or much more, in most cases.

JUDGMENT, AND CONCLUSION

Judgment must be based not only on experience but also on all the evidence and the data, and all items must be reconciled, as demonstrated in this paper. There are valid reasons why 5% base shear buildings do not fall down under 0.5g or more instrumental accelerations. To the extent possible in this short paper, the important reasons have been shown, including the very important empirical record of nine events with high accelerations. The peak instrumental acceleration need not be used directly in design -- that would be wasteful. Means of obtaining effective acceleration have been presented, and these values, converted to design base shear coefficients, are in general accord with modern seismic codes where good judgment and high quality control have been used in both design and construction. However, some damage can be expected on the average, and deviations from the mean values discussed in this limited paper

toward the unsafe side can lead to severe damage and possibly some collapses in major earthquakes. High-risk facilities like nuclear power plants are often subjected to extremely severe and unrealistic seismic conditions in view of all the facts. A much more realistic approach would be to consider the many factors as variables in joint-probability analysis rather than to compound envelope or peak values from item to item (Ref. 10).

This paper is generic in nature and should not be applied to specific cases without proper analysis.

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