COMPARISON OF SEISMIC DESIGN PROVISIONS

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SUMMARY

The newly developed "Tentative Provisions for the Development of Seismic Regulations for Buildings" (Ref. 1) represent a significant departure from present practice as embodied in the model building codes. A full assessment of these changes is beyond the scope of this paper. However, a comparison of the basic design force level in the new provisions is made with that in the Uniform Building Code (Ref. 2), a well known standard. This comparison reveals, for the areas of highest seismicity, that the force levels in the new provisions are about the same for frames detailed for special ductility, somewhat lower for shear walls of reinforced concrete or plywood, somewhat higher for shear walls of reinforced masonry and for steel frames of ordinary ductility, and substantially higher for shear walls of unreinforced masonry and for reinforced concrete frames without special provisions for ductility.

SCOPE

The "Tentative Provisions for the Development of Seismic Regulations for Buildings" (Ref. 1), prepared by the Applied Technology Council, contain many provisions substantially different than found in current building codes. The specified force levels begin with a much higher spectral acceleration, comparable to what would be expected in a real earthquake. Design criteria are based upon significant yield, and the design forces are explicity modified for system ductility and damping. Statement of the hazard level is based upon a probablistic assessment of the ground motions expected in a fixed time frame and is expressed in two new maps. Several new details and procedures of analysis are specified as well. Provisions regarding details of construction and limitations on the use of certain systems in high hazard zones are also somewhat different than current codes. And a much more extensive set of provisions for non-structural elements of buildings is included.

The provisions are so different that an extensive program of comparative trial designs of hypothetical buildings is being executed by the Building Seismic Safety Council (Ref. 3). Once that program is completed in mid 1984, the Council intends to amend the ATC provisions and eventually recommend their incorporation in model building codes. Pending the results of those thorough studies, it is possible to make some simple comparisons between the ATC provisions and current building code provisions for seismic resistance.

The emphasis in this paper is on the specified design force level, commonly known as the base shear. Many engineers will relate very comfortably to comparisons of the design force level, and it is ordinarily directly

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translatable into terms of material quantities and construction costs. Therefore, the reader is cautioned that this is simply one measure of an entire host of differences beteen the ATC provisions and current building codes. Other significant cost impacts (either positive or negative) may arise from new analytical procedures, construction details, system limitations, etc. The basis used for most comparisons is the Uniform Building Code (Ref.2), because the UBC is the most commonly used code in seismically active regions of the USA. A basis for comparison with other model building codes is also established.

DEFINITION OF BASE SHEAR

The ATC defines the base shear by the equation $V = C_s W$, where W is the weight of the building and C_s is an acceleration coefficient. C_s depends on the ground acceleration at the site, the fundamental period of vibration and the ductility of the structure, and the type of soil profile at the site. Figure 1 shows a plot of C_s versus the period of vibration for a structure with no ductility (the elastic response case) and for a site with the maximum ground acceleration given for the USA (40% of gravity). Note that there are three discrete curves for the three type of soil profiles defined. Soil profile type 2 is the default for sites that do not fit the definitions. The kink at four seconds is only for use in design by modal analysis. For static design procedures the curve above four seconds is a smooth extension of the curve below four seconds. Finally, note that the maximum value of C_s for elastic response in the most seismically active regions is 100% of gravity. Although this is realistic, it is far above the design forces specified in current codes. For most buildings, the design force level is much less than 100% of gravity due to allowances for the ability to yield without collapse.

The UBC defines the base shear by the equation $V = Z \cdot I \cdot K \cdot C \cdot S \cdot W$, where W is the weight, Z is a function of ground acceleration at the site, I is a factor to increase the required strength of particularly important buildings, K is a factor related to the ductility of the building system, C is the basic spectral ordinate depending on the fundamental period of vibration, and S is an amplification factor for resonance of the site with the structure. Z is 1.0 for sites in the highest hazard cone, and I and K are 1.0 for ordinary framed buildings. Therefore the product $C \cdot S$ is roughly comparable to C_s from the ATC provisions. Figure 2 is a plot of $C \cdot S$ versus period. First, note that the maximum value is 14% of gravity, far less acceleration than would be expected in a significant earthquake. A substantial allowance for ductility is implicity included in the UBC. Also note that there is a continuous range of values due to the fact that S is a continuous function with limit of 1.0 and 1.5. The various kinks in the curve are due to limits placed upon parameters used to calculate S.

Other building codes in the USA include seismic provisions specified in American National Standard A58-1972 (Ref. 4), which is based upon a much earlier version of the UBC. These provisions define the base shear by the equation $V = Z \cdot K \cdot C \cdot W$, where all terms are as defined in the UBC. The value of C is different, however, and this C is also plotted in figure 2. Note that it is substantially less, particularly for periods less than 1.2 seconds, which includes the bulk of all buildings. This difference is shown more graphically in figure 3. Thus for a building with a period of about 0.5 seconds, the maximum value of $C \cdot S$ from the UBC is about 124% greater than the value of C in A58.1-1972. If the building happened to be a hospital, for which $I = 1.5$, the increase is 235%! The difference between the UBC and A58-1972 is in many cases more substantial than the difference by the UBC and ATC. Therefore, figure 3 provides a means to assess the change from a code based upon A58-1972 to one based upon the ATC provisions.

COMPARISON OF BASE SHEAR

The basic design criteria in the ATC are different than those in the UBC. Consequently, it is more appropriate to compare design forces on an equivalent member strength basis. Thus, for a structure in which the primary lateral force-resisting elements require the same strength by UBC and ATC, the equivalent ATC base shear, V_e, will be identical to the UBC base shear, V_u.

In the UBC, two forms of design criteria are used. For steel, wood, and masonry, the effects, Q, of the unfactored seismic loads (combined with various gravity loads) are measured against the allowable strength, S_a, increased by one-third:

$$Q(V, D, L) = 1.33 \cdot S_a$$

For reinforced concrete, the effects of the factored load (combined with various factored gravity loads) are measured against the nominal strength, S_n times the strength reduction factor, $\emptyset$:

$$Q(f_v \cdot V,\ f_d \cdot D,\ f_l \cdot L) = \emptyset \cdot S_n$$

In the ATC, one form of design criteria is used for all materials. The effects of the unfactored seismic load (combined with various factored gravity loads) are measured against the strength at "significant yield", S_y:

$$Q(V,\ f_d \cdot D,\ L) = S_y$$

For steel, wood, and masonry, S_y is taken as a factor of S_a:

$$S_y = m \cdot \emptyset \cdot S_a$$

S_a is the same S_a used in the UBC, m accounts for the nominal difference between allowable and yield, and $\emptyset$ is a capacity reduction factor analogous to the strength reduction factor $\emptyset$ used by the UBC (and by ACI 318) for concrete. $\emptyset$ depends on the type of member and the mode of failure being considered. For reinforced concrete, S_y is taken as $\emptyset \cdot S_n$, just as in the UBC. With a few exceptions, the $\emptyset$ for concrete is same as specified in ACI 318. For this comparison, the equivalent ATC base shear is established thus:

For steel, wood, and masonry: $V_e = 1.33 \cdot V_a \div (m \cdot \emptyset)$

For concrete: $V_e = V_a \div f_v$

The conversion is imprecise for concrete, due to the difference in load factors on gravity loads. However, this discrepency should not be large for the primary elements resisting lateral force. The value f_v used herein is 1.43. m takes the values 1.7, 2.0, and 2.5 for steel, wood, and masonry, respectively. For nearly all steel components, $\emptyset$ is 0.9. For most wood members, $\emptyset$ is 1.0, however, it is as low as 0.65 for some connections. $\emptyset$ is 0.75 for plywood diaphragms and shear walls because the referenced allowable strength already contains a one-third increase for seismic loads. For masonry $\emptyset$ varies from zero to 1.0. In this analysis, $\emptyset$ is 0.4 for shear in unreinforced masonry and 0.6 for shear carried by reinforcement.

Plots of V_e against V_u for several common building systems are shown in figures 4 through 13. The plots do not show the potential variations due to differences in soil profiles. V_u was determined from the maximum value of S, because that is the default value. Furthermore, even after an extensive geotechnical evaluation, the maximum value is quite often the most appropriate value for common buildings. V_e was determined from the intermediate soil profile type, also because that is the default value.

Figures 1 through 13 are plotted for the maximum seismic hazard. The relative seismic hazard from location to location is not the same for each code, therefore the comparisons here must be adjusted for any given city. Table 1 is included as an illustration of a technique to extrapolate the comparison to a particular city. The ordinates for V_u are to be multiplied by the hazard ratio shown in the "UBC" column, which is the ratio of Z for the particular city to the maximum Z. Likewise, the ordinates for V_a are to be multiplied by the analogous number from the ATC column. Similar ratios for A58-1972 and A58-1982 (which is otherwise very similar to the UBC) are provided in Table 1. Generally, the ATC hazard ratio in lower-risk areas is smaller than the UBC's. This is primarily a result of introducing the probability of occurance as a factor in seismic zoning.

The ATC contains two parameters for defining the seismic hazard level, one related to the effect of nearby earthquakes, the other related to distant events. This makes it possible to change the shape of the spectrum in locations where the effect of large distant events might predominate for longer period structures. Plots for two locations are shown in figures 14 and 15.

REFERENCES

1. Applied Technology Council, Tentative Provisions for the Development of Seismic Regulations for Buildings, (ATC-3-06), National Bureau of Standards-Special Publication 510, Washington, DC, 1977.
2. Uniform Building Code, International Conference of Building Officials, Whittier, CA, 1973 and 1982.
3. "Plan for a Trial Design Program to Assess Amended ATC-3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings," National Bureau of Standards, NBSIR 82-2589, Washington, DC, 1982.
4. Minimum Design Loads for Buildings and Other Structures, American National Standards Institute, A58.1, New York, NY, 1972 and 1982.

TABLE 1: RELATIVE INDEX FOR SEISMIC HAZARD

	'82 UBC	ATC	A58-1972	A58-1982
Boston	0.75	0.25	0.50	0.375
New York City	0.1875	0.25	0.25	0.375
Atlanta	0.375	0.25	0.25	0.375
Chicago	0.1875	0.125	0.25	0.125
Houston	0	0.125	0.25	0.125
Seattle	0.75	0.5	1.0	0.75
Los Angeles	1.0	1.0	1.0	1.0

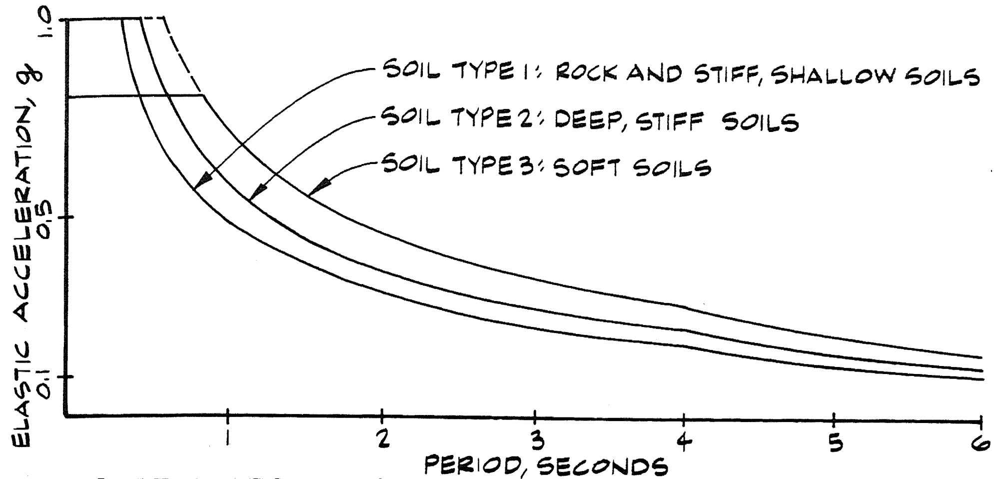

FIGURE 1. ATC DESIGN SPECTRA FOR ELASTIC CONDITIONS

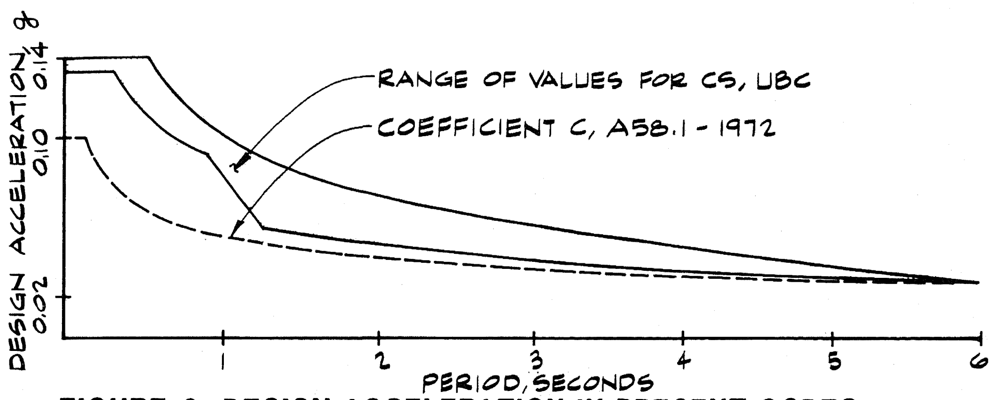

FIGURE 2. DESIGN ACCELERATION IN PRESENT CODES

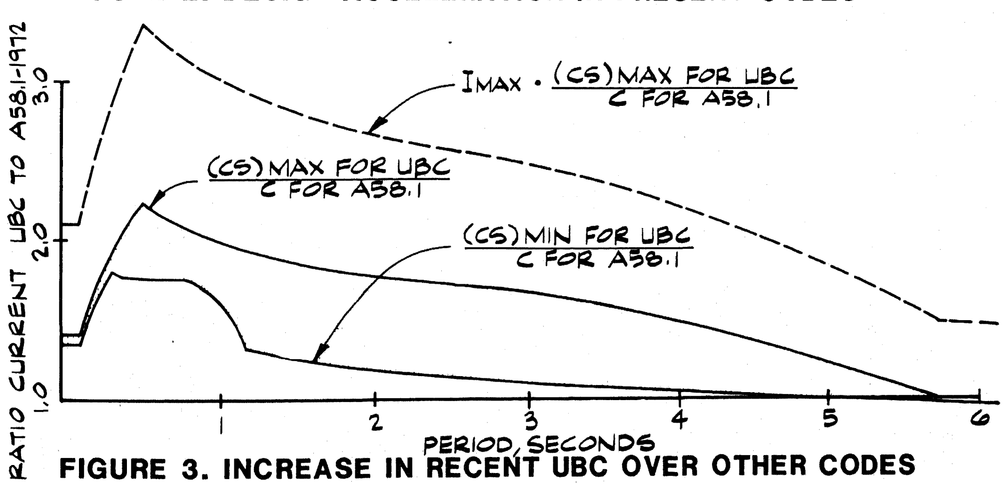

FIGURE 3. INCREASE IN RECENT UBC OVER OTHER CODES

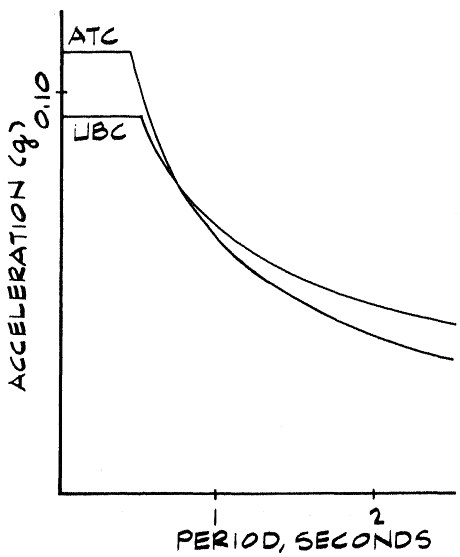

FIGURE 4. DUCTILE STEEL FRAME

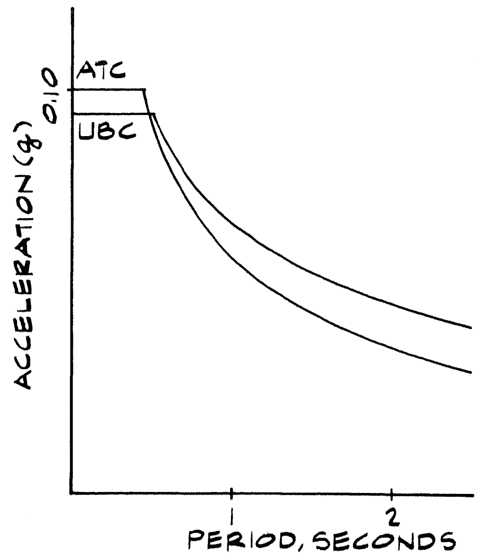

FIGURE 5. DUCTILE CONCRETE FRAME

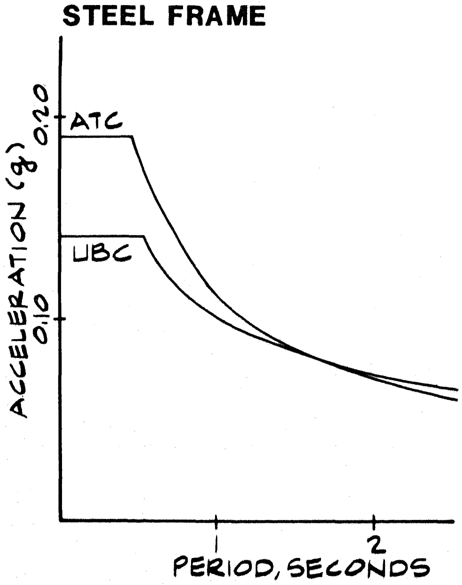

FIGURE 6. ORDINARY STEEL FRAME

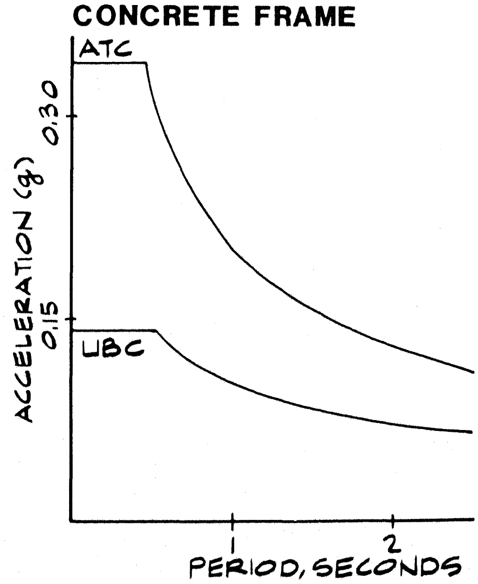

FIGURE 7. ORDINARY CONCRETE FRAME

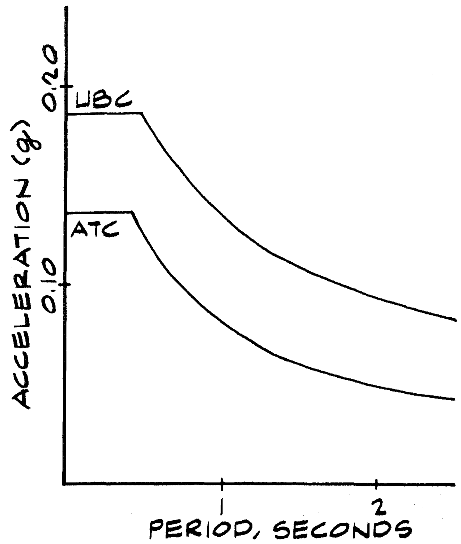

FIGURE 8. PLYWOOD SHEAR WALLS

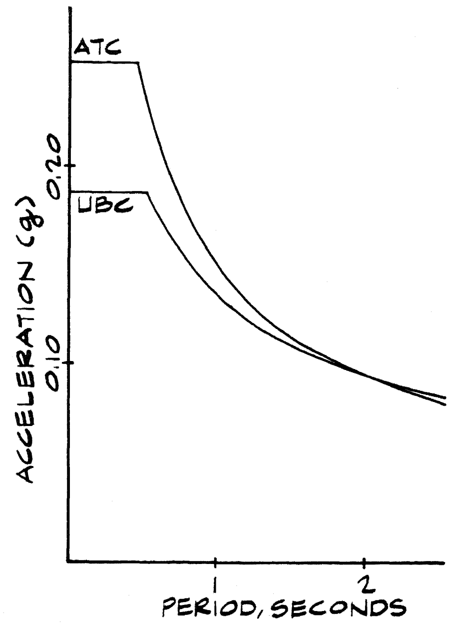

FIGURE 9. REINFORCED MASONRY BEARING WALL

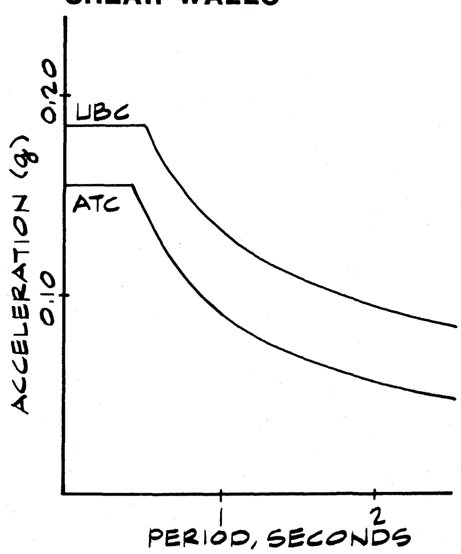

FIGURE 10. REINFORCED CONCRETE BEARING WALL

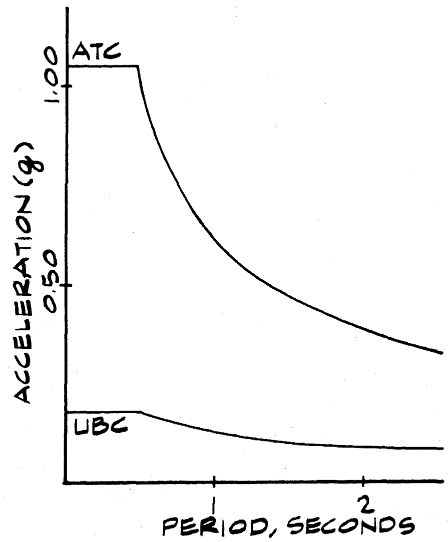

FIGURE 11. UNREINFORCED MASONRY BEARING WALL

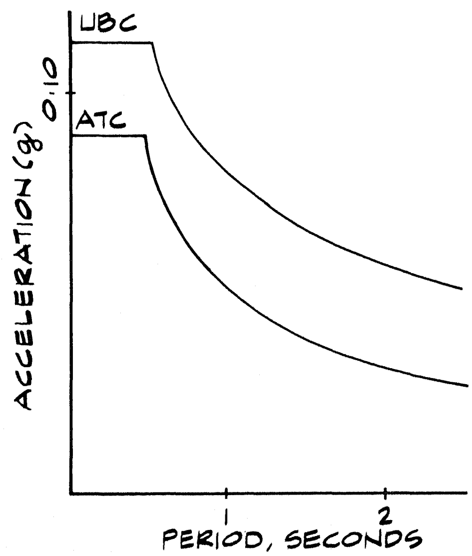

FIGURE 12. DUAL SYSTEM-CONCRETE SHEAR WALL

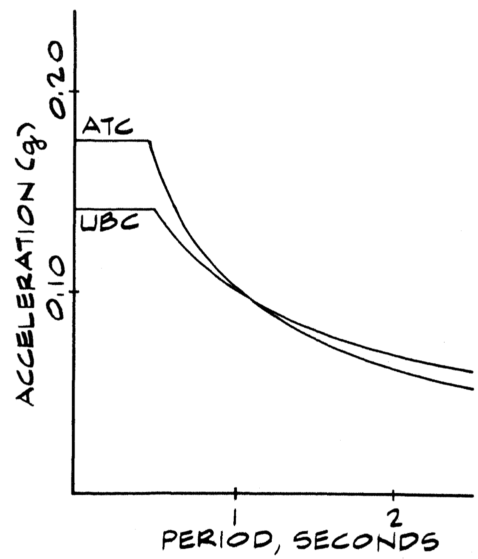

FIGURE 13. BRACED STEEL FRAME

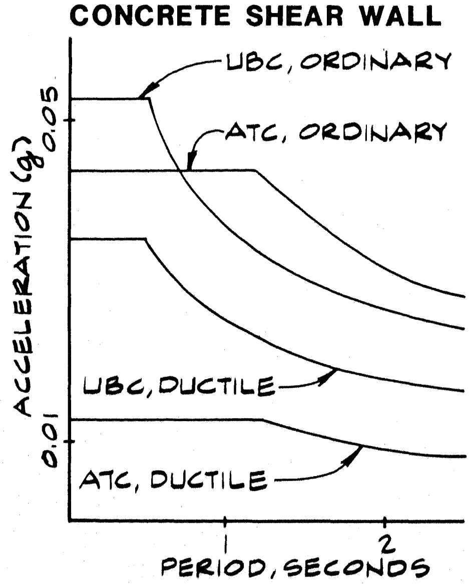

FIGURE 14. CONCRETE FRAMES IN PHOENIX

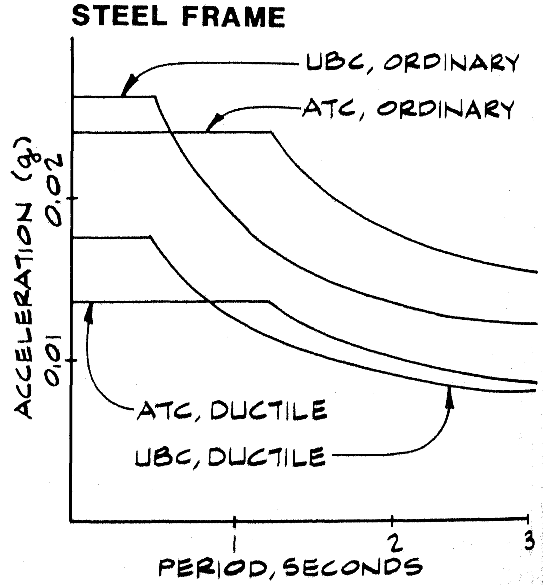

FIGURE 15. STEEL FRAMES IN PHILADELPHIA