



# Suggested Simplifications for the 1997 UBC Seismic Design Provisions

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## Abstract

This paper proposes some simplifying changes to the 1997 UBC Earthquake Design Provisions in order to improve the efficiency and accuracy of application to the design and analysis processes.

## Introduction

In the mad rush of the recent major code changes involving: the conversion to strength design basis, near-source ground motion, redundancy factors, and others, very little time and energy remained to look at some of the existing design requirements that have been in the lateral force provisions for the past 20 to 30 years. The new changes have just been added to these older requirements, many of which are quite subjective and in need of a good "house-cleaning" and modernization to current practice. Also, because of the need for consensus, even some of the new provisions were adopted on a "last minute" compromise basis, rather than for effective design performance, and merit either improvement or deletion.

Given the unhappy but realistic condition that the structural engineering fee allocation for a given project is not likely to change even though new code provisions such as the 1997 UBC can require a considerable increase in design time over that of the previous code, one answer to survivability is to reduce or simplify some of the work intensive provisions that really add very little to seismic performance capability of a structural design. The general justification for this

simplification is that many of the required procedures result in number-crunching and time-consuming refinements that are completely overshadowed by the real subjective uncertainty in the assignment of code specified values such as for the site seismic ground motion and the system reduction factor  $R$ . The objective is to provide sufficient time for the production of quality construction documents and on-site observation, and still allow our resource of knowledgeable and experienced professionals to remain in practice. Also, the simpler we can make things, the less the error of application will occur due to the inexperience and/or haste of the users. The following suggestions and recommendations by the author are a result of experience gained from editing the 1999 Blue Book, and from the written and verbal comments generated by the Seismic Design Manual and its seminars. Fruitful areas of simplification are in:

- load combinations
- the definitions and consequences of structural irregularities
- application of accidental eccentricities
- structural modeling procedures
- dynamic analysis procedures and reduction to design values
- element design provisions

## Load Combinations

**Sections 1630.1.1 and 1612.2.1:** Delete  $E_v = 0.5C_a I D$  from Formula (30-1) so as to be  $E = \rho E_h$  and add  $0.5C_a I D$  to the strength design load combination (12-5) so as to be:

$$(1.2 + 0.5C_a I)D + 1.0 E + (f_1 L + f_2 S) = (1.2 + 0.5C_a I)D + (1.0) \rho E_h + (f_1 L + f_2 S)$$

Reason: The definition of  $E = E_v + \rho E_h$  is confusing since it is a mix of lateral seismic load effects and dead load effects. It would be preferable to just define the  $0.5C_a I$  part of the dead load factor as a representation of the vertical earthquake ground motion response.

**Section 1612.3:** Eliminate the basic allowable stress combinations in Section 1612.3.1. The alternate basic allowable stress combinations of Section 1612.3.2 have long served to provide adequate performance, therefore why use combinations that result in higher requirements because of the prohibited one-third allowable stress increase? The argument that Section 1612.3.1 is needed to better produce designs near to the strength basis designs of Section 1612.2.1 is not valid, particularly for the case of low live load where the combination of  $D + E / 1.4$  in Formula (12-9) would govern the design and there is no allowed stress increase: the equivalent strength requirement would be  $1.7D + 1.2E$ . The harm in keeping the basic combinations is that some enforcement agents may decide to require them because they are more conservative and are alleged to produce designs equivalent to the strength basis.

#### **Determination of Structural Irregularity and Use of Accidental Eccentricity**

**General Discussion:** For the evaluation of stiffness and torsional irregularity conditions it is counter-productive to use the relatively complex and lengthy procedures of dynamic analysis, involving the evaluation of the CQC of each required elastic response deformation parameter, and the search for its maximum value. The implied precision is not consistent with the subjectively assigned limit ratios for the determination of structural irregularity (such as 0.7 for stiffness and 1.2 for plan torsion) and the uncertainties due to the assumption that inelastic deformations can be predicted by the elastic response deformations. The real accuracy of the results is not significantly improved by the precision of the analysis. The following proposed changes are based on the concept that the results of the Static Force Procedure are sufficient for the determination of irregularities and that the results of the dynamic analysis should be required

only for the design force distribution patterns and possibly for the evaluation of the  $\Delta_M$  values of displacement and story drift. The Code is unclear on this matter, and it is quite possible that an enforcement agent would require the dynamic analysis procedure for the evaluation of the deformations needed for the determination of vertical and plan irregularities when dynamic analysis is either selected or required for the design process.

#### **Elements of the Dynamic Analysis Procedure:**

Refer to the Blue Book Commentary Appendix D for the Dynamic Analysis procedures. The appropriate elastic model of the structure should comply with Sections 1630.1.2 and 1631.3 and, when applicable, incorporate foundation flexibility and diaphragm deflections per Section 1633.2.4.

Given the elastic design response spectrum per Section 1631.2 as input loading, the dynamic modal analysis provides the Combined Quadratic Combination (CQC) for each required elastic response parameter. These parameters include not only the element loads and displacements that are to be reduced per Section 1631.5.4 for design requirements, but also include the deformation ratios needed for the determination of structural irregularities in Tables 16-L and 16-M. Although not specifically required by the Code, the correct procedure of modal analysis would require the CQC of each individual mode value of: the story drift ratios needed for Vertical Irregularity Type 1., the extreme end story drift values and their average for Plan Irregularity Type 1., and the extreme end story displacements and their average for the evaluation of the eccentricity amplification factor  $A_x$  by Formula (30-16). While this CQC processing of the individual mode parameters and identification of the governing maximum values can be done (but how many of us would know if the particular computer program that we are using is doing it?), it seems to be overly complex for the simple task of determining irregularities. Also, one drawback in dealing with combined mode response such as CQC is that the maximum response parameters do not all occur at the same time and the designer has neither a visual representation of the total deformed shape of the structure nor the ability to check for statical equilibrium since the maximum CQC forces do not occur at the same time. The Static Force Procedure may have some inaccuracies in

representing the particular elastic response load distribution pattern, but it does provide a picture of the deformed structure, and statics can be used. Therefore the following change is proposed.

**Structural Irregularity Tables 16-L and 16-M:**

For both the vertical and plan structural irregularities in Tables 16-L and 16-M, and related Section 1630.7, specify that the Static Force Procedure (SFP) may be used to evaluate the displacements and story drifts required for the Stiffness irregularity, Torsional irregularity, and Diaphragm discontinuity. Dynamic Analysis as required by Section 1629.8.4 may be used but would not be required for the determination of a particular structural irregularity condition. The irregularity conditions may be based on the results of the Static Force Procedure on the mathematical model of Section 1630.1.2.

**Need for Improvement of Irregularity**

**Definitions:** Since their first appearance in the 1988 Blue Book, the definitions of irregularity have remained pretty much the same even though they were initially quite quickly formulated with rather subjective percentage limits or bounds. In their present form they can unnecessarily trigger dynamic analysis and design penalties for perfectly good systems and may fail to identify deficiencies in vulnerable systems. Furthermore their impact on design is amplified by their nearly verbatim adoption into the NEHRP, FEMA, IBC, and Performance Based Design proposals. They are in definite need for improvement and some suggested changes are as follows.

**Table 16-L, Type 1:** Define Stiffness irregularity-soft story in terms of story drift values rather than story stiffness. Story stiffness is not usually directly available from the lateral force analysis of the mathematical model. The changed definition might be: A soft story is one in which the story drift is greater than  $(1/0.7 = 1.43)$  times that in the story above or greater than  $(1/0.8 = 1.25)$  times the average story drift of the three stories above. The story drifts may be determined at the story centers of mass due to the (SFP) static forces  $F_i$  applied at these centers of mass.

**Table 16-L, Type 4:** In-plane discontinuity or offset is defined in terms of length of elements.

This is rather unclear and the definition should be revised to identify any discontinuity in the load path for forces due to overturning moment. For example, an in-plane offset less than the length of the lateral force resisting elements can result in an unsupported overturning moment reaction

**Table 16-M, Type 2:** Re-entrant Corners irregularity is defined only in terms of the length of the projecting legs. This ignores diaphragm properties and the type and plan layout of the lateral force resisting elements. The particular re-entrant corner conditions that would require extra design strength need to be added to this basic definition. Consideration should be given to the following cases.

- For a rigid diaphragm condition, this plan configuration (particularly for “L” shaped plans) has the possibility of having torsion irregularity. However this would be detected by the Type 1. Torsional irregularity evaluation, and the re-entrant corner loading would be determined from the required torsional analysis.
- For a flexible diaphragm condition, each leg or wing of the re-entrant corner plan must be supported by vertical lateral force resisting elements at the extreme ends and by elements or collector elements at the re-entrant corner. The resistance requirements are the same no matter if load directions cause opening or closing action of the two wings. Also extra safety is provided by the  $\Omega_o E_h$  requirement for collectors per Section 1633.2.6.
- The only case where dynamic load amplification at the re-entrant corner might be applicable is where there is a large modal participation for the wing flapping mode of a multistory nonrigid diaphragm model. Here possibly a large part of the acceleration response forces could occur at the re-entrant corner if the modal period were to be at the larger spectral values. However these forces are reduced to match with the required base shear. It would appear that any increased force intensity from that of the Static Force Procedure would be covered by the  $\Omega_o E_h$  requirement for collectors per Section 1633.2.6.

**Section 1630.7:** The evaluation of the accidental eccentricity amplification factor  $A_x$ , Formula (30-16), should be based on story drift rather than displacement so as to be consistent with the evaluation of Torsional irregularity in Table 16-M. There is no apparent reason to use displacement, and the results used for the torsional irregularity determination can be used directly for  $A_x$ . Also it should be stated that the story drifts may be found by the SFP as allowed for determination of torsional irregularity.

**Section 1631.5.6:** To avoid the mass shift analyses with  $\pm e_{acc}$  for dynamic analyses, use the permitted Static force accidental torsion ( $F_x e_{acc}$ ) couple loading and add the effects to the reduced (Section 1631.5.4) results of the analysis with zero mass shift. The evaluation of the story drifts required for the determination of torsional irregularity and possible amplification factor  $A_x$  that require the  $\pm e_{acc}$  mass shift may be performed by use of the Static Force Procedure.

### Modeling Provisions

**Integrate modeling instructions of Sections 1630.1.2 , 1631.3 , 1633.2.4:** These three separate Sections requiring that the analytical model of the structural system should consider: cracked section properties, elements not part of the lateral force resisting system, three-dimensional representation of torsional irregularity, foundation flexibility and diaphragm deformations should be combined. All of the listed conditions need to be considered for inclusion in the model no matter if the purpose is for the SFP element forces, displacements and story drifts  $\Delta_S$ , dynamic analysis, or deformation compatibility. Only one model is needed, unless the designer needs to evaluate the effects of uncertainty by using upper and lower bound values for conditions such as foundation flexibility, diaphragm flexibility, or the interaction of non-structural elements on the structural system.

**Section 1633.2.4:** Delete requirement that deformations be found from a structural model without the stiffening effects of those elements not part of the lateral load resisting system and compensate by using  $1.2\Delta_M$ . The element loads  $E_h$  and displacements  $\Delta_S$  due to the prescribed lateral forces on the basic structural model of Section 1630.1.2 can then be scaled by the factor

$1.2\Delta_M / \Delta_S$  to provide the element loads induced by the  $1.2\Delta_M$  deformation required to verify deformation compatibility. The Section 1630.1.2 should be expanded to provide instructions for when and how to incorporate foundation flexibility and diaphragm deformability in the mathematical model.

### Element Design Provisions

**Section 1632.2:** Add a special procedure of the determination of the out-of-plane load  $F_p$  for concrete and masonry walls supporting flexible diaphragms. For these walls, find  $F_p$  using Equation (32-2) with  $a_p = 1.25$ ,  $R_p = 3.0$  and  $h_x$  at single story wall panel centroid height. For  $C_a = 0.4$ , the resulting  $F_p = (0.42)W_p = (1.4)(0.3)W_p$  gives parity with the 1994 UBC. For two story wall panels, find  $F_p$  at centroid of each story using  $h_x$  at each story centroid height and  $W_p$  as panel weight for each story.

Reason: Equation (32-2) is intended to represent acceleration force on the center of mass of equipment or elements attached at a given floor level  $h_x$ . For distributed elements such as wall panels attached at two or more elevations it is presently required to use the average of the  $F_p$  values at the lower and upper levels of attachment. The proposed change is simpler and, with the use of  $a_p = 1.25$ , gives a better representation of the dynamic behavior of the distributed mass of the wall panel.

**Section 1633.2.9 , Item 6:** Delete this prohibition of the one-third allowable stress increase since Sections 1630.6 and 7. require torsional moment effects, collectors require  $\Omega_0 E_h$  per Section 1633.2.6, and wall anchorage provisions have been increased by nearly a factor of two.

**Section 1633.2.9 , Item 7:** Any amplified loading that might occur at the re-entrant corner due to a primary opening and closing mode shape in a multi-story building model with semirigid diaphragms would be detected by the dynamic mathematical model required by Section 1631.3. This particular mode cannot occur if the diaphragm is rigid. For the flexible diaphragm condition, each leg or wing of the re-entrant corner plan must be supported by vertical lateral force resisting elements or collector elements. The resistance requirements are the same no matter if

load directions cause opening or closing action of the two wings.

**Flexible Diaphragm Assumption for Wood Frame Panel Construction:** Present Code wording requires the investigation of whether a wood panel diaphragm is to be considered as not flexible as per Section 1630.6 ; and, if not flexible, then a torsional analysis must be performed for the wood panel wall shears and other lateral bracing elements. This is a change from a traditional practice that has assumed that all wood panel diaphragms are considered flexible regardless of the relative rigidity of the supporting wall panels. While there can be cases where the diaphragms would be classed as not flexible based on deformation criteria, and where the resulting computed torsional shears on walls of an irregular plan configuration can result in significant and appropriate design loadings as compared to the loads from the assumed flexible diaphragm condition, there are also many cases where the flexible diaphragm load distribution is adequate when there is a reasonably symmetric plan condition and there is redundancy provided by non-calculated partition walls. Rules need to be formulated in terms of amounts or floor area ratios of partitions and bracing configurations that would permit the use of the flexible diaphragm assumption or at least a tributary load minimum for load distribution. Because of the real uncertainties involved in the as-built stiffness of wood frame construction, general rules may be more appropriate than detailed calculations involving these highly variable stiffness properties.

### Examples

**Example One:** Find the design loads  $E_h$  and deformations  $\Delta_S$  for a concrete SMRF over 240 feet in height such that dynamic analysis is required. Given no vertical irregularity types 2, 3, 4, or 5 ; and no plan irregularity types 2, 3, 4, or 5. It is necessary to check for vertical irregularity type 1, and plan irregularity type 1. Purpose is to show the advantage of using the SFP for determining irregularities when dynamic analysis is triggered.

- Formulate model per proposed change that integrates the modeling requirements.

- Run one dynamic analysis without accidental mass shift, and with required input response spectrum. Output is the dynamic response force and deformation parameters,  $E_D$  and  $\Delta_D$ , and the (Method B) calculated design period  $T_B < 1.3 T_A$ .
- Determine trial design base shear  $V_B$  using  $T_B$  and the corresponding lateral forces  $F_i$ , these can be later reduced by a 0.9 factor per Section 1631.5.4 if no irregularities are detected.
- Check for Table 16-L Vertical Irregularity Type 1 using the  $F_i$  at centers of mass.
- Check for Table 16-M Plan Irregularity Type 1, using the  $F_i$  at  $\pm e_{acc}$
- Assume for this example that there are no irregularities, and therefore the required base shear  $V_R$  to be used for the reduction of the elastic response parameters is  $V_R = 0.9V_B$ .
- Compute, by 0.9 factor, the new set of lateral force  $F_i$  values corresponding to  $V_R$ .
- Compute the load and deformation effects of accidental eccentricity by applying pure couple loading ( $F_i e_{acc}$ ) at each floor level
- Reduce  $E_D$  and  $\Delta_D$  by the ratio of  $V_R / V_D$  and add the absolute values of the ( $F_i e_{acc}$ ) pure couple load effects to determine the design values of  $E_h$  and  $\Delta_S$ .

Note that only one dynamic analysis had to be performed.

**Example Two:** Find the design loads and deformations for a Building Frame System with concrete shear walls and a concrete vertical load bearing frame. The building has no irregularities and qualifies for the use of the Static Force Procedure.

Purpose is to show the advantage of using one single analytical model for both the evaluation of the design load and deformation parameters and for the verification of deformation compatibility. Also, it is desired to show an alternative to the old (pre-computer) practice of designing the "100% V" shear walls as independent from the frame, and then checking the frame for deformation compatibility assuming that the

frame is independent from the walls such that the real interaction effects of the total wall and frame system are ignored.

How do we handle the problem of the designing the shear walls for 100% V, yet consider the real interaction of the frame, and then check the frame for deformation compatibility ?

How do we handle the consideration of accidental torsion effects for both the shear wall and frame design requirements ?

- Formulate model per proposed change for integrated model requirements.
- Calculate design period  $T_B < 1.3 T_A$
- Determine required base shear  $V_R$  using  $T_B$  and corresponding  $F_i$
- Apply the  $F_i$  to the model. Output is:  $\Delta_S$ , shear wall-frame reactions  $E_W$ , individual shear wall base shears  $V_W$ , total of the wall base shears  $\Sigma V_W$ , and frame element loads  $E_F$ .
- Apply accidental eccentricity torsion couples ( $F_i e_{acc}$ ) to model and add absolute effects to  $E_W$ ,  $V_W$ , and  $E_F$ .
- Determine the required 100% shear wall base shear  $V_{RW}$  and corresponding shear-wall frame reactions  $E_{RW}$  by multiplying the  $V_W$  and  $E_W$  (with their accidental torsion effects) by the factor of  $V_R / \Sigma V_W$ . Shear walls are to be designed for these actions.
- Compute the deformation compatibility deformation  $1.2 R \Delta_S$  per proposed change. Check the frame capacity for the effects of this deformation. These effects  $E_{RF}$  can be found by multiplying the ( $E_F$  plus the accidental torsion effects) by the factor of  $1.2 R$ .

Note only one analytical model is required with only two seismic load inputs,  $F_i$  and torsions  $F_i e_{acc}$ . All required wall design loads and the frame loads for deformation compatibility verification can be found by the appropriate ratio multiples of the results.

Designing wall as a cantilever without frame reactions can unnecessarily increase the flexural capacity and thereby promote a shear failure mode. Ignoring the frame reactions when wall and frame are in the same frame line can result in inadequate detailing at the interaction points.

## Conclusion

This paper has been a presentation of a wish list of possible code changes that might make the seismic design process more efficient than its present form. If all or any are deemed useful and do not essentially degrade the life safety objective, then Seismology Committee may wish to consider proposals for their adoption.