University of California at Berkeley
Civil and Environmental Engineering

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CEE 227 -- Earthquake Resistant Design

Homework Assignments

Problem 17 and 18 due April 29, Problems 19 due May 8, 2008

Problem 17 – Initial "Preliminary Re-Design" of 3-story moment frame building

In Problem 13, you developed a table of story shear forces (as well as floor level displacements and interstory drift ratios) corresponding (i) to elastic behavior under earthquake hazards with a 50% probability of exceedence in 50 years and (ii) to a displacement ductility of 6 under the very rare 2% in 50-year hazard level (using Newmark and Hall' or some similar method). The simple spectral analyses done in that problem suggest that the existing structure design is much too weak and flexible for the design criteria considered. We also saw that the lower level earthquake might in fact control the design forces that need to be considered, as the difference between the forces associated with an elastic system subjected to ground motions subjected to the two hazard levels was less than the force reduction factor (R) we could use according to Newmark and Hall or other procedures for $\mu = 6$.

In this problem, we will use the simple preliminary design equations developed in class for moment resisting frames to estimate quickly the size of ideal girders in the first floor level above the ground for the initial configuration of the structure. These girder sizes should allow the structure to satisfy drift and strength requirements for the 2%/50 and 50%/50 excitations. In general, we would use the procedure to design the girders over the height of the entire building, but we want a simple "reality check" at this stage to see if using the initial configuration is feasible (or not). This illustrates the process you would use for the entire building.

> *Note:*
> *For the basic design approach presented in class, we need to identify the trial configuration of the structure (here our 1994 frame configuration), loads associated with the seismic hazard (here from Problem 13) and locations where yielding will be permitted (we will use a strong column – weak girder criteria). Column sizes will be controlled by the beams selected – thus, we do not need to spend too much time on the columns at this stage (other than to assume the ratio of I_g/I_c and to understand that an unnecessarily large beam M_p will require the columns, connections and foundations to be designed for large loads. The beam-column connection forces will depend on the beam moment and the depth of the beams and columns being joined. Thus, deeper members are often preferable, so connection design does not control.*

Current codes often incorporate a redundancy factor that will increase design forces or stipulate minimum numbers of lateral load resisting elements. This term is ignored in the following problems for sake of simplicity.

In this problem, we will assume three 30-ft bays are moment resisting at each end of the building – this is what was assumed in the original 1994 UBC design.

We will use ASTM A992 hot rolled wide flange shapes for the beams and columns, these have a nominal F_y = 50ksi, F_u=65 ksi, R_y = 1.1, R_t = 1.1. ϕ = 0.9 for flexure.
For the ratio of I_b/I_c for this problem, you may use the ratio represented by the girder (beam) and interior column sizes used in the original UBC94 building design. This is just a starting point, and other ratios may be tried later.

For the lowest floor in the building (only):

a. Estimate the gravity loads you will have acting on the perimeter beams (you have this value from previous assignments). We will use ASCE-7 load combinations. Dead and live load moments, M_D and M_L, respectively, can come from your structural analysis, or if you want a quick estimate suitable for this problem use $M_x = w_x L^2/12$, where x represents the unfactored and uniformly distributed dead load and live load, respectively.

   The beams must hold up gravity loads alone (in the absence of earthquake effects) so there is a minimum factored moment that must be considered for gravity, $M_{gravity} = 1.2M_D + 1.6\ M_L$. Determine what plastic section modulus you need to support gravity only (say $Z = M_{gravity}/\phi F_y$). This is a minimum value of Z you can use in the selection of your member.

b. When considering earthquake effects on the beams, the controlling gravity load will generally be: $M_{gravity} = 1.2M_D + 1.0\ M_L$. Thus, compute and save this value for later.

c. For the 50% in 50-year (continued occupancy) level earthquake, compute an acceptable value of Z_{co} so that the structure can be expected to remain functional after the earthquake without structural repair. In this case, compute the moment due to lateral earthquake loads as described in class. Sketch the distribution of this moment, and the moment due to $M_{gravity} = 1.2M_D + 1.0\ M_L$ along the length of one (interior) beam. What is the maximum combined moment (anywhere) along the length of the beam?

   For our acceptance criteria, we could use the yield capacity of the steel and keep the member elastic, but here we will assume that we will get our targeted behavior if our stress criterion is approximated by $1.5R_yF_y$. Note that this is not theoretically correct, as the steel cannot exceed a stress of F_u. Nonetheless, this or similar values are a widely used and expected to lead to acceptably small amounts of inelastic behavior for our performance criteria. That is, we assume that there will be little damage to steel structural elements if they are overstressed by a factor of 1.5 or 2. Thus, for this earthquake excitation level, $Z_{min\text{-}co}$= $(M_{EQco}+1.2M_D+1.0\ M_L)/(1.5R_yF_y)$. It is common in this type of supplemental calculation to assume $\phi = 1$. In practice, you may want to use $Z=M/\ \phi F_y$ or another appropriate criterion developed specifically for your project.

d. Using the elastic design story shears for the 50% in 50 year hazard level, and the appropriate values of R_y and ϕ where needed, as well as an acceptable interstory drift of ¾% (increased from prior problems as it is clear we will have to use nonstructural components that are more accommodating to interstory drift without becoming damaged), compute using the methods described in class:
   i. The minimum depth of the beam such that it will not yield before the drift criteria is reached (you may assume the columns are 17 inches deep (based on a 14 inch column with two 1.5 inch thick flanges), and
   ii. The minimum I_{co} such that the structure satisfies the continued occupancy drift criterion for the 50%/50year event.

   To do this part of the problem, you need an estimate of I_{girder}/I_{column}. This can be identified by trial and error, but do not do this for this problem. You may assume that this ratio would be the same ratio as you find in the 1994 building design for an interior column connection at the first floor level. In reality, you would need to iterate to find an appropriate value.

e. Using the inelastic story shears for the 2% in 50-year hazard level, a maximum ductility factor of 6, and an appropriate collapse mechanism, estimate the minimum values of Z_{cp}. For this, the required ultimate moment capacity M_{cp} would be $1.2\ M_D+1.0M_L+M_{EQcp}$. For this level of shaking, the structure is intended to form a complete collapse mechanism, and as such, Z_{cp} might reasonably be taken as $Z_{cp} = M_{cp}/(1.1(F_y+F_u)/2)$. This reflects strain hardening sufficient to get the steel half way between F_y and F_u (this assumption is used in FEMA 350). AISC 341-05 uses 1.1RyFy as the effective yield strength for such calculations. A value higher or lower could be picked, depending on the value of μ targeted and the degree that inelastic deformations concentrate at one level rather than being uniformly distributed over height.

f. Repeat the basic process in part d for the 2% in 50 year hazard level to determine the minimum value of I_{cp} for a peak interstory drift ratio of 4% (used in FEMA 350 for steel moment frames) and ductility factor of 6. Use methods described in the class notes to identify the optimal depth for the beam (again assuming the column is 17 inches deep). ASCE 41 permits a drift of 5% at collapse prevention, but this is excessive based on evidence in FEMA 350.

g. From the answers above, prepare a combined table of design values for the girder at the first floor above the ground including: