A LIFETIME COST APPROACH TO AUTOMATED EARTHQUAKE RESISTANT DESIGN

by

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SUMMARY

A lifetime cost approach to the design of earthquake resistant multistory steel building frames is presented. The development begins with the design objective: minimized lifetime cost including construction and earthquake-induced damage. Standard design constraints are then formulated for operating loads and a dual design constraint for earthquake loading. Finally, the design problem thus formed is explored through the example of a one-story frame.

INTRODUCTION

The problem addressed here is selection of member sizes for single-bay, multistory, unbraced steel frames with fully rigid connections. Uniformly distributed beam loads and earthquake-generated horizontal ground motion will be considered. A typical member of this class of design problems is shown in Fig. 1. The frame is symmetric about its vertical mid-plane, and members are to be selected from the set of A-36 rolled steel wide flange economy sections. Performance constraints for operating loads will be introduced through typical code requirements, while for earthquake loads a dual criterion based on selection of moderate and strong design earthquakes is adopted. As a design criterion, we take lifetime cost of the structure, which we assume to be composed of initial (construction) cost and the cost of earthquake-induced damage over its lifetime. The following sections briefly sketch a methodology for formulating the problem and supply an example. More detailed information can be found in [1].

DESIGN OBJECTIVE

To compare alternative choices of a given structure, a design objective must be quantified. Here, we choose minimum lifetime cost (LC), a choice obviously dependent upon the selection of design variables (design vector). Only those costs strongly related to the design variables need be calculated; costs which are relatively independent of the design vector merely add a constant to the cost, producing no effect on the outcome of the design process. Obviously, care must be exercised in selecting design variables compatible with the design objective (cost). Here, we select

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moment of inertia of the member cross-section for this purpose.

The LC associated with multistory framed buildings separates into two categories: (1) cost of construction and (2) cost of damage associated with structural overload, here assumed to result from earthquake exposure.

Construction Costs

Design vector dependent construction costs include costs of members, beam-column connections, including welding, transportation, size extra charges, painting, etc. We will indicate the form of these cost functions. If C_s denotes the unit cost of steel, the total frame cost can be written

$$\text{Total Cost} = C_s \, \gamma \sum_{m_i} A_i \, L_i \tag{1}$$

where A_i, L_i denote cross-sectional area and length of each member, γ the unit weight of steel and the summation is taken over all frame members. In many studies of optimal structural design, Eq. 1 represents the design objective function. To account for cost of connections, welding and other member-related charges, it is possible to develop empirical equations relating costs to section properties. Thus, the total construction cost C_c can be expressed in the form

$$C_c = \gamma \sum_{m_i} [C_s \, A_i + C_s' \, f_s \, (A_i)] L_i + \gamma \sum_{g_i} [C_c' \, f_c \, (I_i) + C_w \, f_w \, (I_i)] \tag{2}$$

In Eq. 2, the following definitions have been introduced:

C_s' = unit cost of additional charges for members (transportation, etc.)

C_c' = unit cost of connection steel

C_w = unit cost of welding connections

The functions f_s, f_c, f_w can be determined by curve-fitting, [1]. Symbols m_i and g_i on the summations denote "all members" and "all girders", respectively.

Damage Costs

To develop a model for damage costs resulting from earthquake-induced overload, it is necessary to relate damage to structural response parameters and identify an expected earthquake exposure hazard for the building lifetime. In a complete treatment of costs of future damage, certain economic assumptions dealing with the cost of money, etc., would have to be incorporated. To avoid departing from the main objective of our work here, a "constant dollar", unencumbered by economic considerations, is used.

We assume that damage costs can be divided into three categories: structural damage, non-structural damage, and down-time costs. The definition of structural damage is elusive. Fortunately, for steel framed buildings structural, as opposed to non-structural, damage is relatively unimportant, assuming the design prevents collapse of the structure. A suggested model, based on restoration of member ductility, can be found in [1].

Included in the category of non-structural damage are items such as interior and exterior walls, partitions, glazing, plumbing, electrical fixtures, etc. Taken collectively, the cost of damage for these items is much more significant than structural damage in steel framed buildings. From the above list, the principal contributions are from interior drywalls, glazing and masonry, if present. There is evidence to support the choice of story drift as an appropriate measure of non-structural damage. Utilizing data from [2], it has been found that the damage ratio D_n, defined as the cost of damage repair, divided by the cost of construction of the damaged items can be expressed as [1]:

$$D_n = 8.52\ \delta \tag{3}$$

where δ is the story drift in feet. Using Eq. 3 to compute the non-structural damage ratio, D_n, the cost of damage per story can be developed. The total cost of non-structural damage is then obtained by summing over all the floors.

Repair of non-structural damage frequently requires temporary shutdown or relocation of activities, with resulting costs and revenue losses which affect the LC. Review of data from [3] reveals a range of down-time costs from zero to 300 per cent of the total damage cost. In order to estimate this type of cost, some assessment of the susceptibility of the function of a building to such inconvenience costs must be made.

Lifetime Cost

The damage cost models developed apply to individual earthquakes. To obtain lifetime cost, it is necessary to make assumptions about the intensity and frequency distributions of earthquakes for the particular site and sum the damage costs over all expected earthquakes to obtain a lifetime exposure profile. This is accomplished as follows: we develop a model for the annual frequency of earthquakes with a given peak acceleration at a site utilizing a linear relation between log frequency and magnitude in connection with Housner "affected area" curves [4, 5] for a fixed fault direction. A least squares fit of the resulting simulation gives

$$n = 3.44\ e^{-15.25a} \tag{4}$$

where a is the acceleration normalized by gravity. The constants reflect seismicity appropriate to a Southern California site. To obtain damage costs, we must relate the proposed damage models to structural response, i.e., in Eq. 3, δ is a function of a. The lifetime cost of non-structural damage per story can then be written as

$$C_{NS} = \int_{o}^{a_{max}} Nn\ D_n\ da = \int_{o}^{a_{max}} d_t\ da \tag{5}$$

where n is determined by Eq. 4, N is the structural life in years, D_n is obtained from Eq. 3 and $d_t = Nn\ D_n$ will be called the "lifetime damage profile". As an example, consider a one-story frame with beam and column moments of inertia of 223 in.^{4} and 235 in.^{4}, respectively, with a span and height of 300 inches and 150 inches, assuming 5% of critical damping. Use of Newmark-Hall response spectra [6] gives story drift $\delta = 3.7a$, where δ is in inches. Using this in Eq. 3 with an assumed non-structural cost of 10% of the construction cost and employing Eq. 4 with a 50-year service life yields an expected lifetime damage profile

$$d_t = 4540\ a\ e^{-15.25a} \tag{6}$$

Eq. 6 is shown in Fig. 2. Note that most of the structural damage results from ground accelerations of less than 25% g with the peak in the curve occurring at 6.56% g. The area under this curve is easily computed from Eq. 6, in the general case, however, numerical integration is necessary to obtain the lifetime cost of non-structural damage defined by Eq. 5. For multistory frames, story drifts at each floor level can be found from appropriate dynamic analysis, e.g., employing modal analysis and maximum modal response estimates for the assumed response spectra. Eq. 5 is then evaluated at each story and the total damage cost obtained by summation. This result, together with Eq. 2, provides the design objective in terms of a lifetime cost.

PERFORMANCE CONSTRAINTS

Design limitations are typically imposed via building codes. Here we will treat constraints under operating loads in the usual manner; however, criteria for earthquake loading will follow a different course. Only the general outline of the constraint formulation scheme will be given; details are in [1].

Constraints Under Operating Loads

Maximum moments in members are required to satisfy the condition

$$|M| \leq C\ M_p \tag{7}$$

where M is the moment under operating loads, M_p the section plastic moment and c a reduction coefficient, typically ≈ 0.6. For columns M_p must be modified to reflect axial loading. Limitations on maximum beam deflection are also incorporated.

Because of the lateral strength requirements of earthquake resistant frames, it is assumed here that sidesway stability requirements will not play a prominent role in the design process. This requirement, along with any other limitations thought to be necessary, can be easily incorporated.

Constraints Under Dynamic Loading

For response to earthquake loads, we adopt the following dual design criterion:

(i) The structure should respond elastically to a moderate earthquake of an intensity reasonably anticipated within its lifetime.

(ii) During a maximum credible (strong) earthquake, the structure may yield significantly but must avoid collapse.

Design earthquakes representative of the above conditions are typically selected on the basis of their probability of occurrence. A sample probability of occurrence curve is shown in Fig. 3, generated on the basis of a 50-year life expectancy for a Southern California site [4]. Moderate earthquakes are chosen with a 50-80% probability of occurrence in mind, whereas strong earthquakes are picked in the 5-10% probability of occurrence range. Both are selected on the basis of a 50-70 year building life expectancy. Thus, two peak ground acceleration values, referred to as design earthquakes, are chosen to represent a moderate and strong earthquake. This is consistent with analysis procedures which employ response spectra. In the specification of dynamic constraints, dead/live load effects on the beams are accomodated in addition to those resulting from the earthquake. No reduction of the live load from that specified for the static operating constraints is introduced. For a moderate earthquake, the structure is to respond elastically, hence, the maximum member moments throughout must be less than each corresponding member yield moment, M_y.

In general, the same form of constraints on maximum beam and column moments carries over to the case of the moderate earthquake, i.e., constraints have the form of Eq. 7 where now M is obtained by combining the separate effects of operating loads and earthquake loading.

The strong earthquake design criterion requires avoidance of structural collapse. We adopt the strong column-weak girder design constraint and utilize the ductility ratio defined as the maximum total end rotation of a member divided by its elastic limit end rotation. In terms of ductility ratio, the strong column-weak girder philosophy means that the ductility demands of each member must be less than some specified allowable, which for columns is close to unity. Let M_T represent the total maximum moment (i.e., the sum of the static and dynamic moments) in a particular member. Then the form of strong earthquake constraints can be written

$$M_T \leq \mu M_p, \qquad (8)$$

where μ is the allowable ductility. As can be seen, this equation is identical in form to Eq. 7 with $c = \mu$. Hence, all of the constraint developments for the moderate earthquake apply to the strong earthquake with c equal to the allowable ductility in each member.

EXAMPLE

To illustrate the methodology, we select a one-story frame with the span and height used to obtain Eq. 6. The intent is to identify the optimal design easily and to illustrate the characteristics of the objective and constraint functions. The beam supports a 40 kip distributed load. Moderate and strong design earthquakes are taken to have 0.12 g and 0.35 g peak ground accelerations (corresponding to 80% and 5% probabilities of occurence, Fig. 3), respectively. In Eq. 7, reduction coefficients c are given values 0.60 for operating loading and 0.85 for dynamic loading associated with the moderate earthquake. A deflection of one inch is permitted at the center of the beam span under operating loads. For the strong design earthquake ductility factors μ of 1 for columns and 6 for beams are assigned. Typical construction cost rates for California are assumed, along with allowances of 10% of construction cost for overhead and profit and 10% of total damage cost for down-time costs. Structural damage is not accounted for, on the basis of earlier computational experience [1].

Using these assumptions, the design space is shown in Fig. 4. Hatched lines denote system constraints with the unhatched side of the curves representing usable designs. Constraint c corresponds to a static (operating load) beam constraint, while a and b are column constraints. For clarity, only constraints which bound the usable design region of the design space are shown. The cost lines given in the figure are computed as follows:

$$\text{Cost} = \frac{100\ [LC(X) - LC(X^*)]}{\text{Construction Cost at Optimal}} \tag{9}$$

where LC(X) is the lifetime cost as a function of the present design vector X and X* is the optimal (minimum LC) design vector.

The following features deserve comment: the optimal design is unconstrained. This would not be expected for multistory frames where strong earthquake column constraints would become active. Another interesting feature is that the objective function has the rough appearance of an uncoupled function. That is, the principal directions in the cost surface (eigenvectors of the Hessian of the cost function) are nearly parallel to the axes. This characteristic grows especially strong as the optimal is approached. Since coupling is greatest between adjacent members in a structure, this uncoupled feature of the objective function should become stronger as multistory frames are considered. Finally, the constraint functions are also nearly uncoupled. That is, each constraint depends essentially upon one variable, lying approximately parallel to one axis or the other. This feature in conjunction with the uncoupled objective funtion leads to the important conclusion that the sizing of the various members can take place nearly independently of one another, i.e., member sizing decisions are uncoupled. This has major ramifications in the selection of an automated design procedure, to be presented in a forthcoming paper.

In conclusion, we note that the frame designed on the basis of least weight (equivalent to minimum initial construction cost) is 25% cheaper in terms of construction cost, but when its lifetime cost is considered, it is actually 23% more expensive than the LC optimal frame. Thus, a clear choice in design philosophy exists.

ACKNOWLEDGMENT

This research was sponsored by the National Science Foundation under grants to the University of California, Berkeley.

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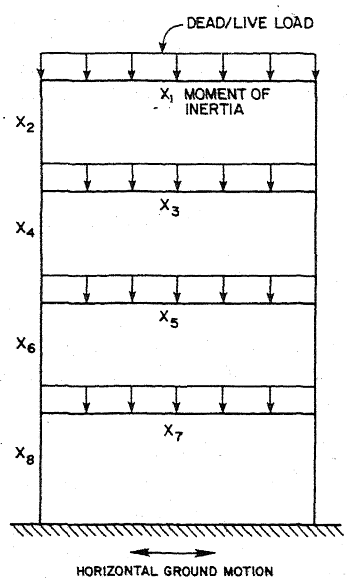

FIG. 1. STRUCTURE WITH LOADS AND MEMBER DESIGN VARIABLE LABELS

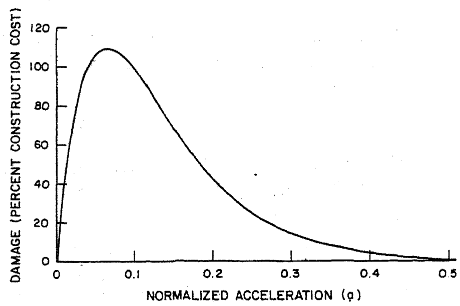

FIG. 2. LIFETIME DAMAGE PROFILE FOR ONE STORY FRAME

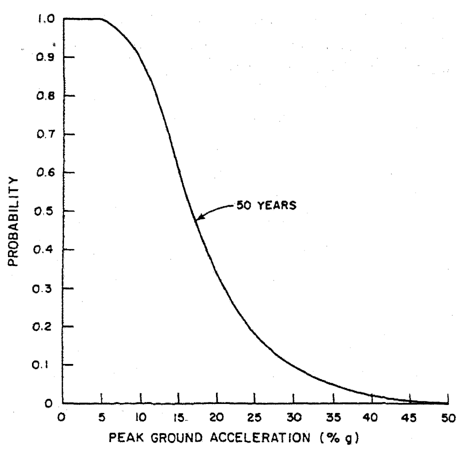

FIG. 3. PROBABILITY OF OCCURRENCE OF PEAK GROUND ACCELERATION

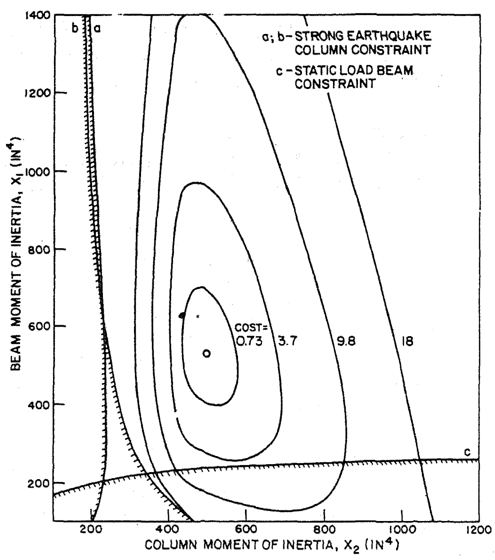

FIG. 4. ONE-STORY FRAME DESIGN SPACE