EARTHQUAKE RESISTANCE OF MULTISTORY BUILDING STRUCTURES DESIGNED FOR WIND

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SUMMARY

This investigation attempts to determine, through inelastic dynamic response history analyses, how much strength and/or inelastic deformability would be required in various structural members if a building proportioned for wind were to be subjected to earthquake input motions of moderate intensities. It is shown that properly designed wind-resistant structures can resist moderate earthquakes, if the beams are allowed to yield within the limit of acceptable damage, and if the columns and walls are strengthened somewhat to remain elastic throughout their seismic response. The additional strength required in the columns and walls is usually available at no extra cost.

INTRODUCTION

In 1972 the National Science Foundation and the National Bureau of Standards initiated a cooperative program in Building Practice for Disaster Mitigation. As a part of this program, in 1978, the Applied Technology Council (ATC), associated with the Structural Engineers Association of California, issued a state-of-the-art document (Ref. 1) entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings." This document is currently being reviewed by the building community under the direction of the Building Seismic Safety Council, in anticipation of its possible use as a resource document for model codes.

If and when the ATC provisions are adopted and incorporated into legal codes, some degree of seismic risk, however small, will have to be considered in the design of a structure located anywhere in the country. This will be a departure from current practice, and may have significant economic repercussions. This investigation was prompted by a concern about these possible repercussions.

By far the largest volume of construction in this country is in areas of relatively low seismicity. Structures in these areas are currently designed to resist wind; earthquake resistance is usually not considered. The purpose of this paper is to show that multistory building structures, properly designed and proportioned to resist wind, do possess a degree of earthquake resistance that is available at no extra cost. The performance of these structures, when subjected to moderate to intense earthquakes is also investigated.

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THE ANALYTICAL INVESTIGATION

A typical 22-story office building is first designed for gravity loads and wind. It is then subjected to inelastic dynamic response history analyses under various intensity levels of a carefully chosen earthquake input motion. In these analyses, the beams are permitted to go into the inelastic range, but the columns and the walls are kept elastic throughout their seismic response to ensure structural stability. The analyses lead to the determination of:

(a) The extent of inelasticity in the yielding members with increasing levels of earthquake intensity, and

(b) The increase of the moments, shears and axial forces in the elastically responding columns and walls with increasing intensities of input motion.

The Building

A specific hypothetical building configuration with a rectangular core and peripheral four-bay frames is considered (Fig. 1a). The slab system consists of one-way joists spanning between the core and the periphery, and two-way waffle slabs of the same depth in the four corner bays. The 8-in. (203 mm) wide ribs of the joists and waffles are spaced 5 ft. (1.524 m) on centers.

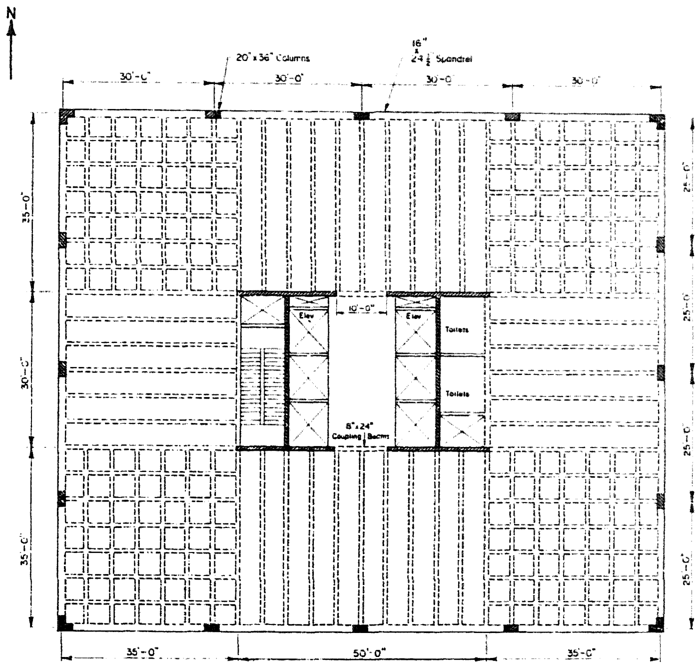
The Lateral Load-Resisting System

In the E-W direction, the direction considered in this paper, the two I-segments of the core walls are coupled by beams at every floor level. The lateral load-resisting system in this direction consists of the wall segments bending about their minor axes in interaction with the four-bay peripheral frames at the north and south faces of the building. The frames are connected with the core walls through the floor slabs.

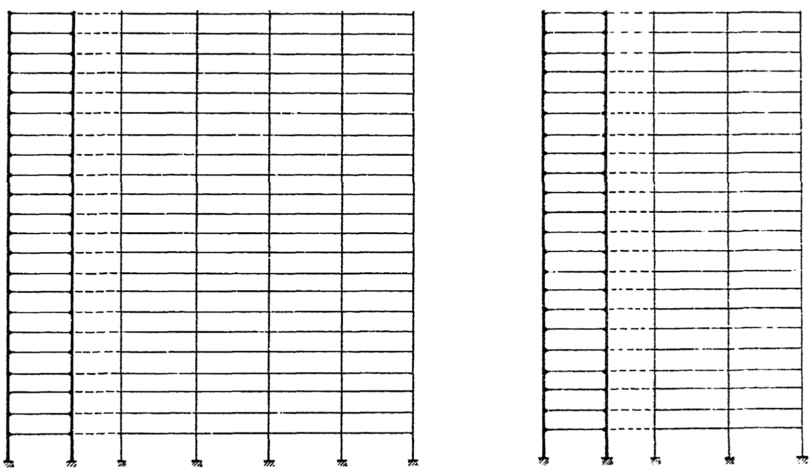
Modeling for Static and Dynamic Analysis

Because of symmetry of the structural system, only half the building on either side of the E-W axis needs to be analyzed. Two structural walls, each as wide as the flange of an I-segment of the central core, and each with half the area and half the moment of inertia (about the minor axis) of an I-segment of the central core, are connected through links representing one of the two coupling beams at every floor level (Fig. 1b). These coupled walls are connected to one of the two four-bay peripheral frames on the north and south faces through flexible links at every floor level. These flexible links simulate the coupling through slabs which transmit little bending.

The model of Fig. 1b is reduced by taking advantage of the symmetry of the frame itself. Two bays of the four-bay frame, with altered member properties for the central column stack, are connected through flexible links with a coupled wall system having half the properties of the central core in the more extended model (Fig. 1c). The static lateral loads on the reduced model should be half the corresponding loads computed for the extended model.



PLAN



ANALYTICAL MODELS

Concrete strength: columns and walls, bottom 4 stories - 6 ksi;
stories 5 - 8 - 5 ksi;
Other columsn, walls, and horizontal members - 4 ksi;
Specified yield strength of steel - 60 ksi;

Fig. 1: Building Studied

The amounts of damping in the two models, and the dynamic excitations on them, are kept the same. The masses at every floor level of the reduced model are half the corresponding masses in the extended model.

Static Analysis for Code Wind Forces

In accordance with the Uniform Building Code, 1979 Edition (Ref. 2)*, 30-psf Map Area wind forces are considered on the building in the E-W direction. These wind forces are typical of the midwestern states which include cities like Chicago, Indianapolis, Des Moines, Omaha, and Kansas City. An elastic static analysis of the lateral load resisting system under these wind forces is carried out. The resulting bending moments in the coupling beams and the spandrel beams are illustrated in Fig. 2. The strengths assigned to the beams for purposes of further analyses vary stepwise along the height of the building in accordance with standard design practice, rather than from floor to floor (Fig. 2).

Dynamic Analysis

Dynamic inelastic response history analysis, by the computer program DRAIN-2D (Ref. 3), is used to determine the internal forces and inelastic deformations caused by earthquakes in the various structural members. DRAIN-2D accounts for inelastic effects by allowing the formation of concentrated "point hinges" at the ends of elements where the moments equal or exceed the specified yield moments. The moment vs. end rotation characteristics of elements are defined in terms of a basic bilinear relationship which develops into a hysteretic loop with unloading and reloading stiffnesses decreasing in loading cycles subsequent to yielding. The modified Takeda model (Ref. 4) is utilized in the program to represent the above characteristics. Viscous damping in the form of a linear combination of mass-proportional and stiffness-proportional components is used. Five percent of critical damping in the fundamental and the second modes is assumed.

Definition of Ductility

The ductility discussed in this paper is based on rotations over the hinging regions of structural members. The hinging region is the length of a member over which the bending moments, while on a loading cycle, exceed the specified yield moments. Rotational ductility is defined as θ_{max}/θ_y where θ_{max} is the maximum rotation, as the hinging region goes through many cycles of response, and θ_y is the rotation corresponding to yielding. Yielding is defined as corresponding to the intersection between the initial elastic and post-yield branches of the moment-rotation diagram of the hinging region.

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*This study was initiated before the publication of the 1982 Edition of the Uniform Building Code. The wind design provisions of UBC-82 are substantially different from those of previous codes. The UBC-82 design wind forces corresponding to a basic wind speed of 80 mph (Chicago, Des Moines, Kansas City, Oklahoma City) and Exposure B (terrain with buildings, forest or surface irregularities) are lower than the UBC-79 30-psf Zone design wind forces considered in this investigation. UBC-82 design wind forces corresponding to a basic wind speed of 90 mph (Houston, Pensacola, Norfolk) and Exposure B would be comparable to the UBC-79 30-psf Zone wind forces.

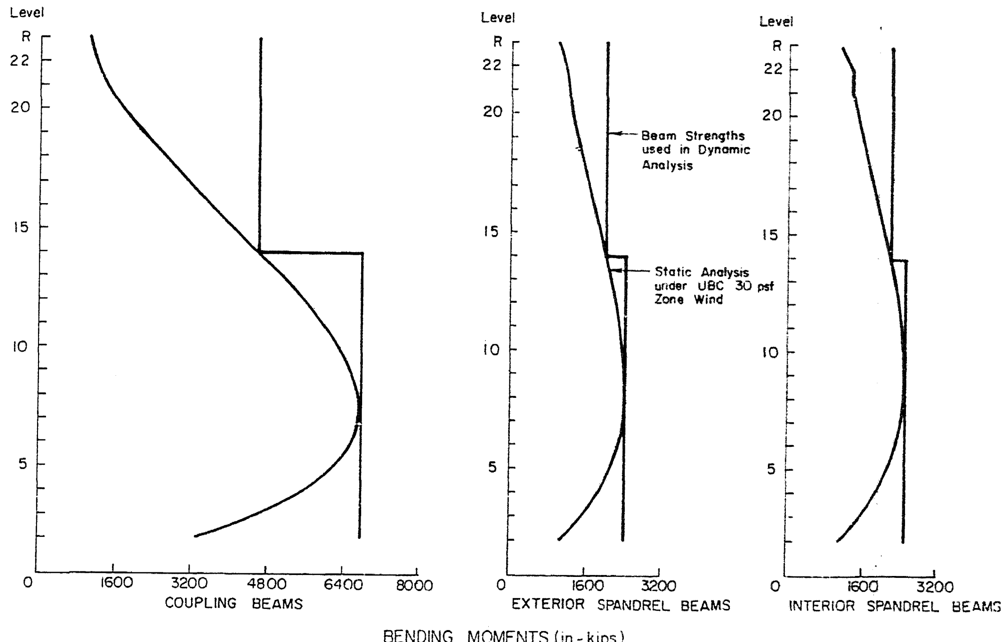

Fig 2: Bending moments caused by factored wind forces in beams, moment capacities assigned to beams

Input Motion

Preliminary analyses using five different input motions indicated that the El Centro, 1940, E-W component is the critical input motion for the E-W direction of the structure.

Inelastic dynamic response history analyses under the chosen input motion, normalized to peak ground accelerations of 0.05g, 0.10g, 0.15g, and 0.20g, are carried out for the structure with indicated strength levels assigned to the beams, and with very high (virtually unlimited) strength levels assigned to the columns and walls. The results yield the elastic strength requirements for the columns and walls, and the inelastic deformability requirements for the beams.

RESULTS OF ANALYSIS

The ductilities required in the coupling beams and the spandrel beams to resist earthquakes with peak ground accelerations of 0.05g to 0.20g are shown in Fig. 3. Hardly any ductility is required to resist a peak ground acceleration of 0.05g. Structural response caused by an earthquake of the above magnitude is virtually elastic. The ductility requirements naturally increase with earthquake intensity. Test results (Ref. 5, 6) indicate that the limit of available ductility may be set between 8 and 10 for slender, properly reinforced beams (span-to-depth ratio >4). Short beams (span-to-depth ratio <2), which must be diagonally reinforced against shear failure,

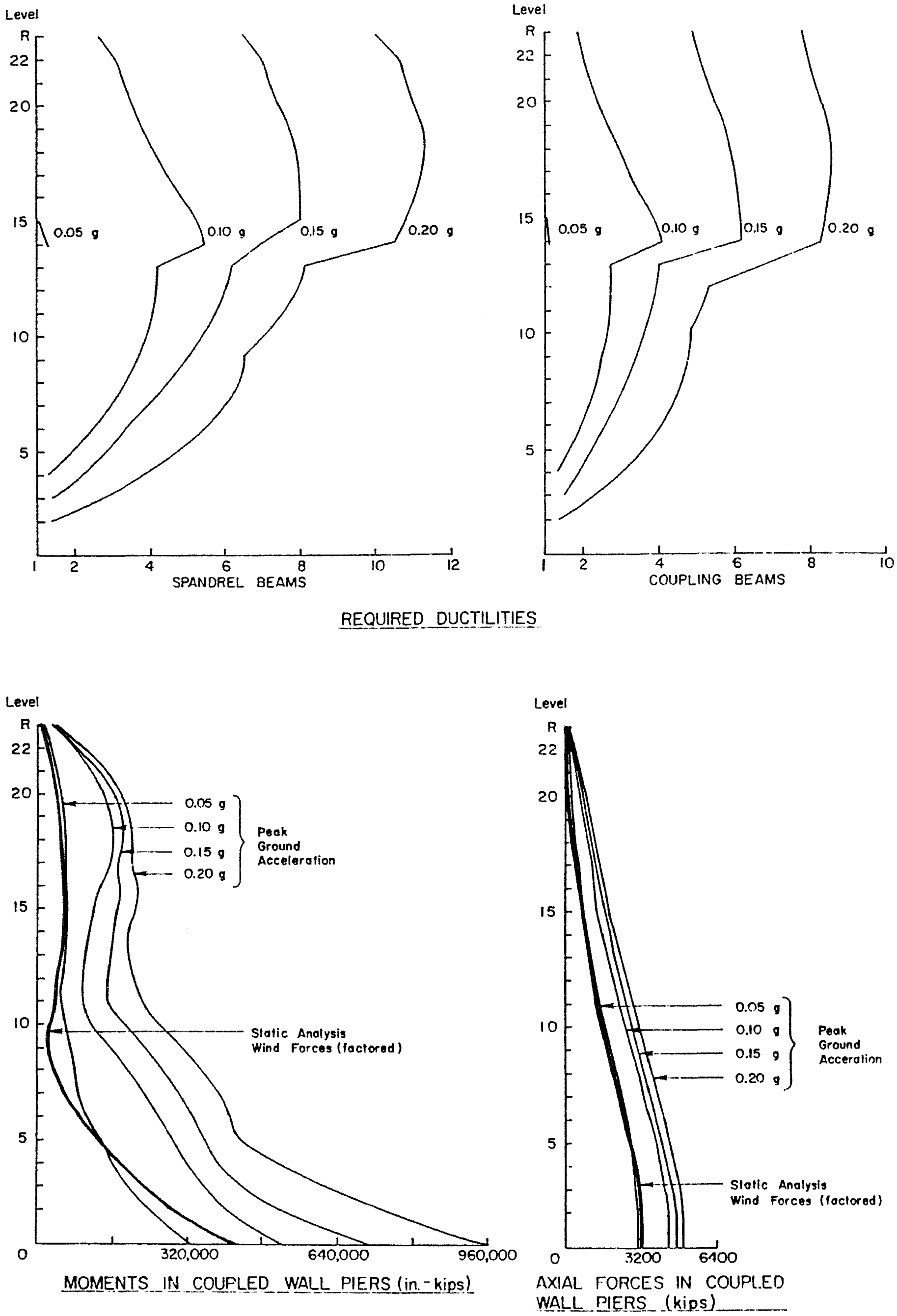

Fig. 3: Envelope values of various response quantities under earthquakes of increasing intensity

may have somewhat higher ductilities because of the special reinforcement (Ref. 7). It is thus possible for the structure under consideration to withstand a peak ground acceleration of 0.10g with little or no modification of the standard reinforcement details. A peak ground acceleration of 0.15g requires ductilities that are at or just beyond the limit of availability. This intensity can be withstood with minor upward adjustments in the beam strength levels which would bring the ductilities back within available limits. A 0.20g peak ground acceleration, however, appears to be beyond the capabilities of the wind-designed structure to withstand.

Figure 3c shows the bending moments in the coupled wall piers caused by the earthquakes of increasing intensity. It is apparent that the 0.05g earthquake does not require any strengthening of the walls beyond what is required by the factored wind forces. Higher earthquake intensities require progressively higher degrees of strengthening. It should be noted that, except in the top few stories, considerable excess moment capacity may be available in the walls and the columns because of minimum reinforcement requirements and because of the presence of compressive axial loads.

Figure 3d shows the axial forces in the coupled wall piers caused by earthquakes with peak ground accelerations from 0.05g up to 0.20g. The 0.05g earthquake causes axial forces that are comparable to those caused by the factored wind loads. Larger earthquake intensities cause higher, but not significantly higher, axial forces.

CONCLUSIONS

Based on this limited study, it appears that buildings designed to elastically resist factored wind forces typical of cities like Chicago (according to the Uniform Building Code, 1979 edition) will also behave with only minimal inelasticity under an earthquake with a peak ground acceleration of 0.05g.

Increasing the earthquake intensity to 0.10g causes the beams (designed for wind) to yield. The inelastic deformation capacities available in beams should be adequate to accommodate the extent of such yielding, provided that proper attention is paid in detailing to: minimum reinforcement requirements, web reinforcement, and anchorage. To prevent yielding of the wind-designed columns and walls, their strengths need to be increased. However, the inherently available reserve moment capacities may be sufficient, except in the upper stories where low axial forces result in limited moment capacities.

The increase in column strength relative to beam strength assures yielding of the beams, rather than of the columns, during seismic response. The columns are thus protected against larger moments and an excessive build-up of axial forces. The shear in the beam-column joints is also limited to the level corresponding to the yield strength of the beam reinforcement. Therefore, the difficult detailing requirements for ductile beam-column joints can be relieved. The columns need to be designed only for elastic behavior.

For acceleration levels of 0.15g, the ductility demands in the beams are at the limit of, or just in excess of, the available ductilities. With minor upward adjustments in beam strengths, such ductility requirements can be lowered and the earthquake intensity level resisted without excessive damage or collapse.

For earthquake intensity levels approaching and exceeding a peak ground acceleration of 0.20g, the structure should be designed for a level of lateral forces substantially greater than that for wind.

It should be pointed out that precise limits on ductilities available in structural members can be established only in correlation with degrees of observed deformations, cracking and damage in laboratory tests of such members. Much work still needs to be done in this area.

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