DESIGN OF SHEAR WALL SYSTEMS FOR NON-ELASTIC BEHAVIOR DURING A MAJOR EARTHQUAKE

by

Theodore Zsutty[I]

SYNOPSIS

This proposed design method presents two new concepts. First, the ultimate design base shear is made a function of a computed measure of the total amount of non-elastic, cyclic, deformation that would occur in the entire wall system during a major earthquake. Second, controlled distributed diagonal cracking is introduced as an acceptable form of non-elastic energy dissipation. This region of shear capacity which exists between first cracking, and the yield of the shear reinforcement, represents the mode of non-elastic participation for short wall elements which are unable to develop flexural hinge action.

INTRODUCTION

Reinforced concrete shear wall buildings have an excellent record of providing damage control and collapse resistance during major earthquakes. When the wall configuration is reasonably stable, and good construction details tie all structural components together, the shear wall building ranks as one of the safest types of earthquake resistant structures. However, in order to remain economically competitive with other, perhaps less reliable materials and structural systems, it is necessary that shear walls be designed for lateral forces representative of non-elastic structural response. These ultimate strength design forces being of the order of one tenth of full elastic response acceleration forces, should be based on the type and amount of non-elastic, cyclic, energy dissipation that a particular wall system is able to provide.

TYPES OF SHEAR WALLS AND THEIR RESPECTIVE BEHAVIOR DURING A MAJOR EARTHQUAKE

Shear wall systems can occur in various configurations of plan and elevation form. They may be constructed with significantly different methods and types of concrete materials: poured-in-place, reinforced concrete with keyed and dowelled joints; precast, wall panels and floor slabs with various methods of poured-in-place joints; "slip-formed", utility towers with pre-cast floor slabs on pre-cast concrete frame systems; and masonry (brick or block) walls with poured-in-place or pre-cast floor slabs, are some common examples.

[I] Professor of Civil Engineering, California State University at San Jose, California.
Chairman: 1971 Subcommittee on Shear Walls, and 1972 Subcommittee on K and C Factors, of SEAONC Seismology Committee.

Present code specified ultimate base shear values such as

$$V_B = UKCW = (1.4)\ (1.33)\ \frac{0.05}{\sqrt[3]{T}}\quad W \cong 0.15W$$

where T = Building Period, and W = Building Weight, assume that an almost identical amount of non-elastic action takes place in a building, irrespective of the type of wall configuration and method of construction. However, if the shear wall system has been designed for these code lateral forces, representative of non-elastic structural response to major earthquake ground motion, a variety of different behavior patterns may occur during the actual quake. Three particularly different cases will be discussed:

(1) Box-type walls with few, dispersed wall openings; and thick-walled utility towers with pre-cast slabs and frames, have very high elastic strength values, and may develop nearly full-elastic response acceleration forces of the order of 100% gravity. These forces are much larger than the code ultimate design forces, and severe distress may occur in slab diaphragm connections to the walls, in wall construction joints, or in the foundation.

(2) Multi-story, spandrel coupled shear walls, with good flexural and shear reinforcing in both the spandrels and vertical piers, are able to develop a substantial amount of non-elastic action in the coupling spandrels without loss of structural stability. This type of structure may have actual lateral forces fairly close to the code design values and will exhibit satisfactory performance.

(3) Tall, single cantilever walls of masonry, or pre-cast construction, having minimum specified reinforcing and connection details; or stiff solid upper-story walls supported by the slender piers or columns of a relatively open first story wall, can develop collapse behavior. Quake acceleration forces build up to the ultimate design capacity of the cantilever wall base, or the first story supporting piers, and subsequent non-elastic deformation reversals can destroy the vertical load capacity of these important stability elements.

The following design procedure is presented to enable the designer to better predict the forces and behavior pattern of the wall system at peak quake demand. A redesign, or a back-up frame system can therefore be provided if a collapse pattern is indicated.

THE PROPOSED DESIGN PROCEDURE

The use of non-elastic base shear values for design implies the existance, during a major quake, of repeated cyclic deformation beyond the elastic limit of the wall elements. While the resulting amount of deformation, and stiffness and strength degradation cannot be predicted with any great degree of precision, we can approximate an order of magnitude for these effects; and most important, we can recognize and incorporate their existance in our design procedure. It is felt that various simplifying assumptions are in order because observed major failures have not been caused by the lack of precise prediction of non-elastic, cyclic deformation effects, but by the complete nonrecognition of their possible existance when present code seismic load levels are used for design.

The following types of information are available for the seismic lateral load analysis of any given shear wall building structure:

(1) The basic structural dimensions and trial section size for any wall pier or spandrel element i.

H_i = Height of pier or length of spandrel i
D_i = Width of pier or depth of spandrel i
t_i = Thickness of element i
I_i = Moment of Inertia of the section of element i.

(2) The trial section strength capacities, either from the vertical load design, or from first estimates of wall thicknesses and reinforcement. This preliminary design information permits the calculation of the non-elastic or participation level for each pier and spandrel section.

a) $Vp_i = 2\sqrt{f'_c}D_i\ t_i$, for short elements with $H_i/D_i < 2$

b) $Mp_i = 7/8\ D_i\ A_s\ F_y$, for slender elements with $H_i/D_i > 2$

(3) The results of a fully-elastic response, dynamic analysis for major quake ground motion.

a) V_{Ei} = resulting shear in a short wall element with $H_i/D_i < 2$.

b) M_{Ei} = resulting moment in slender wall elements.

c) P_{Ei} = resulting axial load in vertical load bearing members.

The dynamic analysis may range from: a statically equivalent "code" method using a base shear of $V_{BE} = U_1$ (10CW), (where U_1 is a load or "uncertainty" factor such as 1.4, and the 10 factor is used to convert the code "C" factor spectrum to an elastic response spectrum.); to a site conditioned, soil response ground excitation on a mathematical model of the structure, and refined stress analysis by computer. A lower load factor U_1 may be employed if the more detailed methods are judged likely to give more accurate results. The values V_{Ei} and M_{Ei} are not the actual section loads in the non-elastic structure, but they provide a measure of the non-elastic cyclic deformation beyond the participation levels V_{Pi} or M_{Pi}. The above information allows the computation of a "non-elastic participation ratio" for each wall element i; $\frac{V_{Ei}}{V_{Pi}}$ for short Piers or Spandrels, and $\frac{M_{Ei}}{M_{Pi}}$ for Slender Piers or Spandrels.

Maximum allowable limits may be specified for these participation ratios: for example, adequately reinforced concrete elements can have $\frac{V_{Ei}}{V_{Pi}} \le 20$, while masonry, or pre-cast wall elements with poor joint details might be restricted to $\frac{V_{Ei}}{V_{Pi}} \le 5$. Values exceeding these limits would indicate destructive amounts of participation, and therefore a redesign would be required. The measures of non-elastic energy dissipation are: $V_{Ei}/V_{Pi}\ H_i D_i t_i$, representing the safe usable amount of controlled distributed shear cracking deformation which exists between the first cracking load and the steel yield shear capacity of a short element i; and $M_{Ei}/M_{Pi}\ I_i H_i$ is energy due to flexural yield deformation in slender wall elements.

Finally, the ultimate design base shear V_{BD}, and the ultimate design section strength requirements V_{Di}, M_{Di}, P_{Di} are found by dividing the fully elastic response values V_{BE}, V_{Ei}, M_{Ei}, P_{Ei} by the "average non-elastic participation ratio".

For all Short Elements   For all Slender Elements

$$\overline{P} = \frac{\sum \frac{V_{Ei}}{V_{Pi}} H_i D_i t_i}{\sum H_i D_i t_i} + \frac{\sum \frac{M_{Ei}}{M_{Pi}} I_i H_i}{\sum I_i H_i} - 1$$

This equation is intended to provide a measure of the relative amount of non-elastic participation in the total structure. Therefore, any V_E/V_P or M_E/M_P less than 1 is to be set equal to 1, such that $\overline{P}$ will have a lower limit of unity. This is to provide full elastic response force values when no non-elastic participation occurs. Upper limits on $\overline{P}$ may be set so as to have the minimum value of base shear V_{BD} equal to the present code design levels for ultimate base shear.

Note that the specification of good reinforcing and connection details which merit higher allowable limits on the individual element participation ratios, can result in a higher $\overline{P}$ values, with the resulting economic advantage of lower design values.

The entire procedure leading to the wall ultimate design values, has purposely been kept simple and in terms of quantities which are readily available to the designer. This is an essential requirement for any improved method of non-elastic design, since only a few designer man-days for seismic analysis are available in any economically competitive engineering budget.