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PROPORTIONING STRUCTURES FOR ADEQUATE DUCTILITY

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ABSTRACT

The seismic design provisions contained in most U.S. building codes are based upon the premise that during a major earthquake, substantial inelastic deformations will limit the demands placed on most structures to acceptable levels. However, while requiring most structures to be designed with ductile lateral load resisting elements, these same codes do not require that structures be designed so that the ductile elements actually yield. This paper discusses the need to proportion structures so that they perform as intended. Specifically, the importance of mobilizing the inelastic deformation capabilities of ductile elements while limiting the forces sustained by non-ductile elements is emphasized using both real and hypothetical examples of poorly proportioned structures.

KEYWORDS

proportions, element ductility, system ductility, weak-link

INTRODUCTION

For many years, the seismic design provisions of most United States (U.S) building codes have been based on the ability to sustain the demands of earthquakes having return periods of approximately 475 years. In areas of moderate to high seismicity, structures are seldom designed to sustain such demands elastically. Instead, structures are designed to withstand the demands of major seismic events via inelastic deformations that, among other things, dissipate energy, mobilize additional strength, and alter dynamic characteristics of the structure.

To provide the necessary levels of ductility, U.S. building codes usually include special detailing requirements for certain elements of lateral load resisting systems. The intent is that specially detailed elements will be capable of sustaining the required inelastic deformations while maintaining a minimum level of load carrying capacity. In typical structures, ductile elements consist of vertical components such as shear walls, moment frames, and braced frames. Other elements of lateral load resisting systems such as diaphragms, foundations, collectors, and most connections are seldom designed to sustain significant inelastic deformations. Therefore, it is essential to proportion structures so that yielding of ductile elements precludes failure of elements with limited ductility. In other words, ductile elements must constitute the "weak-link" in each load path.

Unfortunately, proper system proportioning is not explicitly required by most U.S. building codes. In fact, most U.S. codes tacitly allow, and in some instances promote, the design of systems in which the failure of non-ductile elements would preclude the intended yielding of carefully detailed ductile components.

Through his activities as a peer reviewer, an instructor, and via informal discussions, the author has discovered that many engineers do not realize that inelastic deformation is an integral part of most seismic designs. Even those engineers who appreciate the importance of inelastic deformation sometimes do not understand the significance of certain building code provisions. These circumstances lead to situations where designers of new structures seek only to satisfy the letter of the applicable building code. As a result, many structures have been designed with specially detailed walls and/or frames whose inelastic deformation characteristics will never be mobilized. When excited to the extent anticipated by the applicable building codes, such structures will not perform as intended. Instead, failure of non-ductile elements and/or connections will often result in greater than expected damage and greater than expected hazards to life safety.

GENERAL CONCEPTS

Typical U.S. building code seismic design provisions include the following two steps:

1. Provide a specified level of elastic capacity (typically a small fraction of the associated elastic seismic demand).

2. Follow special detailing requirements for certain elements in order to provide a minimum level of ductility *in those elements.*

As indicated in the wording of step 2, ductility requirements are usually specified in terms of individual elements, not the structure as a whole. Unfortunately, the provisions of most building codes allow ductile elements to be incorporated in ways that prevent mobilization of any ductility.

For example, in the 1991 Uniform Building Code (UBC), the simplified design procedure requires the design (working stress) lateral load capacity of a structure to be greater than the base shear (V) as determined using Formula (34-1). Since this minimum design force is used as a basis for designing both ductile and non-ductile structural elements, proportioning problems may arise when the actual capacities of ductile elements exceed the required minimum. In such instances, non-ductile components such as diaphragms, foundations and many connections that are designed for forces derived from V, may not be strong enough to develop the yield capacities of the ductile elements.

Figure 1 represents a simple load path consisting of the five elements shown. Conformance with typical U.S. seismic design provisions would simply require that this load path have an elastic capacity that exceeds a nominal design load (V) which is typically one-eighth to one-fourth of the corresponding elastic seismic demand, and that the *frame* possess a certain amount of ductility. Figure 2 summarizes characteristics of two load paths (A and B) that are identical except for the strengths of the frames. Both systems satisfy the letter of typical U.S. seismic design codes in that the strengths exceed the specified minimum, and both *frames* are highly ductile. However, differences in proportions result in significant differences in seismic response characteristics. While System A is slightly stronger than System B, System A will fail early during a design level seismic event, the first time the non-ductile diaphragm/frame connection is overloaded. In contrast, System B will remain intact, resisting lateral loads and dissipating energy as intended. The key to this difference in performance is proportions. To ensure optimum performance, ductile elements must constitute the weak links in a given lateral load resisting system.

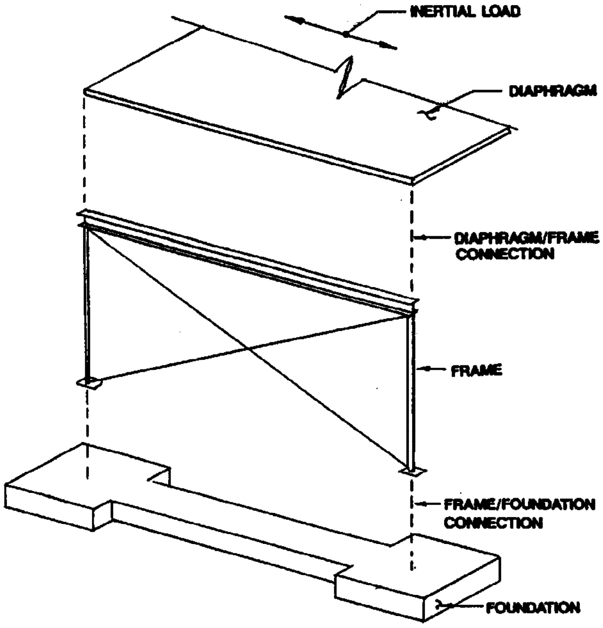


Fig. 1: Simple load path

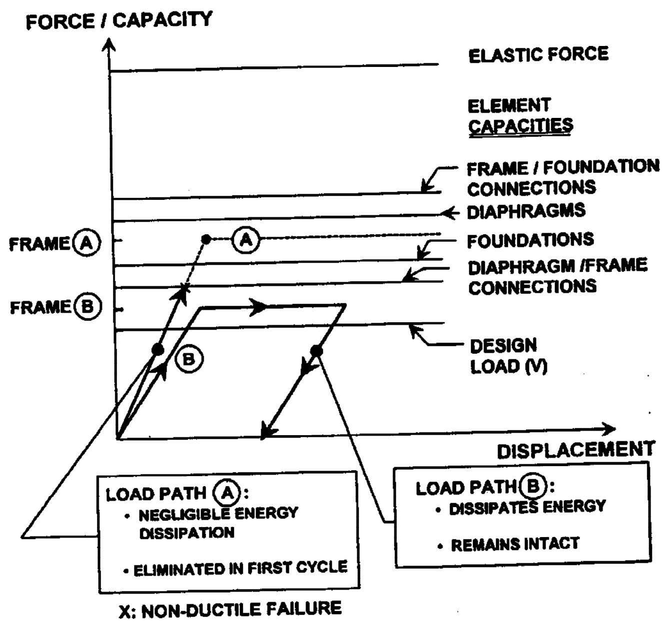


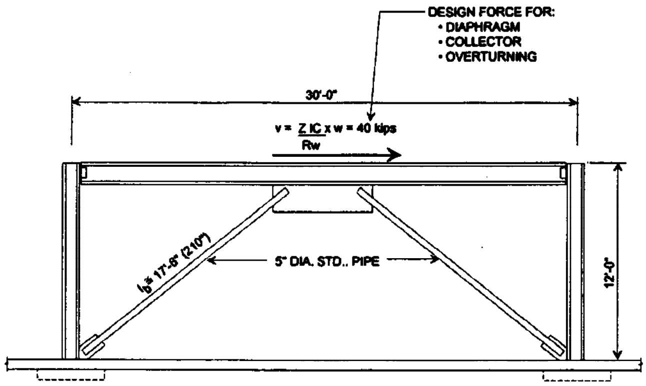
Fig. 2: Proportioning issues

EXAMPLES OF PROPORTIONING PROBLEMS

The braced frame shown in Fig. 3 was designed to satisfy the provisions of the 1991 UBC. As indicated, the design lateral load (V) is 40 kips and the design overturning moment is 480 foot-kips. Where tributary dead load is used to resist overturning, the 1991 Uniform Building Code (UBC) requires only that dead load tributary to each column (reduced by 15 percent) resist the design overturning moment. Similarly, design of the diaphragm and its attachment to the frame can be based on the 40 kip design load, even when the frame is much stronger than the design load would require.

The frame design is summarized in Fig. 3. As indicated, the frame's ultimate capacity of 123 kips exceeds the design load by a factor of 3. Unless the non-ductile portions of the load path are proportioned accordingly, the system will not perform as intended.

As shown in Fig. 3, chevron bracing designed according to the 1991 UBC must have an elastic capacity that is 50 percent greater than the nominal design load. However, similar increases in the strengths of non-ductile elements are not required. In this way, the 1991 UBC actually increases the potential for proportioning problems to impair the effectiveness of lateral load resisting systems that use chevron bracing.



PIPE PROPERTIES:

l/r = 112	≤ 120: AS REQ'D BY UBC
Fa = 11.3 ksi x 1 1/3 = 15.1 ksi	
ß = .70	= UBC EQN (10-5)
ß Fa A pipe = 45 kips	= ALLOWABLE LOAD
Fcr A pipe = 80 kips	= ULTIMATE CAPACITY

FRAME CAPACITIES:

Allowable = 70 kips > 1.5 x40 = 60 kips : Satifies code requirement for chevron bracing

Ultimate = 123 kips

Ultimate Capacity / Design Force = 3:1

Fig. 3: Design load vs. ultimate strength

Manufacturing and warehouse structures include some of the most extreme examples of structural proportioning problems. The exterior walls of such buildings often consist of reinforced concrete tilt-up or precast concrete wall panels with very few openings. Shear wall capacities that exceed code specified minimums by factors of 5 to 10 are not uncommon. In other words, shear wall capacities often exceed the elastic demand of the design earthquake. While this means a structure will experience lateral accelerations that are much higher than typical code-specified minimum design values, most U.S. building codes permit the design of non-ductile components to be based upon the smaller and, in this case, entirely unrealistic minimum design accelerations. As a result, many low-rise, commercial structures will experience failures in critical areas such as diaphragms, roof-to-wall connections, or foundations, even during moderate earthquakes.

Even structures utilizing ductile moment frames are prone to proportioning problems. Limitations on interstory drift often govern the design of moment frames. In addition, the ultimate strength of most moment frames is much greater than the load sustained at first yield (i.e., the design strength). As a result, moment frames usually exhibit ultimate capacities that are much greater than code-required minimum design forces.

POOR PROPORTIONING CASE STUDIES

A few examples of proportioning problems that have been encountered by the author are summarized below. In each case, the lateral load resisting system was proportioned so that the intended response (yielding of ductile elements) would be precluded by the failure of another element in the load path.

Overturning: Peer Review - An eccentrically braced frame (EBF), similar to that shown in Fig. 4, was encountered in a design review. As indicated, the yield capacity of the frame was 414 kips and the dead load tributary to each column was 172 kips. Although the dead load was sufficient to resist the design loading overturning moment, comparison of the overturning moment associated with yielding of the link beam to the overturning stability provided by the dead load, showed that the frame would rock rather than yield in the link beam.

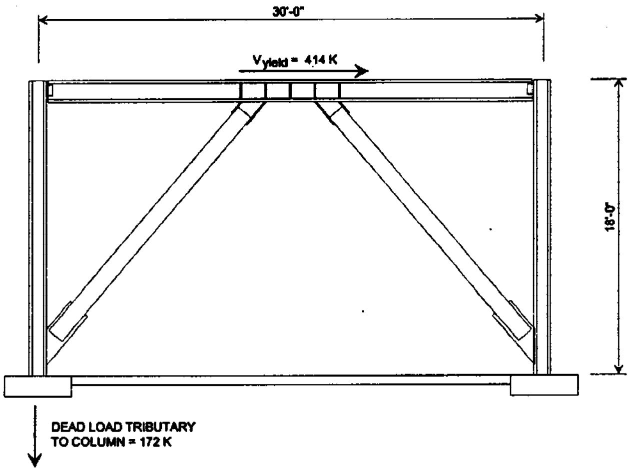


Fig. 4: A link beam that will not yield

Overturning: Northridge Earthquake

Two reinforced concrete shear walls in a three-story building sustained only minor cracking and essentially no inelastic deformation while the mat foundations supporting the walls rotated. Failure of the foundations to resist actual overturning forces severely reduced the effectiveness of these walls. As a result, additional damage was sustained by other portions of the building that were forced to resist demands that should have been carried by these walls. Examples included other shear walls (supported by more substantial foundations), non-ductile columns, and architectural elements.

Connection: Northridge Earthquake

In a building in Simi Valley, the connection of the roof diaphragm to a moment frame collector consisted of a wood nailer that was bolted to the collector element. During the Northridge Earthquake, the nailer bolts failed before the elastic capacity of the moment frame was reached. Thus the frame was effectively prevented from resisting additional seismic demands.

Diaphragm: Northridge Earthquake

The metal deck roof diaphragm of a one-story manufacturing facility failed while the adjacent shear wall remained undamaged. Although the diaphragm satisfied 1991 UBC seismic design provisions, it could not perform its intended function of delivering load to the shear wall throughout the duration of the earthquake.

Multiple Problems: Northridge Earthquake

At the location shown in Figure 5, a diaphragm failed near its attachment to a shear wall, effectively removing the wall from the lateral load resisting system. An example of inadequate foundation capacity is shown in Figure 6. Rotation of the wall not only prevented the wall from developing its full capacity (the wall was virtually uncracked), it also caused considerable damage to the girder above. A diaphragm-to-wall connection is shown in Figure 7. Failure of this connection prevented the wall from further participation in the lateral load resisting system.

Fig. 5: Diaphragm failure limited effectiveness of shear wall

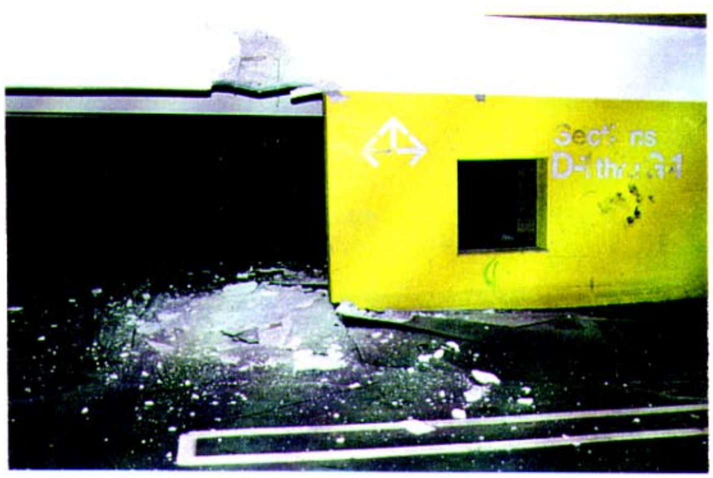


Fig. 6: Foundation failure limited effectiveness of shear wall

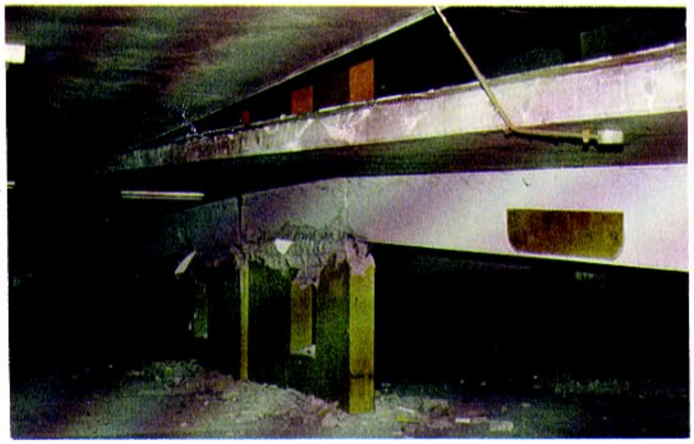


Fig. 7: Failure of diaphragm-to-wall connection limited effectiveness of shear wall

This structure was clearly not proportioned to develop the capacities of its primary, ductile lateral load resisting elements. As a result, a diaphragm, several diaphragm-to-wall connections, and foundations failed when the structure's elastic capacity was exceeded. The associated displacements and redistribution of forces caused considerable damage to elements that were not intended to participate significantly in the resistance of lateral loads. Figure 8 shows some columns that were severely damaged as a result of the proportioning problems discussed above.

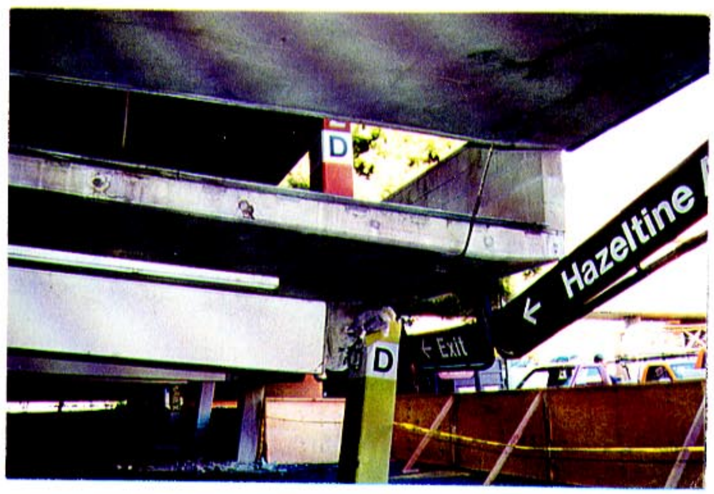


Fig. 8: Column damage related to proportioning deficiencies

SUMMARY AND CONCLUSIONS

In areas of moderate to high seismicity, typical structures are not designed to sustain the demands of major earthquakes elastically. Therefore, the elastic capacity of one or more components of such structures will be exceeded during large or even moderate earthquakes. When elastic capacity is a small fraction of elastic seismic demand, successful performance requires "overloaded" elements of the lateral load resisting system to have considerable ductility. While most U.S. building codes require certain lateral load resisting elements to be highly ductile, these same codes do not explicitly require systems to be designed so that the ductile elements are the ones that are actually "overloaded". In other words, *element* ductility is required while *system* ductility is not.

It is always beneficial for engineers to understand the intent of building code provisions. It is especially important where such provisions are related to life-safety issues. Unfortunately, many structures that conform to the letter of specific seismic design provisions are not capable of performing as those provisions intended. A combination of insufficient training and building code loopholes (such as no explicit proportioning requirements) has made this possible.

Over the last 25 years, the United States has made great progress in mitigating seismic hazards. As long as interest in earthquake engineering continues, the engineering community's ability to provide capable seismic resistant structures will improve. Both long term measures (such as education and training) and short term measures (such as building code improvements) should be actively promoted.