Chapter 16
Supplemental Vertical Support as a Means for Seismic Retrofit of Buildings

Craig D. Comartin

Abstract A large number of concrete buildings in seismically active areas through-out the world exhibit a common deficiency. Weak columns, in one or more stories, lose vertical load-carrying capacity as a result of lateral distortion. This chapter presents a conceptual strategy for retrofit comprising the installation of supplemental vertical supports to prevent collapse. This procedure utilizes a risk-based perspective based on recent research on the realistic capacity of concrete columns, dynamic instability, and the effects of in-cycle degradation of strength in building systems and components. An example building is used to illustrate the application of the concept.

16.1 Introduction and Background

Buildings subject to earthquake shaking have a potential to reach a point of dynamic instability at which they collapse due to theoretically unlimited lateral displacement. Many buildings, however, lose the ability to support vertical loads and collapse at smaller levels of shaking intensity than that which would otherwise cause lateral dynamic instability (see Fig. 16.1). The procedures proposed here are intended to identify buildings prone to preemptive vertical load collapse and improve their safety by the installation of supplemental vertical supports. Maintaining the capability to support vertical loads changes the critical collapse mechanism to lateral dynamic stability which occurs at larger and less probable lateral displacements (see Fig. 16.2).

Experience in past earthquakes around the world indicates that concrete frames infilled with unreinforced masonry (URM) have been particularly prone to collapse [1]. This most often is due to a weak first story caused by the omission of all or a substantial portion of the infill to allow for retail, parking, or other uses conducive to

C.D. Comartin (✉)
CDComartin, Inc., Stockton, CA, USA
email: ccomartin@comartin.net

Fig. 16.1 Story collapse
vertically downward without
significant lateral movement
(note vertical alignment with
adjacent uncollapsed wing)
(See also Plate 19 in Color
Plate Section on page 467)

Fig. 16.2 Side sway
mechanism with incipient
collapse (See also Plate 20 in
Color Plate Section on
page 467)

open spaces. The infill in upper stories restrains frame action and forces the flexible
lower floor to absorb most of the energy demand and drift.

There are several alternatives for retrofit strategy that have been implemented in
the past:

1. The infill on upper floors could be removed and replaced with less stiff and less
   strong materials.
2. The infill on upper floors could be isolated from the structure by installing joints
   with gaps to prevent interaction with the frame.

3. The lower floor, or floors, could be strengthened with new structural walls. These measures (Items 1, 2, and 3) are effective in reducing the ductility demand in the weak story or stories. However, all of them are costly and intrusive. In the US, it is not unusual for the costs of these types of retrofit to exceed half of the replacement value of the building.
4. Wrap the columns in the weak story with jackets of steel, concrete, fiberglass, or carbon fiber.

The fourth option can increase both the strength and ductility of the columns enough to reduce the collapse risk for most buildings. The cost is somewhat less than for the first three alternatives; however the overall performance would also be less with more damage focused in the weak floor. Column jacketing is popular in the US, but the costs can still be high. In developing countries, the advanced technologies for some types of jackets may not be readily available.

The procedure proposed here incorporates another alternative that has been used in the US to reduce collapse risk. This strategy is to provide supplemental vertical supports designed to prevent preemptive vertical load collapse (see Fig. 16.3). These are intended to support loads that are transferred from shear critical columns as they are damaged and begin to fail. The supplemental supports are typically steel shapes, pipe shoring, or timber shores. They are often installed near individual columns, but

Fig. 16.3 Steel column "pre-shoring" examples from Stanford University and University of California, Berkeley (See also Plate 21 in Color Plate Section on page 468)

also can be placed beneath capable horizontal framing. This technique is effective in reducing collapse risk by avoiding the preemptive vertical collapse mode. The intensity of shaking required for lateral dynamic instability is generally higher and less likely to occur. The installation of supplemental vertical supports is relatively inexpensive. The functional use of the spaces may be affected by the installation, but often this is minor compared to other alternatives. Installations in the US have been made for a very small percentage of the replacement costs for the building.

The following Section 16.2 of this chapter presents the conceptual procedure for determining the benefits of supplemental vertical supports based on recent research. The proposed procedure is illustrated with an example application in Section 16.3. The chapter concludes with a summary Section 16.4 and list of references cited in Section 16.5.

16.2 Conceptual Procedure and Background

Assessment of the potential for vertical load collapse in buildings with weak stories and the effectiveness of supplemental vertical supports can accomplished with the steps outline in this section.

16.2.1 Horizontal and Vertical Load Characteristics of Weak Story Components

Engineers in seismically active areas of the US are using FEMA 356 [2] increasingly for the evaluation and retrofit of existing buildings. In 2006 this document was adapted as a national standard ASCE 41 [3]. Over the past decade important research relating to the inelastic behavior of reinforced concrete has been produced at the Pacific Earthquake Engineering Research Center (PEER). The Earthquake Engineering Research Institute (EERI) in conjunction with PEER presented a seminar summarizing the practical implications of this work for engineers [4]. These implications included the prospect of significant improvement of acceptability criteria for several types of concrete components in FEMA 356 and ASCE 41. Consequently, a supplement to ASCE 41 was proposed and accepted [5]. FEMA 356/ASCE 41 provisions are suitable to determine the post elastic horizontal and vertical behavior characteristics of the weak story components as long as they are properly applied in accordance with the recent supplement.

Column components are characterized by the force and displacement relationship depicted in Fig. 16.4. In the figure, the parameter "a" is the plastic rotation at which horizontal force resistance capacity diminishes significantly to a level defined by parameter "c." The column fails for vertical loads at a plastic rotation represented by parameter "b." In FEMA 356/ASCE 41 and Supplement, the assignment of these parameters depends first upon the mode of inelastic behavior for the column

**Fig. 16.4** Force and displacement relationship for concrete columns from FEMA 356/ASCE 41
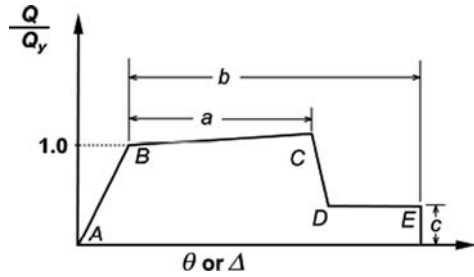
(i.e. flexure, flexure-shear, and shear). Columns are classified in accordance with Table 16.1 depending on the detail of transverse reinforcing. For each "condition" parametric limits are tabulated based on ultimate shear stress, transverse reinforcement ratio, and axial load intensity. The Supplement provides improved parametric limits as illustrated in Fig. 16.5 for flexurally control columns classified as "condition ii."

Recent research [6] demonstrates that the force-displacement relationship for degrading components and systems is a capacity surface which cannot be transgressed during analysis or experiment. It is different than the hysteretic envelope (often termed a "backbone") resulting from a dynamic analysis or experiment. The hysteretic envelope is dependent on loading protocol, whereas the force-displacement capacity boundary is not. This distinction is critically important when determining the inelastic properties of components. The mistaken use of a hysteretic envelope in place of a capacity boundary can result in under-prediction of vertical load capacity and dynamic instability.

Table 16.1 Classification of columns by mode of inelastic behavior and transverse reinforcing details

Inelastic behavior mode	Transverse reinforcement details		
	ACI conforming details with 135° hooks	Closed hoops with 90° hooks	Other (including lap-spliced transverse reinforcement)
Flexure $V_p/(V_n/k) \leq 0.6$	i[a]	ii	ii
Flexure-Shear $1.0 \geq V_p/(V_n/k) > 0.6$	ii	ii	iii
Shear $V_p/(V_n/k) > 1.0$	iii	iii	iii

[a] To qualify for condition i, columns must have p" ≥ 0.002 and s/d ≤ 0.5 within the flexural plastic hinge region. Otherwise column shall be considered as condition ii.
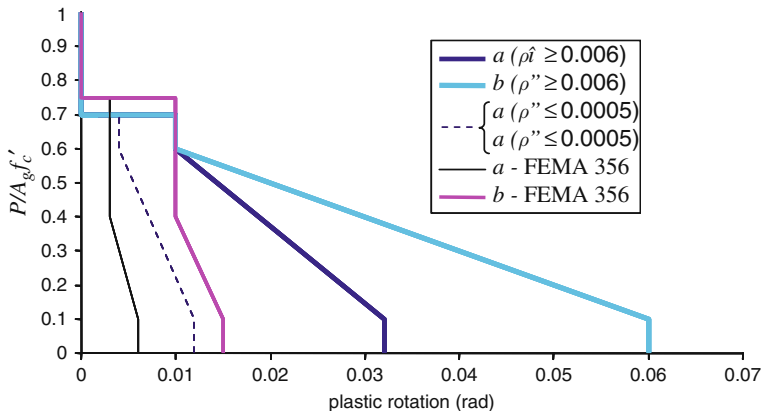
Fig. 16.5 Comparison of modeling parameters for Condition ii and previous *FEMA 356* parameters for columns "controlled by flexure" with nonconforming transverse reinforcement and $v \leq (f_c')^{1/3}$ (See also Plate 22 in Color Plate Section on page 468)

16.2.2 System Capacity Boundary (Pushover)

Figure 16.6 depicts a conceptual analytic model that can be used to determine the probability of vertical load collapse and lateral dynamic instability. In this case, the first-floor is the weakest of the floors and the inelastic response of the structure is assumed to occur at that level. The upper floors are rigid compared with the columns of the first-floor. Consequently the model represents the upper floors as completely rigid. As for most relatively rigid buildings, the flexibility of the foundation is an important factor in its dynamic behavior. The model assumes that the foundation and supporting soils deform elastically imparting a rigid body rotation to the response of the structural model. The rotational stiffness of the foundation should be adjusted to account for the fact that much of the inertial lateral load is applied above the 1st level resulting in larger overturning moments. The response of this model to ground motions represents an approximation of actual response based on a single-degree-of-freedom (first floor drift, θ_{1st}) oscillator. When considering the distortion imparted to the first floor, it is important to account for the rigid body foundation rotation as shown in Fig. 16.6b.

The component capacity boundaries for the weak story, generated as discussed in the previous section, can be assembled using nonlinear static procedures (pushover) to generate a global representation of the force-displacement capacity boundary of the system (see Fig. 16.7).

16.2.3 Simplified Dynamic Analysis

The next step in the process is to determine the intensities of shaking at which vertical load collapse and lateral dynamic instability occur. An incremental dynamic analysis (IDA) is a type of response history analysis which involves subjecting

Fig. 16.6 Analytical model

(a) Assumptions

(b) Rotational displacements
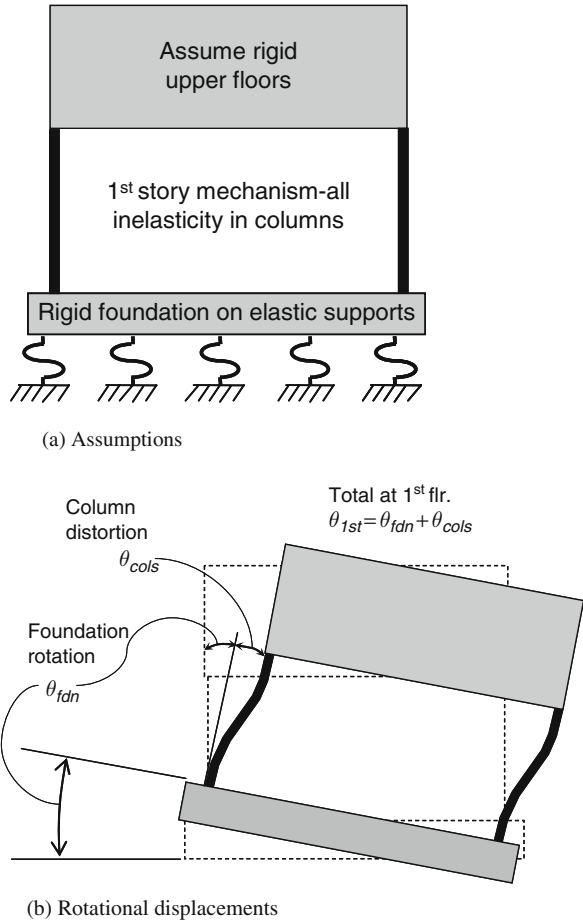
a system to a ground motion record successively scaled to increasing levels of intensity, as measured by an intensity measure IM (e.g. spectral acceleration at the fundamental period of the system, S_{aT}), until global dynamic instability is observed [7].

Dynamic instability appears as a rapid, nearly infinite increase in the response of the system, measured by the engineering demand parameter (e.g. rotation at the weak floor, θ_{1st}), for a small increment in intensity. Essentially this means that the model cannot withstand any more increase in the applied intensity and thus responds with infinitely large displacements, indicative of collapse due to dynamic instability. The procedure is normally done with a number of ground motions to gain probabilistic perspective of the potential for collapse as illustrated in Fig. 16.8. In the figure, the force parameter is the spectral acceleration at the period of the SDOF system, normalized by the spectral acceleration at the yield point of the system, $R = S_{aT}/S_{ay}$. The displacement is represented by the ductility demand, $\theta_{1st}/\theta_{yield}$.
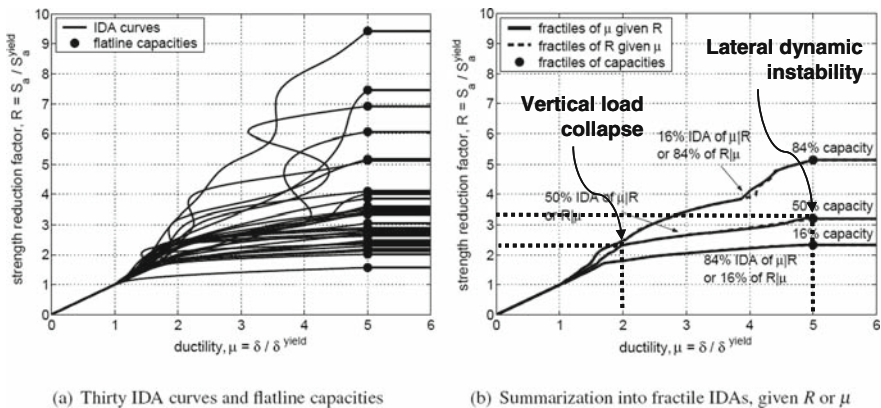
Fig. 16.7 Force displacement capacity boundary from nonlinear static pushover analysis
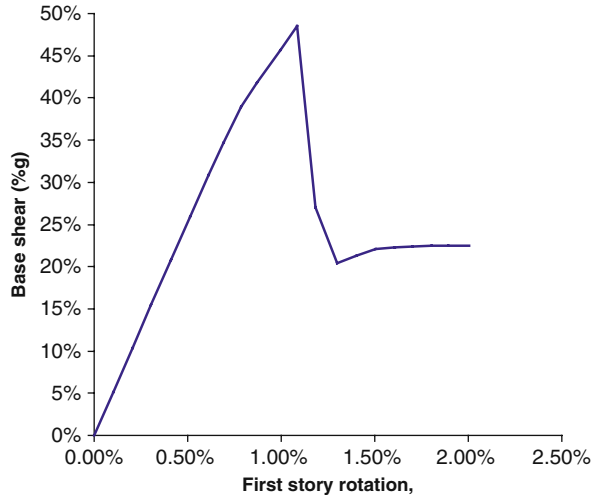
In Fig. 16.8b, lateral dynamic stability is defined by the intensity and ductility at which the IDA curves reach horizontal. The capacity for vertical load is determined by plotting the ductility at which collapse is expected and finding the associated intensity by intersecting the IDA curves.

16.2.4 Collapse Mode Risks

The information from the simplified dynamic analysis defines the probability of collapse due to each mode conditioned on intensity as shown in Fig. 16.9. The combined probability of collapse due to either mode is expressed by Eq. 16.1 [8]. The combined risk of collapse is expressed in Eq. 16.2.

(a) Thirty IDA curves and flatline capacities

(b) Summarization into fractile IDAs, given R or μ

Fig. 16.8 Incremental dynamic analysis

Fig. 16.9 The combined probability of collapse, $P(C_{comb.})$, due to vertical load collapse, $P(C_{VLC.})$, and lateral dynamic instability, $P(C_{LDI.})$
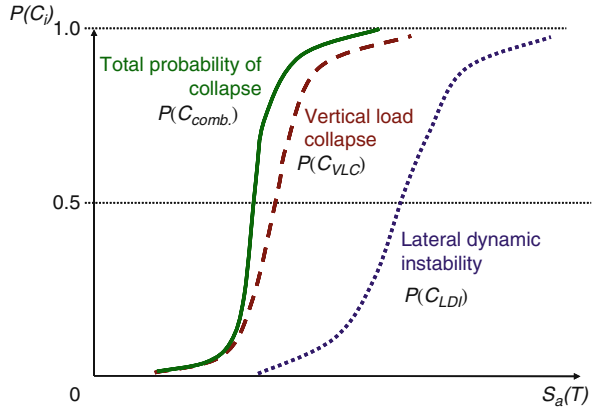
$$P\left(C_{comb.}\right) = P\left(C_{VLC}\right) + P\left(C_{LDI}\right) - P\left(C_{VLC}\right)P\left(C_{LDI}\right) \qquad (16.1)$$

$$\lambda_c = \int_{S_a} P(C_{comb.}) \left| d\lambda_{S_a} \right| = \lambda_{S_a}(S_{aT}) \exp\left(\frac{1}{2}k^2\beta^2\right) \qquad (16.2)$$

The benefit of supplemental vertical supports is measured as the reduction in the risk of collapse that they would provide. To quantify the benefit, the following steps are taken:

1. Assess the probability of each mode of collapse using the procedure outlined in the previous section.
2. Determine the combined risk of prior to the installation of supplemental supports using Eqs. 16.1 and 16.2.
3. Reassess the probability of vertical load collapse after the installation of the supplemental supports.
4. Determine the combined risk after installation using Eqs. 16.1 and 16.2.

In many cases, these procedures can be simplified by assuming that the supplemental supports eliminate the probability of vertical load collapse. Also, a seismic hazard curve can be used to estimate the integration in Eq. 16.2. This is illustrated in the example which follows.

16.3 Example Application

This section demonstrates the application of the procedure to quantify the benefits of supplemental vertical supports to a typical five story concrete frame residential building with unreinforced masonry in-fill partitions in the upper stories and a

Fig. 16.10 Example five story concrete frame building with unreinforced masonry infill
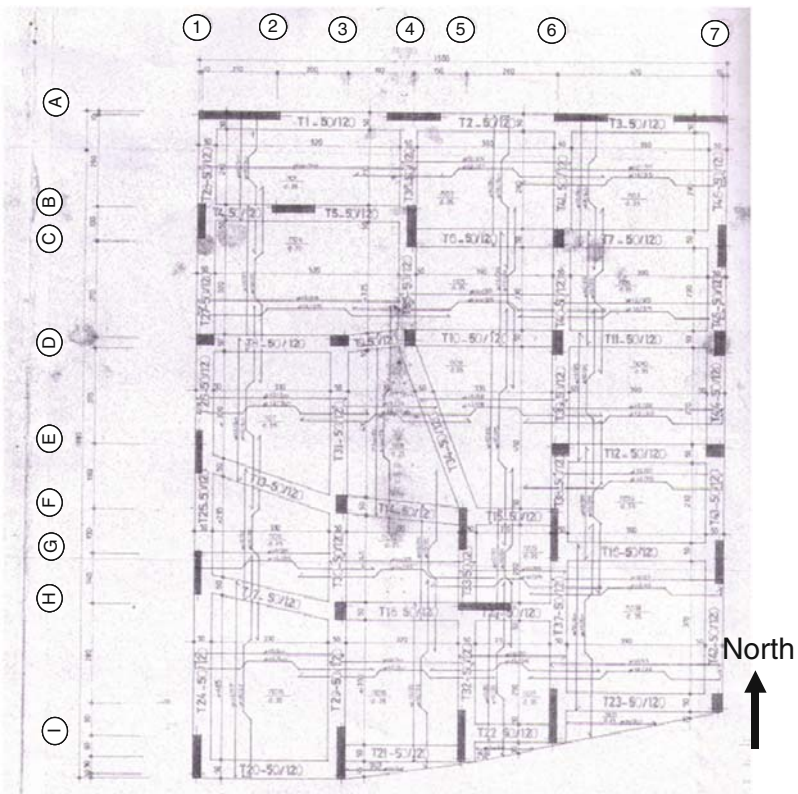
Fig. 16.11 Example building plan

weak open first-floor (see Fig. 16.10). This building is quite typical of construction in Turkey for multi-unit residential construction. Building and many others like it were subject to extensive damage and collapse during the Kocaeli earthquake in 1999. This building was investigated by a team from the Applied Technology Council including the author. The investigation team obtained design drawings from the owner. Although it did not collapse, the building was severely damaged. The most severe damage was found in the columns of the lower floor which had few unreinforced masonry partitions compared to the upper floors.

The columns in the lower floor of the example building are shown in Fig. 16.11. Transverse reinforcing is very light resulting in many shear and flexure-shear critical column components. Component capacity boundaries were generated using the FEMA 356/ASCE 41 procedures outlined in Section 16.2.1. The global capacity boundary for SDOF model of the building generated by a nonlinear static pushover is shown in Fig. 16.12. The data in the FEMA 356/ASCE 41 Supplement lead to a vertical load collapse limit controlled by horizontal distortion within the first floor. This limit translates to a total 1st story rotation including foundation flexibility of approximately 1%. The Supplement limits were developed to produce reliability against collapse of 85%. In this example, the vertical load collapse rotation was increased to 1.2% as an estimate of the median value.
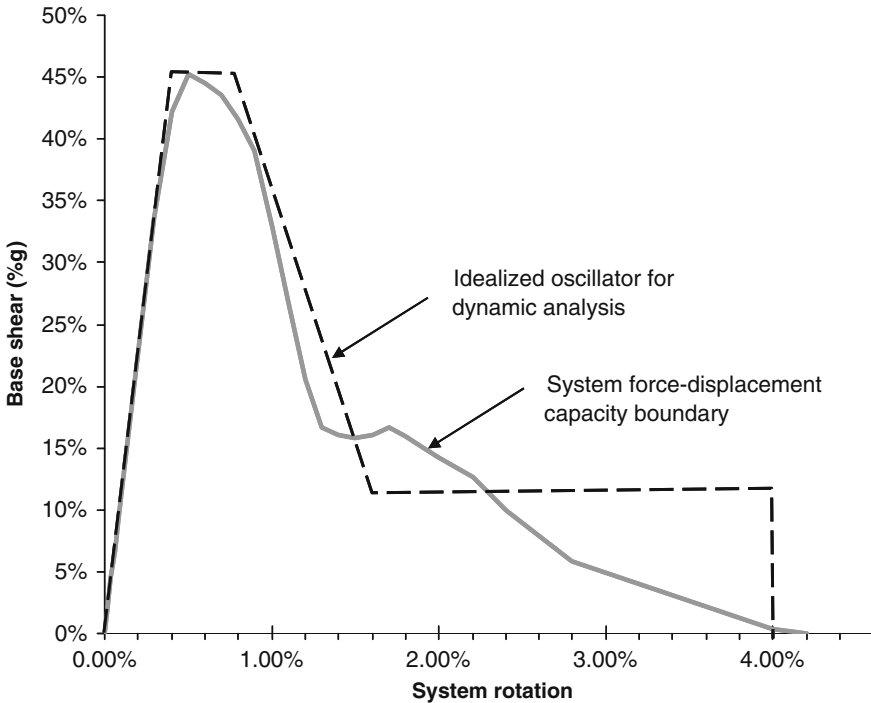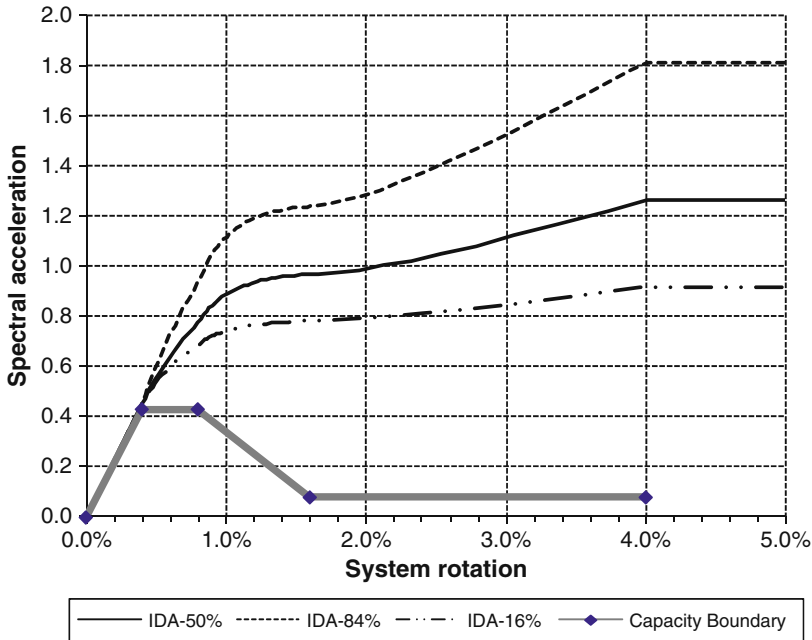

**Fig. 16.12** Capacity boundary for example building model

Fig. 16.13 Capacity boundary and incremental dynamic analysis results for example building model for a long residual strength plateau

The capacity boundary was used to represent a SDOF model of the building and subjected to an incremental dynamic analysis using a version of the software tool SPO2IDA [9]. The results are illustrated in Fig. 16.13. The capacity boundary is also shown in the figure.

As mentioned in the previous section the median values of each collapse mode demand can be used to estimate the associated risks. The intensities (R axis in Fig. 16.13) are converted to spectral acceleration at the period of the model, S_{aT}, by multiplying by the spectral acceleration at yield, S_{aYield}. These then are plotted on a seismic hazard curve for the site as shown in Fig. 16.14. This comparison indicates that supplemental vertical supports reduce the median risk of collapse in this example by a factor of about three.

This conclusion must be viewed cautiously as it is dependent upon some critical assumptions. The conclusions from ATC 62 include the observation that post-elastic characteristics of the capacity boundary control lateral dynamic instability. These parameters are:

1. the rate of degradation, $\alpha K_{initial}$,
2. the magnitude of the residual strength plateau,
3. the length of the plateau defining the point at which there is no lateral resistance, *max.* θ.
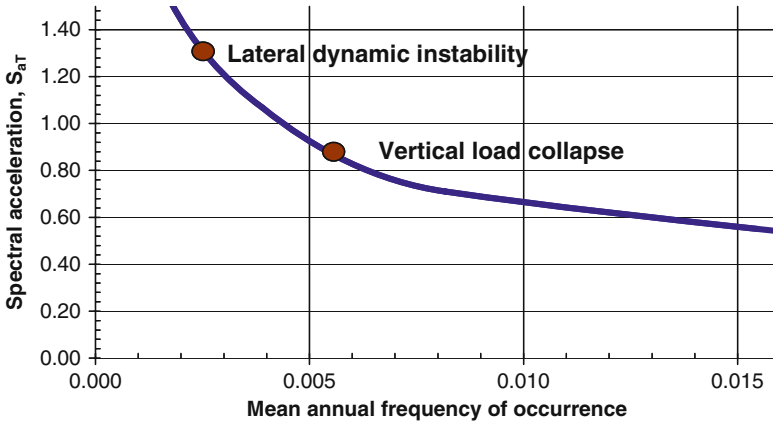
Fig. 16.14   Seismic hazard and annual frequency of vertical load collapse and dynamic instability

The rate of degradation is reasonably estimated from the capacity boundary. The magnitude and length of the residual strength plateau are relatively elusive. Buildings similar to the example retain some horizontal resistance at very high drifts even when discounted for P-delta effects. Supplemental vertical supports themselves do not contribute significantly to raising or extending the lower strength plateau. However, they might be augmented with beams or braces to do so. At present, there is no definitive process for estimating these parameters.

16.4 Summary and Conclusions

The use of supplemental vertical supports in building prone to loss of vertical load capacity caused by seismic distortion appears to an effective and economical retrofit strategy. They virtually eliminate preemptive vertical load collapse and reduce the risk of collapse to that associated with sidesway collapse due to lateral dynamic instability. The magnitude of risk reduction, and therefore the effectiveness of the retrofit, is dependent upon the risk of lateral dynamic instability. Improved information on the controlling post-elastic characteristics (i.e. rate of degradation, magnitude of residual strength plateau, length of residual strength plateau) are essential to more accurate assessment of lateral dynamic instability.

The use of supplemental vertical supports has the prospect of dramatically improving the safety of thousands of buildings in many areas of the world (e.g. Turkey, Central Asia, India, and Pakistan). Beyond the conceptual technical procedure presented here, realization of this goal requires the following:

1. The seismic hazard in regions where such buildings are prevalent needs to determined.
2. Prototype building models representing local and regional design and construction practice should be developed.

3. Parametric studies of the prototypes can be used to identify generalized behavior characteristics and to assess the cost effectiveness of supplemental vertical supports.
4. Simplified design guidelines addressing local and regional conditions can developed to facilitate implementation.
5. Coordinated local strategies for funding retrofit should be developed.

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