

# Dynamic and Nonlinear Load Increase Factors for Collapse Design and Analysis

Aldo McKay, Protection Engineering Consultants  
830/438-7772, amckay@protection-consultants.com

Kirk Marchand, Protection Engineering Consultants  
830/438-7772, kmarchand@protection-consultants.com

Eric Williamson, University of Texas, Austin  
512/475-6175, ewilliamson@mail.utexas.edu

Brian Crowder, NAVFAC Atlantic  
757/322-4228, brian.crowder@navy.mil

David Stevens, Protection Engineering Consultants  
830/438-7772, dstevens@protection-consultants.com

## Abstract

The US Department of Defense (DoD) and the General Services Administration (GSA) require that multistory buildings be designed to resist progressive collapse. Both organizations have developed their own design requirements and criteria, which are currently being merged to provide a unified federal approach to progressive collapse design. The merged requirements will incorporate both direct and indirect design procedures. The main direct design procedure will be the Alternate Path (AP) method, in which a structure is analyzed for collapse potential after the removal of a column, section of wall or other vertical loadbearing element. Different analytical procedures may be used, including Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). The use of static procedures in AP analyses requires that factors be applied to the loads and structural capacities to account for dynamic and non-linear effects. The factors currently used with static procedures in the existing guidelines tend to generate inconsistent designs when compared to the NLD procedure.

Consequently, an important aspect of the new unified progressive collapse criteria is the determination of appropriate modification factors such that the solutions and designs for the simpler Linear and Nonlinear Static approaches better approximate the more accurate and realistic Nonlinear Dynamic results. To develop these factors, reinforced concrete and steel moment resisting frames were modeled and analyzed with existing structural design software. The results of these analyses are presented in this paper and revised factors are proposed.

## Introduction

Over the last 10 years, two United States government agencies have developed guidelines for the design of their structures to resist progressive collapse: 1. The General Services Administration, "Progressive Collapse Analysis and Design Guidelines," (GSA Guidelines) and 2. The Department of Defense Unified Facilities Criteria 4-023-03 "Design of Buildings to Resist Progressive Collapse" (UFC 4-023-03). Although both documents incorporate some of the same approaches, there are notable differences in the application of these procedures. In an effort to develop a common federal design requirement, both guidelines are being revised and will eventually be merged into one technical guidance document.

Within both approaches, the main direct design procedure is the Alternate Path (AP) method, in which a structure is analyzed for collapse potential after the removal of a column or section of wall. Different analytical procedures may be used, including Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). Typically, NLD procedures give better and more accurate results, but are more complicated and expensive. As a result, designers often choose static procedures which tend to be simpler, requiring less labor. As progressive collapse is a dynamic and nonlinear event, the load cases for

the static procedures require the use of factors to account for inertial and nonlinear effects, similar to the approach used in the “Prestandard and Commentary for the Seismic Rehabilitation of Buildings” by the Federal Emergency Management Agency (FEMA 356). It is important that the new combined design requirements incorporate appropriate dynamic and nonlinear factors such that the linear static and nonlinear static designs are more representative of the actual nonlinear and dynamic response of the structure.

### **Design Approaches to Resist Progressive Collapse**

Prevention or mitigation of progressive collapse can be achieved using two different methods: indirect design and direct design. The indirect method consists of improving the structural integrity of the building by providing redundancy of load paths and ductile detailing. Currently, only the UFC 4-023-03 allows the use of indirect methods. The direct method is divided into two methods: Specific Load Resistance (SLR) method and Alternate Path (AP) method, with the latter being the most widely used in the US. The new guidelines will incorporate a combination of indirect and direct methods with AP being the main direct design procedure.

### **Proposed Alternate Path Method**

In the AP method, the designer must show that the structure is capable of bridging over a removed structural element and that the resulting extent of damage does not exceed the damage limits. In the revised UFC 4-023-03, an AP analysis may be performed using one of three procedures: Nonlinear Dynamic, Nonlinear Static, or Linear Static procedures. Each of these proposed procedures is briefly described in the following paragraphs.

Linear Static (LS): This is the simplest procedure of the three. After the linear static model is created, the applied load is enhanced by a “Load Increase Factor” that approximately accounts for both dynamic and nonlinear effects. The enhanced load is applied to the linear static model that has been modified by removal of a column, wall section or other vertical load bearing member. The calculated internal member forces (or actions or demands) due to the enhanced loads are compared to the expected member capacities. For deformation controlled actions, the expected member capacities are increased by a capacity increase factor (CIF, similar to the “m-factor” in FEMA 356 and the Demand Capacity Ratio, DCR, in the GSA Guidelines) that accounts for the expected ductility and the resulting value is compared to the internal action or demand. For force-controlled actions, the calculated demand is directly compared to the un-modified member capacity.

Nonlinear Static (NLS): After the materially- and geometrically-nonlinear model is built, the loads are magnified by a dynamic increase factor that accounts for inertia effects and the resulting load is applied to the model with the removed vertical load-bearing element. For deformation-controlled actions, the resulting member deformations are compared to the expected deformation capacities; for force-controlled actions, the member strength is not modified and shall not be less than the maximum internal member forces (demands).

Nonlinear Dynamic (NLD): In this case, the un-modified load case is directly applied to a materially- and geometrically-nonlinear model of the structure. In the first phase of the dynamic analysis, the structure is allowed to reach equilibrium under the applied load case. In the second phase, the column or wall section is removed almost instantaneously and the software tool calculates the resulting motion of the structure. As with the NLS case, the resulting maximum member deformations are compared to the expected deformation capacities and for force-controlled actions, the member strength is compared to the maximum internal member forces. Dynamic nonlinear analysis explicitly includes nonlinearity and inertial effects and therefore no correction factors are needed.

## Inconsistency of Existing Factors

As mentioned earlier, the linear static procedure requires the use of a load increase factor (LIF) to account for both dynamic and non-linear effects. The nonlinear static procedure requires a dynamic increase factor (DIF) to account for just the inertial effects. For linear and nonlinear static analysis methods, both the UFC 4-023-03 and the GSA Guidelines currently use the same load multiplier of 2.0 which is applied directly to the progressive collapse load combination. Four major issues have been identified in the static procedures.

### 1. The same load enhancement factor is used for Linear Static and Nonlinear Static analyses.

To approximate the actual nonlinear and dynamic response of a damaged structure, the load on a LS model must be increased by a factor that accounts for both effects. For a NLS model, the load must be increased by a factor that accounts only for the dynamic effects, as the nonlinear behavior has already been addressed. The current UFC 4-023-03 and GSA Guidelines use the same increase factor of 2.0 for both types of analyses, which is incorrect.

### 2. The dynamic increase factor of 2.0 is not appropriate for the majority of LS and NLS cases.

As is well known from structural dynamics, the maximum dynamic displacement of an instantaneously applied, constant load in a linear analysis is twice the displacement achieved when the load is applied statically. If a structure is designed to remain elastic, a factor of 2.0 would be appropriate. However, in extreme loading events, it is more economical and typical to design structures to respond in the nonlinear range. Thus, as will be shown later for the buildings that were analyzed, the dynamic increase factor (DIF) that allows a Nonlinear Static solution to approximate a Nonlinear Dynamic solution, is typically less than 2. On the other hand, the load increase factor (LIF) for a Linear Static analysis must be greater than 2, since dynamic **and** nonlinear effects are present.

### 3. Load enhancement factors do not vary with the performance level

The current guidelines apply the same multiplier to the loads independent of the performance level being used in the design. In other words, a structure is assigned a load enhancement factor of 2.0 regardless of whether the designer wants to allow significant structural damage (Collapse Prevention, as described in FEMA 356) or very little damage (Immediate Occupancy in FEMA 356). As will be shown later, the load enhancement factors can be defined as functions of the desired building performance level and the building characteristics.

### 4. Inconsistency of Capacity Increase Factors (CIF) in LS procedures

UFC 4-023-03: UFC 4-023-03 uses a CIF (m-factor) of 1.0. A CIF (m-factor) of 1.0 combined with a dynamic multiplier of 2.0, can produce overly-conservative designs as the resulting double-span condition after the removal of a vertical load bearing element is required to carry 2 times the progressive collapse load.

GSA Guidelines: GSA uses CIFs (or DCRs) between 1.0 and 3.0. As shown by Ruth 2004, the design could either be overly-conservative or un-conservative, depending on the DCR:

GSA LS Acceptance Equation: **Dyn. Multiplier \* (PC load) < (DCR) x (Capacity)**

$$\text{DCR} = 3: 2 \times (\text{PC load}) = (3) \times (\text{Capacity})$$

$$(2/3) \times (\text{PC load}) = (\text{Capacity})$$

$$\text{DCR} = 1: 2 \times (\text{PC load}) = (1) \times (\text{Capacity})$$

$$(2) \times (\text{PC load}) = (\text{Capacity})$$

As seen in the previous equations for a DCR of 3.0, the members are designed using (2/3) of the actual load, which is un-conservative. Conversely, for a DCR of 1.0, the equation becomes similar to the UFC 4-023-03 case.

### Procedure for Determining Dynamic Factors

As a result of these inconsistencies, a study was undertaken to investigate the factors needed to better match the LS and NLS static procedures to the NLD procedure. The variation of the enhanced load with respect to structure deformation was investigated. As in FEMA 356, structural deformation is considered to be the best metric for approximating structural damage. Constant material over-strength factors were employed. The factor applied to the load was the only parameter adjusted to match a given deformation level. The ASCE 7 extreme event load case was used for all analyses; ignoring wind and snow loads, this load combination is  $1.2D + 0.5L$ , where D is the dead load and L is the live load.

The procedure to determine the load increase factors (LIFs for LS analyses) and dynamic increase factors (DIFs for NLS analyses) consists of 3 steps:

- 1- Starting with a baseline model designed using conventional design loads, a NLD was performed using the ASCE 7 extreme event load case without any enhancement; the values of plastic rotation and displacement at the column removal location were recorded.
- 2- Using the same design and model from Step 1, a NLS analysis was performed, with a trial DIF applied to the ASCE extreme event load case. The DIF was adjusted and the model was re-run until the maximum plastic rotation matched the rotation measured in Step 1.
- 3- Using the same design and model from Step 1, a LS analysis was performed. A trial LIF was applied to the ASCE extreme event load case. The LIF was adjusted and the model was re-run until the maximum displacement matched the displacement that corresponds to plastic rotation measured in Step 1.

After a value for the DIF and LIF has been determined for one design, the building was re-designed to produce a different maximum plastic rotation and Steps 1 through 3 are repeated. This process is illustrated in Figure 1 .

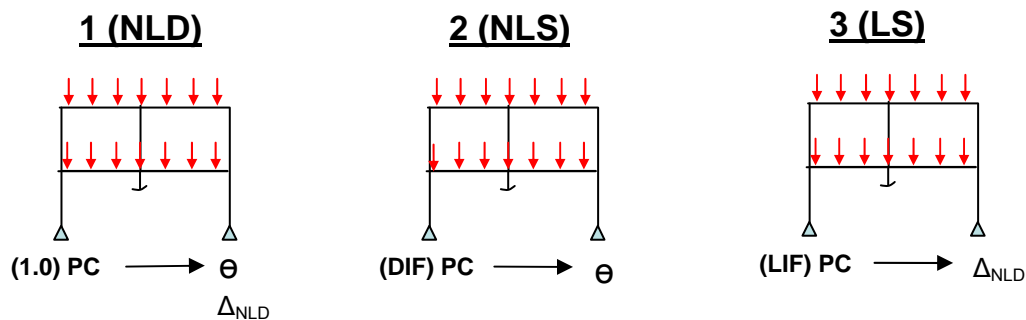


Figure 1. Procedure to Determine Load and Dynamic Increase Factors.

The ranges of nonlinear structural deformation used in this study were taken from the acceptance criteria given in FEMA 356; these values will largely be the basis for the allowable performance levels in the new unified progressive collapse criteria, although some modifications are anticipated. The deformation limits for Life Safety for steel buildings were taken identical to those values in Table 5-6 of FEMA 356. However, for reinforced concrete (RC) buildings, the Life Safety values in Table 6-7 were increased by a factor of 3.5. Within the seismic community, the RC limits in FEMA are considered to be conservative (EERI/PEER 2006) and, in the blast-design community, the allowable deformation criteria in FEMA 356 are much smaller than indicated by test data from blast- and impact-loaded RC structural members. In addition, the conservative FEMA 356 RC criteria are based on backbone curves derived from cyclic testing of members and joints, whereas only one half cycle is applied in a progressive collapse event. Thus, a factor of 3.5 is applied to the Life Safety values in FEMA 356.

## Analytical Models

All buildings were analyzed as 3-dimensional structures, using *SAP2000*, a popular and widely used software package for conventional structural design. All lateral resisting elements were modeled with full moment connections and the connections at the foundations were modeled as pinned. Secondary members were not included.

### Nonlinear Dynamic Analysis

As mentioned earlier, the NLD procedure is the most comprehensive and realistic method of analysis for progressive collapse. The important modeling parameters include the damping ratio, time step, column removal time and plastic hinge definitions. For these analyses, the damping ratio was set to 1% and the column removal and time step were taken as 1/20 and 1/200 of the natural period, respectively, for the structural mode corresponding to the column removal.

The hinge definition for the steel buildings was taken from the pre-set options available in *SAP2000* corresponding to the hinge definition given in Chapter 5 (Steel Frame Structures) of FEMA 356. A graphical representation of this hinge definition is shown in Figure 2.

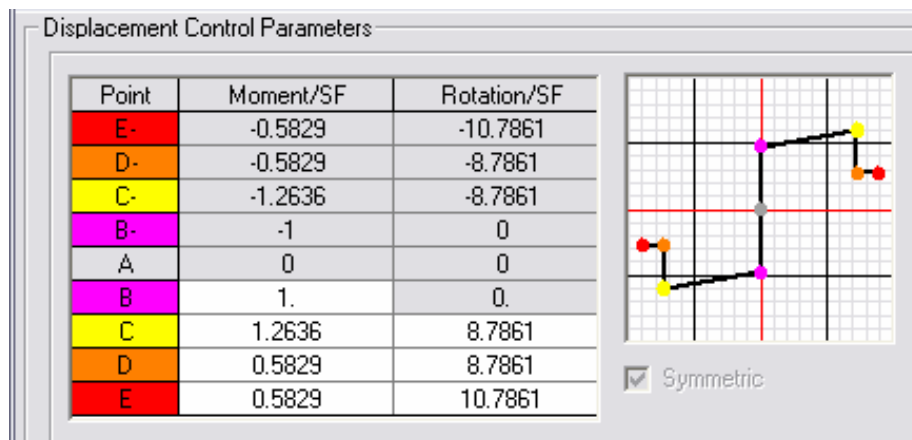


Figure 2. Steel Frame Building Hinge Definition (FEMA 356).

For reinforced concrete structures, the hinge definition (Figure 3) was design to allow strain hardening of 5% at the point expected to be the maximum allowed rotation (0.07 radians). This differs from the 10% hardening at 0.025 radians used in FEMA 356. The reason for this difference is the larger allowable rotations used in the progressive collapse analyses. In other words, if the same slope used in the FEMA 356 hinge definition from points B to point C (Figure 3) was used in this analysis, this would result in an increase in moment capacity of approximately 30% at the point of maximum allowed displacement (0.07 radians) which is unrealistic.

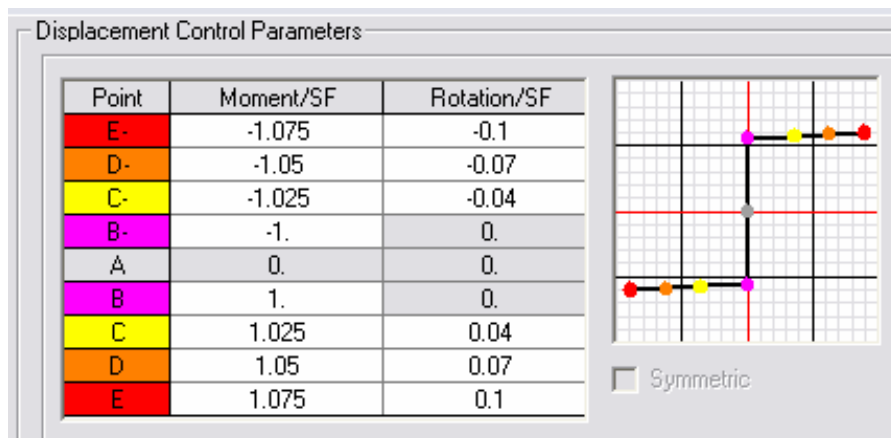


Figure 3. Reinforced Concrete Hinge Definition.

Plastic hinges were located at both ends and mid-point of every beam element and at both ends of the column elements. No hinge off-sets were used.

To simulate the instantaneous removal of a given column, the column was replaced with equivalent reactions obtained from a static analysis of the building using the progressive collapse load applied to the entire structure. These loads were then removed over time to simulate the removal of the column. This process is shown in Figure 4.

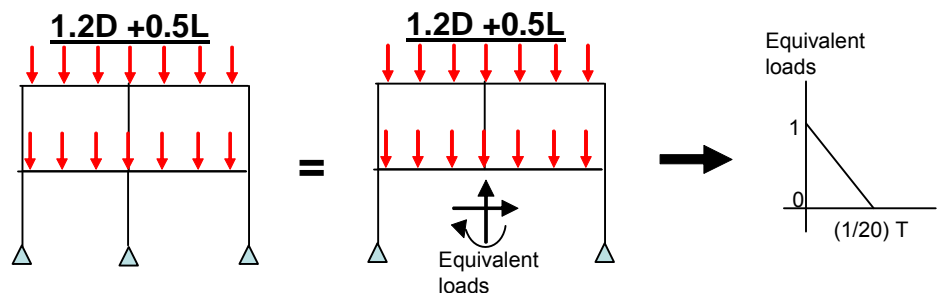


Figure 4. NLD Analysis Procedure.

After the equivalent column loads were removed, the building was allowed to deform until it settled and the maximum plastic rotation was recorded for all hinges formed during the analyses. (See Figure 5)

Nonlinear Static Analysis

In the NLS analysis, non-linearity was modeled identically as with the NLD model just discussed. However, to simulate the column removal, the “non-linear staged construction” feature in *SAP2000* was used (Figure 6). The model was analyzed in three stages using 100 steps per stage.

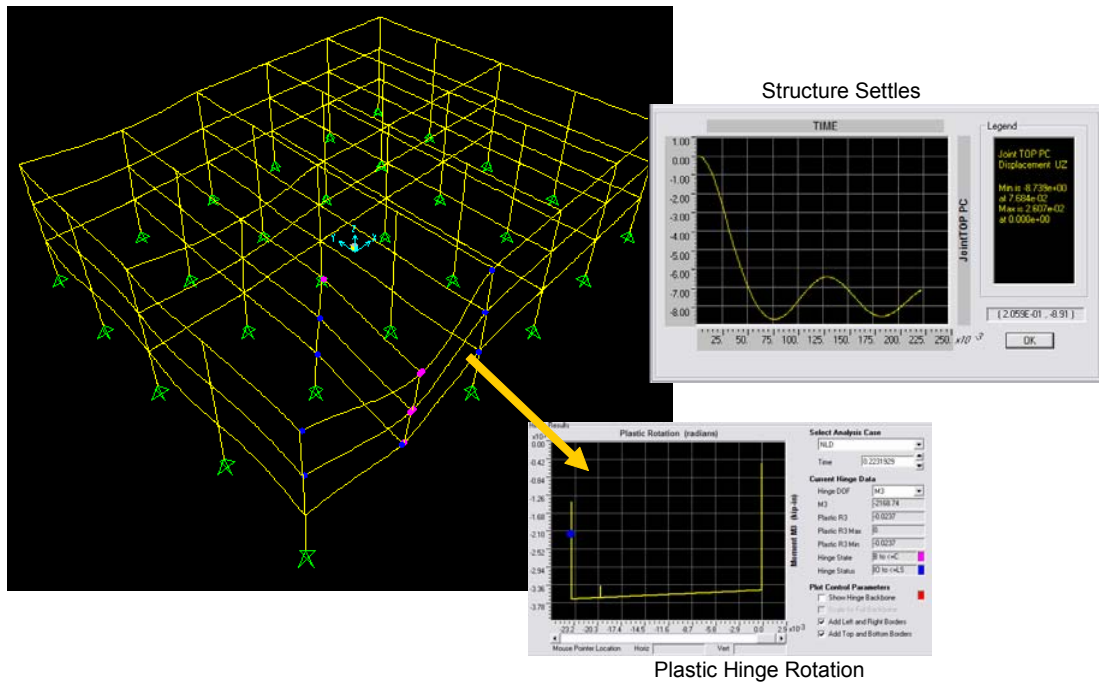


Figure 5. Results of NLD Procedure.

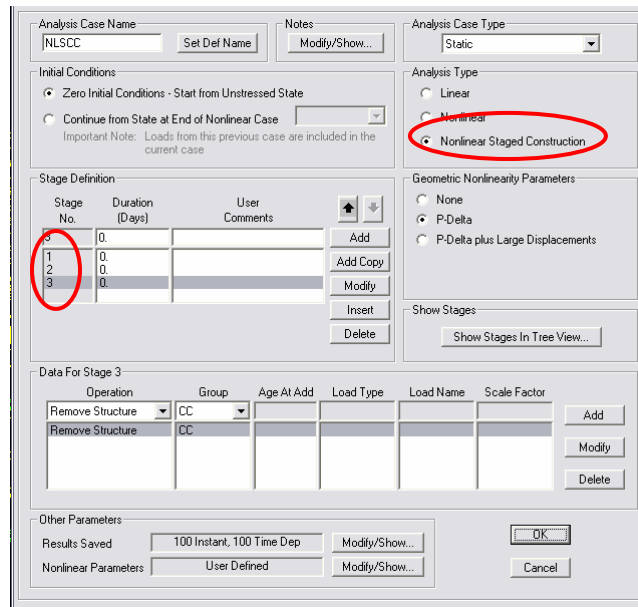
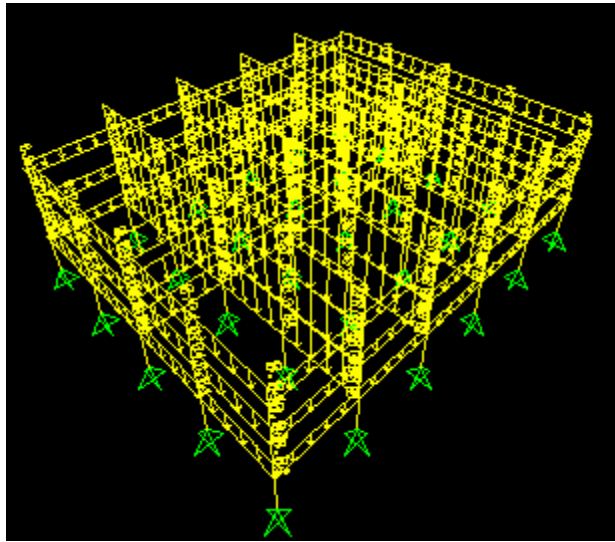


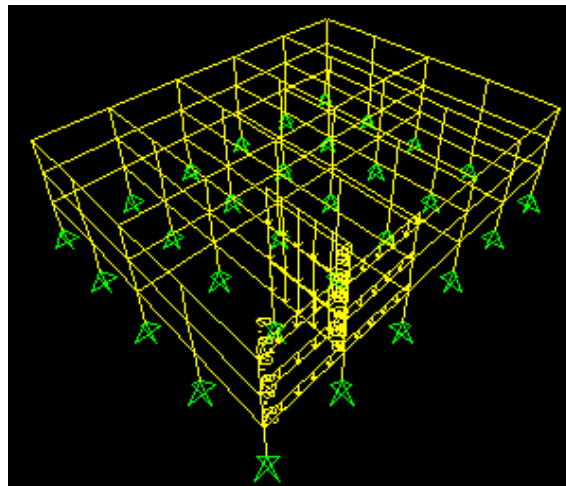
Figure 6. Stage Construction Set up.

In the first stage, the progressive collapse load case was applied to all elements; see Figure 7.



*Figure 7. NLS Analysis Stage 1.*

In the second stage, only the bays around the loss location were loaded with the progressive collapse load, multiplied by the trial DIF, as shown in Figure 8.



*Figure 8. NLS Stage 2, Load Around Loss Location.*

In the final stage, the column was removed and the analysis was run until the building settles; see Figure 9. After the building has settled, the maximum plastic hinge rotations were recorded in a similar manner to the NLD case. If the maximum plastic rotation was not equal to the plastic rotation from the NLD analysis, the DIF was adjusted and the analysis was repeated, until the plastic rotations from the NLD and NLS analyses matched within a few percent.

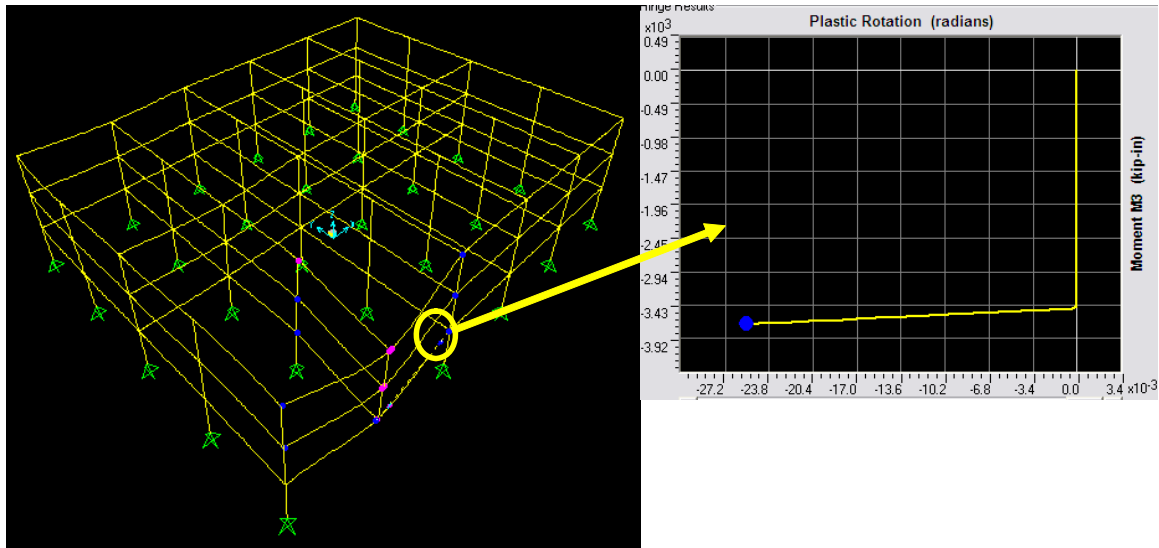


Figure 9. NLS Stage 3, Nonlinear Analysis of Structure Response.

Linear Static Analysis

The linear static procedure is simpler in that it does not require the use of dynamic and non-linear parameters such as time step, damping ratio, plastic hinges, etc. In these analyses, two sets of loads were applied to the building model, from which a column has been removed: one set of loads was applied to the whole structure, and the other set of loads which includes the trial LIF was applied only around the column removal locations as directed in UFC 4-023-03. The analysis was run using the linear elastic option in *SAP2000* and the displacement was measured at the loss location. If the displacement did not match the displacement from the NLD procedure, the trial LIF was adjusted and the analysis was run again.

**Building Designs**

The baseline design for each building was taken from the examples in the current UFC 4-023-03.

Steel Frame Buildings

Analyses for Interior, perimeter and corner column removals were performed on 3- and 10-story steel frame buildings. Each building has a bay size of 25 ft and floor plan geometry as illustrated in Figure 10. The loads and material strengths used in the analysis are given in Tables 1 and 2. Wind loads were not included in the progressive collapse study. However, the baseline design did include wind loads.

Table 1. Load Conditions.

DL	49 psf	Includes self weight of members not modeled
SDL	35 psf	Includes partitions, ceiling weight and mechanical equipment
CL	15 psf	Cladding load, only in the perimeter
LL	50 psf	Live load

Table 2. Material Properties.

$f_y$	52.5 ksi	Includes 1.05 over-strength factor
E	29,000 ksi	Modulus of elasticity

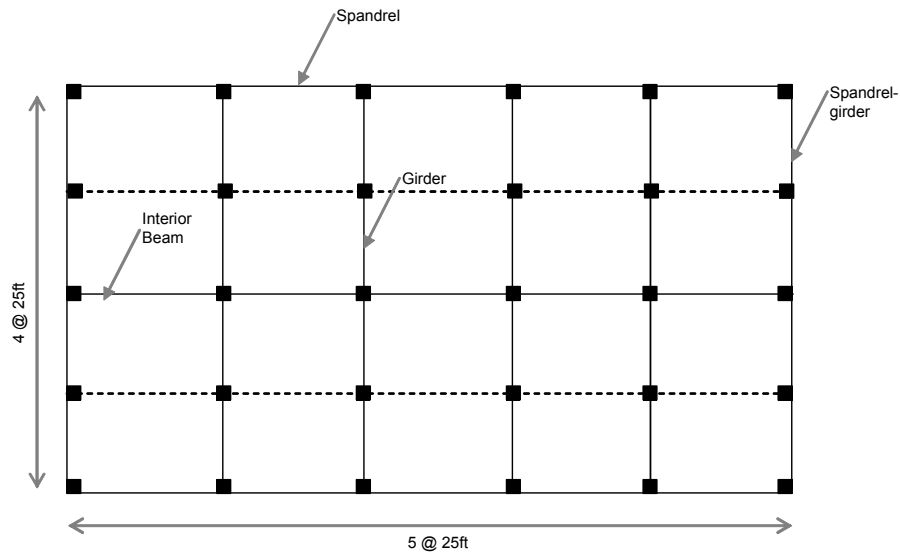


Figure 10. Typical Floor Plan.

As indicated in the example of the current UFC 4-023-03, member sizes were designed using the maximum moments for all floors. Therefore, the member groups are identical at all floors. However, the columns and some other members of the 10-story building were modified to support the extra weight of the additional floor levels. Table 3 presents a summary of the member sizes corresponding to the baseline design for the 3- and 10-story buildings.

Table 3. Baseline Design - Member Sizes.

<b>Member Group</b>	<b>3-Story</b>	<b>10-Story</b>
Spandrels	W18x35	W18x55
Interior Beams	W18x35	W18x35
Girders	W18x55	W18x55
Spandrel-Girders	W18x40	W14x90
Bottom Columns	W14x145	W24x162
Top Columns	W14x68	W24x68

**Reinforced Concrete Buildings**

Interior column removals were performed on a 3- and 10-story reinforced concrete building. The floor plan of the building was identical to those for the steel building. The material properties used in the analyses are given in Table 4 and the preliminary member sizes are presented in Tables 5 and 6.

Table 4. Reinforced Concrete Material Properties.

$f_c$	6.25 ksi	Includes 1.25 over-strength factor
$f_y$	75 ksi	Reinforcing steel w/ 1.25 over-strength factor

Table 5. 3-Story Reinforced Concrete Bldg. - Member Sizes.

Member Group	b (in)	d (in)	As (in <sup>2</sup> )	A's (in <sup>2</sup> )
Spandrels	24	20	1.76	2.40
Interior Beams	24	20	1.76	2.40
Girders	30	20	2.42	4.48
Spandrel-Girder	30	20	2.20	3.25
Bottom Columns	20	20	8.00	
Top Columns	20	20	6.32	

Table 6. 10-Story Reinforced Concrete Bldg. - Member Sizes.

Member Group	b (in)	d (in)	As (in <sup>2</sup> )	A's (in <sup>2</sup> )
Spandrels	24	20	1.76	2.40
Interior Beams	24	20	1.76	2.40
Girders	30	20	2.42	4.48
Spandrel-Girder	30	20	2.20	3.25
Bottom Columns	24	24	24.96	
Top Columns	26	26	15.24	

### Analysis Results

The results of the study are presented in the following tables. Each table lists the different structural designs that were evaluated for the particular building and column removal location. The first column in each table indicates the “design” number. As mentioned earlier, a baseline design model was developed using standard structural design software and then modified (Re-Design 1 through X) by changing the beam, spandrel, girder, and spandrel-girder cross-sections to acquire different displacements and plastic rotations. Next, the tables provide the section properties and geometry of the structural elements framing into the loss location for that particular “re-design”. Finally, the tables show the displacement and/or plastic rotation measured with the NLD analysis and the values of DIF and LIF obtained from the NLS and LS analysis of that particular “re-design.”

### Steel Buildings

Table 7. 3-Story Steel Building - Corner Column Removal Results.

Run #	Frame Section	Section	Zx	Ix	Weight	NLD, Disp.	NLD Plastic Rotation	NLS DIF	LS LIF
	<i>sap name</i>		<i>in<sup>3</sup></i>	<i>in<sup>4</sup></i>	<i>lb</i>	<i>in</i>	<i>rad</i>		
Re-Design 3	Spandrel	w18x60	123	984	60	4.93	0.0019	1.74	1.78
	Spn-Gird	w18x65	133	1070	65				
Re-Design 2	Spandrel	w21x44	95.4	843	44	5.47	0.0075	1.62	1.80
	Spn-Gird	w21x50	110	984	50				
Baseline	Spandrel	w18x35	66.5	510	35	9.44	0.0222	1.42	2.17
	Spn-Gird	w18x40	78.4	612	40				
Re-Design 1	Spandrel	w14x38	61.5	385	38	13.45	0.0337	1.36	2.43
	Spn-Gird	w14x43	69.6	428	43				

Table 8. 3-Story Steel Building - Interior Column Removal Results.

Run #	Frame Section	Section	Zx	Ix	Weight	NLD, Disp.	NLD Plastic Rotation	NLS DIF	LS LIF
	<i>sap name</i>		<i>in<sup>3</sup></i>	<i>in<sup>4</sup></i>	<i>lb</i>	<i>in</i>	<i>rad</i>		
Re-Design 7	Girder	w24x76	200	2100.	76	3.77	0.0021	1.82	1.80
	Int. Beam	w18x60	123	984.	60				
Re-Design 6	Girder	w21x73	172	1600.	73	4.76	0.0080	1.68	1.84
	Int. Beam	w16x57	105	758.	57				
Re-Design 5	Girder	w24x62	153	1550.	62	4.80	0.0103	1.60	1.90
	Int. Beam	w21x44	95.4	843.	44				
Re-Design 4	Girder	w24x55	134	1350.	55	6.42	0.0167	1.44	2.12
	Int. Beam	w18x40	78.4	612.	40				
Re-Design 3	Girder	w18x60	123	984.	60	8.99	0.0260	1.38	2.37
	Int. Beam	w16x40	72.9	518.	40				
Baseline	Girder	w18x55	112	890.	55	11.44	0.0349	1.29	2.84
	Int. Beam	w18x35	66.5	510.	35				
Re-Design 1	Girder	w16x57	105	758.	57	16.91	0.0532	1.23	3.49
	Int. Beam	w14x38	61.5	385.	38				

Table 9. 3-Story Steel Building - Perimeter Column Removal Results.

Run #	Frame Section	Section	Zx	Ix	Weight	NLD, Disp.	NLD Plastic Rotation	NLS DIF	LS LIF
	<i>sap name</i>		<i>in<sup>3</sup></i>	<i>in<sup>4</sup></i>	<i>lb</i>	<i>in</i>	<i>rad</i>		
Re-Design 3	Spandrel	w21x50	110	984	50	4.13	0.0058	1.73	1.86
	Girder	w21x73	172	1600	73				
Baseline	Spandrel	w18x35	66.5	510	35	8.74	0.0237	1.38	2.35
	Girder	w18x55	112	890	55				
Re-Design 1	Spandrel	w14x38	61.5	385	38	12.52	0.0371	1.33	2.65
	Girder	w14x61	102	640	61				
Re-Design 2	Spandrel	w12x40	57.5	310	40	16.80	0.0514	1.31	2.95
	Girder	w12x65	96.8	533	65				

Table 10. 10-Story Steel Building - Corner Column Removal Results.

Run #	Frame Section	Section	Zx	Ix	Weight	NLD, Disp.	NLD Plastic Rotation	NLS DIF	LS LIF
	<i>sap name</i>		<i>in<sup>3</sup></i>	<i>in<sup>4</sup></i>	<i>lb</i>	<i>in</i>	<i>rad</i>		
Baseline	Spandrel	w18x55	112	890	55	4.73	0.0067	1.65	1.70
	Spn-Gird	w14x90	157	999	90				
Re-design 1	Spandrel	w14x53	87.1	541	53	6.89	0.0138	1.55	1.80
	Spn-Gird	w12x79	119	662	79				
Re-Design 1	Spandrel	w14x38	61.5	385	38	10.61	0.0300	1.41	2.10
	Spn-Gird	w12x58	86.4	475	58				
Re-design 2	Spandrel	w14x30	47.3	291	30	20.68	0.0620	1.27	3.23
	Spn-Gird	w12x45	64.7	350	45				

Table 11. 10-Story Building - Interior Column Removal Results.

Run #	Frame Section	Section	Zx	Ix	Weight	NLD, Disp.	NLD Plastic Rotation	NLS DIF	LS LIF
-	<i>sap name</i>		<i>in<sup>3</sup></i>	<i>in<sup>4</sup></i>	<i>lb</i>	<i>in</i>	<i>rad</i>		
Re-Design 8	Girder	w21x93	221	2070	93	3.7	0.0037	1.75	1.81
	Int. Beam	w18x71	145	1170	71				
Re-Design 7	Girder	w21x83	196	1830	83	3.92	0.0070	1.70	1.83
	Int. Beam	w21x57	129	1170	57				
Re-Design 6	Girder	w21x68	160	1480	68	5.03	0.0116	1.57	1.85
	Int. Beam	w18x50	101	800	50				
Re-Design 5	Girder	w18x76	163	1330	76	5.69	0.0120	1.55	1.85
	Int. Beam	w14x61	102	640	61				
Re-Design 4	Girder	w21x62	144	1330	62	5.73	0.0146	1.50	1.95
	Int. Beam	w18x46	90.7	712	46				
Re-Design 3	Girder	w21x57	129	1170	57	8.48	0.0244	1.39	2.30
	Int. Beam	w12x53	77.9	425	53				
Baseline	Girder	w18x55	112	890	55	11.45	0.0360	1.28	2.90
	Int. Beam	w18x35	66.5	510	35				
Re-Design 9	Girder	w16x57	105	758	57	16.82	0.0536	1.22	3.60
	Int. Beam	w14x38	61.5	385	38				
Re-Design 1	Girder	w10x77	97.6	455	77	38.15	0.1230	1.18	5.40
	Int. Beam	w10x45	54.9	248	45				

Table 12. 10-Story Steel Building - Perimeter Column Removal Results.

Run #	Frame Section	Section	Zx	Ix	Weight	NLD, Disp.	NLD Plastic Rotation	NLS DIF	LS LIF
-	<i>sap name</i>		<i>in<sup>3</sup></i>	<i>in<sup>4</sup></i>	<i>lb</i>	<i>in</i>	<i>rad</i>	-	-
Baseline	Spandrel	w18x55	112	890	55	4.69	0.0079	1.60	1.86
	Girder	w18x55	112	890	55				
Re-Design 1	Spandrel	w18x46	90.7	712	46	6.14	0.0132	1.47	2.05
	Girder	w18x46	90.7	712	46				
Re-Design 2	Spandrel	w16x40	72.9	518	40	10.57	0.0289	1.30	2.72
	Girder	w16x40	72.9	518	40				

Reinforced Concrete Buildings

Table 13. 3-Story RC Building - Interior Column Removal Results.

Run #	Frame Section	Top. As	Bot. As	Steel %	NLD,Plastic Rotation.	NLS DIF	LS LIF
	<i>sap name</i>	<i>in^2</i>	<i>in^2</i>	<i>increment</i>	<i>in</i>		
Baseline	Girder	4.48	2.42	---	---		
	Int. Beam	2.40	1.76	---			
Re-Design 1	Girder	7.84	4.24	75%	0.021	1.14	4
	Int. Beam	4.20	3.08	75%			
Re-Design 2	Girder	7.48	4.04	67%	0.028	1.09	5.5
	Int. Beam	4.01	2.94	67%			
Re-Design 3	Girder	7.17	3.87	60%	0.042	1.06	8
	Int. Beam	3.84	2.82	60%			
Re-Design 4	Girder	7.03	3.80	57%	0.053	1.05	10
	Int. Beam	3.77	2.76	57%			
Re-Design 5	Girder	6.94	3.75	55%	0.064	1.05	13
	Int. Beam	3.72	2.73	55%			
Re-Design 6	Girder	6.81	3.68	52%	0.086	1.04	17
	Int. Beam	3.65	2.68	52%			
Re-Design 7	Girder	6.76	3.65	51%	0.097	1.04	19
	Int. Beam	3.62	2.66	51%			

Table 14. 10-Story RC Building - Interior Column Removal Results.

Run #	Frame Section	Top. As	Bot. As	Steel %	NLD,Plastic Rotation.	NLS DIF	LS LIF
	<i>sap name</i>	<i>in^2</i>	<i>in^2</i>	<i>increment</i>	<i>in</i>		
Baseline	Girder	4.48	2.42	---	---		
	Int. Beam	2.40	1.76	---			
Re-Design 1	Girder	8.96	4.84	100%	0.015	1.25	2.7
	Int. Beam	4.80	3.52	100%			
Re-Design 2	Girder	7.17	3.87	60%	0.060	1.05	11
	Int. Beam	3.84	2.82	60%			
Re-Design 3	Girder	7.03	3.80	57%	0.080	1.05	15
	Int. Beam	3.77	2.76	57%			
Re-Design 4	Girder	6.94	3.75	55%	0.099	1.04	18.5
	Int. Beam	3.72	2.73	55%			

### Data Analysis and Proposed Application

To develop a larger baseline for comparison of results, additional data was generated using simple 2-dimensional double-span models analyzed with a Single Degree of Freedom (SDOF) approach. These results were compared with data obtained from the 3-dimensional models presented above and additional result from similar 3-dimensional SAP models to determine trends in the data. Plots were generated with proposed normalized factors for static procedures.

#### Linear Static Analysis

The LS approach will often be used in concept development of complex structural systems required to satisfy AP requirements. The practitioner will “pick” an LIF and a CIF from charts provided in the final UFC guidance. The CIF will depend on the acceptance criteria being used for the design or analysis. The CIF value will be a direct multiplier on the expected component strengths given in the final unified PC guidelines, which will correspond to the existing values for acceptance criteria on FEMA 356.

However, the results obtained in this study demonstrated that LIFs are a function of section properties and geometry, particularly for RC sections, where stiffness can vary significantly based on rebar placement and section aspect ratio. For steel structures, the LIFs results were found to be less dependent on the selected section. Figures 11 and 12 illustrate the variation of LIFs with the total rotation, which is a function of the sections used in the building design.

The plots show LIF plotted versus total rotation for selected analysis cases from the results presented in the previous sections. The dispersion of data points in Figure 11 represents the strong dependence on section properties for the concrete members. Scatter is also seen in the data plotted in Figure 12 for steel buildings. Because the LIFs need to be applied consistently to different structural elements regardless of their stiffness or shape, the data above was normalized using the elastic stiffness in the form of calculated yield rotation. The plots in Figures 13 and 14 show the same LIF data when normalized by yield rotation.

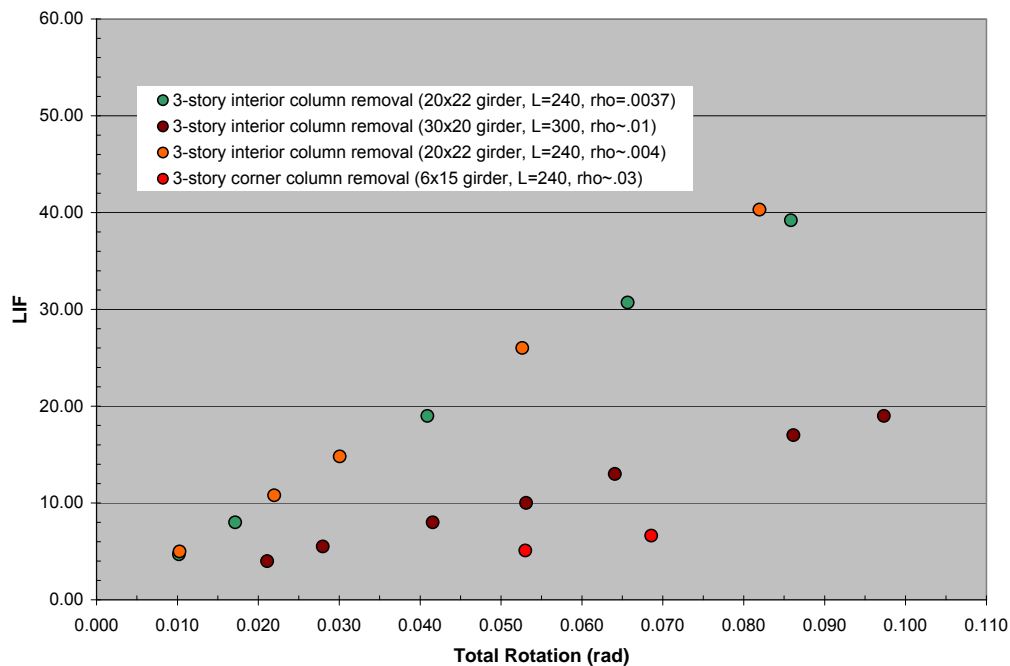


Figure 11. LIFs for RC Buildings, Not Normalized

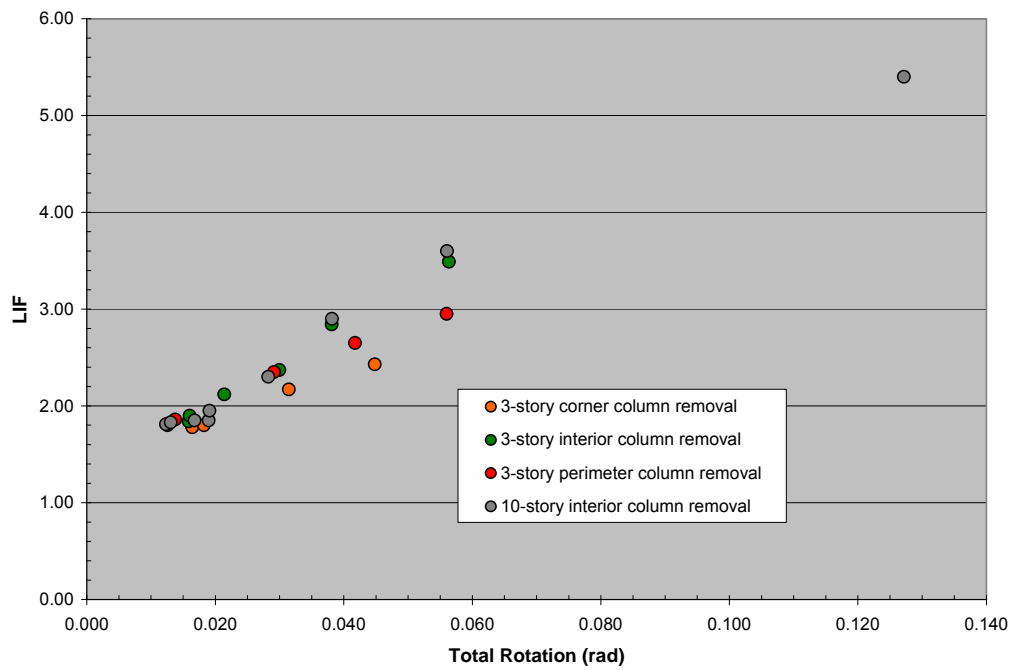


Figure 12. LIFs for Steel Buildings, Not Normalized

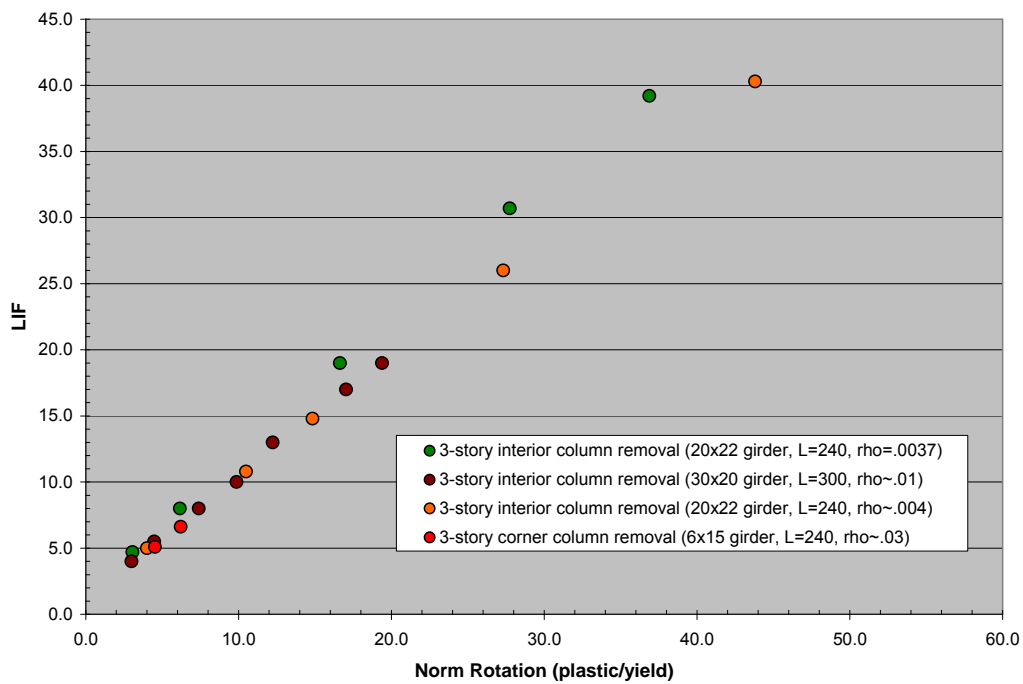


Figure 13. LIFs for RC Buildings, Normalized

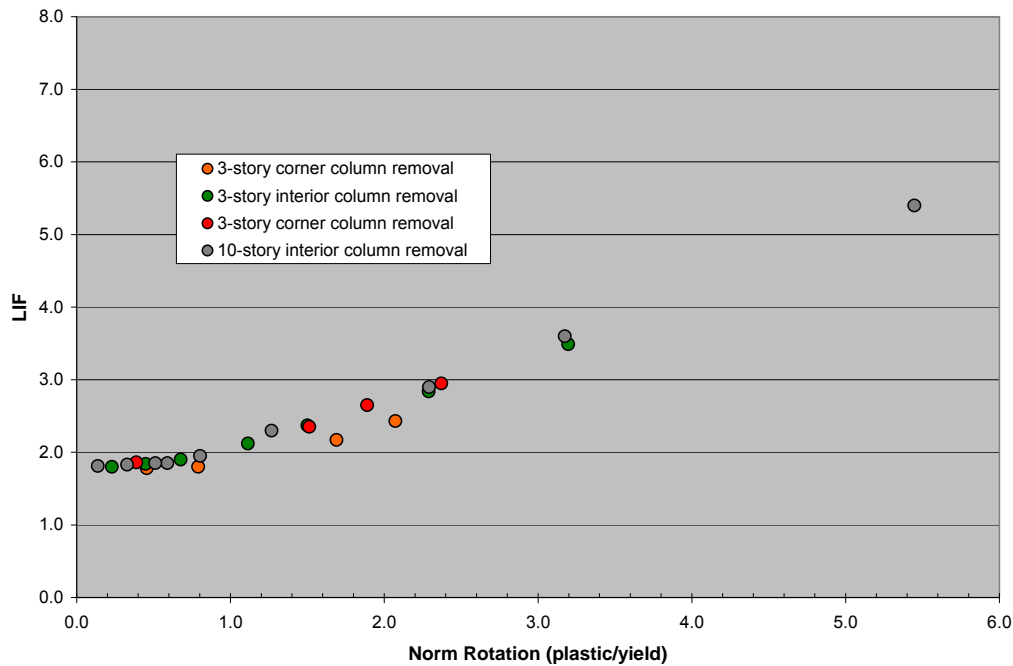


Figure 14. LIFs for Steel Buildings, Normalized

The normalized values presented in Figures 13 and 14 represent the values of LIFs to be used in LS procedures and can be chosen using an equation developed from the relationships illustrated above for the selected rotations, which depend on the acceptance criteria. The practitioner would calculate the LIF for his selected configuration, using either averaged or minimum properties per floor and beam-column intersection.

Although these final values for LIFs seem very high, it should be considered that the true “multiplier” on the static load case for LS analysis is the LIF divided by the CIF:

$$\text{General Equation: } (\text{LIF}) \times (\text{PC load}) < (\text{CIF}) \times (\text{Capacity})$$

$$\text{LS: } (\text{LIF/CIF}) \times (\text{PC load}) = (\text{Capacity})$$

As mentioned earlier, the CIF (i.e., m-factors in FEMA 356) values in the revised UFC 4-023-03 are currently being re-evaluated and modified. For reinforced concrete, the proposed CIF will nominally range from 10 to 20 for concrete after adjustments are made to account for the conservatism in the existing concrete criteria. For steel, the range of CIF will be a function of the component but will not be significantly different the existing FEMA 356 criteria, i.e., 1.5 to 4. Hence, final “load multipliers” for LS analysis will generally vary from 1.0 to 2.0, but will likely be less than 2.0 in most cases.

Nonlinear Static Analysis - Dynamic Increase Factors

In NLS procedures, non-linearity is explicitly accounted for in the model by use of plastic hinges and the capacity of the members does not need to be adjusted using CIFs. Therefore, the values of DIFs obtained in this study are a direct representation of the dynamic multiplier on the load. Normalization of the data was still needed as fixed rotation limits involve differing ratios of elastic to plastic deformation. The application of the DIFs is demonstrated below:

General Equation:  $(DIF) \times (PC \text{ load}) < (CIF) \times (\text{Capacity})$

NLS:  $(DIF) \times (PC \text{ load}) = (1) \times (\text{Capacity})$

$(DIF) \times (PC \text{ load}) = (\text{Capacity})$

The results showed a range of variation in DIFs with respect to plastic rotation from 1.20 to 1.85 for steel buildings and from 1.00 to 1.40 for concrete buildings as illustrated in Figures 15 and 16, respectively. In the new PC guidelines, the practitioner would select a value of DIF to be applied to the load combination in NLS procedures based on the ratio of yield rotation to allowable plastic rotation specified in the acceptance criteria.

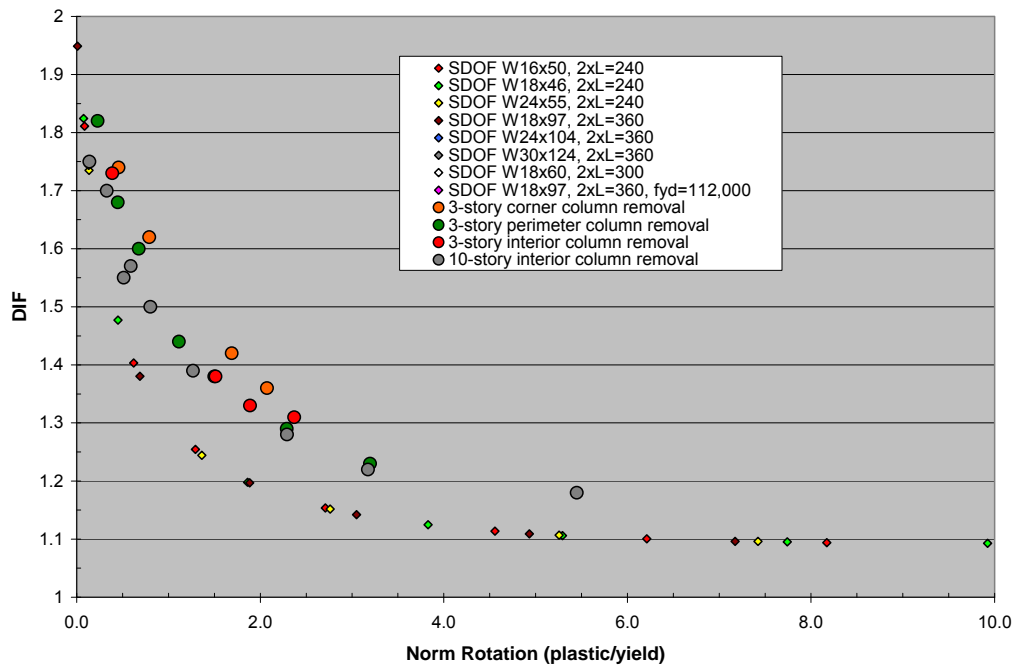


Figure 15. DIFs for Steel Buildings

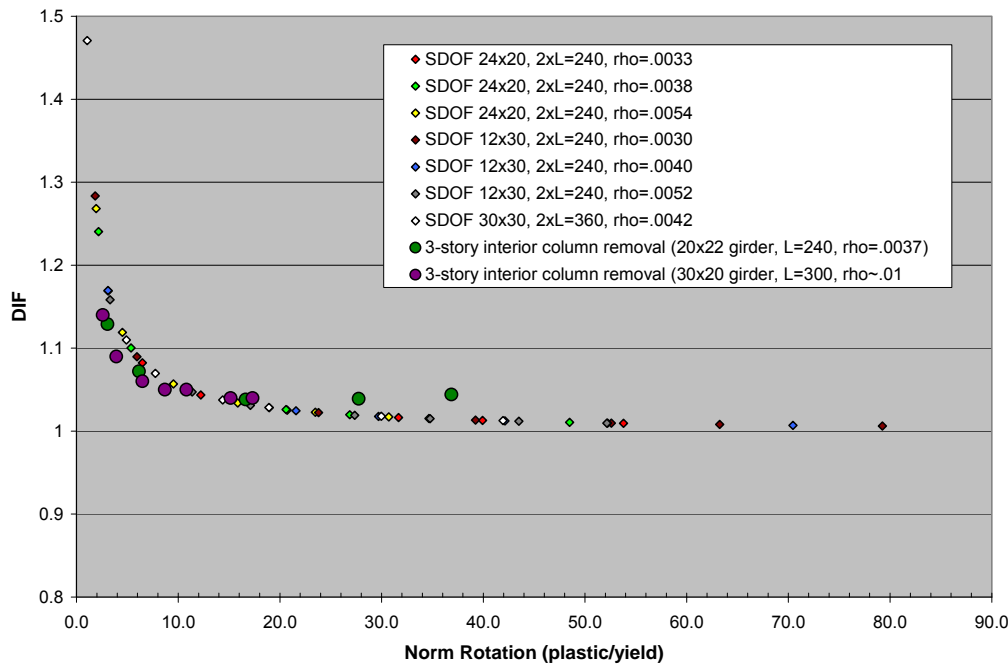


Figure 16. DIFs for Concrete Buildings

## Conclusions and Recommendations

The dynamic multiplier on the load of 2.0 currently used in NLS procedures for AP analysis can produce overly-conservative designs. This is particularly true for cases where large deformations are allowed. The results of this study showed that for RC buildings, the dynamic multiplier on the load (DIF) ranges from 1.05 to 1.4, which is significantly less than 2.0. Similarly, for steel buildings analyzed, the dynamic multiplier on the load (DIF) ranged from 1.2 to 1.8.

Additionally, the results from the analyses of LS procedures demonstrated that LIFs depend on the total deformation as was expected, but are also strongly dependent on section properties. As there are no LIFs in the current guidelines, there is no direct comparison that can be made between the conservatism of the existing approach and the proposed use of the LIFs. However, the current DoD and GSA procedures use a load multiplier of 2.0, which is effectively defined to be a DIF, and capacity increase factors (DCRs) of between 1.5 and 2.0. So, while the LIFs presented in this work are significantly greater than 2.0 in almost all cases, the CIFs (m-factors) are much, much larger than the DCRs used in the current criteria, hence, it is the ratio, or effective multiplier that can be compared to evaluate relative conservatism. With a multiplier of 2.0 and a DCR of 2.0, the effective multiplier in the current GSA criteria, for example, is 1.0. For an LIF of 38 in the current research, for example, coupled with a CIF (m-factor) of 21, the effective multiplier is 1.8, significantly larger than 1.0 as used in the current criteria. This also reinforces that the current GSA Guidelines could be under- or over-conservative, depending on the structure details; the UFC 4-023-03 could likewise be considered slightly overly conservative.

The DIF and LIF curves proposed in this paper were developed based on approximations of the results from static procedures and the more complex and accurate NLD procedures and represent an improvement over the existing guidelines. However, because the LS and NLS approaches provide only an approximation of the NLD results, it is recommended that additional conservatism be added to these curves. This would, in effect, reward a designer who uses a NLD procedure that produces a more efficient

and cost-effective structure design relative to the LS and NLS procedures. A more detailed assessment of the correct level of conservatism is ongoing and, when complete, will be implemented into UFC 4-023-03.

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