Title:	Proposed Methodology To Determine Seismic Performance Factors For Steel Diagrid Framed Systems

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PROPOSED METHODOLOGY TO DETERMINE SEISMIC PERFORMANCE FACTORS FOR STEEL DIAGRID FRAMED SYSTEMS

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Abstract

This paper presents a proposed methodology for the reliable determination of seismic performance factors for steel diagrid framed seismic force-resisting systems. The paper focuses on developing an analytical methodology using preliminary analytical methods and modeling of representative archetype diagrid framed systems. As current model building codes do not explicitly address the seismic design performance factors for this new and emerging structural system, the purpose of these studies is to propose a sound and reliable basis for defining such seismic design parameters. This study and recommended methodology was based on the approach as developed using the ATC-63 project (90% Draft, 2008), *Quantification of Building Seismic Performance Factors*, subsequently published as FEMA P695 (2009). The methodology referred to herein as "ATC-63" represents a broad knowledge base of standard building code concepts, structural systems, relevant research and technologies utilizing state-of-the-art nonlinear dynamic analysis and collapse simulation to reliably quantify system performance and response parameters for use in seismic design.

Parameters of interest in the analysis and design of steel diagrid framed structural systems in regions of high seismicity include the effects of variation of geometrical properties and configurations of diagrid structures, such as, variation of building height-to-width aspect ratios, angle of sloped columns with intervals of diagrid elements over story heights, concentration of energy absorption demands and capacities, combined flexure and axial post-buckling component hysteretic behavior, as well as, level of system redundancy. For the purpose of this study, it is intended that steel diagrid framed systems be considered as a new seismic force-resisting system suitable for use in model building codes designed using linear methods of analysis to achieve equivalent safety margins against collapse as intended by code compliant seismic force resisting systems.

Steel Diagrid Framed System

In recent years, new and emerging architectural building designs have been put forward consisting of geometrical and structural system frame definitions consisting of triangulated sloped column and spandrel beam frame configurations called "diagrids" shown in Figure 1. These triangulated diagrid frames are most often placed on the building perimeter creating efficient structural systems in resisting both gravity and lateral wind loads. The purpose of this paper is to investigate their apparent superior structural efficiency with respect to performance and behavior as seismic force-resisting systems in high seismic regions. Under moderate to extreme earthquake

Lotte Supertower,   Jinling Tower,
Seoul, Korea   Nanjing, China
*Figure 1. Steel Diagrid Framed Systems
(Skidmore, Owings & Merrill LLP)*

ground shaking demands, typical seismic force-resisting systems must provide sufficient ductility and energy dissipation characteristics to provide life safety against collapse while undergoing inelastic frame deformations. In the typical triangulated configuration of the steel diagrid framed system, both gravity and lateral loads are distributed in the sloped column and spandrel beam elements primarily in axial compression and tension. Under these axial compression conditions, the diagrid frame elements are designed to remain linear elastic with appropriate factors of safety using current AISC 341-05 Seismic Design Provisions (AISC, 2005a).

Typically, steel diagrid framed systems are configured as a dual system for the seismic force-resisting system and their response modification coefficients (R-factors) are selected from 5.5 to 8.0 without further justification since the systems are typically composed of exterior steel diagrid frames in combination with ductile reinforced concrete core-wall frames. However, for a diagrid system that is not combined with a ductile core-wall frame as a dual system, the exterior diagrid frames become a standalone lateral force-resisting system. This standalone seismic force-resisting system may be classified as a "Bearing Wall System" per UBC 97 (ICBO, 1997) code provisions, or as an "Undefined System" based on the current ASCE 7-05 (ASCE, 2005) provisions. Therefore, as an undefined system, it is necessary to establish a methodology to determine appropriate seismic performance factors using a more rigorous and reliable procedure.

Overview of Historical Methods for Seismic Performance Factor Estimation

A brief overview of historical seismic analysis and design methods using code based seismic response modification factors is shown in Table 1. The 1961 Uniform Building Code (UBC) introduced six K-factors as horizontal force factors used to address all seismic force resisting systems. The K-factors distinguished ductile and non-ductile systems from regular systems. Simply, a K-factor of unity was specified for regular systems, and a K-factor of 0.67 for ductile moment resisting frames, while a K-factor of 1.33 was specified for a bearing wall system considered as a non-ductile system. In subsequent codes, while response modification factors were further modified based on detailing and expected performance, the K-factors remained until the R-factor, assuming working force levels, was introduced in the 1988 UBC based on the earlier ATC 3 (ATC, 1978). Today, current R-factors embodied in ASCE 7-05 have been principally calibrated to the strength levels based on the 1997 UBC defined systems, such that, 83 different seismic force-resisting systems including dual systems are included in ASCE 7-05 provisions. However, during the process, the R-factors for "newly defined" or "undefined" systems are typically determined using "engineering judgments" and qualitative comparisons to achieve "equivalent" behavior with the factors of previously code defined systems. Based on the general observed behavior of previously code defined systems, the steel diagrid framed system used in combination with ductile core-wall systems can be considered as a dual system having a range of R-factors depending on the "executive engineering judgment". While satisfying this general code level approach, for significantly complex building systems, a performance based design approach is typically undertaken to substantiate conformance with equivalent code provisions.

Table 1. Short History of Seismic Base Shear

Code (Seismic Performance Factor)	Base Shear
UBC 61 – UBC 85: K-factor	$V = ZIKCSW$: Working Stress Level
ATC-3 (1978): R-factor	Seismic Performance Factor "R" is introduced.
UBC 88: R_w-factor	$V = ZICW/R_w$: Working Stress Level
UBC 97: R-factor	$V = C_v IW/RT$: Strength Design Level
ASCE 7-05: R-factor	$V = S_{D1} IW/RT$: Strength Design Level

ATC-63 Project

Intent of current ASCE 7 code provisions permit a new "undefined" system to be used with seismic performance factors of equivalent systems if the system demonstrates "equivalent" energy dissipation capacity and dynamic characteristics comparable to existing systems. However, model codes do not specify how to verify the equivalent performance other than analytical and test data. The ATC-63 project presents a rigorous methodology to provide a rational and reliable basis for determining building system performance parameters in order to provide "equivalent" safety against collapse for different seismic force-resisting systems with an acceptably low probability of structural collapse.

Critical in the development of seismic performance factors utilizing the ATC-63 methodology is the determination an acceptably low probability of collapse under Maximum Capable Earthquake (MCE) ground motions with reliable collapse margin ratios. The level of conservatism in developing appropriate seismic performance factors (R, Ω_o, C_d) is directly related to the consideration of uncertainties including variability of ground motions and seismic hazard, the accuracy of design procedures, comprehensive test data and nonlinear analysis modeling. Since the seismic performance factors include the effects of inherent system ductility, seismic energy dissipation capacity, mode of failure mechanisms, and past performance, the factors are developed considering

Figure 2. Key elements of the ATC-63 Methodology (ATC, 2009)

established design practice, level of testing and analytical studies in parallel with a comprehensive peer review process. These key elements of the ATC-63 methodology are illustrated in Figure 2.
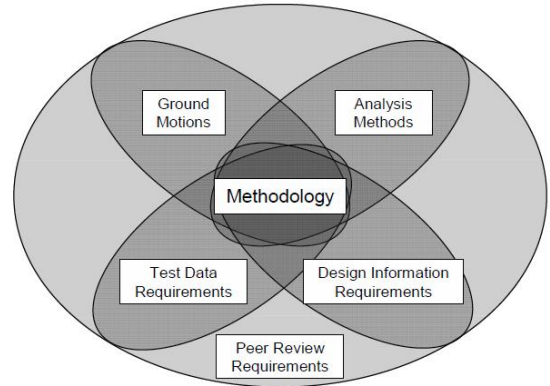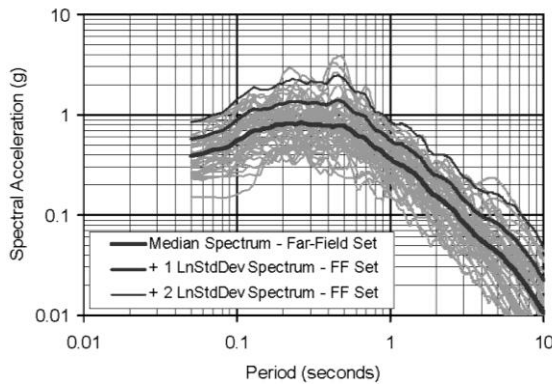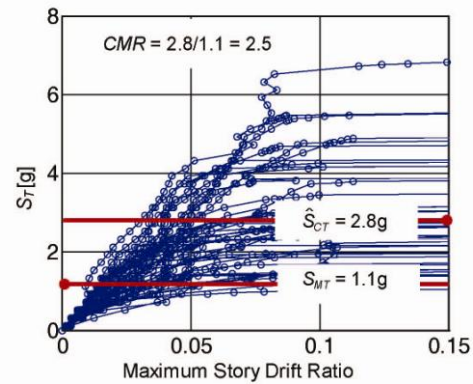

(a) Definition in FEMA 450   (b) Definition in ATC 63

Figure 3. Seismic Performance Factors (ATC, 2009)

Performance Evaluation Procedure. The performance evaluation assessment steps inherent in the ATC-63 methodology are outlined as follows:

(1) Develop a structural system concept including detailed design requirements and proposed seismic performance factors (R-factor) for evaluation. The system concept shall consider use of construction materials, system configurations, mode of failure mechanisms, and other limitations defined by ASCE 7-05 provisions. The detailed design requirements shall ensure the minimum strength and detailing of elements and connections to achieve the expected performance of the system in conformance with AISC 341-05 seismic design and detailing requirements.

(2) Develop structural system archetypes representing prototypical application of the system by identifying the range of design parameters, system attributes and behavior characteristics. Typical characteristics to consider include ground motion intensity, building height, fundamental period, framing configurations, gravity load intensity, etc intended to quantify performance for an entire class of buildings.
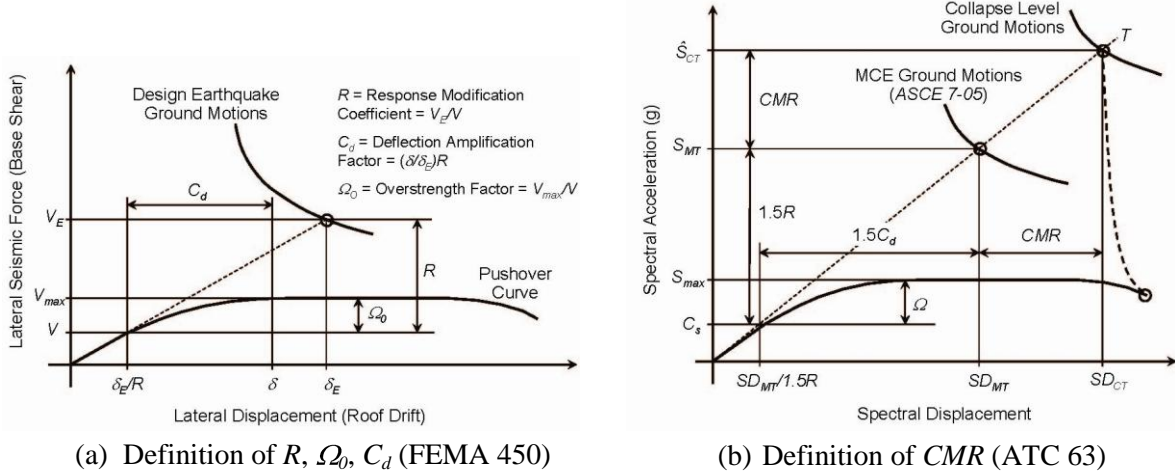

(a) Definition of R, Ω_0, C_d (FEMA 450)   (b) Definition of CMR (ATC 63)

Figure 4. Seismic Performance Factors (ATC, 2009)

(3) Develop index archetype models providing the generic idealization of the archetypical characteristics and capturing significant behavior modes and collapse performance of the proposed seismic force-resisting system.

(4) Perform nonlinear static pushover analysis to validate the behavior of the nonlinear model and to estimate the system overstrength factor (Ω_o) and ductility capacity (μ_T) using a nonlinear static pushover curve as shown in Figure 4(a).

$$\Omega_0 = V_{max}/V \tag{1}$$
$$\mu_T = \delta_u/\delta_{y.eff} \tag{2}$$

(5) Perform nonlinear response history analysis using a set of Far-Field Maximum Considered Earthquake (MCE) level ground motions consistent with ASCE 7-05 provisions. In each analysis, the ground motion intensities are systematically increased using incremental dynamic analysis techniques to assess median collapse capacities and collapse margin ratios as shown in Figure 3(b) and Figure 4(b) for the index archetype model.

(6) Estimate the adjusted collapse margin ratio (*ACMR*) of the system which is calculated using the collapse margin ratio (*CMR*) and the spectral shape factor (*SSF*). The collapse margin ratio is defined as the ratio of median collapse level spectral acceleration ($\hat{S}_{CT}$) to the MCE ground motion demand (S_{MT}) at the fundamental period (T) of the structure. The median collapse level spectral acceleration ($\hat{S}_{CT}$) is defined as the median value of the estimated spectral accelerations at collapse (S_{CT}). Additionally, the unique characteristics of spectral shape for rare ground motions are captured using a spectral shape factor (*SSF*) which is a function of the fundamental period of the structure, ductility capacity and the seismic design category. Such that, the *ACMR* of the system is calculated as follows:

$$CMR = \hat{S}_{CT}/S_{MT} \tag{3}$$
$$SSF = exp\big[\beta_1(T) \times \big(\bar{\varepsilon}_0 - \bar{\varepsilon}(T)\big)\big] \tag{4}$$
$$ACMR = SSF \times CMR \tag{5}$$

Where,

$$\beta_1(T) = 0.14 \times (\mu_T - 1)^{0.42} \leq 0.317$$

$$\bar{\varepsilon}_0 = 1.0 \text{ for } SDC \text{ B and C, } 1.5 \text{ for } SDC \text{ D, } 1.2 \text{ for } SDC \text{ E}$$
$$\bar{\varepsilon}(T) = 0.6 \times (1.5 - T), \bar{\varepsilon} = 0.6 \text{ if } T \leq 0.5 \text{ and } \bar{\varepsilon} = 0.0 \text{ if } T \geq 1.5$$

(7) From the quality of the knowledge base associated with key elements of the system, estimate uncertainties for each parameter including (a) the variability between ground motion records ($\beta_{BTR} = 0.1 + 0.1 \times \mu_T \leq 0.4$ for $\mu_T \geq 1$), (b) the uncertainty in the nonlinear structural modeling assumptions (β_{MDL}), (c) the quality and extent of test data used to calibrate the element models (β_{TD}), and (d) the quality of the structural system design requirements (β_{DR}). Uncertainty definitions are defined as, Superior: $\beta = 0.10$, Good: $\beta = 0.20$, Fair: $\beta = 0.35$ and Poor: $\beta = 0.50$ except β_{RTR}. Since the uncertainties are statistically independent, the total collapse uncertainty parameter (β_{TOT}) is derived using the square root of the sum of the squares (SRSS) of each parameter as shown in Equation (6).

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{MDL}^2 + \beta_{TD}^2 + \beta_{DR}^2} \tag{6}$$

(8) Define the performance objectives by estimating the acceptable collapse margin ratio of the system. ATC-63 methodology defines two levels of collapse performance objectives: (a) a collapse probability of 20% or less for each archetype building ($ACMR_{20\%}$), and (b) a collapse probability of 10% or less on average for each performance group ($ACMR_{10\%}$) at a total collapse uncertainty (β_{TOT}) based on the lognormal distribution assumption of collapse level spectral intensities as shown in Figure 5 based on FEMA P695 Table 7-3 (ATC, 2009).
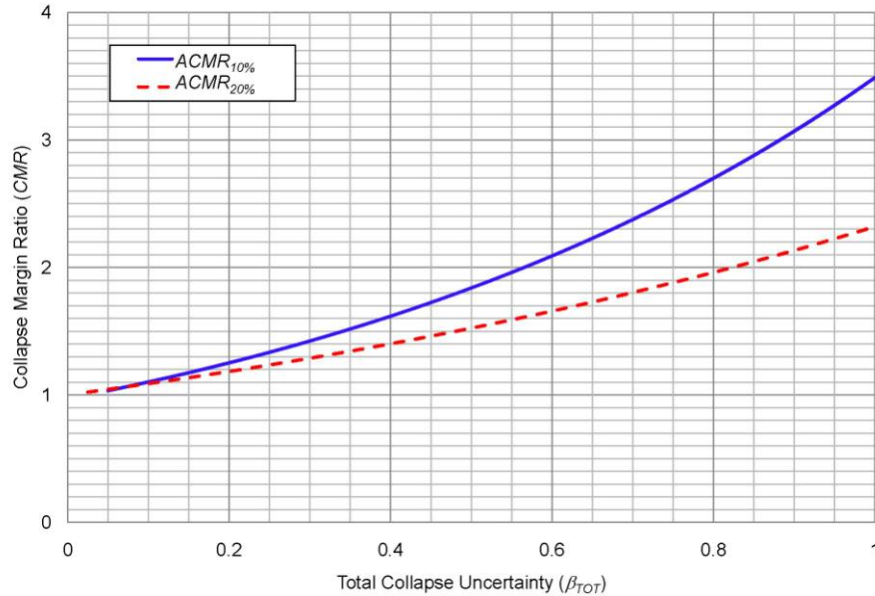

Figure 5. Acceptable Collapse Margin Ratio
(Ref. Table 7-3, ATC, 2009)

(9) Evaluate if the calculated *ACMR* is greater than the performance objective limits ($ACMR_{20\%}$ and $ACMR_{10\%}$). If acceptable, the seismic performance factors meet the collapse performance objectives. If the system does not meet performance objectives, the structural system concept and system design requirements must be redefined and reevaluated by either (a) modifying *R*-factor, (b) adjusting structural system design requirements, (c) improving the quality of test data, and/or (d) reducing the uncertainty in the structural modeling.

Proposed Methodology for Steel Diagrid Framed System

Since the ATC-63 methodology is expected to predict "equivalent" safety against collapse in a seismic event in a more reliabe and consistent manner, this study has adopted the ATC-63 approach in the estimation of appropriate seismic performance factors for steel diagrid framed systems. And again, recognizing the intent is to address a model building code system definition, it is assumed that the definition will be suitable to allow linear elastic analysis and design using code based methods for systems expected to perform with significant inelastic demands. Therefore, the proposed methodology in this study (a) identifies the seismic force-resisting system, (b) develops structural system archetypes and index archetype model, (c) estimates R-factors iteratively using the method shown in FEMA-450 (FEMA, 2003), and (d) confirm the estimated seismic performance factors to meet the performance objectives to achieve an equivalent safety against collapse with the seismic-force resisting systems in the model building code.

System Identification. For this study, it is assumed that archetype steel diagrid framed structural systems considered are located in a region of high seismicity with a seismic design category of "D" ($SDC = D$). Also, it is assume that the exterior diagrid frames are the only seismic lateral force-resisting system. Typically, configurations of exterior diagrid frames provide regularity creating redundant load paths for both gravity and lateral loads. Architecturally, it is of interest to consider structural system that will have variations in geometrical properties and configurations, such as, variation of building height-to-width aspect ratios, and, angle of sloped columns with intervals of diagrid elements over story heights as shown in Figures 6 and 7. The seismic energy applied to the system is expected to be dissipated by a combined flexure and axial post-buckling/tension yielding hysteretic behavior of diagonal members. The system is expected to follow the framework of ASCE 7-05 for system analysis and design requirements and AISC 341-05 for detailing requirements.
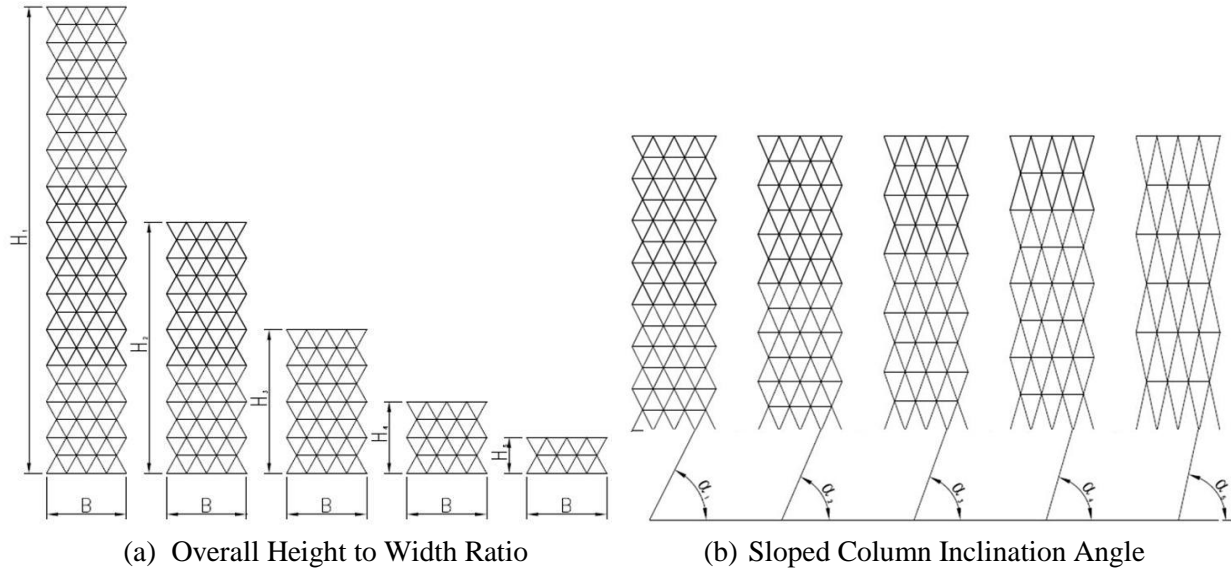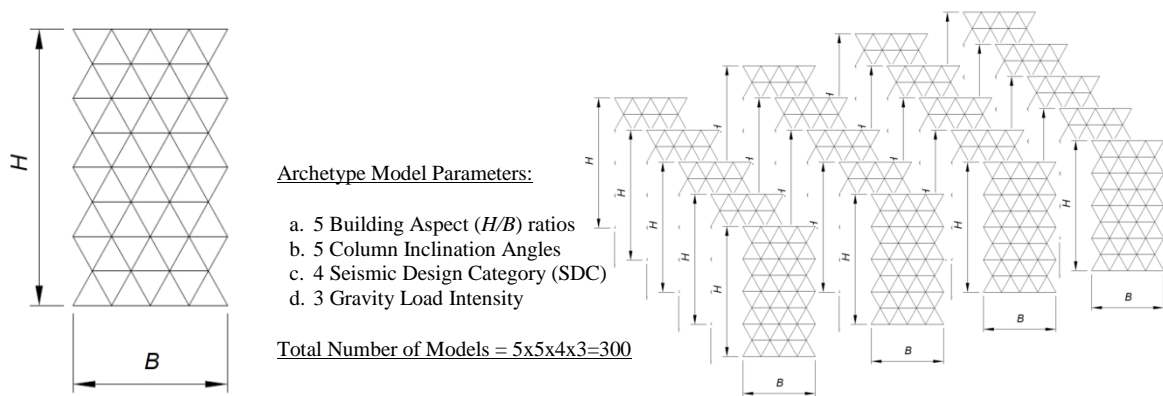

    (a) Overall Height to Width Ratio   (b) Sloped Column Inclination Angle

Figure 6. Parameters Of Interest For Diagrid

(a) Index Archetype Model   *(b) Number of Archetype Models*

Figure 7. Development of Archetype Models

Archetype Models. Initial building analysis model investigations have considered the development of a representative index archetype model, as shown in Figure 7(a). Additional archetype models may be investigated in terms of key performance parameters including varying building aspect (*H/B*) ratios, column inclination angles, seismic design categories (*SDC*), and variations in gravity load intensity and associated member sizes. The index archetype model will be used for generating a set of archetypical configurations consistent with the design requirements and general application of the system.

Example Archetype Model Evaluation. An 8-story steel diagrid framed building is selected as an archetype model based on generic frames defined in the FEMA program (FEMA-355C, 2000). Since the 8-story building can be considered as a mid-rise building in the FEMA model buildings, it follows to use this model as a basis for characterizing both high rise building and low-rise building frames in future archetype modeling. The plan dimension is 150 feet by 150 feet on grid lines with 12 inches of slab overhang beyond the grid lines as shown in Figure 10(a) with an assumed story height of 15 feet at all levels for the simplicity. Typical column configuration has a fixed slope (15 feet horizontal and 30 feet vertical, approximately 63.4 degree to ground level plane). As perimeter steel diagrid frames are the only seismic force-resisting frame, there are no additional columns within the diagrid frame with a clear span to interior core gravity columns as shown Figure 10(b). Typical diagrid frame elements are W14 steel rolled shapes (Gr 50 ksi). The estimated typical dead load and live loads are 130 psf and 80 psf respectively are assumed for the study. Typical dead loads include assumptions for architectural exterior wall, partitions, as well as, other superimposed dead loads including ceiling, mechanical and electrical building systems.

The building is assumed to be located at high seismic zone, having design spectral acceleration parameters at short periods (S_{DS}) and at a period of 1 second (S_{D1}) is 1.0g and 0.6g based on the ASCE 7-05, respectively, and frames are analyzed based on linear elastic dynamic response spectrum at the Design Earthquake level per ASCE 7-05. The structural member design was performed using ETABS program and designed per AISC 341-05 provisions. The nonlinear analysis was undertaken using the PERFORM-3D program based on its superior capability to model the material nonlinearity of individual structural elements using fiber element modeling. The inclined column elements are modeled using a "Column, Inelastic Fiber Section" based on material properties defined as "Inelastic Steel Material, Buckling" in PERFORM-3D modeling space. This definition of section properties captures the tension yielding in a manner of idealized elasto-perfectly plastic yielding while the buckling strength of members is idealized based on the limited compression stress of $0.60F_y$. The force-deformation relationship of typical column element modeling is shown in Figure 8 for both axial stress-strain and flexural moment relationships. The horizontal beam spandrel elements have been modeled as linear elastic elements providing lateral bracing

for column elements and diaphragm collectors but not considered as participating in the energy dissipation of earthquake loads.
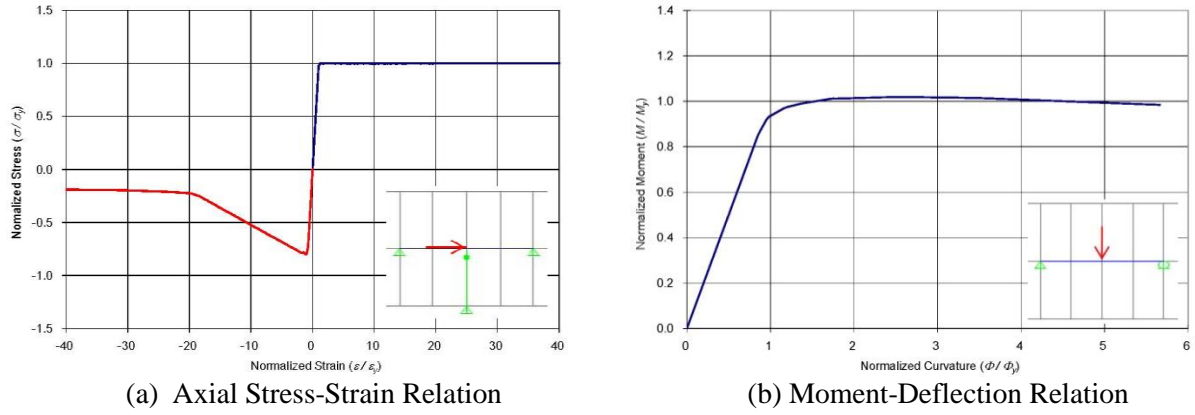

(a) Axial Stress-Strain Relation (b) Moment-Deflection Relation
Figure 8. Element Behavior in Analysis Model
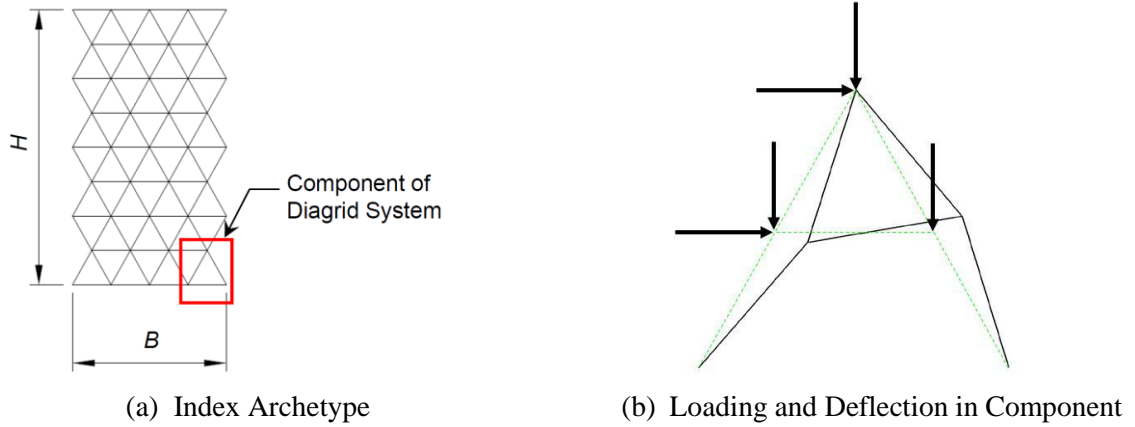
(a) Index Archetype (b) Loading and Deflection in Component
Figure 9. Minimum Component of Diagrid Frame

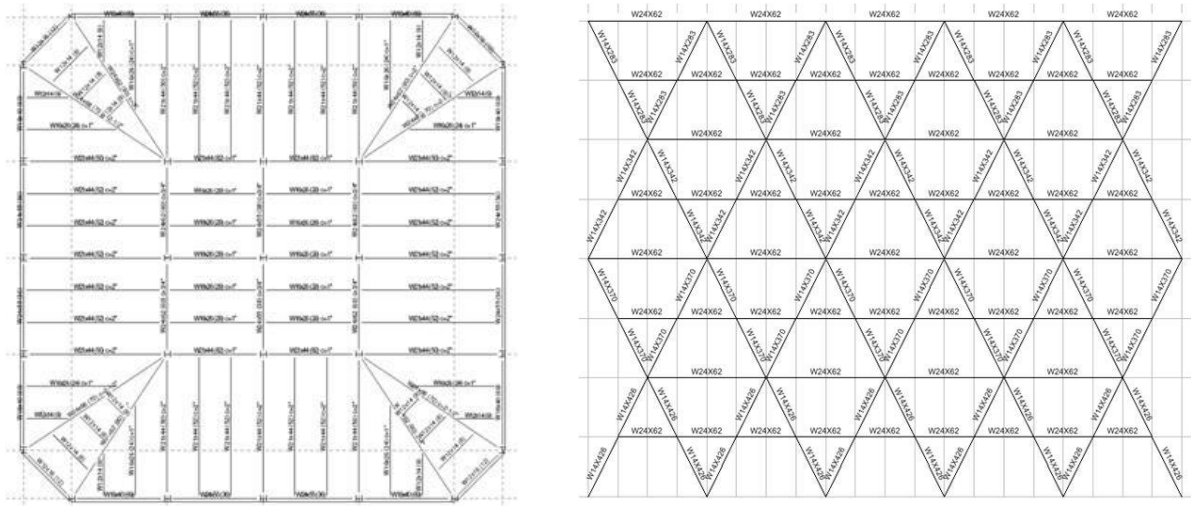(a) Typical Floor Plan Framing (b) Typical Exterior Framing Elevation
Figure 10. Typical Framing of 8-Story Archetype Model

Derivation of R-Factor

Since the steel diagrid framed system is not a prescribed seismic force-resisting system per ASCE 7-05 provisions, the value of the R-Factor is estimated for initial design purposes to perform nonlinear response history analysis. The initial R-Factor estimate is performed using nonlinear static analysis methods based on FEMA 450 procedures. An iterative procedure is utilized for a given index archetype model with assumed detailing and system design requirements until the R-factor converges as shown in Figure 11. With a derived R-factor, the overstrength factor (Ω_o) and period-based ductility (μ_T) can estimated from the static pushover curve drawn from the nonlinear static analysis as shown in Figure 12.

Summary of Results

Seismic Response Factor, $R = 3.64$ from Figure 11
Fundamental Period, $T = 0.8$ sec
Base Shear, $V = 0.11$g, $V_{max} = 0.16$g, $0.8V_{max} = 0.13$g from Figure 11
Displacement, $\delta_{y,eff} = 3.8$ inch, $\delta_u = 14.0$ inch from Figure 11
System Overstrength Factor, $\Omega_o = V_{max}/V = 0.16/0.11 = 1.5$
Period-based ductility, $\mu_T = \delta_u / \delta_{y,eff} = 14/3.8 = 3.7$
Spectral Shape Factor, $SSF = exp[0.212 \times (1.5 - 0.42)] = 1.26$ from Equation (4)

$$\beta_1(0.8) = 0.14 \times (3.7 - 1)^{0.42} = 0.212$$
$$\bar{\varepsilon}_0 = 1.5 \text{ for } SDC\ D$$
$$\bar{\varepsilon}(0.8) = 0.6 \times (1.5 - 0.8) = 0.42$$

Collapse Uncertainties

The individual and total uncertainties used in the example seismic performance evaluation are based on the following assumptions:
(a) variability between ground motion records, $\beta_{RTR} = 0.40$ (period-based ductility, $\mu_T \geq 3$)
(b) uncertainty in the nonlinear structural modeling, $\beta_{MDL} = 0.20$ (modeling related uncertainty is judged good)
(c) quality of test data used to calibrate the element models, $\beta_{TD} = 0.20$ (high confidence level in test results and most important testing issues are addressed)
(d) quality of the structural system design requirements, $\beta_{DR} = 0.20$ (high confidence level in basis of design requirements and reasonable safeguards against unanticipated failure modes)
(e) total collapse uncertainty parameter, β_{TOT}, using Equation (6)

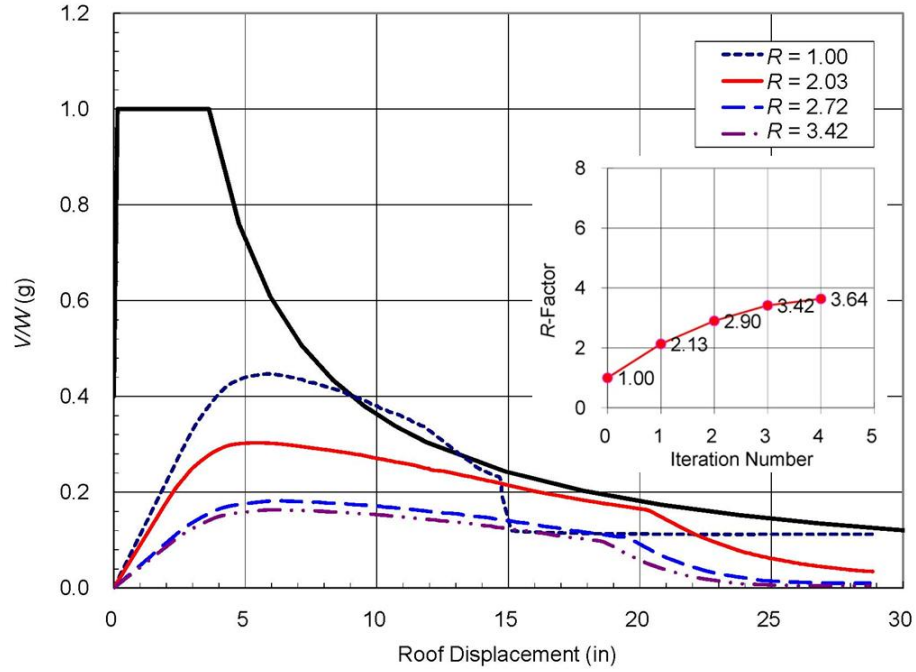$$\beta_{TOT} = \sqrt{0.40^2 + 0.20^2 + 0.20^2 + 0.20^2} = 0.53$$

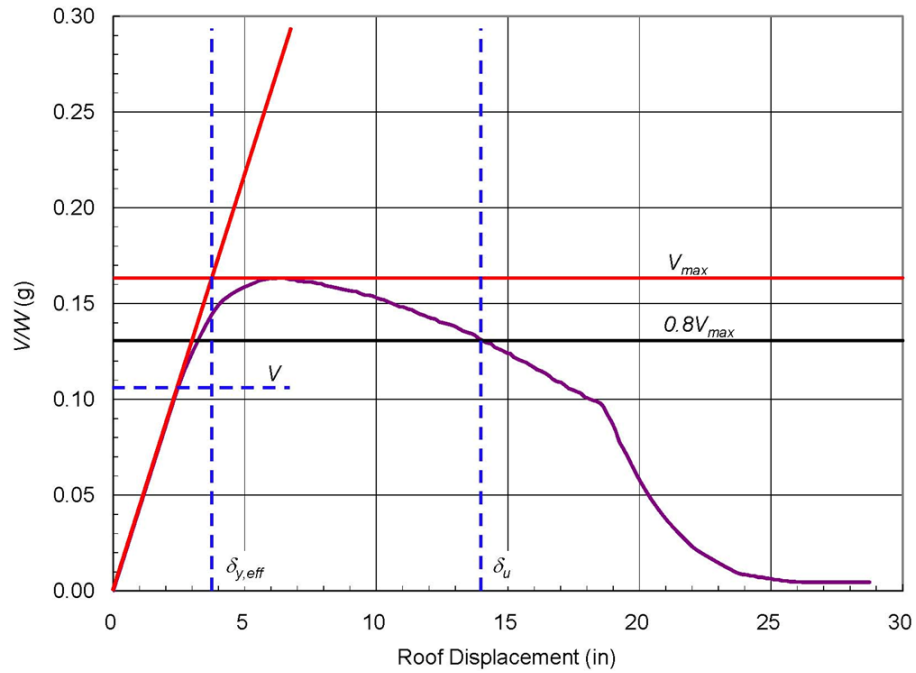*Figure 11. Iterative Estimation of R-Factor*

Figure 12. Nonlinear Static Pushover Curve

Evaluation of Collapse Margin Ratio Criteria

The collapse capacity of the system is estimated from the results of nonlinear response history analysis using a set of Far-Field MCE level with increasing intensity of ground motions such as incremental

dynamic analysis techniques. If the collapse margin ratio, *CMR*, is calculated as a 1.60 from the nonlinear response history analysis results, the *ACMR* is calculated as follows:

$$ACMR = SSF \times CMR = 1.26 \times 1.60 = 2.0$$

The collapse demand of the system is estimated on the basis of the collapse performance objectives of ATC-63 methodology. Since the current study addresses the performance of only a single archetype model, the governing collapse performance objective is a collapse probability of 10% for each performance group ($ACMR_{10\%}$) at the total collapse uncertainty (β_{TOT}). The $ACMR_{10\%}$ at the $\beta_{TOT} = 0.53$ can be read as 1.96 from the Figure 13. While the expected collapse margin ratio is larger than the target collapse margin ratio at the collapse probability of 10% or less, the estimated seismic performance factors for steel diagrid framed system can be used in a similar "equivalent" manner as the factors prescribed in the building code provisions.
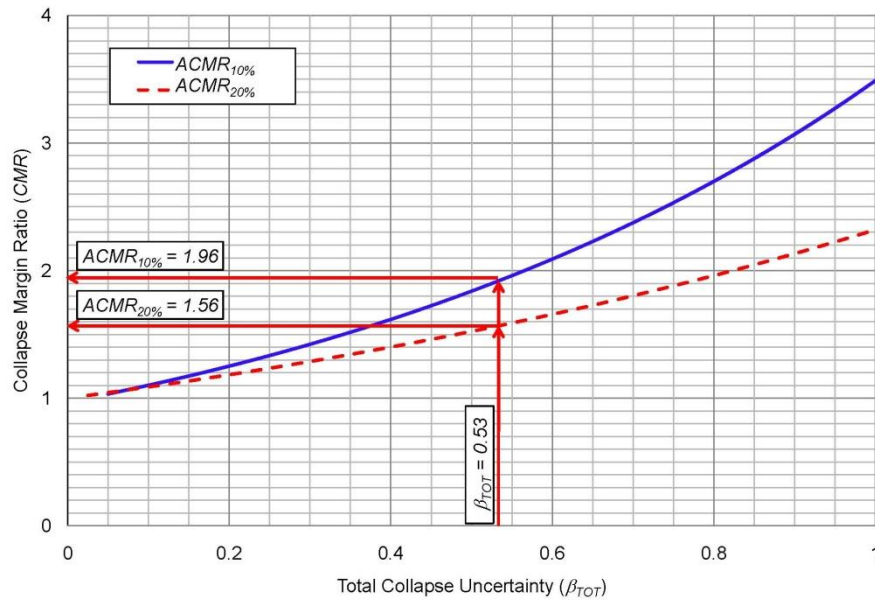

Figure 13. Estimation of Collapse Margin Ratio Criteria
(Ref. Table 7-3, ATC, 2009)

Evaluation of Project Specific Buildings

While the purpose of this study was to investigate the application of ATC-63 methodology to define seismic performance factors for steel diagrid framed systems as "new undefined" systems for inclusion in model building codes, it may be desirable to evaluate expected seismic performance of individual specific building projects. Alternatively, the methodology may also be used to evaluate project specific structural systems designed to meet code provisions to achieve intended seismic performance objectives as outlined in Appendix F of the ATC-63 (ATC, 2009).

Summary and Conclusions

This paper investigated a proposed methodology to determine seismic performance factors for steel diagrid framed systems by introducing the ATC-63 methodology addressing the application of steel diagrid framed systems located in regions of high seismicity.

The current study proposes a methodology that adopts and bridges the technologies of two related and established methods contained in both FEMA-450 and ATC-63. The FEMA 450 methodology provides a

reasonable initial R-Factor based on analytical assumptions only, while the ATC-63 methodology provides a more reliable basis to confirm the estimated R-Factor using a well defined and reproducible evaluation procedure. The ATC-63 methodology provides a sound basis incorporating key element uncertainties to ensure reliable estimation of probabilities against collapse to establish "equivalent" performance with well established and reliable systems defined in model building codes. For an undefined seismic force-resisting system, such as steel diagrid framed systems, several iterations are required to determine acceptable seismic performance factors (R, Ω_o, C_d).

Additionally, this investigation suggests an overall approach that allows a significant reduction in computational effort through an iterative approach to establish R-Factors based on a nonlinear static analysis rather than a more cumbersome iterative approach using nonlinear dynamic response history analysis.

Limitations and Recommended Further Investigations

The scope of this study is limited. Recommended further investigations are required to study further the general application of the seismic performance factors for steel diagrid framed systems suitable for use in a model building code. Items requiring additional investigation may include the following:

1. Influence of the frame extension in vertical direction.
2. Influence of the strain-hardening effect in tension, post-buckling behavior in compression, and strength degradation under cyclic loading.
3. Influence of discrepancy of material properties between nominal and expected properties.
4. Influence of modeling uncertainties including correlation and applicability with available component test data and additional testing.
5. Dynamic response history analysis including increased ground motions using incremental dynamic analysis techniques.
6. Consideration of varying ground motion intensities.
7. Reliable modeling of nonlinear component behavior based on results obtained from rigorous testing of all applicable archetype elements and subassemblages.
8. To fully define seismic performance factors for a steel diagrid framed system suitable for use in a model building code, extensive archetype model evaluations are required that represent reasonable range of possible application.
9. Peer review at each stage of development with sufficient expertise and oversight addressing key elements of the methodology including ground motions, testing, nonlinear modeling and design procedures.

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