Earthquakes and Structures, *Vol. 10, No. 4 (2016) 735-755*
DOI: http://dx.doi.org/10.12989/eas.2016.10.4.735

Evaluation of seismic performance factors for steel DIAGRID structural system design

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(Received June 6, 2014, Revised November 18, 2015, Accepted December 3, 2015)

Abstract. This article presents a proposed analytical methodology to determine seismic force-resisting system *R*-values for steel 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 this study is to provide a sound and reliable basis for defining such seismic design parameters. An approach and methodology for the reliable determination of seismic performance factors for use in the design of steel diagrid framed structural systems is proposed. The recommended methodology is based on current state-of-the-art and state-of-the practice methods including structural nonlinear dynamic analysis techniques, testing data requirements, building code design procedures and earthquake ground motion characterization. In determining appropriate seismic performance factors (R, Ω_O, C_d) for new archetypical building structural systems, the methodology defines acceptably low values of probability against collapse under maximum considered earthquake ground shaking.

Keywords: diagrid; seismic performance factor; structural system; analytical methodology

1. Introduction

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 beam frame configurations called diagrids (Mele *et al*. 2012, Moon *et al*. 2007, Elnashai and Sarno 2008). These triangulated diagrid frames are most often placed on the building perimeter creating efficient structural systems in resisting both gravity dead and live loads, as well as, resisting lateral wind load requirements (Petrini and Ciampoli 2012). The triangulated sloped and varying geometries made up of column and beam frame elements are typically efficiently constructed from structural steel wide-flange, box or circular rolled shapes and welded plate connections. Computational design and automation of one-of-a-kind building systems provides a particularly challenging problem realizing the promising diagrid framed systems with varied

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http://www.techno-press.com/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

geometries.

Unique to varying steel diagrid framed system configurations is that both gravity and lateral loads are distributed in the triangulated sloped column and beam elements. The load path of the frame elements consists primarily of axial compression and tension loading. Under these load conditions, according to current AISC (AISC 360-05, 2005) provisions, the diagrid frame elements are designed to remain linear elastic with appropriate factors of safety. However, under moderate to extreme earthquake ground shaking demands, the lateral frame seismic force-resisting system must provide sufficient ductility and energy dissipation characteristics of the structural system to provide life safety against collapse while undergoing inelastic frame deformations (Hejazi *et al*. 2013).

In the consideration of steel diagrid framed systems, current model building codes do not explicitly address the seismic design performance factors for this new and emerging structural system. Moreover, seismic design criteria are commonly decided by the experience of the earthquake on the basis of experimental studies and the type of structures. Due to the axially loaded sloped column elements of the steel diagrid frame system subjected to sustained gravity loads, it is expected that the system will exhibit low-ductile behavior under combined axial and pinned-connected post-buckling loading. Factors that will affect the ability of a steel diagrid frame system to exhibit adequate ductility and energy dissipation behavior under seismic loads include the level of seismic design force reduction (*R*-value, i.e., response modification factor), component detailing, slenderness effects (Kl/r, here Kl: effective length of the column, r: radius of gyration of the cross section about the axis of bending) of sloped column elements and redundancy of the layout of structural system.

Today, perimeter steel diagrid type braced frame configurations are often combined with framed building cores that provide building code permitted dual systems (Winter 2011, Lee *et al*. 2012, Lee *et al*. 2014a, Lee *et al*. 2014b). Core frames may consist of ductile steel moment and braced frames, reinforced concrete wall and steel-reinforced concrete composite systems that serve to provide ductile behavior and redundancy in resisting seismic loads (Elnashai and Di Sarno 2008). For the purpose of the proposed methodology to determine appropriate *R*-values for steel diagrid frame systems, it is assumed the building frame system consists of a single diagrid framed system in each building principal direction rather than dual systems combined with special moment resisting systems as permitted by the building code. The approach encompasses standard seismic analysis and design procedures relying on established consensus based seismic design standards and steel design specifications including ASCE 7-05 (2005), "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-05), consistent with the provisions of the "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" (FEMA 450, 2003) and the International Building Code - 2006 Edition (IBC 2006). Structural steel design procedures conform to the minimum requirements of the American Institute of Steel Construction, "Specification for Structural Steel Buildings" (ANSI/AISC 360-05, 2005), and, "Seismic Provisions for Structural Steel Buildings" (ANSI/AISC 341-05, 2005).

Typical building code (IBC 2006) seismic force-resisting systems as defined in ASCE 7 (AISC 7-05, 2005) provisions Table 12.2-1 provide code prescribed seismic performance factors for design of new building structures. These include a Response Modification Coefficient, *R*-value; System Over-strength Factor, Ω_O; and, Deflection Amplification Factor, C_d. These parameters provide a measure of system reduction in elastic load response levels due to inherent system ductility and energy dissipation capacities (Oosterhuis and Biloria 2008, Nuti *et al*. 2010, Dougka *et al*. 2014). These systems are generally well-defined with expected system and component behavior under inelastic seismic deformations. Also, as shown in Table 12.2-1, limitations are provided including restrictions on building height depending on level of seismic hazard as defined by the Seismic Design Category (SDC), where SDC “B” & “C” are generally categorized as “low seismic” and SDC “D” typically as “high seismic”.

The purpose of the code-based methodology is to substantiate that R-values greater than 1.0 for steel diagrid frame systems are reliable by characterizing component behavior based on test data, establishing design provisions, defining archetype models representing a range of geometric diagrid frame systems, and, assessing probability of collapse under MCE ground motions using nonlinear incremental dynamic analysis techniques. Thus, if properly implemented, the methodology can be utilized to define model building code level seismic performance factors (R *is* response modification factor, Ω_O is overstrength factor, and C_d is deflection amplification factor.) for steel diagrid frame system with an acceptably low probability against collapse under maximum considered earthquake ground shaking.

This study is divided into 6 Sections. In Section 2, steel diagrid structural systems are described conceptually considering design parameters and archetype models. Section 3 presents ATC-63 (2007) procedures and definition of seismic performance factors as a key to the proposed methodology to determine seismic force-resisting system R-values. In Section 4, analytical archetype modeling and numerical experiments are applied to an illustrative case study of the proposed methodology, satisfying technical design conditions shown in Section 3. The conclusions are presented in Section 5.

2. Steel DIAGRID structural systems (SDSS)

2.1 Design parameters

According to height-to-width ratio, steel diagrid frame system is shown in Fig. 1.

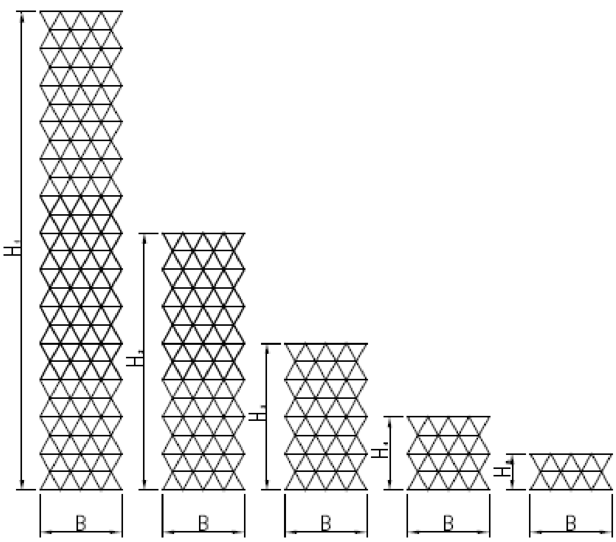


Fig. 1 DIAGRID frame system height-to-width ratios

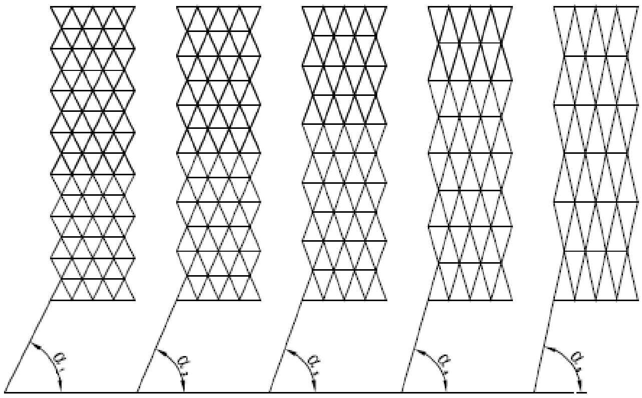


Fig. 2 DIAGRID frame configurations with varying sloped column inclination angles

In order to estimate steel diagrid framed systems, design parameters of interest can be described as follows.

- Overall height (H) to width (B) building aspect ratio (Fig. 1)
- Sloped column inclination angle (Fig. 2)
- Archetype analysis model (Fig. 3)
- Structural component behavior

In considering the number of required archetype models, the effectiveness of the diagrid frame configuration as a function of varying sloped column inclination angles is addressed. The optimum behavior under elastic gravity and wind loads may differ from seismic inelastic demands.

2.2 Archetype analysis models

From the consideration of varying diagrid framed systems and design parameters of interest, a series of possible archetype models is defined. Each archetype model is then designed to meet the applicable seismic design provisions using ASCE 7-05 (2005) requirements. The archetype models are selected based on a range of applications and expected seismic behavioral aspects of the system. Development of the archetype models begins with definition of an idealized model that reflects the expected behavior that impact the collapse response of the structural system.

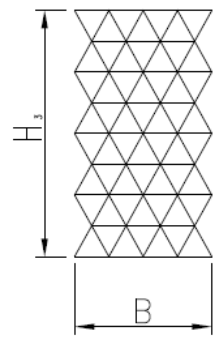


Fig. 3 DIAGRID frame archetype analysis model

3. Defining seismic performance factors

Technical approach of ATC-63 methodology considers key elements (Kircher and Heintz 2008) in Fig. 4 including MCE ground motions, nonlinear dynamic analysis (Dorvash *et al*. 2013), test data requirements, design information requirements, and peer review requirements. Flowchart (Deierlein 2007) of ATC-63 methodology and the present computational analysis procedures are shown in Fig. 5. This computational procedure concentrates on assessing collapse performance metric of archetype models through analytical approach to determine reliable *R*-value by carrying out both linear elastic response analysis of ETABS (Computers and Structures, Inc., 2000) and nonlinear static pushover analysis of Perform-3D in turn.

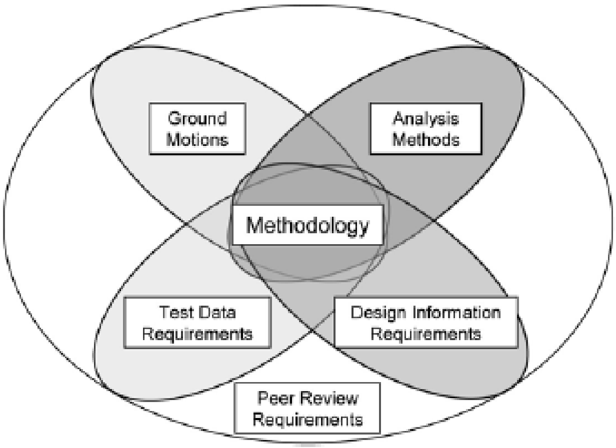
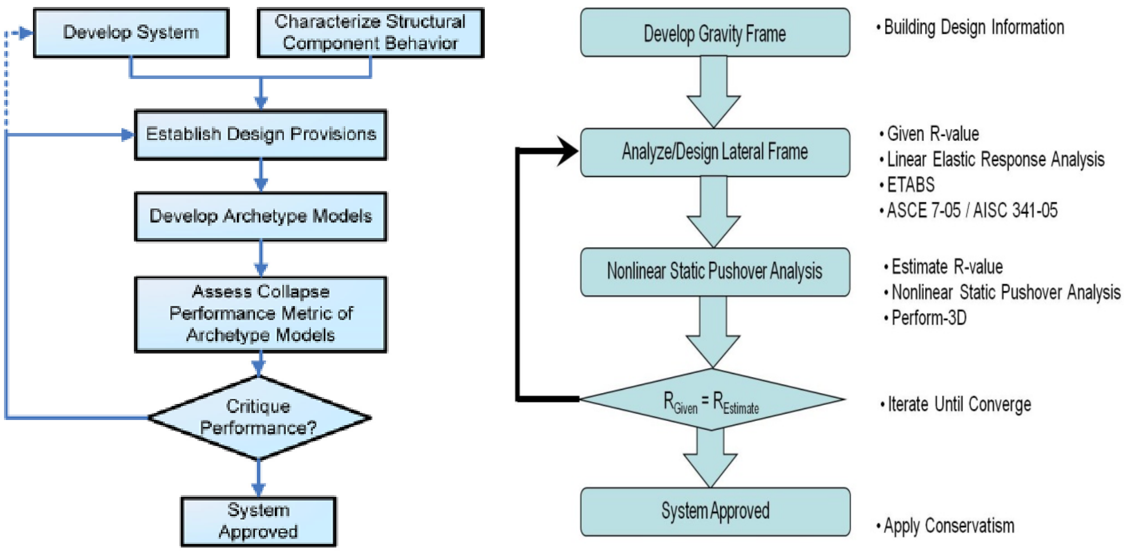


Fig. 4 Key elements of the ATC-63 methodology



(a) ATC-63 methodology (b) Analytical studies: analysis procedures

Fig. 5 Flowchart of ATC-63 methodology and the present computational analysis procedures

The validated test data can be used in conjunction with improved numerical models to help reduce uncertainties (Jiang and Adeli 2008, Lee *et al*. 2008) in the predicted response associated with modeling assumptions. Physical testing can also be utilized in the development of improved detailing of element components, sub-assemblages and connections for more predictable and reliable seismic performance factors (R, Ω_O, C_d).

Physical testing can be used to further validate modeling assumptions and requirements for steel diagrid framed systems. It is critical to characterize the post-buckling behavior of these steel member components including bi-axial bending as well as longitudinal axial load and local buckling effects.

Current nonlinear analysis computer programs such as, Perform3D (CSI 2007) and OpenSees (UC Berkeley 2006), utilize element yield surface fiber representations of member sections to capture triaxial P-M-M interaction including large-displacement buckling effects. Fiber element modeling may consider varying member section types, such as steel rolled wide-flange (WF), built-up plated box, and circular shapes.

Testing and archetype analysis modeling may also consider a range of diagrid beam-column element slenderness parameters including Kl/r=60 and Kl/r=180 depending on design of archetype models. For example, correlation of experimental testing on tubular steel tubular brace members with a slenderness ratio of 80 (Black *et al*. 1980) is considered, while analytical hysteretic modeling using OpenSees (UC Berkeley 2006) of the pin-ended tubular element is applied.

Limitations on available test data may require additional physical testing as necessary to validate component and frame archetype behavior. An example test frame set-up for a series of special concentric braced frame (SCBF) tests was conducted at UC Berkeley (Uriz and Mahin 2008) and by other researchers (Di Sarno and Elnashai 2009, Chen 2011, Sarma and Adeli 2002). The frame test set-up consists of a full-size two-story single bay chevron configuration SCBF.

Preliminary proposed testing for a diagrid archetype model may also consist of a full-size two-story single bay frame. Alternatively, a 1/5th scale four-story diagrid frame configuration may be tested based on the previous UC Berkeley test frame.

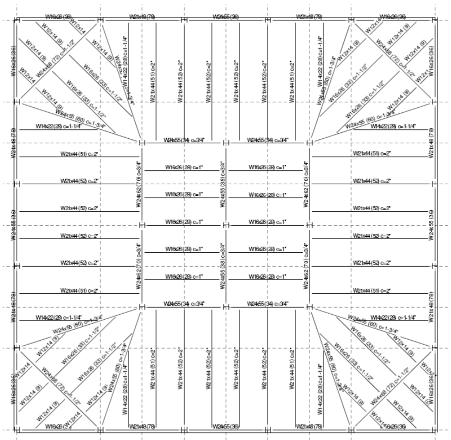
4. Analytical archetype modeling and numerical applications

4.1 General

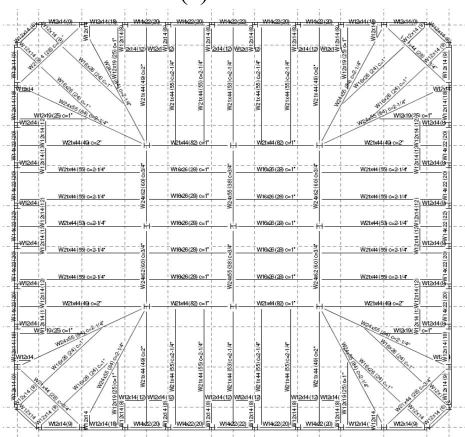
This study is a progressive research presenting a methodology to develop seismic performance factors, including seismic response modification coefficient (R-value), system overstrength factor (Ω_O) and deflection amplification factor (C_d), for a steel diagrid framed system. With the seismic performance factors, the equivalent seismic performance would be provided to new building to other buildings having seismic force resisting system provided in the model building code.

The seismic performance factors represent the inherent system ductility, seismic energy dissipation capacity, failure mechanism, past performance and so on. Therefore, the factors shall be developed through review of past performance and design practice, full scale sub-assemblage tests, analytical studies and peer reviews.

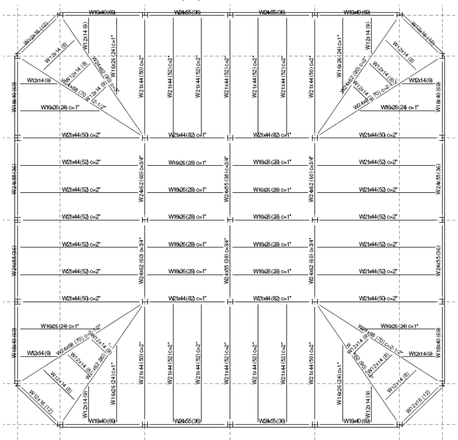
This study concentrates on the analytical studies. The parameters considered in the study are (1) the ratio of height-to-width (H/B), (2) the inclination of column, (3) the existence of secondary lateral force-resisting frame, (4) the existence of gravity column within diagrid frame, (5) the post-buckling stiffness of column, (6) the analysis methodology of structure, and (7) the combination of parameters above mentioned.



(a) Plan A



(b) Plan B



(c) Plan C

Fig. 6 Typical floor framing

4.2 Archetype model: 8-story building

An 8-story steel diagrid framed building is selected as an archetype model based on the generic test model of FEMA program (FEMA-355C, 2000). Because the 8-story building is considered as a mid-rise building in the FEMA model buildings, it can be expanded to high rise building and low-rise building in the future study. The plan dimension is 45.7 m by 45.7 m on grid lines with 30.4 cm of slab overhang beyond the grid lines. The story height is 4.6 m at all levels for the simplicity. The column has a fixed slope (4.6 m horizontal and 9.1 m vertical, ~63.4º to ground level), the steel diagrid frame is a lateral force- resisting frame, and there is no column within the diagrid frame as shown in the elevation. The estimated typical dead load and live loads are 6.2 kN/m^2 and 3.8 kN/m^2 respectively. The typical floor framing plans and typical exterior framing elevations are shown in Figs. 6 and 7. Typical floor plan and bay size are 45.7 m×45.7 m and 9.1 m×9.1 m, respectively. Typical story height is 4.6 m.

It is assumed that the building is located at San Francisco, CA, which is classified as a high seismic zone, and the building is sitting on the Site Class D soil condition with stiff characteristics, not soft, which is based on IBC 2006. The resulting design spectral acceleration parameters at short periods (SDS) and at a period of 1 second (SD1) are 1.000 g and 0.602 g based on the ASCE 7 (2005), respectively, and the response spectrum at the Design Basis Earthquake level with ground motion of 10% per 50 years for total 475 years is shown in Fig. 8.

Per the recommended methodology of ATC-63 (2007), lateral analysis is performed using ETABS (Computers and Structures Inc., 2000) through the elastic response spectrum analysis procedure per ASCE 7 (2005) with a trial *R*-value (*R*=1).

The demand-to-capacity ratios of the diagrid members are evaluated per AISC 360 (2005) assuming the other seismic force transfer system including foundation has enough capacity. One of the resulting frames is shown in Fig. 7.

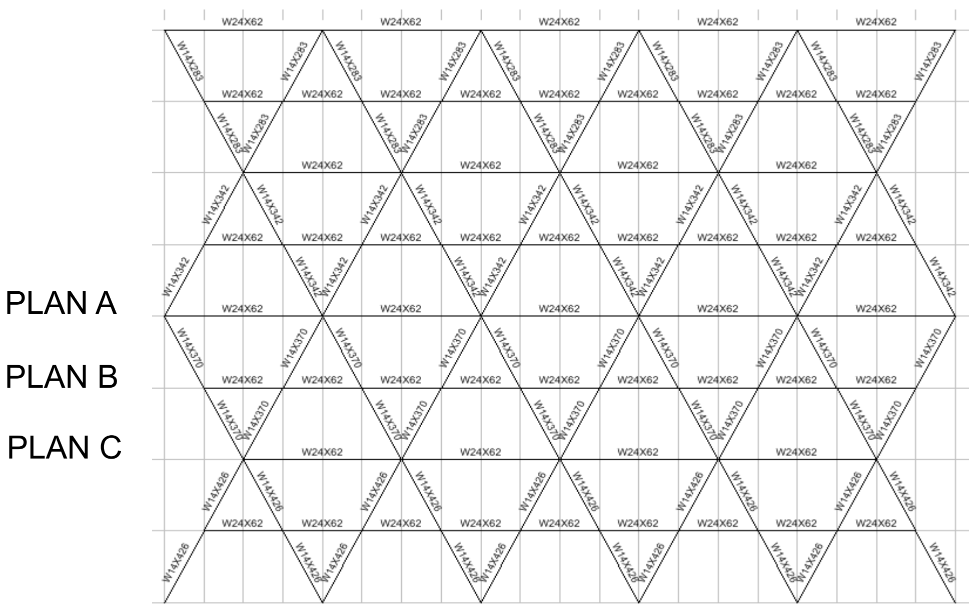


Fig. 7 Framing elevation of DIAGRID frame with *R*=1

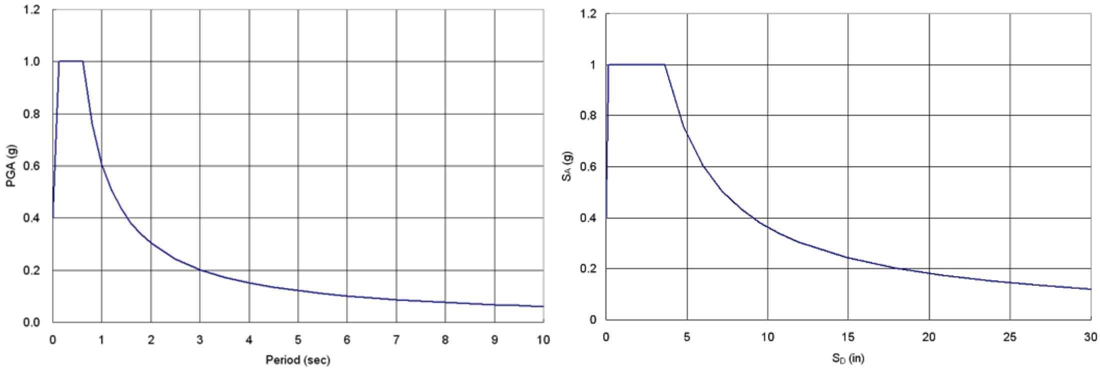


Fig. 8 Response spectrum-design basis earthquake (PGA: predicting ground motion, SA: spectral acceleration, SD: spectral displacement)

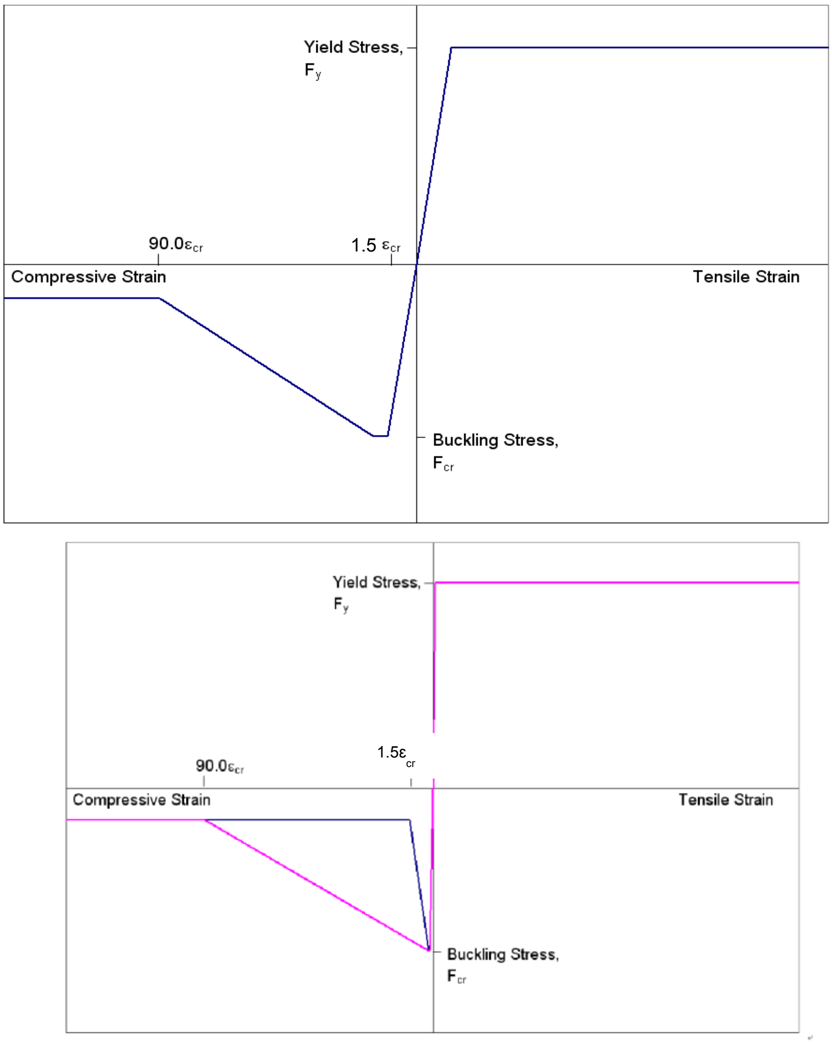


Fig. 9 Assumption of material properties

After R-value is evaluated, the structure can be re-framed with an updated R-value. From the study in the elastic frame analysis and design, the members at the upper level have more demand in moment (D/C=0.680) than demand in axial force (D/C=0.328) and the members at the lower level have more demand in axial force (D/C=0.730) than demand in moment (D/C=0.217).

4.3 Nonlinear finite element of computer software

As members of other seismic force-resisting frame dissipate the seismic force in nonlinear behavior, the stress in the diagrid frame members is also expected to behave beyond linear elastic limit. The material properties of sloped column in the diagrid frame is idealized as a linear elastic perfectly plastic in tension with a yield strength of F_y=345 MPa and an elastic modulus of E=199,948 MPa. For the compressive stress, the column of no-compact section is idealized as a linear elastic buckling at an assumed critical stress and strain of $\varepsilon_{cr}=E/F_{cr}$ and $1.5\varepsilon_{cr}$ at $F_1=F_{cr}=0.80F_y$, and $90\varepsilon_{cr}$ at $F_2=0.20F_{cr}$ as shown in Fig. 9. The beam is idealized as a linear elastic material since the beam is not considered as earthquake energy dissipaters. In diagrid system, all members are dealt with as beam-column.

The structures are evaluated with a nonlinear static analysis of pushover analysis (Fajfar 2005) with PERFORM-3D (CSI 2006). Nonlinear analyses can be static and/or dynamic, and can be run on the same model. Loads can be applied in any sequence, such as a dynamic earthquake load followed by a static pushover. The inclined column section is modeled with a "Column, Inelastic Fiber Section" and the section uses material properties called "Inelastic Steel Material, Buckling". This definition of section properties catches the tension yielding in a manner of linear elastic and perfectly plastic yielding but the compression buckling of member is simulated by limiting the compression strength.

The behaviors of material and element are verified with the simple element models. Fig. 10 shows the behavior of axial tension and compression in a simple tension and compression analysis model.

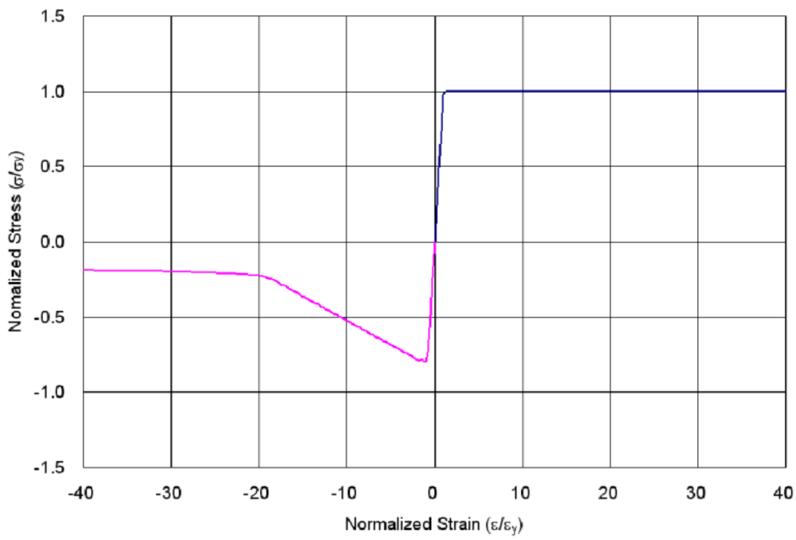


Fig. 10 Axial stress-strain relationship in the simulation model

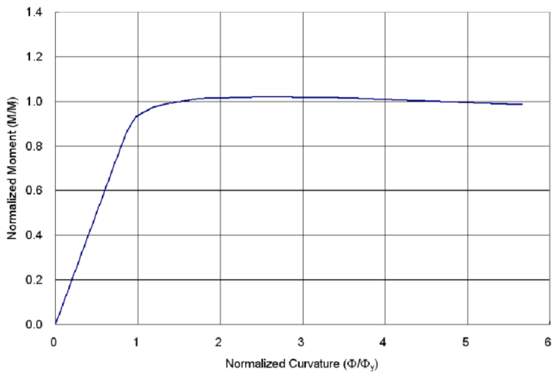
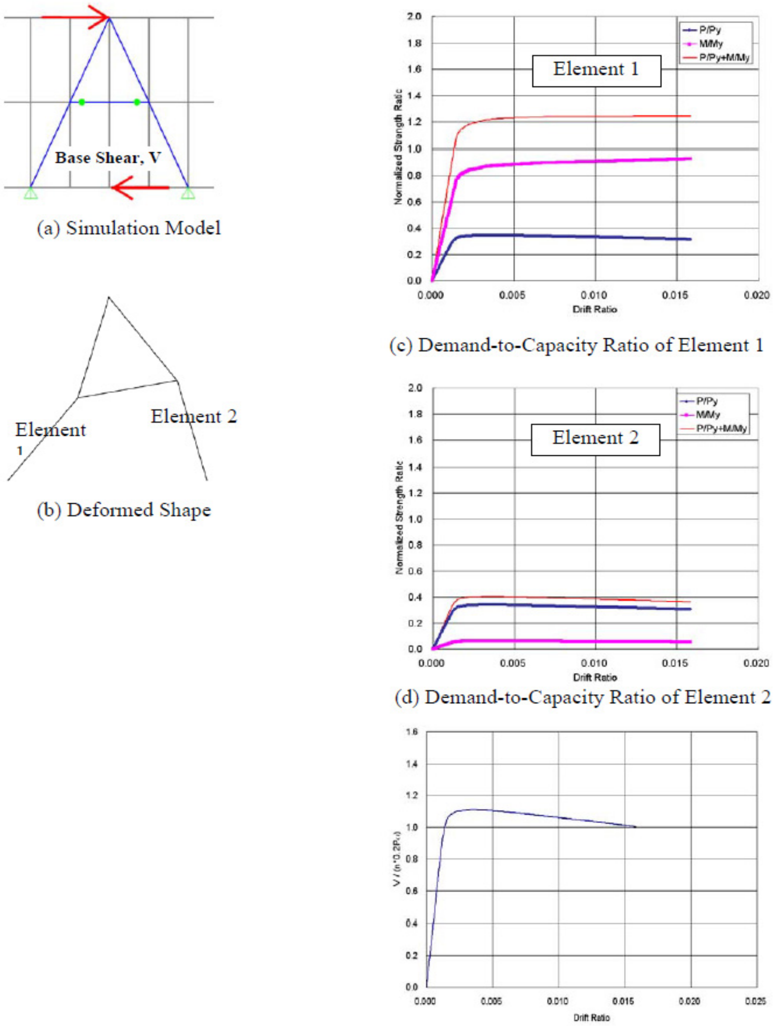


Fig. 11 Flexural moment-deflection relationship in the simulation model



(a) Simulation Model

(b) Deformed Shape

(c) Demand-to-Capacity Ratio of Element 1

(d) Demand-to-Capacity Ratio of Element 2

(e) Base Shear-to-Column Yield Stress Ratio

Fig. 12 Sub-assemblage model and analysis results

Fig. 11 shows the moment-curvature relation in a beam model with a single center loading. This may prove that all members are modelled as beam-column. It is found that the flexural strength is smaller than the nominal flexural strength of the beam since the compression strength of material properties is limited to the buckling strength of element.

The axial model and flexural model is combined to examine the beam-column behavior of the diagrid members as shown in Fig. 12. The un-deformed and deformed shapes are shown in Figs. 12(a) and 12(b). Fig. 12(c) shows the normalized strength ratio of Element 1, which is taking a combination of axial tension and moment. Fig. 12(d) shows the normalized strength ratio of Element 2, which is taking a combination of axial compression and moment. In the two Figs., it is shown that the Element 1 have more demand in moment than the demand in axial force and the Element 2 have more demand in axial force than the demand in moment. The maximum strength is reached when the axial compressive demand reached the buckling strength of the Element 2 as shown in Fig. 12(e).

4.4 Nonlinear behavior of archetype model

Nonlinear static analyses have performed with the 8-story building archetype model shown in Fig. 7 using a Perform-3D. The diagonal members are idealized with 24 inelastic fiber sections and the each fiber section has material properties of Fig. 9. The diagonal members are considered as a single element over the two stories and have idealized pinned boundary conditions when the diagonal members are cross each other. In this study, initial imperfection is not considered for all members due to design simplicity. The beam elements are idealized as a linear elastic material and have idealized pinned boundary conditions at each end. The gravity load is applied uniformly on the beam members considering the approximate tributary width (6.7 m-15.2 cm). The pushover analysis is performed after a code-based combined load of 1.2 Dead Load+0.25 Live Load is applied. Analyses are performed to check the influence of the lateral load patterns, triangular load pattern and uniform load pattern. The typical deformed shape of the frame under the triangular load pattern is shown in Fig. 13(a). The mechanism occurs at the roof level. Fig. 13(b) shows the typical deformed shape of the frame under the uniform load pattern. The mechanism occurs at the first level.

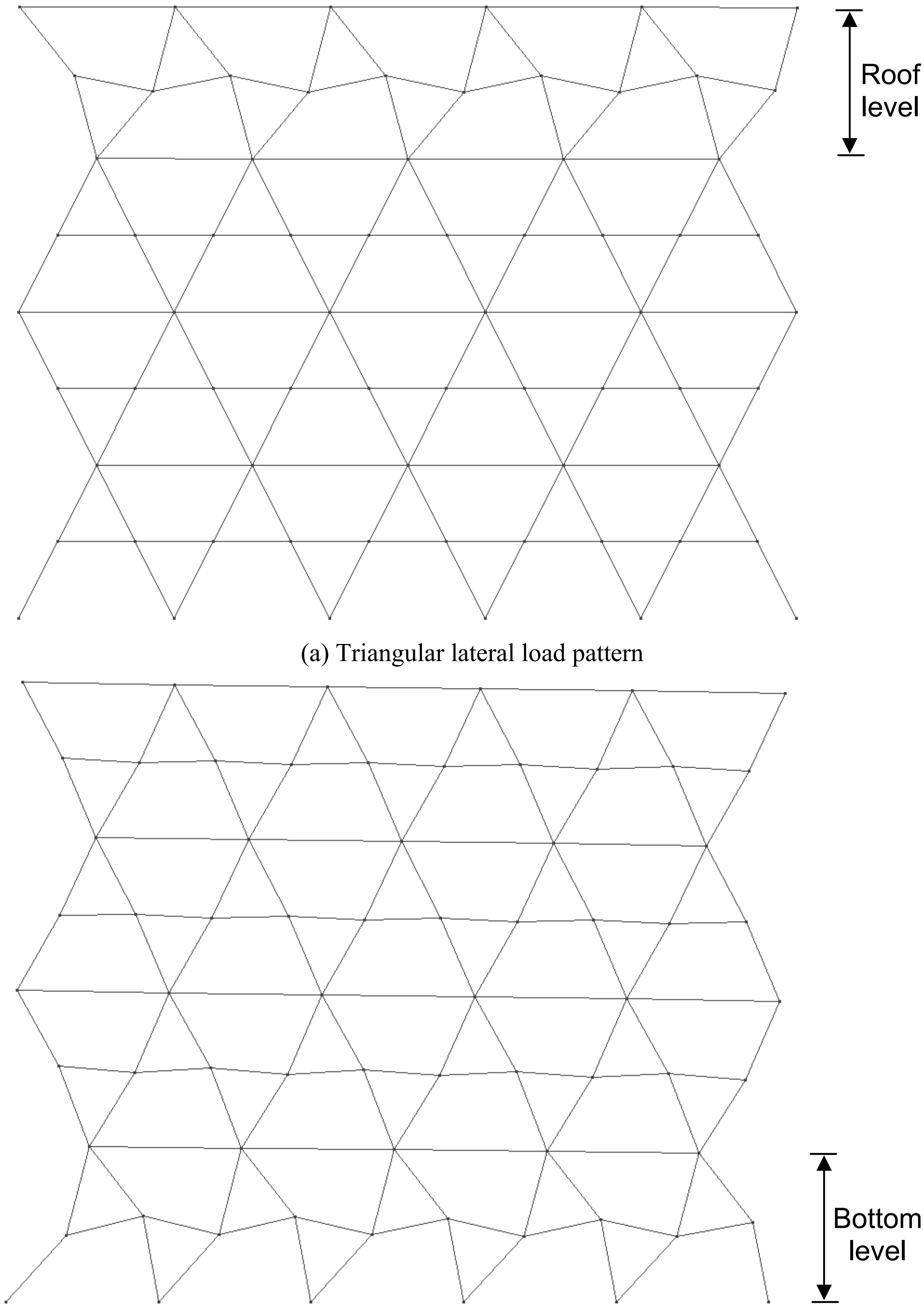
Fig. 14 shows the pushover curves with respect to the pattern of uniform load and triangular load.

Analyses are performed to investigate the influence of the gravity loads. In the pushover curves shown in Fig. 15, the solid lines are the analysis results without the gravity load. The initial stiffness is almost identical but the ultimate strength and post-buckling behavior are reduced by the gravity load.

The influence of the post-buckling stiffness of diagonal members is investigated. The material property of the diagonal members under the combined loading of axial compression and bending, the post buckling behavior is idealized as shown in Fig. 9.

The solid lines in Fig. 16 represent the sharp reduction of compressive strength after buckling. As shown in Fig. 16, the performance of the lateral frame is greatly influenced by the post-buckling behavior of the members with material A and B relying on slenderness and compact section. Therefore, it is required to verify the behavior through the sub-assemblage test.

The influence of the building height is investigated. Figs. 17 to 19 show the lateral framing using R=1 and the deformed shape at the mechanism. Fig. 20 summarizes the influence of building height to the performance of the building behavior and shows that there is a difference performance factor for the different height of the building.



(a) Triangular lateral load pattern

(b) Uniform lateral load pattern

Fig. 13 Deformed shape at the failure mechanism

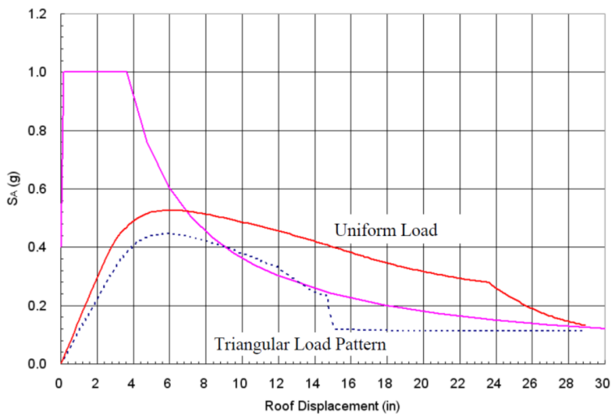


Fig. 14 Pushover curves of 8-story model

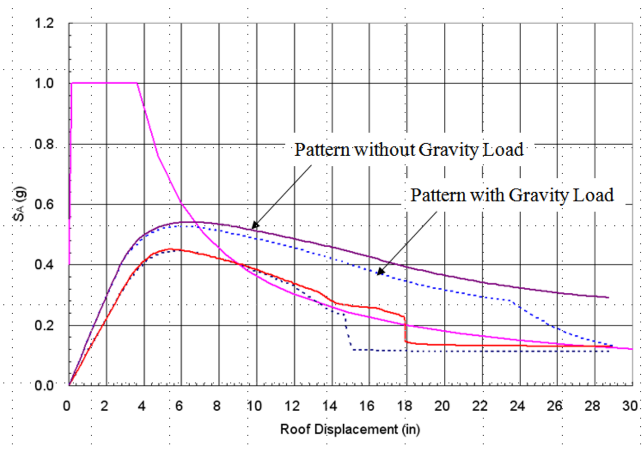


Fig. 15 Pushover curves of 8-story model : influence of gravity load

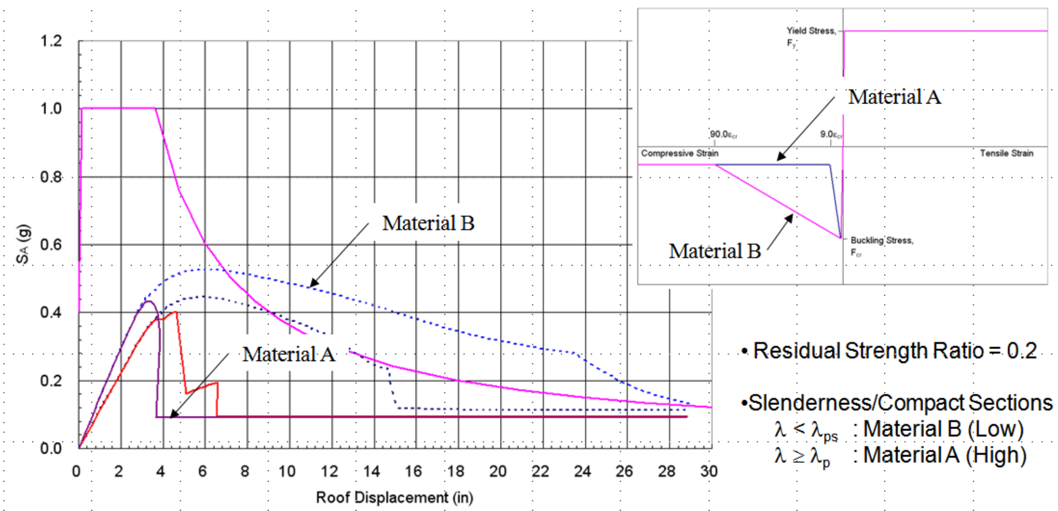
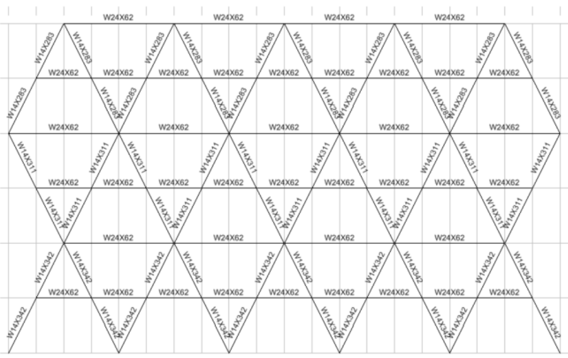
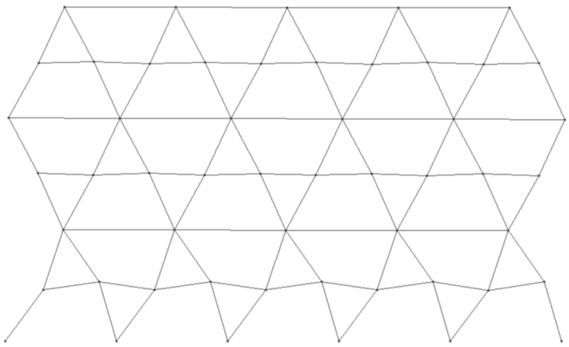


Fig. 16 Pushover curves of 8-story model: influence of post-buckling behavior

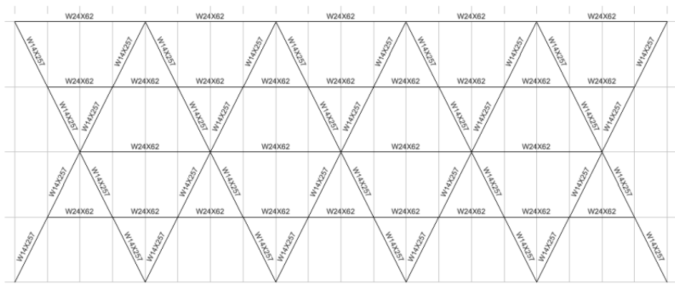


(a) Framing elevation of DIAGRID frame with *R*=1

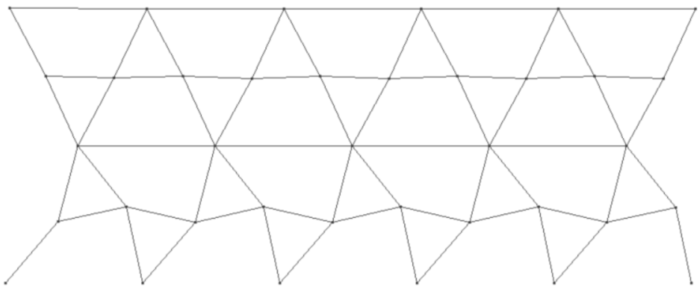


(b) Deformed shape at the failure mechanism

Fig. 17 6-story diagird frame analyse

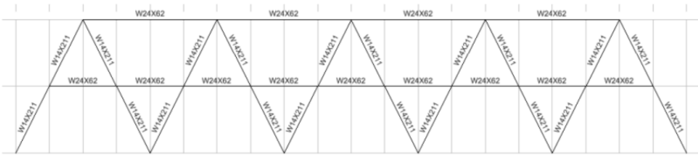


(a) Framing elevation of DIAGRID frame with *R*=1

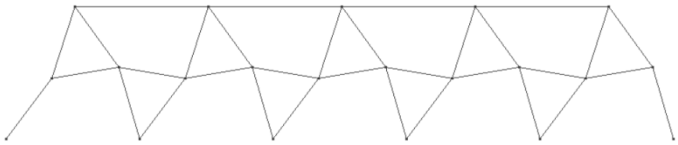


(b) Deformed shape at the failure mechanism

Fig. 18 4-story DIAGRID frame analyses



(a) Framing elevation of DIAGRID frame with *R*=1



(b) Deformed shape at the failure mechanism

Fig. 19 2-story DIAGRID frame analyses

4.5 Strength reduction factor: R-factor

The strength reduction factor (*R*-value) is calculated using the analysis result of the archetype model as shown in Fig. 14. Fig. 20 describes pushover curves depending on building heights. From the Fig. 21, the *R*-values are estimates as follows:

• Uniform Load Pattern: *R*=0.73/0.36=2.03
• Triangular Load Pattern: *R*=0.64/0.30=2.13

Since the initial *R*-value was assumed one, it is required to adjust the archetype model using the updated *R*-value, which can be 2.0. Using a new *R*-value, the diagrid frame is re-analyzed and designed for the new seismic model and is performed the pushover analysis with new member size. After four cycles of iterations as shown in Fig. 20, the *R*-value reaches to *R*=3.75 in case uniform load pattern and corresponding over-strength factor is estimated as Ω*o*=1.5 as shown in Fig. 22 and the related framing elevation is shown in Fig. 23.

• Uniform Load Pattern: *R*=0.45/0.12=3.75
• Triangular Load Pattern: *R*=0.40/0.11=3.63
• Over-strength Factor: Ω*o*=0.18/0.12=1.5

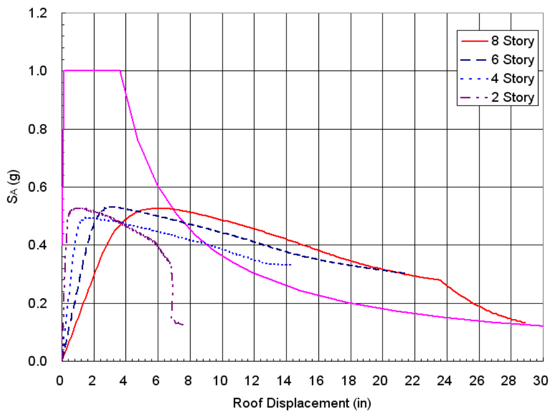


Fig. 20 Pushover curves: influence of building height

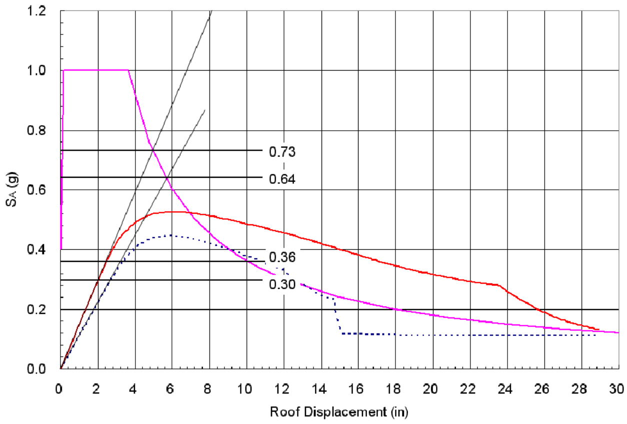


Fig. 21 Estimation of *R*-value: initial version

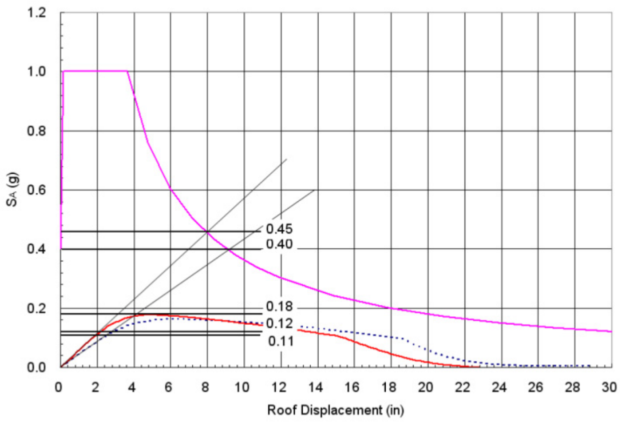


Fig. 22 Estimation of *R*-value: final version

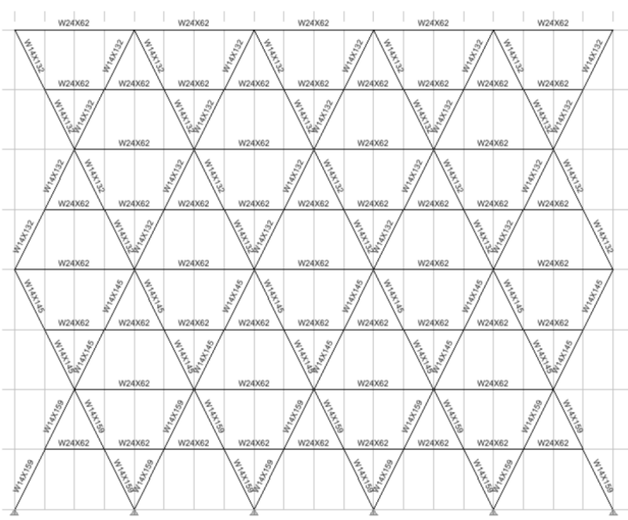


Fig. 23 Framing elevation of DIAGRID frame with *R*=3.75

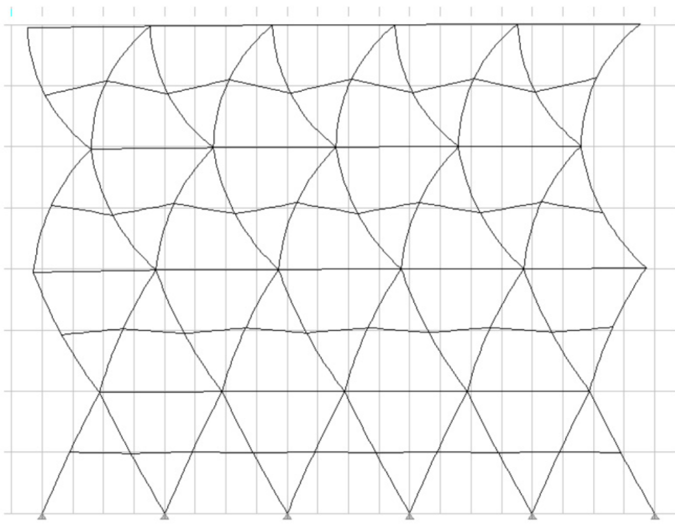


Fig. 24 1st mode shape of 8-story archetype model with R=3.75

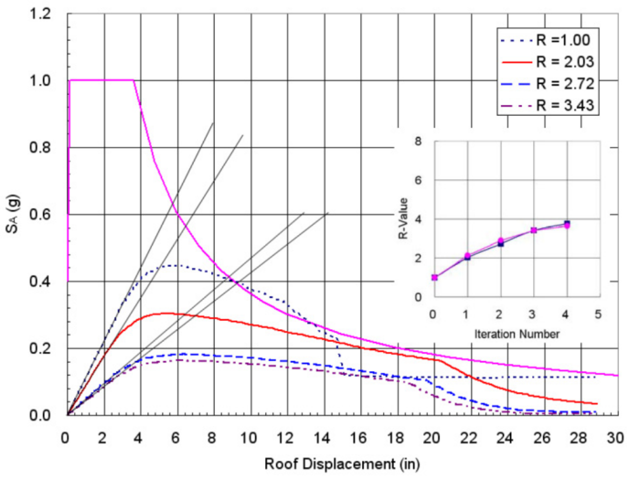


Fig. 25 R-value of 8-story archetype model (S_{DS}=1.000 g, S_{D1}=0.602 g, SDC=D)

From the Fig. 25, it is concluded that the R-value is reaching to R=4.0. Fig. 24 shows the modal deformed shape of 1st mode of 8-story archetype model of steel diagrid framed system. The ordinary steel concentrically braced frames of ASCE 7 have R=3.25. Considering the assumptions made for this study, the estimated R-value is considered as reasonable.

4.6 Limitations and recommended future study

It is expected to develop the seismic performance factors of the steel diagrid framed system for the 8-story building archetype model with assumptions above mentioned per FEMA-450 (2003). However, it is necessary to study further for general applications of the seismic performance factors as other seismic performance factors in model building code. The items needed to be investigated are listed as following:

• Influence of the frame extension in vertical direction such as shown in Fig. 20.

• Influence of the strain-hardening effect in tension, post-buckling behavior in compression, and strength degradation under cyclic loading as shown in Fig. 16.

• Influence of discrepancy of material properties between nominal and expected properties.

• Influence of modeling uncertainties including correlation and applicability with available component test data.

• Introduction of collapse margin ratio (CMR) with incremental dynamic analysis (IDA) of ATC-63 in the tall buildings.

5. Conclusions

An approach and methodology for the reliable determination of seismic performance factors for use in the design of steel diagrid framed structural systems is proposed. The recommended methodology is based on current state-of-the-art and state-of-the practice methods including structural nonlinear dynamic analysis techniques, testing data requirements, building code design procedures and earthquake ground motion characterization. In determining appropriate seismic performance factors (R, Ω_O, C_d) for new archetypical building structural systems, the methodology defines acceptably low values of probability against collapse under maximum considered earthquake ground shaking.

The approach encompasses standard seismic analysis and design procedures based on established consensus based seismic design standards and steel design specifications including ASCE 7-05 (2005), AISC 360-05 (2005) and AISC 341-05 (2005). Use of these design standards allow for more accurate nonlinear modeling consistent with expected component behavior under tested conditions. Critical in the development of seismic performance factors utilizing the ATC-63 methodology is the determination of an acceptably low probability of collapse under maximum capable earthquake (MCE) ground motions with reliable collapse margin ratios, in which this study is academically an alternative to classical MCE definition. The level of conservatism in developing appropriate seismic performance factors (R, Ω_O, C_d) is directly related to the consideration of uncertainties including the accuracy of design procedures, comprehensive test data and nonlinear analysis modeling.

Note that the proposed analytical methodology in this study represents a sound strategy that can be used to estimate seismic performance factors for various steel diagrid system archetype models for a range of regions of seismicity. However, it should be emphasized that although the proposed methodology is rigorous, depending on the level of certainty in analytical modeling assumptions, it may be necessary to lower (more conservative) the estimations or conclusions in determining seismic performance factors that may be deemed acceptable for practical building code applications.

As part of the ATC-63 methodology there are inherent limitations in determining reliable seismic performance factors (R-values) which depend on the level of certainty in four primary areas including (1) ground motion data; (2) analytical modeling capability and assumptions; (3) applicability of design standards; and, (4) correlation of analytical modeling assumptions with empirical test data results.

The detailed analytical studies require complex assumptions based on the reliability of the above inherent uncertainties including several idealizations, such as material properties, building shapes and loadings. The degree of uncertainties on these assumptions may lower the confidence level of the estimates, whereas conversely, the less uncertainty in assumptions made can lead to more reliable estimates. Then, in order to overcome the limitations of analytical results depending on uncertainty, wavelet-based control algorithm for motion control of buildings may be applied in the future study.

Acknowledgements

This research was supported by a grant (2013R1A1A2057502 and 2014R1A1A3A04051296) from the National Research Foundation of Korea (NRF) funded by the Korea government. In addition, POSCO and SOM are also appreciated for this study sincerely.

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