SEISMIC EVALUATION, REHABILITATION, AND IMPROVED DESIGN OF SUB-STANDARD STEEL CONCENTRICALLY BRACED FRAME BUILDINGS

By

DEREK SLOVENEC

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Thesis Advisor: Prof. Michael Pollino

Civil Engineering Department
CASE WESTERN RESERVE UNIVERSITY
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CASE WESTERN RESERVE UNIVERSITY
SCHOOL OF GRADUATE STUDIES

We hereby approve the thesis of

Derek Slovenec
candidate for the degree of Master of Science *.

Committee Chair
Dr. Michael Pollino

Committee Member
Dr. Brian Metrovich

Committee Member
Dr. YeongAe Heo

Date of Defense
11/13/2015

*We also certify that written approval has been obtained
for any proprietary material contained therein.
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Seismic Evaluation, Rehabilitation, and Improved Design of Sub-Standard Steel Concentrically Braced Frame Buildings

by

DEREK SLOVENEC

Abstract:
Seismic design of multi-story buildings requires capacity design principles that allow for distributed damage (plastic member deformations) to occur over the building height while preventing soft-story failure mechanisms that may lead to collapse. Seismic evaluation of steel concentrically braced frame (CBF) buildings has revealed that they exhibit soft-story behavior due to non-uniform brace degradation and non-ductile failure modes. This research proposes a rehabilitative design procedure for existing buildings that uses a stiff rocking core to redistribute plastic deformations along the structure’s height. Additionally, an improved design procedure for braced frame columns is proposed for new frame design. Several representative frames were designed and evaluated using nonlinear transient seismic finite element analysis and large-scale hybrid experimental testing. Predicted, analytical, and experimental response results show reasonable agreement, and the proposed techniques are believed to be reliable for achieving desirable seismic performance in low- to mid-rise steel braced frame structures.
Section 1: Introduction

Steel concentrically braced frames (CBFs) are popular seismic lateral force resisting systems (LFRSs) for multi-story buildings due to their efficient load path where the lateral loads are primarily resisted by the bracing members in tension and compression. Seismic design of steel CBFs has evolved over the years to include special member and connection detailing considerations with the intent of achieving desirable ductile response through tension yielding and compressive buckling of the brace members. All CBF beams, columns, and connections today are intended to be capacity designed to resist the maximum expected brace forces to relegate all damage to the braces during an earthquake. Even well-designed, ductile bracing members undergo non-symmetric, degrading cyclic force-deformation (hysteretic) behavior primarily due to compressive buckling which can pose challenges in design. The intended response of multi-story buildings to seismic demands is a nearly uniform inter-story drift and damage (through plastic member deformations) over the building height.

Many steel CBFs designed prior to modern seismic specifications are still in service, and the LFRSs in these structures exhibit undesirable behaviors during seismic response which can lead to drift and damage concentration in a single story. Seismic deficiencies might include poor lateral strength or stiffness distribution (irregularities) in the original design and/or lack of member or connection ductility resulting in brittle failures. Similarly, unforeseen irregularities in the vertical distribution of mass, strength, and/or stiffness in modernly designed CBFs can lead to non-uniform degradation of
braces along the frame’s height resulting in drift/damage concentration akin to that observed in older CBFs. This concentration can lead to a soft- or weak-story failure where one story is highly damaged and the remaining stories are relatively undamaged. This is an undesirable response because the LFRS cannot utilize the strength and stiffness of all stories (as intended) due to soft-story formation, potentially resulting in collapse. Such seismic deficiencies are known to exist in multi-story buildings—steel or otherwise—throughout the world. Additionally, modernly designed CBFs have been found to perform poorly under the maximum considered earthquake (MCE) hazard level often resulting in soft-story formations with drifts exceeding code-prescribed allowable drift levels.

A rehabilitation technique that uses a stiff rocking core (SRC) is investigated in this thesis. This technique utilizes a stiff elastic body or “spine” which would likely be added in-line with the existing sub-standard LFRS to help re-distribute lateral demands to all stories when building drift (and the resulting damage) starts to concentrate in a single story to prevent soft-story formation and premature structural failure. The SRC features a base connection that prevents it from participating in seismic response during uniform building drift (desirable response) however the high inter-story stiffness of the SRC provides corrective resistive forces to enforce a uniform drift profile at the onset of soft-story formation effectively preventing further concentration of damage on that story. Connection of the SRC to the existing framing with steel yielding energy dissipation elements can be incorporated into the SRC rehabilitation design to further strengthen and stiffen existing sub-standard frames to achieve an acceptable seismic drift performance.
The SRC rehabilitation technique offers several advantages over traditional rehabilitation including:

- limited interruption of normal business operations within the building during construction
- limited additional demands to the existing foundation and structural framing
- potential self- or re-centering through post-tensioning or hydraulic jacking following an earthquake to eliminate residual building drift (not considered in this study).

In addition to investigating the SRC rehabilitation technique, improved design of columns in new CBF design are proposed in this study. Based on previous research which noted similar soft-story failures of CBFs under the MCE hazard level, recommendations for improved CBF column design arose naturally once a quantitative understanding of the soft-story failure phenomenon was established.

The primary objectives of this research included: (1) investigating the fundamental seismic structural behavior of steel CBFs with deficiencies and cyclic degradation, (2) develop design procedures for the SRC-rehabilitation technique for sub-standard steel CBFs and (3) examine the effectiveness of SRC-rehabilitation and the proposed design procedures. The research methods include use of nonlinear transient seismic analyses of detailed 2D CBF frame models and static hybrid experimental testing of approximately 1/3-scale, 3-story frames. These experimental tests were performed to explore the fundamental behaviors of a degrading CBF and its interaction with the SRC and for system-level confirmation of the analytical models.
This thesis first discusses the design and seismic response of existing CBFs through a review of design standards and previous research in these fields. Preliminary research into the SRC-rehabilitation technique which served as the basis for this study will also be discussed. Next, prototype buildings are designed and detailed to represent existing, sub-standard or modernly designed low- to mid-rise steel CBF buildings. Design procedures for the SRC-rehabilitation technique (and improved CBF design) are then proposed using principles of structural mechanics and dynamics to achieve a specified drift performance objective. Advanced finite element analysis (FEA) models of all existing and rehabilitated prototype buildings are developed and used to perform nonlinear transient seismic analyses to assess the performance of each frame configuration and compare critical predicted values for design to analytical results. A large-scale hybrid experimental testing program performed during this research is described, and results from these experiments are compared with those obtained analytically to verify the accuracy of FEA models, which examined a wider range of parameters than could reasonably be tested experimentally. Finally, some conclusions and recommendations for future research on the topics examined in this study are discussed.
Section 2: Literature Review

Concentrically braced frames (CBFs) are commonly employed in low- to mid-rise steel structures to resist lateral demands including those induced by earthquakes. CBFs can have bracing members arranged in many configurations including X-bracing, 2-story X-bracing, and chevron (among others) which affect the internal force distribution and nonlinear seismic behavior of the CBF. The chevron configuration (example shown in Figure A-2) has historically been and continues to be the preferred bracing configuration primarily due to architectural advantages. This study focuses exclusively on low- to mid-rise steel CBFs with a chevron bracing configuration and square or rectangular HSS tubes as the bracing members. To understand the seismic performance of this type of frame, a review of modern and older CBF design methodologies is necessary. Previous research on the seismic response of these frames has indicated inadequate performance which may result from vertical irregularities in the structure and would require rehabilitation. The rehabilitation technique described herein was proposed as part of an ongoing collaborative project, and previous work completed on this project will be summarized at the end of this section.
2.1 Review of Seismic Design of Steel Concentrically Braced Frames

2.1.1 Modern Code-Prescribed Loading for Design

Modern seismic loading provisions define the seismic loading under the DBE hazard level using an idealized design spectral acceleration curve can be developed per ASCE 7-10, and design-basis spectral acceleration values can be determined based on the structure’s calculated natural periods (or some estimation thereof). In the equivalent lateral force procedure (ELFP), the product of spectral acceleration and seismic mass yields a design base shear force that is a function of the building location and site characteristics, earthquake hazard level, and dynamic properties of the structure. A sample ELFP calculation is provided in Appendix 1. The design base shear force can be converted into an equivalent set of lateral forces acting at each floor elevation whose relative magnitudes are determined based on the structure’s vertical distribution of mass. For a low-rise building with uniform floor mass over its height, the vertical distribution of lateral seismic forces has an inverted triangular shape, i.e. the lateral force at a given floor elevation can be determined by a linear equation with its maximum at the roof elevation and a value of zero at the structure’s base. Braced frame members can then be designed to resist these lateral demands, along with any other forces in the controlling load combination. The design of all members and connections within the frame are prescribed by the American Institute of Steel Construction (AISC) Specifications (ANSI/AISC 360-10, 2010) and AISC Seismic Provisions (ANSI/AISC 341-10, 2010).

CBF braces in each story are often designed to resist the entire inter-story shear demand resulting from the equivalent lateral seismic forces via axial tension and compression with the compression strength controlling brace sizes. The surrounding
beams, columns, and connections are then designed to elastically carry the maximum expected brace capacity forces to ensure that all inelastic behavior is relegated to the braces, often referred to as capacity-based design. As such, determining the expected brace capacity forces is essential for proper beam, column, and connection design as described in the following subsection.

### 2.1.2 Modern Design of Special Concentrically Braced Frames (SCBF)

Concentrically braced frames designed with highly ductile brace members and connections according to AISC 341 (2010) are designated as special concentrically braced frames (SCBF). The system ductility is achieved through prescribed ductile detailing requirements primarily in the bracing members and capacity design of the surrounding framing. Seismic SCBF design must utilize this ductility to allow for more practical design lateral loads and must also be detailed and designed to undergo significant plastic deformations. The reduction of seismic loads from elastic force levels to design levels is included via the response modification factor (R) which has a value of 6 for SCBFs.

Braces in SCBFs must be selected within strict section (b/t) and member (kl/r) slenderness limits due to their influence on nonlinear behavior (buckling). Additionally, net section reinforcement is required at the connected ends of the brace members such that the net effective area (including shear lag) of any cross section along the member’s length is greater than or equal to the gross section area. Gusset plates must be designed to accommodate a clear edge-to-edge (perpendicular to the longitudinal axis of the brace) span extending beyond the termination of the brace-to-gusset connection by a length equal to twice the thickness of the plate (see Figure A-3) to allow for repeated ductile
folding (plastic rotation at the end connection) of the plate about this region as the attached brace buckles out-of-plane in compression. With these ductile detailing requirements in mind, appropriate brace members can be designed to resist the calculated inter-story shear demand due to equivalent seismic lateral loading (obtained using ELFP) according to AISC 360 (2010).

Strength design of all beams, columns, and connections in an SCBF must consider the maximum expected brace force (or combination of brace forces) including the effects of material over-strength and degradation due to cyclic tensile yielding/compressive buckling behavior. Brace material over-strength (with respect to the nominal minimum specified yield strength for a given material) is incorporated via the $R_y$ factor. As such, the expected tension (ET) force in a brace is given as $F_{ET} = R_y F_y A_g$. The $R_y$ factor is also utilized in conjunction with an additional factor of 1.14 applied to the nominal buckling load ($P_n$) to compute the maximum expected buckling (EB) force: $F_{EB} = 1.14 F_{cre} A_g$, where $F_{cre}$ is the critical buckling stress defined by AISC 360 but using the expected material yield stress $R_y F_y$. Compressive strength degradation due to the inelastic cyclic loading expected during a significant ground motion is considered via calculation of the expected post-buckling (EPB) force, which is taken as thirty percent of the expected buckling capacity: $F_{EPB} = 0.3 F_{EB}$. Brace connections are designed for the more critical expected brace force (compression or tension).

Beams of an SCBF in a chevron configuration must have adequate flexural capacity to resist the combined vertical force resultant (unbalanced force) from the braces connected to the beam undersides. This combined vertical brake force, $F_{BV}$, has a maximum expected value of $F_{BV} = (F_{ET} + F_{EPB}) \sin(\theta_{br})$ where $\theta_{br}$ is the angle of the
braces with respect to horizontal. Note that this design force considers the compression brace to be at \( F_{EPB} \) rather than \( F_{EB} \) since it results in a larger (upper bound) unbalanced vertical load on the beam. Assuming the beams are pin-connected near the columns (often at the column face or outside the gusset plate region) and the longitudinal centerlines of both connecting braces intersect with the beam centerlines, the resulting moment demand on the beam is \( (F_{BV})(L)/4 \) where \( L \) is the length of the beam between pin connection points.

SCBF columns are designed based on forces from an analysis where all tension braces are at \( F_{ET} \) and all compression braces are at \( F_{EB} \) or all compression braces are at \( F_{EPB} \). This often yields demands on the columns which are primarily axial forces with little shear and bending since all braces are assumed to degrade uniformly (all at \( F_{EB} \) or \( F_{EPB} \)). The AISC Seismic Provision states that column moment demands can be ignored if all applied loads act at locations of lateral support, which is the case for ELFP seismic design where lateral loads act at the floor elevations where beams provide lateral support for the columns. As such, SCBF column design is controlled by axial compression force demand. However, if brace degradation is concentrated on a single story such that its compression brace is at \( F_{EPB} \) while all other compression braces are at \( F_{EB} \), significant column shear forces and bending moments can develop. This non-uniform strength degradation should be expected in all CBFs subjected to seismic loading because degradation is dependent on member and section slenderness as well as the load-deformation history, and these controlling parameters will not be identical for all braces in a given CBF. Non-uniform brace degradation is not considered in current SCBF design practice and is believed to be a critical factor leading to significant concentrated
drift (soft-story response) of modern CBFs at least during an MCE event due to the resulting irregularities in strength and stiffness as discussed in Section 2.3. A method for including the non-uniform brace strength degradation in a proposed improved SCBF design procedure is discussed in Section 3.6.

2.1.3 Design of Older, Non-Ductile CBFs

Many existing CBF structures located in high seismic regions were designed prior to the development of modern code-prescribed ductile detailing requirements and capacity design considerations discussed in Section 2.1.2. Seismic demands for CBF design have traditionally been determined using similar procedures to the ELFP described in Section 2.1.1 (see 1959 SEAOC procedure in Appendix 1) however beams, columns, and connections were not capacity designed for the maximum expected brace forces resulting in undesirable seismic response. A survey of braced frame structures on the west coast of the United States built prior to 1988 was conducted by Sloat et al. (2013) to identify the prevalence and severity of seismic deficiencies in these older, non-ductile CBF structures. Demand-to-capacity ratios (DCRs) were calculated by comparing the as-built strength capacity of various deficient connection, member, and frame limit states to the required capacity design forces. These DCR values were grouped into bins and evaluated statistically to produce the histograms shown in Figure 2.1. Note that net section fracture at brace-to-gusset connections was identified as the most commonly under-designed limit state with over 80% of the surveyed frames lacking adequate capacity, while severely insufficient beam bending capacity was found in about 75% of the frames. These deficient details must be considered when evaluating the seismic response of older CBF structures, and the survey results provided by Sloat et al.
were used in this study to design representative non-ductile CBFs for both analytical and hybrid experimental seismic evaluation. Additionally, Sloat et al. identified a typical beam-to-column connection detail (Figure 2.2) in these older structures that is no longer used for seismic CBF design due to its non-negligible moment capacity which can attract unwanted demands to the beams and the undesirable deformation kinematics it produces. An approximate consideration of this connection detail was included in the analytical models used in this study (see Section 4).

2.2 Review of Seismic Behavior and Response of Steel Concentrically Braced Frames

2.2.1 Code-Prescribed Performance Objectives and Quantitative Measures

In the United States, seismic design uses qualitative performance objectives for buildings depending on the level of seismic hazard considered for design (ASCE 7-10). For ordinary building structures, a “life-safety” (LS) performance objective is prescribed for a seismic hazard with a 10% probability of exceedance in 50 years (design basis earthquake, DBE) while “collapse prevention” (CP) is prescribed for a seismic hazard with a 2% probability of exceedance in 50 years (maximum credible earthquake, MCE). A third performance objective of “immediate occupancy” (IO) is prescribed for more frequent earthquakes. However, most ordinary structures are only evaluated for seismic loading under the DBE hazard level, since it is all that is technically required by the building code, thus assuming that the building code provisions provide adequate performance under a rarer (MCE) or more frequent seismic event. ASCE 41-06 defines quantitative performance measures for seismic rehabilitation of building structures based
on peak inter-story drift and for CBFs prescribes peak inter-story drift limits at 0.5%, 1.5%, and 2.0% to meet the IO, LS, and CP performance objectives, respectively.

### 2.2.2 Seismic Response of Modernly-Designed CBFs

Past research has shown that modern seismic braced frame designs do not perform adequately under the MCE hazard due to the formation of a soft-story mechanism (Sabelli, 2001; McCormick, 2007; Uriz, 2008; and Sanchez-Zamora, 2013). Research has shown CBFs behave adequately (meet the life safety performance objective) under DBE level ground motions possibly when only limited concentrated brace strength degradation occurs. In fact, most CBFs meet drift performance objectives quite easily under the DBE hazard due to their high levels of initial stiffness. However, these research studies have shown that CBFs in a maximum-considered earthquake (MCE), which has a probability of exceedance of two percent in fifty years, do not meet the collapse prevention drift performance objective. This is often due to a combination of non-uniform brace strength degradation and inadequate column design within the CBF resulting in soft-story formation. MacRae et al. (2004) identified the fact that CBF columns are not designed for moment demands potentially resulting in soft-story formation but showed that this deficiency is not likely to affect seismic performance if the columns are continuous over the frame’s height. However, that study did not consider vertical irregularities due to non-uniform brace degradation, distribution of mass, stiffness, and/or strength, or any combination thereof which likely result in greater column demands than those observed by MacRae et al. Ground motion frequency content characteristics can also induce concentrated ductility demands particularly in the lower stories of structures as described by Alavi and Krawinkler (2004) in their study of
modern (post-Northridge) moment frames subjected to near-fault (NF) ground motions. The effects of NF ground motion characteristics on the seismic response of CBFs are less known and are further investigated in this study.

The effect of continuous gravity framing on seismic response was studied by Foutch and Yun (2002) for steel moment frames. Their study found that continuous gravity frame columns reduced maximum inter-story drifts by about 10%. The moment frames considered had a tendency to exhibit soft-story response when subjected to seismic loads however this phenomena was somewhat mitigated by the inclusion of continuous gravity framing due to the alternate load path it provides to redistribute lateral demands away from the soft story. They concluded that the contribution of continuous gravity framing should be included to accurately capture seismic response, especially for frames susceptible to soft-story response. While this study focuses exclusively on braced frames, the concept of redistributing lateral forces away from a soft-story via continuous gravity frame members is applicable to any LFRS. Accordingly, the contribution of continuous gravity framing to seismic response was investigated for CBFs in this study.

2.2.3 Behavior and Seismic Response of Older, Non-ductile CBFs

Older braced frame structures designed according to pre-modern seismic standards were often designed with non-ductile member and connection details that do not follow capacity design principles and thus results in non-ductile failure mechanisms. Such designs quickly result in soft-story mechanisms as the critical weak story fails which imposes large demands on the columns (which were not designed for such loads) and generates concentrated drift and eventual collapse on the weak story. The deficiently designed details identified by Sloat et al. (2013) discussed in Section 2.1.3 can result in
this type of undesirable non-ductile failure, especially if brace net section fracture occurs. This type of non-ductile connection failure is referred to as “brittle connection” (BC) throughout this study. Additionally, the severely under-designed beams bending limit state identified by Sloat et al. can result in plastic hinging at the center of beams during seismic response as braces approach their expected forces and degrade due to cyclic tensile yielding and compressive buckling. This “weak beam” (WB) behavior causes undesirable damage in beams and can accelerate compression brace degradation due to the additional deformations it must undergo as the beam is pulled down by the tension brace. These undesirable behaviors exhibited by older, non-ductile braced frames prevent them from achieving the performance objective under the DBE hazard level.

2.3 Vertical Irregularities in Low- to Mid-Rise Structures

Vertical irregularities have been found to induce soft-story failures by several researchers. These irregularities might include the distributions of mass, stiffness, and/or strength along a structure’s height as well as vertical and/or horizontal geometric irregularities. Research performed by Vijayabaskar et al. (2012) used incremental dynamic analysis (IDA) to evaluate the effects of these vertical irregularities on the seismic response of a mid-rise steel structure. Both single- and multi-story irregularities were investigated. In general, it was found that strength irregularities have the most detrimental effect on seismic response followed by stiffness then mass irregularities. Irregularities in the middle stories were found to have the least effect on seismic response followed by upper stories then lower stories. A weak/soft story near a structure’s base can act as a “fuse” where damage is concentrated on the deficient story and all stories
above remain relatively undamaged. Seismic inter-story shear demands are greatest in the lower stories of a structure and must be carried by brace members in a CBF. This can lead to strength and/or stiffness degradation in the braces resulting in a vertical irregularity that is highly detrimental to the structure’s seismic performance.

An older study performed by Al-Ali and Krawinkler (1998) investigated the effect of vertical irregularities from a demand standpoint. Several linear-elastic and nonlinear dynamic analyses were performed using frames with strength, stiffness, and/or mass irregularities, as well as a baseline frame with no irregularities for comparison purposes. Demands including inter-story shear forces, overturning moment, and plastic deformations were compared to determine how each type of irregularity (and any combination of irregularities) affected seismic demands. Their findings were consistent with those of Vijayabaskar et al.; strength irregularities have the greatest effect on seismic demand concentrations followed by stiffness then mass. Varadharajan et al. (2012) quantified vertical irregularities considering both the severity, location, and type of irregularity in their study of multi-story reinforced concrete structures. They proposed an irregularity factor calculated as the sum of the ratios of participation factors for each mode in an irregular and equivalent regular structure. This study highlights the importance of considering all modes when determining the seismic response of irregular buildings and cautions against single-degree-of-freedom (SDOF) approximations that neglect higher mode response.
2.4 Review of Previous Research for the SRC-Rehabilitation Project

The research presented in this thesis is part of an ongoing project investigating a seismic rehabilitation technique for low- to mid-rise structures that uses a stiff elastic rocking core, or SRC, to redistribute lateral loads away from deficient stories and prevent soft-story failure. Previous research completed as part of this project is summarized in a paper by Pollino et al. (2013). Prior to the work performed for this research, the SRC-rehabilitation technique was being conceptually investigated using a preliminary version of the finite element analysis (FEA) models described in Section 4 of this thesis. This preliminary model was developed by Sanchez-Zamora (2013) who performed sensitivity analysis to determine appropriate modeling parameters that realistically capture the expected behaviors of modern CBFs subjected to seismic loading. The CBF designs modeled by Sanchez-Zamora were developed by Sabelli (2001) as braced frame analogues to the prototype moment frame structures developed as part of the SAC steel project.

The SRCRehab concept was inspired by similar seismic retrofit strategies proposed by Alavi and Krawinkler (2004), Günay et al. (2009), and Qu et al. (2012). All of these studies focused on moment frames (mostly reinforced concrete) and proposed a similar rocking wall system which redistributes lateral loads to prevent soft-story failure. These previous studies that proposed similar rehabilitation approaches were limited to conceptual studies or rehabilitation of particular buildings and utilized nonlinear transient seismic analysis for evaluation of response. However, fundamental understanding of the mechanisms that result in soft-story formation and procedures for rehabilitation design to achieve a particular seismic performance objective were lacking. Additionally,
experimental evidence of the complex interaction of a degrading sub-standard frame and a rehabilitation approach of this type have never been performed.
Figure 2.1: Demand to Capacity Ratios for Connection and Frame Limit States in Older CBFs (Adapted from Sloat et al., 2013)
Figure 2.2: Typical Brace-Beam-Column Joint Connection in Older CBFs

(Adapted from Sloat et al., 2013)
Section 3: Seismic Evaluation and Rehabilitation Design Approach for Sub-Standard Steel Concentrically Braced Frames

Successful seismic rehabilitative design of steel concentrically braced frames (CBFs) that are susceptible to soft-story failure under seismic loading requires an understanding of their expected hysteretic behavior both before and after rehabilitation. In braced frames, soft-story formation occurs when braces reach their ultimate strengths in tension and compression and column shear demands exceed the force required to form a plastic panel mechanism of a given story. Developing the plastic mechanism strength of the columns significantly limits the ability to re-distribute lateral loads vertically over the building height to engage the lateral strength of other stories and concentrates plastic drift on the weak story that can then lead to global collapse.

An understanding of the intended seismic behavior of CBFs and assumptions inherent to both modern and older non-ductile seismic braced frame design is necessary to determine where these procedures are deficient and for proposing improved design or rehabilitation methods. A summary of current CBF seismic design and critical details of older non-ductile CBFs was discussed in Section 2.1. This section discusses calculations of critical fundamental behavior quantities (plastic mechanism strengths, internal forces, drift predictions) that are needed for rehabilitated design of sub-standard CBFs. A set of prototype buildings are used to illustrate the calculations and are used later for finite element analysis (Section 4) and as the prototype frames for experimental hybrid testing (Section 5). First, the frames’ plastic mechanism strengths are calculated to determine
total frame strength and also to assist in quantifying the inter-story capacity design forces which are essential for rehabilitation design. The plastic mechanism strengths of the prototype frames are determined considering:

- uniform drift response
- soft-story response with:
  - brace yielding/buckling behavior
  - weak beam and brace buckling behavior
  - brace brittle fracture behavior

Based on the observed behavior of sub-standard CBFs in past research and preliminary analysis performed in this research, a rehabilitation approach for such frames is proposed that uses a stiff rocking core (SRC) to allow for lateral load re-distribution and enforce a nearly uniform drift distribution over the building height. The SRC rehabilitation approach is first introduced conceptually and then design methods for achieving a target seismic drift performance and strength design of the SRC are presented. A recommended process for performing the iterative SRC-rehabilitation design is then described.

Finally, recommendations for an improved design approach for columns in newly designed CBFs are presented and discussed. While the focus of this research study is on rehabilitation of sub-standard CBFs, the recommendations for improved column design for new CBFs seemed to naturally derive once the plastic mechanics of CBFs were understood. Appropriate internal bending moment and axial force demands for capacity design of CBF columns are proposed.
3.1 Prototype Buildings and Performance Objectives

Prototype buildings considered in this study were based on the concentrically braced frame buildings designed by Sabelli (2001) as analogues to the example steel moment frames developed for the SAC steel project (Gupta and Krawinkler, 1999). Three- and six-story buildings with wide flange beams and columns and square HSS tubes in a chevron bracing configuration were designed by Sabelli for office use in downtown Los Angeles. Symmetry in the floor plans and braced frame locations in these buildings allowed the total structural response to be determined while only considering one braced frame (and its tributary gravity framing and mass) per building. In the original Sabelli frames, braced frame members were designed according to the 1994 AISC Load and Resistance Factor Design Specifications (ANSI/AISC 360-94) and the 1997 AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-97).

3.1.1 3SCBF and 6SCBF

Beams in the three- and six-story frames from Sabelli (2001) were found to have insufficient bending moment capacity to resist the out-of-balance vertical force acting at mid-span when the attached braces reached their expected tensile (ET) and expected post-buckling (EPB) strengths. These inadequate beams were redesigned in this research to modern seismic provisions per AISC 341-10, and the resulting frames are representative of modernly-designed low- and mid-rise braced frame structures in high seismic regions of the United States. The modern three- and six-story prototype braced frames are referred to as 3SCBF and 6SCBF, respectively, throughout this thesis and are shown in Figures 3.1 and 3.2.
3.1.2 3NCBF-WB and 3NCBF-BC

Additional three-story prototype braced frames—referred to as 3NCBFs (non-ductile CBFs)—were designed for this study using a historic seismic design procedure and several under-designed limit states based on the review of existing CBF designs by Sloat et al. (2013) to represent existing structures that predate modern design standards. The prototype 3NCBF frame is detailed in Figure 3.1.

Lateral seismic demands have historically been determined using a procedure similar to the modern ELFP described in Section 2.1.1, and similar design forces are calculated using either procedure. These procedures (and their differences) are described in detail in Section 5.3.2 (and Appendix 1) in the context of a representative pre-modern braced frame design for hybrid experimental testing. As was the case for modern SCBF design discussed in Section 2.1.2, braces in older braced frames are designed to carry all of the inter-story shear demand imposed by a design-basis earthquake through axial tension and compression. However, ductile detailing considerations were not required in older braced frames, and beams and columns were not capacity designed based on the maximum expected brace forces. Instead, a linear-elastic static analysis of the frame with the seismic lateral loads applied or a rational approximate analysis method was used to determine member and connection forces for design.

Deficiencies in NCBF designs become apparent when contrasting the modern and pre-modern design procedures. Design forces for beams, columns, and connections outside of the braces are obtained based on linear-elastic static analysis of the proposed frame design and do not consider the expected capacity of each brace or effects of degradation, resulting in a frame whose beams, columns, and connections are not
adequately designed for the demands imposed by braces behaving inelastically. NCBF column design results in even smaller sections relative to SCBF design that are likely even more susceptible to soft-story formation. In SCBF design, beams are designed to resist the maximum expected $F_{BV}$ force in a given story resulting from one brace at $F_{ET}$ and the other brace at $F_{EPB}$. NCBF designs do not account for this “pull-down” force, and because of this the controlling frame plastic mechanism can include beam mechanisms during a seismic event as brace degradation occurs. As the compression brace loses strength over load-deformation cycles, the difference between the vertical components of each brace’s axial force grows, potentially reaching the beam’s pull-down force, $F_{PD} = 4M_{pb}/L$. Assuming the compression brace is at $F_{EPB}$, the maximum axial force that can develop in the tension brace, $F_{TEPB}$, is calculated using Equation 3.1 which was developed based on the free body diagram shown in Figure 3.3 and is less than the expected tensile strength of the brace.

$$F_{TEPB} = \frac{4M_{pb}}{L \cdot \sin(\theta_{br})} + F_{EPB}$$

where: $M_{pb} =$ beam plastic moment capacity, $L =$ length of beam, $\theta_{br} =$ angle of brace with respect to horizontal, $F_{EPB} =$ expected post-buckling axial force in compression brace.

A frame exhibiting this type of beam pull-down plastic mechanism is referred to as NCBF-WB, or “weak beam” NCBF.

Brace-to-gusset and gusset-to-beam connections were identified by Sloat et al. (2013) as commonly under-designed details in older braced frame structures. Failure of a brace-to-gusset connection is particularly detrimental to frame performance because it
severs the primary load path of the inter-story shear force demand carried by the braces. Brace connection fracture also dramatically reduces the strength and stiffness of that story, likely resulting in soft-story formation and collapse. These types of older, non-ductile CBFs are referred to as NCBF-BC (“brittle connection) frames throughout this thesis.

3.1.3 CBF Seismic Performance Objectives

ASCE 41-06 defines three seismic performance objectives for seismic rehabilitation of buildings based on maximum inter-story drift: immediate occupancy (IO), life safety (LS), and collapse prevention (CP). For a steel braced frame, these objectives are achieved at 0.5%, 1.5%, and 2% maximum inter-story drift, respectively. Buildings of varying importance—quantified by the importance factor, I—are expected to achieve different performance objectives under each hazard level considered. For example, essential structures (with greater I factors) are expected to achieve the life safety performance objective during a maximum-considered earthquake (MCE) event, while ordinary structures need only achieve the collapse prevention objective. For design-basis earthquake (DBE) events, immediate occupancy and life safety are the desired performance levels for essential and ordinary structures, respectively. For more frequent earthquakes, essential buildings are expected to remain operational (small inter-story drift; no limit is defined), while ordinary structures should achieve the immediate occupancy objective. These inter-story drift limits provide a metric for evaluating a structure’s global performance to earthquake hazards.

The prototype buildings considered in this study were designed for office use with an importance factor of one, designating them as “ordinary” structures. As such, they are
expected to remain within 2% inter-story drift during an MCE event and 1.5% inter-story drift during a DBE event. Seismic performance evaluations performed by other researchers (Sabelli, 2001; McCormick, 2007; Uriz, 2008; and Sanchez-Zamora, 2013) demonstrated that the 3-story and 6-story braced frame buildings designed by Sabelli (2001), which are similar to the 3SCBF and 6SCBF considered in this study but with inadequate beams for chevron brace configurations according to modern seismic codes, achieve the DBE performance objective but fail to achieve the MCE objective, often due to the formation of a soft-story mechanism for which SRC rehabilitation is effective. A study performed by Alavi and Krawinkler (2004) suggested that multi-story buildings may be more susceptible to soft-story mechanism formation especially in the lower stories when subjected to near-fault (NF) ground motions.

For this study, existing, SRC-rehabilitated, and improved braced frame column design configurations of the modernly-designed frames (3SCBF and 6SCBF) are designed to meet the stated performance objectives for the DBE hazard level in Section 3 (this section) and then evaluated analytically and experimentally for MCE and NF hazard levels in later sections. Additionally, multiple 3NCBF frame configurations were designed considering both “weak beam” and “brittle connection” behaviors and evaluated under DBE and MCE hazard levels to assess the effectiveness of SRC rehabilitation for limiting peak inter-story drift in older structures to comply with modern seismic rehabilitative performance objectives.
3.2 Prototype Frame Configurations

Multiple configurations were considered for each prototype frame as described below and illustrated in Figures 3.4 through 3.6. For the 3SCBF and 6SCBF, original and improved braced frame column design cases (without SRC) were considered, with the improved column design procedure detailed in Section 3.6. Only the original braced frame columns were considered for the 3NCBF prototypes. Rehabilitated cases for the three-story frames (3SCBF and 3NCBF) used the original braced frame columns and full-height SRC, and both pin-pin- and fix-fix-ended braced frame-to-SRC links were considered separately. For the six-story frame (6SCBF) with original braced frame column design, two- and six-story SRCs were considered. For the two-story SRC cases, only pin-pin links were considered because a partial-height SRC with fix-fix-ended links can over-strengthen the rehabilitated stories and cause a soft story mechanism in stories above the SRC. Note that all calculations and design procedures for a partial-height SRC with pin-pin links are identical to those discussed for a full-height SRC in the following sections. The six-story SRC cases considered both pin-pin- and fix-fix-ended links. For each frame and case described above, three gravity framing cases were considered: no gravity framing contribution to LFRS, continuous gravity frame columns assuming weak-axis orientation, and continuous gravity frame columns assuming strong-axis orientation. The contribution of gravity frame columns in resisting differential story shears—the difference between inter-story shear demands due to vertical distribution of base shear force and inter-story shear capacity of braces in each story—and soft story formation is questionable as it requires an adequately designed load path through the diaphragm and
into the gravity columns (loads for which it likely was never designed). Therefore, these gravity frame cases are simply included to investigate a potential range of gravity column contribution. Equivalent single column properties representing all tributary gravity framing for each gravity column contribution assumption are given in Table 3.1. A total of 36 BF frame configurations were investigated, and more detailed breakdowns of these configurations are shown in Figures 3.4 through 3.6 and described in the finite element analysis section (Section 4).

3.3 CBF Plastic Mechanism Strengths

The lateral strength of a CBF can be quantified by the plastic base shear force which is the total applied lateral load to reach the ultimate plastic strength of the entire frame. The plastic base shear strength can be calculated using the principle of virtual displacements where external work caused by an inverted triangular vertical distribution of lateral seismic forces (reasonable assumption for low- to mid-rise buildings) through a prescribed virtual deformation is equated to the internal plastic work done by CBF members through the same deformation.

3.3.1 CBF with Uniform Drift Mechanism

For a well-designed low- to mid-rise braced frame structure, inter-story drifts are expected to be approximately uniform over the height of the structure due to reasonable distribution of mass, stiffness, and strength over the building height and design of mechanisms to re-distribute lateral forces vertically over the building height. A plastic mechanism forms when all braces are at capacity and columns hinge at the base (if a
fixed connection is present) due to the imposed lateral loads. An illustration of the plastic mechanism and virtual work calculation to determine the plastic base shear of this mechanism is shown in Figure 3.7 for a three-story frame. This figure is useful for identifying notation utilized in all calculation procedures described in this section, as well as critical response quantities from finite element analysis and experimental testing discussed in Sections 4 and 5, respectively. $V_p$ is the plastic base shear force, which is formed due to the lateral inertia forces distributed along the height of the structure by the $C_n$ coefficients, where $n$ is an integer value indicating the floor (above the structure’s base) at which each force acts. For the case of uniform floor masses and building story heights for a three-story frame, values of $C_1$, $C_2$, and $C_3$ are 1/6, 1/3, and 1/2, respectively, resulting in an inverted triangular lateral force distribution. The maximum expected axial tensile and buckling capacities of the braces are designated as $ET$ (expected tension) and $EB$ (expected buckling), respectively, and are defined in Section 2.1.2 based on the AISC Seismic Provisions. The horizontal components of individual brace axial forces are referred to as $F_{bh}$, while the horizontal component of the combined axial force in both braces within a story is referred to as $F_{BH}$.

The plastic base shear calculation in Figure 3.7 equates external and internal work terms. The external work ($W_E$) is calculated as the summation of the product of the imposed lateral forces ($C_iV_p$) and their respective virtual floor displacements ($\theta_iH_i$). Internal work done by the braces ($W_{Ib}$) on each story is the expected brace capacity forces multiplied by the relative displacement between adjacent floors ($F_{bh}\theta_iH_i$). If fixed connections are used at the base of each column, flexural hinges must also form at these locations and the resulting internal work of the columns ($W_{Ic}$) must also be included.
Equating internal and external work yields the following equation for the plastic base shear capacity:

\[
V_{p,\text{Ex.UD}} = \frac{\sum_{i=1}^{N} \left( F_{bhETi} + F_{bhEBi} \right) \cdot h_i + 2M_{pc_i}}{\sum_{i=1}^{N} \left( H_i \cdot C_i \right)}
\]  

(3.2)

where:  
i = story or floor,  
N = total number of stories,  
F_{bhET} = horizontal component of brace expected tension force,  
F_{bhEB} = horizontal component of brace expected buckling force,  
h = story height,  
M_{pc} = column plastic moment capacity,  
H = floor elevation,  
C = coefficients distributing plastic base shear to each floor elevation.

This is the plastic base shear strength for the preferred plastic mechanism of a CBF because it utilizes the plastic strength of all bracing members over the height and has a uniform drift profile reducing the concentration of damage in any single story. A summary of critical existing prototype braced frame quantities—including the controlling (soft-story) plastic mechanism base shear forces and expected brace forces—for SRC-rehabilitative and improved column design is provided in Table 3.2.

### 3.3.2 SCBF with Soft-Story Mechanism

Evaluation of the strength capacity of SCBFs with soft-story mechanisms can be accomplished using the virtual work approach discussed in Section 3.3. This procedure is illustrated in Figure 3.8, which shows a three-story SCBF with a soft first story. This is the controlling plastic mechanism for existing SCBF designs when either an inadequately
designed weakened story exists or non-uniform brace degradation occurs. External work done by the lateral forces is only considered at floor elevations above the soft story because all plastic deformation occurs within the soft story. Only the braces within the soft story contribute to the internal work term, and the required work to form flexural column hinges in the soft story is equal to $4M_{pc}\theta_i$ unless the top story is degraded ($n_{PB} = N$, where $N$ is the total number of stories and $n_{PB}$ designates the degraded story), in which case the pinned beam-to-column connections in the third story only require one hinge to form in the bottom of each column to develop the plastic mechanism ($2M_{pc}\theta_i$). The plastic base shear force for the controlling soft-story mechanism expected to form in SCBF structures subjected to seismic loading is given by Equation 3.3.

$$V_{p,Ex,SS} = \left( F_{bhET_{npB}} + F_{bhEPB_{npB}} \right) : h_{npB} + \begin{cases} 2M_{pcN} & \text{if } n_{PB} = N \\ 4M_{pc_{npB}} & \text{otherwise} \end{cases} \sum_{i = n_{PB}}^{N} C_i$$

(3.3)

where all terms have been defined previously.

The plastic base shear force is determined by calculating $V_p$ for a soft-story mechanism in each story individually to determine the minimum (controlling) mechanism strength.

### 3.3.3 Older, Non-Ductile Concentrically Braced Frame Soft-Story Mechanisms

The plastic base shear strength calculation procedure for NCBFs is largely identical to that presented in the previous subsection for SCBFs with the exception of the internal brace work term due to differences in the expected behavior of braces in each
type of CBF. For “weak beam” NCBF behavior, beam internal work must also be included when calculating $V_p$.

### 3.3.3.1 3NCBF-WB

For calculation of the NCBF-WB (discussed in Section 3.1.2) mechanism’s strength, the compression brace on the degraded floor is assumed to be at $F_{EPB}$ which does work through both the lateral frame deformation and vertical beam deformation as shown in Figure 3.9. The tension brace on the degraded floor does not contribute to total brace internal work because beam pull-down occurs before this brace reaches yield and the flexural hinge that forms in the beam prevents additional force from developing in the brace. Beam pull-down can accelerate brace degradation and low-cycle fatigue effects due to the increased plastic deformations in the compression brace caused by the lateral drift and vertical beam deformations. The flexural resistance of the beam web at the column connection is assumed to be negligible, and the work done by these hinges is ignored in the calculation of total internal work done by the pulled down beam. The column internal work and external work terms for a soft-story NCBF-WB are identical to those discussed for SCBF designs with soft stories in Section 3.3.2. The expression for plastic base shear of an existing NCBF-WB design is given by Equation 3.4.
\begin{equation}
V_{p.W.B.ss} = \frac{F_{bhEPB_{npB}} \cdot h_{npB} + F_{bvEPB_{npB}} \cdot \frac{L}{2} + 2M_{pb_{npB}} + \begin{cases} 2M_{pcN} & \text{if } n_{PB} = N \\ 4M_{pc_{npB}} & \text{otherwise} \end{cases}}{h_{npB} \cdot \sum_{i=npB}^{N} C_i} \tag{3.4}
\end{equation}

where:  
- $F_{bvEPB} =$ vertical component of brace expected post-buckling strength,
- $M_{pb} =$ beam plastic moment capacity, and all other terms have been defined previously.

### 3.3.3.2 3NCBF-BC

The plastic mechanism of the 3NCBF-BC is illustrated in Figure 3.10. For this mechanism, the braces are assumed to not contribute to the total internal work. When the frame reaches a sufficient deformation, a tension brace connection will fracture, and the resulting drop in story strength and stiffness (and associated change in dynamic properties) will cause further damage to concentrate on this story. The remaining brace fractures once a sufficient drift in the opposite direction occurs, resulting in a story with almost no lateral force carrying capacity. The external work and column internal work terms are identical to those discussed for CBF soft-story and NCBF-WB designs. Plastic base shear for the expected soft-story mechanism in an NCBF-BC design is given by Equation 3.5.
where all terms have been defined previously.

The soft-story plastic mechanism strength of the prototype 3NCBF-BC frame shown in Table 3.2 shows this frame’s severe lack of lateral strength.

### 3.4 Seismic Rehabilitative Design of Sub-standard Steel Concentrically Braced Frames

An approach for seismic rehabilitation of sub-standard steel braced frame multi-story buildings was proposed for the prototype frames described previously with existing vertical strength and stiffness irregularities based on nominal properties (NCBF) and irregularities resulting from hysteretic degradation (SCBF). A stiff rocking core (SRC) was proposed that would likely be added in-line with the existing sub-standard frame to help re-distribute lateral differential shear that develops from the existing CBF and develop the preferred uniform drift plastic mechanism described previously. The approach is conceptually illustrated in Figure 3.11. A properly-designed SRC can re-distribute seismic forces along the core height, creating a more uniform drift and ductility demand distribution. The core re-distributes seismic ductility demands from weaker floors that would otherwise have significant concentrated ductility demands, potentially
preventing collapse. Displacement-based steel yielding links (SYL) can be implemented between the SRC and the existing framing to further reduce overall building drift.

The rehabilitation approach is believed to have advantages compared to more traditional rehabilitation methods including:

- Focused rehabilitation effort on the outside of the building or in an area of the building which limits business interruption. Traditional seismic rehabilitation often involves replacing existing members and/or connections within the LFRS.

- Limited additional demands to the existing foundation and structural framing. A pin-pin connected SRC is intended to re-distribute lateral shear and thus all internal loads could be directed through the floor diaphragm (which likely has excess in-plane capacity) and the SRC base reaction would simply include a horizontal shear force (at least for pin-pin links). Other rehabilitation techniques which involve supplemental members or framing often require proportional strengthening to the surrounding framing and foundation including uplift forces on the foundation. SRC with SYL does however require resistance of uplifting forces at the base of the SRC and their connecting column (reaction column).

- In general, seismic rehabilitation with a continuous elastic body or “spine” provided by the SRC is likely to provide a more resilient design as it limits the sensitivity of seismic response for any structural system which might develop non-uniform strength or stiffness degradation.

- Inclusion of an elastic spine within a structure allows for potential self-centering or re-centering through post-tensioning or hydraulic jacking following an earthquake and hence can eliminate residual drift in a building which can be quite
difficult to repair in typical building structures. However, self- or re-centering was not considered in this research study.

The SRC can be thought of simply as a continuous elastic body that must be designed to re-distribute loads. The SRC can be designed in various configurations to achieve acceptable rehabilitated performance. A few potential configurations shown in Figure 3.12 were originally considered as part of this research project (Pollino et al. 2013) with each configuration having various advantages and disadvantages in terms of the strength, hysteretic behavior with SYL, and ability for potential self- or re-centering as noted in the figure.

As discussed in Section 2.1.2 and previously in this section, modern CBF designs does not account for the effects of non-uniform brace degradation which can place excessive shear demands on the columns and form a panel (soft-story) mechanism. Formation of this undesirable plastic mechanism can be prevented simply by improving the design procedure for columns in modern SCBF specifications to account for the maximum expected shear, axial, and bending demands resulting from non-uniform brace degradation. However, it is often not practical to replace LFRS columns in existing soft-story-prone structures. For these structures, supplemental rehabilitative systems such as the SRC must be designed to effectively resist the expected inter-story shear forces for which the columns were not adequately designed. These rehabilitative efforts can prevent soft-story mechanism formation, resulting in a desirable uniform drift controlling plastic mechanism pictured in Figure 3.7.

The SRC must be designed for both strength and stiffness to achieve target drift performance objectives with the demands placed on the SRC largely dependent on the
inter-story stiffness and plastic capacity of the existing building and the dynamic properties of the combined system. The SRC is designed to remain elastic as a capacity designed member, and the SRC is assumed to impart a uniform drift profile on the building. However, prediction of overall building drift (average inter-story drift or roof drift) and prediction of the forces for rehabilitation design of the SRC is critical. In some cases, an SRC with pin-pin connecting links to the existing CBF (no SYL) will produce adequate performance by simply eliminating the concentration of drift on a single story. Steel yielding links (fix-fix end connections) can be used between the SRC and existing framing (attached to what is referred to herein as the “reaction” column, or RC) to increase the overall building stiffness and strength for buildings in which the SRC-rehabilitated overall drift without SYL is unacceptable. The SYL considered here is conceptually similar to an eccentrically braced frame (EBF) link with bolted end connections. For the SRC rehabilitation technique, it seems that the links can always be designed as “short” links with shear-dominated yielding behavior. This subsection details proposed calculation procedures for seismic response prediction and SRC-rehabilitation design. First, calculations for evaluating the drift response of each frame configuration—existing, SRC-rehabilitated with pin-pin connecting links, and improved column design (discussed in Section 3.6)—are described. A procedure for designing the SRC system with SYL to further reduce predicted drift to a target value is then proposed. Finally, strength design of the SRC and reaction column (RC) is described. Note that the SRC design procedure described in subsequent subsections is an iterative process due to the design forces and drift dependence on the calculated strength and dynamic properties of the rehabilitated structural system. SRC, SYL, and RC designs for all prototype
frames considering the MCE hazard level are given in Table 3.3 and were obtained using the procedures described in the following subsections. Full details of all calculation procedures described in this section are provided in Appendix 2.

3.4.1 Seismic Drift Prediction of Existing and SRC Rehabilitated CBFs

Maximum drift predictions for each type of frame behavior (SCBF, NCBF-WB, or NCBF-BC) and configuration (existing, with SRC, with SRC and SYL, improved braced frame columns) are necessary to assess seismic performance without performing extensive nonlinear dynamic finite element analysis. Principles of structural dynamics and knowledge of earthquake engineering (particularly the equal energy rule; Newmark and Hall, 1982) can be applied to predict peak seismic drift response of the nonlinear structural systems considered here. If the maximum drift is deemed unacceptable per the drift performance objectives for rehabilitative design given by ASCE 41-06, yielding members can be incorporated into the SRC-rehabilitation design to increase the internal work required for plastic mechanism formation, thereby strengthening the system and reducing predicted drift to an acceptable value.

Concepts of response spectrum analysis are used to predict maximum drift for a given frame. A design spectral acceleration curve must be constructed for the hazard and location of interest. To calculate peak (inelastic) displacement due to first mode response, the equal energy or equal displacement rules (Newmark and Hall, 1982) are applied depending on which range of applicability the structure’s first modal period falls into. All prototype frames considered in this study were within the equal energy range. The equal energy concept states that internal strain energy of an elastic system ($A_e$) with a particular natural period is equal to the total energy ($A_p$, which includes elastic and
plastic strains) of a nonlinear system with the same natural period. This concept is illustrated in Figure 3.20 can be used to calculate the maximum deformation ($\Delta_{\text{max}}$) of a system with given stiffness and strength (approach taken for existing frames, SRC with pin-pin connected links, and improved column design) or to determine the required stiffness and strength to achieve a target displacement or drift (for SRC with SYL design).

For structures with first mode periods within the equal energy range ($0.125s < T_1 < T_s$, where $T_s$ defines the end of the constant spectral acceleration region on the design spectral acceleration curve), the conservative plastic base shear value (using $F_{\text{EPB}}$ forces) calculated for the frame of interest (see Figures 3.8 through 3.10 and 3.14 through 3.19) defines the yield strength of the equivalent elastic-perfectly-plastic SDOF system with elastic stiffness calculated according to the single-degree-of-freedom (SDOF) approach described in Section 3.4.2. The product of spectral acceleration and modal mass divided by this same SDOF stiffness yields the spectral displacement of the system assuming linear elastic behavior. The area under this force-deformation curve is set equal to the area under the elastic-perfectly-plastic curve and the nonlinear spectral displacement can be calculated from this equation. If the first period is in the equal displacement range ($T_s < T_1 < 3s$), the inelastic spectral displacement is equal to the maximum displacement calculated for the elastic system.

Higher mode spectral displacements can also be added to the primary (first) mode response (however they typically contribute less than 10% of the total deformation) and are assumed to be elastic. Higher mode spectral displacements can be combined with the inelastic first mode displacement using an SRSS modal combination rule. Total spectral
displacement can then be multiplied by the first mode shape vector and participation factor to determine the maximum displacement at each floor elevation. Drifts can then be calculated using this maximum displacement profile and compared to code requirements to assess the seismic performance of a given frame configuration. This procedure is valid for predicting drifts for the improved braced frame column SCBF and any SRC-rehabilitated frame because their expected maximum drift response resembles the first mode shape (uniform drift). While adding the higher mode deformation response yields a more accurate prediction, it requires calculation of the higher mode dynamic properties which significantly adds to the design effort. Estimates of drift during design iterations assuming only first mode behavior with an assumed first mode shape consisting of uniform drift (rehabilitated frames) or complete soft-story (existing frames) is more practical for design and more exact calculations can be performed for final design confirmation.

A slightly modified approach is required to predict maximum drift of existing frames assuming a soft-story collapse mechanism because the plastic response (soft-story formation) does not resemble the elastic first mode response (uniform drift). The approach taken here for calculation of appropriate modal properties was to introduce a dummy stiffness matrix with very low stiffness in the soft story relative to the other stories. Solving the eigenvalue problem using this stiffness matrix and a lumped mass matrix yields orthogonal mode shapes, the first of which exhibits a soft-story shape. The modal mass calculated for this mode will essentially equal the entire frame mass and must be used in the equal energy/displacement calculation approach described previously to obtain maximum first mode (inelastic) displacement. The soft-story plastic mechanism
strength and SDOF elastic stiffness (see Section 3.4.2) are used in this equal energy/displacement calculation. The spectral displacement multiplied by the soft-story mode shape vector will produce the desired maximum displaced shape.

3.4.2 Frame Dynamic Property Predictions

A reasonable estimation of the dynamic properties of the existing and rehabilitated structure is essential for predicting seismic response. Characterization of mass and stiffness are necessary to determine all pertinent dynamic properties used for seismic response prediction and/or rehabilitative design. Mass can be considered concentrated on each floor, resulting in a diagonal mass matrix of size N by N, where N is the number of stories in the frame. Both shear and overturning stiffness of the CBF must be considered to calculate the frame stiffness and stiffness matrix, especially for taller structures where overturning flexibility can significantly reduce the total lateral stiffness. For SRC with pin-pin connected links, the first mode properties are not significantly affected however adding fix-fix-ended SYL can significantly increase the total system stiffness. Procedures for calculating the stiffness of existing, improved column design, and SRC-rehabilitated structures are included in the calculation sheets provided in Appendices 2.1 through 2.4 for prototype 3SCBF, 6SCBF, 3NCBF-WB, and 3NCBF-BC structures, respectively.

Existing CBF Stiffness Calculation

In a braced frame, brace members contribute the majority of the lateral shear stiffness. As such, only the horizontal projection of brace axial stiffness is considered when calculating existing braced frame shear stiffness per story.
\[
k_{\text{ExF},i} = 2 \cdot \frac{E_s \cdot A_{bi}}{\sqrt{0.65 \cdot \left( \frac{L}{2} \right)^2 + \left( h_i \right)^2}} \cdot \left( \cos \left( \theta_{bi} \right) \right)^2
\]

where: \( i \) = story, \( E_s \) = modulus of elasticity for steel, \( A_{bi} \) = brace cross-sectional area, 0.65 = factor to account for rigid joint offsets at brace ends, \( L \) = frame width,

\( h \) = story height, \( \theta_{bi} \) = brace angle with respect to horizontal.

To calculate overturning stiffness, the moment area method is used to calculate per-story deflections under unit lateral forces applied to an equivalent flexural member representing the overturning stiffness of the braced frame columns. The moment of inertia of this flexural frame representation can be calculated by applying the parallel axis theorem about the frame’s (vertical) centerline using the cross-sectional areas of the braced frame columns. A unit base shear force is applied to the flexural frame as an inverted triangular set of point load forces acting at each floor elevation. The resulting moment diagram has a value of zero at the roof elevation and increases to its maximum value at the base. The moment diagram between any adjacent floors is linear, and the slopes of these lines increase moving down the structure due to the increase in inter-story shear. The moment area method can then be applied to calculate the per-story flexural (overturning) deformation of each story. This is accomplished by considering only the triangular portion of the moment diagram in a given story to eliminate additional deformations calculated due to overturning in the above stories. This procedure yields the following equation:
where: i = story, \( E_s \) = modulus of elasticity for steel, \( I_O \) = moment of inertia of columns about frame’s vertical centerline, \( M_{BF} \) = maximum moment in story due to unit base shear, \( h = \) story height.

The shear and overturning stiffness terms for each story are added as “springs in series” to determine the total existing frame stiffness per story:

\[
k_{ExF_i} = \frac{E_s \cdot I_O}{\left(1 \cdot M_{BF_i} \cdot h_i \cdot \frac{2}{3} \cdot h_i \right)}\]

(3.7)

where: i = story, \( k_{ExFs} \) = story shear stiffness, \( k_{ExFf} \) = story flexural stiffness.

Once the existing frame’s per-story stiffness terms have been calculated considering shear and overturning deformations, dynamic properties can be calculated in two ways: a single-degree-of-freedom (SDOF) approximation or a multi-degree-of-freedom (MDOF) approach considering one horizontal degree of freedom per floor (N by N mass and stiffness matrices). For the SDOF approach, a single stiffness term can be calculated considering all per-story stiffness terms by calculating roof drift per unit base shear force. This stiffness must be increased by an effective modal height factor—the ratio of the total structure height to the effective modal height—which can be calculated or assumed to account for the fact that SDOF stiffness was calculated based on
deformation at the roof while a dynamically equivalent SDOF system has a modal height less than the full height of the structure. The SDOF stiffness is given by:

\[
k_{0EAF} = \frac{f_{hEx}}{7} \left( \sum_{i=1}^{N} k_{ExF_i} \right)
\]

(3.9)

where: \( f_{hEx} \) = modal height factor, \( N \) = total number of stories, \( k_{ExF} \) = total story stiffness considering both shear and overturning flexibilities.

The effective first modal mass can be calculated with a known (lumped) mass matrix and assumed (or determined from analysis) first mode shape. The fundamental period can be calculated using these effective SDOF mass and stiffness values. If an MDOF approach is taken, a banded diagonal stiffness matrix can be formulated using the per-story stiffness terms. This matrix can be used in conjunction with the known mass matrix to solve the eigenvalue problem and obtain natural periods and mode shapes. Both approaches provide a reasonable estimation of the fundamental period for existing and improved column design structures as discussed in Section 4, however the SDOF approach is more useful for design purposes and was found to more accurately calculate FEA model periods.

**SRC with SYL Rehabilitated Frame Stiffness Calculation**

Rehabilitated frames requiring added stiffness and strength from an SRC with SYL requires calculation of the SRC/SYL/RC system’s supplemental stiffness to determine the total first mode stiffness. A SDOF approach is taken to calculate SRC, link, and reaction column added lateral stiffness for prediction of the fundamental period for SRC-rehabilitated frames including yielding links. Note that separate consideration of dynamic properties for SRC-rehabilitated frames without yielding links is not
necessary because the first mode is expected to be unaffected by SRC inclusion without SYL. Both shear and overturning stiffness are considered similar to the procedure for braced frame stiffness calculations. Frame shear stiffness is calculated considering bending in each column (reaction column and SRC columns), axial stiffness of the SRC braces, and yielding link flexural stiffness including shear flexibility. For consideration of overturning stiffness, the SRC, yielding links, and reaction column are treated as an equivalent moment frame as shown in Figure 3.13. The hinged truss SRC columns are treated as an equivalent single column at the SRC’s geometric vertical centerline with a moment of inertia calculated using the parallel axis theorem. The cross-sectional area of this equivalent SRC column includes SRC brace and beam properties and is given by:

\[
A_{eq} = \frac{N \cdot \left( \sum_{i} h_i \right)}{\sqrt{b_{SRC}^2 + \left(\frac{h_i}{2}\right)^2} + \frac{b_{SRC}}{A_{SB}^i} + \frac{2 \cdot \left(\frac{h_i}{3}\right)^3}{3 \cdot A_{SC} \cdot b_{SRC}^2}}
\]  

(3.10)

where:  
\(i\) = story, \(N\) = total number of stories, \(h\) = story height, \(b_{SRC}\) = SRC width,

\(A_d\) = cross-sectional area of SRC brace, \(A_{SB}\) = cross-sectional area of SRC beam, \(A_{SC}\) = average cross-sectional area of SRC columns.

Roof deformation due to a unit base shear is then calculated using conventional means for a moment frame consideration. An equivalent SDOF stiffness can be calculated by considering shear and overturning stiffness acting in series and applying the modal height factor discussed previously. Existing (SDOF) braced frame stiffness is added (springs in parallel) to the SDOF SRC, SYL, and reaction column system stiffness to determine the equivalent SDOF stiffness for the SRC-rehabilitated frame. This term is used in
conjunction with the first mode’s effective mass (obtained assuming a uniform drift first mode shape or determined from analysis) to calculate the period of the first mode. The higher modes are typically neglected due to their negligible effect on seismic drift response as described in the previous section.

3.4.3 SRC Rehabilitation Design to Achieve a Target Drift

This subsection details the design procedure for SRC-rehabilitation to achieve a target drift using steel yielding links (SYL). As discussed in Section 3.4.1, maximum drift prediction using the equal energy rule requires the structure’s elastic (SDOF) stiffness and controlling plastic mechanism strength. The elastic stiffness of an existing frame rehabilitated with SRC, SYL, and RC is dependent on all members within the rehabilitative “equivalent moment frame” (SRC, SYL, and RC; see Section 3.4.2 and Figure 3.13), however the controlling plastic mechanism strength only depends on properties of the existing frame and SYL. The increase in stiffness after rehabilitation can be assumed, and the required plastic mechanism strength to achieve the target drift can be calculated using the equal energy rule and this assumed stiffness. The plastic strength of each link can be solved for using this required plastic mechanism strength and the expected plastic behavior of the braced frame after rehabilitation. Once links are selected based on their required strength, the RC can be capacity designed to remain elastic for the maximum expected link forces (Section 3.4.3.2). For strength design of the SRC, maximum expected inter-story shear forces in the SRC due to plastic mechanism formation and higher mode response are calculated using the chosen SYL, braced frame properties, and assumed dynamic properties after rehabilitation (Section 3.4.4). Once a complete rehabilitative design has been proposed (SRC, SYL, and RC), the rehabilitated
frame’s stiffness is calculated according to the procedure described in Section 3.4.2 and compared to the assumed stiffness used to calculate the required plastic mechanism strength. This procedure requires iteration to obtain a design with a combination of elastic stiffness and controlling plastic mechanism strength to achieve the target drift. A description of the SRC-rehabilitation design process is given in Section 3.5 after the design of each rehabilitative component has been detailed in the following subsections.

3.4.3.1 Design of Steel Yielding Links

Yielding links between the SRC and reaction column can be incorporated into the rehabilitative design to further reduce drift. These links must be designed to reduce the building drift by adding stiffness and strength while also transferring the interaction forces between the SRC and existing framing. Calculation procedures for yielding link design are included near the end of Appendices 2.1 through 2.4. The conservative plastic mechanism strength and predicted maximum drift for the SRC-rehabilitated frame without yielding links are needed to determine the extent of added stiffness and strength necessary to achieve a target drift. A specified target drift and maximum allowable link plastic rotation (taken as 0.08 radians; typical for an eccentrically braced frame shear yielding link) will determine the required plastic strength and length of the links, respectively. Maximum predicted drift is a function of both elastic stiffness and plastic mechanism strength in the equal energy region as discussed in Section 3.4.1. The only difference in the calculation of controlling plastic mechanism strength for SRC-rehabilitated frames with and without yielding links is the internal work term pertaining to the yielding link forces acting through their virtual deformations. An SRC with SYL and reaction column is considered to stiffen the system by a factor, \( \omega_k \). As such, yielding
link design to achieve a target drift requires the assumption of either the factor by which
the system’s stiffness will increase ($\omega_k$) or the resulting plastic mechanism strength, $V_{pRL}$.
Plastic mechanism strength is the more critical parameter in the prediction of maximum
drift therefore the stiffness factor is assumed and the required plastic strength of the links
can be calculated using an equal energy approach.

The required controlling plastic strength for the SRC-rehabilitated frame with
yielding links to achieve a target drift as a function of the stiffness factor, $\omega_k$, is equal to:

$$V_{pRL}(\omega_k) = \frac{2}{3} \omega_k \cdot \theta_t \cdot H_N \cdot k_0ExF - \omega_k \cdot k_{0ExF} \cdot \sqrt{\frac{4\theta_t^2 H_N^2}{9} - \left( \frac{W_{RL1} \cdot S_{a1}}{gB_{d0}} \right)^2}$$  (3.11)

where: $\omega_k = $ stiffness factor, $\theta_t = $ target drift, $H = $ floor elevation, $N = $ total number of
stories, $k_{0ExF} = $ existing braced frame SDOF stiffness, $W_{RL} = $ effective modal
weight including SRC and SYL, $S_a = $ spectral acceleration, $g = $ gravitational
acceleration, $B_{d0} = $ factor to adjust $S_a$ for damping.

This equation can be derived based on the equal energy consideration for predicting
maximum drift illustrated in Figure 3.20. The maximum predicted displacement is
expected to occur at the roof elevation with a uniform drift equal to the target value:
$\Delta_{max} = \theta_t H_N$. The base shear force is calculated as the product of spectral acceleration,
$S_{a1}$ (adjusted for damping using the $B_{d0}$ factor), and the first modal mass for the
rehabilitated system, $W_{RL1}/g$. The stiffness of the system is taken as the existing frame
stiffness, $k_{0ExF}$ with an applied factor $\omega_k$. Equation 3.11 is obtained by equating the area

3-29
under the elastic force-deformation curve to the area under the elastic-perfectly plastic curve and solving for $V_{pRL}$.

The difference between the required plastic mechanism strength to achieve target drift, $V_{pRL}$ (as a function of the stiffness factor), and the controlling plastic mechanism strength with SRC but not yielding links, $V_{pR}$, is set equal to the link’s contribution to the equation for $V_{pRL}$ (see Figures 3.17 through 3.19) since, as previously discussed, this term is the only difference between the two plastic mechanism strength calculations. Solving this expression for the plastic strength of each link, $V_{pL}$, results in Equation 3.12 which gives the required plastic strength of each link to achieve the target drift as a function of the stiffness factor.

$$V_{pL}(\omega_k) = V_{pRL}(\omega_k) - V_{pR}$$

where: $\omega_k = \text{stiffness factor}$, $i = \text{floor}$, $N = \text{total number of stories or floors}$,

$C = \text{coefficients for distributing plastic base shear to each floor elevation}$,

$H = \text{floor elevation}$, $b_{SRC} = \text{width of SRC}$, $e_L = \text{length of link}$, $V_{pRL} = \text{plastic base shear with SRC and SYL}$, $V_{pR} = \text{plastic base shear with SRC}$.

This equation can be solved for varying frame stiffness ratios, $\omega_k$, and link strengths, $V_{pL}$. Ideally, a design with the smallest $V_{pL}$ is desirable since it will result in less strengthening of surrounding framing and foundations and less force for design of the SRC and reaction column. However, practical limits also exist on the added stiffness of a frame. It might be necessary to add multiple SRCs per line of framing in an actual application however the calculations are presented here for a single SRC.
A design link axial force can be determined considering equilibrium of lateral forces about a section cut through each SRC story (and links). Again, this design force must be enveloped to determine the maximum axial force demand in any link considering any story as degraded \((n_{PB})\). Iteration is required between link design and SRC strength design (Section 3.4.4) because the maximum link axial force is a function of the links’ plastic strength. The maximum link axial force can be determined using the following expression:

\[
A_{L,nPB} = \begin{cases} 
A_{L1,nPB} & \text{for } nPB \in 1,2..N \\
& \text{for } j \in N, N - 1..1 \\
V_{SRC,nPB} & \text{if } j = N \\
V_{SRC,nPB} & \text{otherwise} \\
A_{L1,nPB} & \sum_{k=j}^{N} A_{L1,nPB} 
\end{cases}
\]

(3.13)

where: \(nPB = \) story with compression brace at post-buckling strength, \(N = \) total number of stories, \(j = \) floor where link is attached, \(V_{SRC} = \) design SRC inter-story shear force (including SRSS of higher mode demands).

In EBF shear link design, the maximum axial force must not exceed half the axial force capacity \((A_y F_y)\) of the section. As such, the link must have a total cross-sectional area to resist twice the calculated maximum axial demand, along with a required shear capacity to achieve the required link plastic strength \(V_{pL}\).

The length of the links, \(e_L\), can be calculated based on the prescribed maximum allowable link rotation, \(\gamma_L\), set equal to 0.08 radians based on AISC Seismic Provisions (2010). Link rotation, \(\gamma_L\), is related to frame drift, \(\theta_F\), assuming a rigid SRC and reaction column based on deformation compatibility by:
where: \( b_{SRC} = \text{width of SRC} \), \( e_L = \text{length of link} \), \( \theta_F = \text{frame drift (uniform)} \)

Link length can be solved for by inputting the maximum allowable link rotation (0.08 radians) and the target drift as the frame drift into Equation 3.14. A minimum link length equal to the depth of the link section was assigned since link lengths could get quite short in this application and the behavior of shorter links \((e_L < d)\) is unknown since they are not common for EBFs.

### 3.4.3.2 Reaction Column Design

For SRC rehabilitation without yielding links, the link members can be connected directly to the existing braced frame, possibly with some local reinforcement of the existing column to ensure it can handle the interaction forces. A reaction column between the SRC and braced frame is likely necessary when the existing braced frame column cannot adequately handle the expected forces from yielding link members or does not have the necessary stiffness to achieve the target added frame stiffness \((\omega_k k_{0Ex})\). The reaction column can be designed as a pinned-base continuous beam-column with yielding links attached to one side and simple pin-pin-ended truss members on the other at each floor elevation connected to the existing frame as shown in Figure 3.21. A trial reaction column section can be calculated considering the necessary compressive strength for the yielding links \((\Omega_{LV_{PL}})\). The controlling loading state for this member occurs during peak (uniform) drift when all links have yielded (and hardened to their maximum expected strength). The maximum axial force demand is taken as the summation of all
link plastic shear forces including over-strength/hardening. The maximum moment demand in this loading condition is the maximum link end moment \( \Omega_L V_{pL} e_L \) for shear yielding links. The reaction column will be in double curvature over each story during this controlling load state, therefore a \( C_b \) factor of 1.67 can be used to effectively reduce moment demand in the controlling interaction equation. The reaction column can be designed as a beam-column considering axial-moment interaction, and its cross-sectional area can be increased (if necessary) in successive iterations to achieve the target SRC stiffness.

### 3.4.4 SRC Strength Design

The procedure for strength design of the SRC depends on the expected plastic behavior of the existing frame and changes in the dynamic properties of the frame. The SRC must be designed to redistribute forces along the height of the existing frame to engage all LFRS braces. The significant added stiffness provided by the SRC in the higher modes due to the continuous elastic framing over the height results in significant higher mode forces that must be designed for if the SRC is to remain elastic. Such continuous elastic framing introduces dynamics which are fundamentally different than that in typical LFRS in which the primary inter-story lateral stiffness is lost as the primary lateral force resisting members yield. The approach for combining the forces to form the complete desired plastic mechanism and higher mode forces is described here.

Four SRC strength design calculation procedures are provided in Appendices 2.5 through 2.8 for all prototype frames in the following order: 3SCBF, 6SCBF (includes a column splice and taller first story), 3NCFM-WB, and 3NCFM-BC. Note that each
procedure is nearly identical but incorporates the expected brace behaviors for each frame type discussed in Section 3.1.

To determine plastic mechanism forces, a uniform drift profile is assumed for the controlling mechanism since the design intent of the SRC is to enforce a uniform drift. The difference between inter-story shear demands due to an inverted triangular distribution of the plastic base shear and the expected combined lateral brace force capacity in each story \( F_{BH} \) is defined as the differential shear, \( \Delta V \). This is the inter-story shear demand that must be carried by the braced frame columns, SRC (and reaction column), and continuous gravity framing (if considered) to enforce a uniform drift profile for the controlling plastic mechanism. Considering all compression braces at either EB or EPB results in little \( \Delta V \) demand, therefore each story’s braces must be separately considered degraded to ET/EPB (or EPB with beam pulldown for NCBF-WB; fractured for NCBF-BC). The resulting demand forces must be enveloped for design. These rehabilitated plastic mechanisms considering non-uniform brace degradation are illustrated for SCBF, NCBF-WB, and NCBF-BC frame behaviors (without yielding links) in Figures 3.14 through 3.16. When yielding links are included in the SRC-rehab design, the plastic mechanism calculation is based on that illustrated Figures 3.17 through 3.19.

The high inter-story stiffness of a hinged truss SRC results in the development of non-negligible forces during higher mode response. The higher mode forces are calculated based on principle elastic dynamics to determine the effective modal forces applied to the frame. Inter-story shear demands for each higher mode can be calculated based on the shape of each mode and the SRC’s dynamic properties (Equation 3.15), and
the inter-story shears due to higher mode response are combined using an SRSS modal combination rule.

\[ F_{i,m} = W_f \cdot (\phi_m)_i \cdot \frac{\Gamma_m}{W_m} \cdot V_m \]  

(3.15)

where:  \( i \) = degree of freedom (floor), \( m \) = mode, \( W_f \) = floor weight, \( \phi_m \) = mode shape vector, \( \Gamma_m \) = modal participation factor, \( W_m \) = effective modal weight, \( V_m \) = modal base shear (product of seismic response coefficient and modal effective weight).

These forces are added to the \( \Delta V \) forces to obtain the total inter-story shear demands that must be resisted by the braced frame columns, SRC, reaction column, and gravity framing (if considered). Multiplying this total inter-story shear demand by the SRC’s relative supplemental stiffness factor (discussed in the next paragraph) gives the SRC inter-story shear. While the higher mode forces are independent of which story is considered degraded (\( n_{PB} \)), the \( \Delta V \) term is highly dependent on \( n_{PB} \) therefore each story must be separately considered degraded and design SRC shear forces in each story are taken as the maximum from all \( n_{PB} \) considerations. Therefore, the inter-story shear forces for a given \( n_{PB} \) can be calculated by:

\[ V_{SRC_j} = P_s \cdot \left[ \Delta V_j + \sqrt{\sum_{m=2}^{N} \left( F_{j,m} \right)^2} \right] \]  

(3.16)

where:  \( j \) = story, \( P_s \) = relative supplemental story stiffness provided by SRC,

\( \Delta V = \) differential inter-story shear (demand minus brace capacity), \( m = \) mode,

\( N = \) total number of stories, \( F = \) higher mode inter-story shear.
The total inter-story shear demands considering (first mode) plastic mechanism forces and SRSS of the higher mode response forces can be distributed to the braced frame columns, SRC, reaction column, and gravity framing according to their relative inter-story stiffnesses however the SRC is expected to carry a significant portion of this force due to its relatively high stiffness and was found to carry about 85% of this force for cases considered. All columns (braced frame, reaction, gravity, and in the SRC) consider their flexural stiffness over each story when subjected to a unit lateral displacement with boundary conditions assumed based on member connectivity. For the hinged truss SRC configuration examined in this thesis, the horizontal projection of the axial stiffness of SRC braces must also be considered. The total supplemental inter-story stiffness values are calculated as the summation of these individual stiffnesses on each story and represents the total stiffness of lateral force resisting elements acting in parallel with the primary LFRS members (braced frame braces). SRC inter-story stiffness is divided by the total supplemental inter-story stiffness to determine what relative percentage of the total inter-story shear demand ($\Delta V$ plus SRSS of higher mode forces) will be taken by the SRC on each story.

Once appropriate SRC shear design forces are determined considering plastic mechanism forces combined with higher mode response forces, the SRC can be designed accordingly. Since the SRC is designed to remain elastic as a capacity protected “element”, its design can be performed using conventional approaches for elastic design of braced frames such as the uniform force method. For a hinged truss SRC design, the braces on each story can be assumed to carry the entire inter-story shear demand through
axial tension and compression. Beams and columns can be designed for resulting brace forces as shown in Figure 3.21.

### 3.5 Rehabilitative Design Procedure

This section briefly describes the SRC-rehabilitation design process which results in a rehabilitated braced frame structure that achieves the target drift performance objective under the seismic hazard of interest. This process is depicted as a flowchart in Figure 3.22 and described as follows:

1. Evaluate existing frame assuming a soft-story controlling plastic mechanism
   a. Calculate controlling plastic mechanism strength separately considering each story as soft (Section 3.3)
   b. Calculate the frame’s elastic stiffness and dynamic properties (Section 3.4.2)
   c. Predict the maximum drift using the equal energy rule (Section 3.4.1). If predicted drift is less than the desired performance objective limit (Section 3.1.3), rehabilitation is not necessary. Otherwise, proceed to step (2)

2. Evaluate SRC-rehabilitated frame without SYL assuming a uniform drift controlling plastic mechanism (no added strength or stiffness)
   a. Calculate controlling plastic mechanism strength considering uniform drift response (Section 3.3.1)
   b. Predict the maximum drift using the equal energy rule (Section 3.4.1). If predicted drift is less than the desired performance objective limit (Section 3.1.3), SYL are not necessary and the SRC can be capacity designed for
strength (Section 3.4.4); proceed to step (4). Note that SRC strength design might require iteration due to the design forces’ dependence on the dynamic properties of the SRC-rehabilitated frame (higher mode forces), which themselves depend on the chosen SRC members. If the predicted drift is unacceptable, proceed with SRC/SYL/RC design (step 3).

3. Determine required controlling plastic mechanism strength to achieve target drift using the equal energy rule and an assumed rehabilitated stiffness factor, $\omega_k$, which can typically be assumed to be between 1 and 3.

   a. Choose links to provide required additional plastic strength and select link length based on geometry and maximum allowable rotation (0.08 radians; Section 3.4.3.1). It is desirable to choose the minimum link strength for a given stiffness factor ($\omega_k$) to minimize forces imparted onto the SRC, RC, and foundation.

   b. Capacity design reaction column for maximum expected link forces considering axial-moment interaction (Section 3.4.3.2).

   c. Design a trial SRC for strength considering plastic mechanism forces and assumed higher mode forces (or previously calculated higher mode forces if performing iteration of SRC strength design). For example, plastic mechanism forces can be multiplied by 1.3 to assume the higher modes add 30% of the plastic mechanism forces for SRC strength design (Section 3.4.4).

      i. Calculate the stiffness and dynamic properties of the SRC-rehabilitated frame (Section 3.4.2)
ii. Check SRC strength design including calculated higher mode forces and iterate (3.c) if necessary until the SRC has adequate capacity to remain elastic

d. Compare calculated stiffness increase after rehabilitation to assumed trial stiffness factor

i. If calculated stiffness factor does not agree with assumption, return to step (3) and choose a more reasonable trial stiffness factor. Note that the calculated stiffness factor can be increased without complete redesign/iteration by increasing the cross-sectional areas of SRC and RC members however this may not be adequate to achieve convergence

ii. If calculated and assumed stiffness factors reasonably agree, calculate maximum axial force in links based on SRC inter-story shear demands (Equation 3.13). If links do not have sufficient cross-sectional area to resist the axial force, return to step (3.a) and select a new link section. Otherwise, proceed to step (4).

4. At the conclusion of the preceding steps, design of an SRC, SYL (if needed), and RC (if included) that provide the required controlling plastic mechanism strength and elastic stiffness to achieve the target drift under the seismic hazard of interest will be complete. This rehabilitative design should be checked for strength, stiffness, and maximum predicted drift performance considering the final selected member sizes, dynamic properties, and higher mode effects to ensure this iterative procedure has converged and the resulting structure will behave as intended.
3.6 Improved Column Design for New Special Concentrically Braced Frames

An improved braced frame column design procedure for new SCBF design is proposed in this section. The improved column design procedure requires an estimate of the peak inter-story differential shear resulting from non-uniform brace degradation, as was done for SRC design. However, unlike SRC design, higher mode forces were not considered for improved braced frame column design because the low stiffness of columns bending (relative to a hinged truss SRC in shear) results in very small braced frame column forces during higher mode response.

The full calculation procedures are given in Appendices 2.9 and 2.10 for a three-story (3SCBF) and six-story (6SCBF) structure, respectively. Note that these procedures are identical with the exception of a column splice and non-uniform story heights (first story is taller) for the 6SCBF. As discussed in previous sections, the intent of this improved column design procedure is to produce a frame whose controlling plastic mechanism maintains the uniform drift profile shown in Figure 3.7 given the expected brace forces (including non-uniform degradation) shown in Figure 3.8. The plastic base shear for this mechanism is calculated using Equation 3.2 except the “nPB” story’s internal brace work term uses ET and EPB forces. To determine lateral demands, the calculated plastic base shear for this mechanism is then distributed as an inverted triangular set of point forces acting at each floor elevation. With known lateral external forces and expected internal brace forces, column axial, shear, and moment demands can be determined using equilibrium about section cuts through each story as shown in Figure 3.23. Shear and bending moment demands are assumed to be equally distributed to each braced frame column, and static pushover analysis confirms this assumption to be reasonable. Column
axial force demands are required for design and to calculate the column moments from equilibrium:

\[
A_n = \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \left[ \sum_{i=n+1}^{N} \left( F_{bEPB_i} \cdot \sin(\theta_b) \right) \right. \\
\left. \begin{array}{c}
\text{if } i = n_{PB} \\
\text{if } n < N \\
0 \text{kip otherwise}
\end{array} \right]
\]  

(3.17)

where: \( n = \text{story}, \ i = \text{story or floor}, \ N = \text{total number of stories or floors}, \ P_D = \text{gravity load tributary to braced frame columns}, \ F_{BV} = \text{vertical component of combined brace forces}, \ F_{bEPB} = \text{axial brace expected post-buckling strength}, \ \theta_b = \text{angle of braces with respect to horizontal}, \ n_{PB} = \text{story with compression brace at post-buckling strength}, \ F_{bEB} = \text{axial brace expected buckling strength}.

The moment diagram of each column can be obtained using the calculated column moments (at the top of each story; Equation 3.18), shear forces (constant over a given story; Equation 3.19), and story heights, which can be used to calculate column moments at the bottom of each story (Equation 3.20)

\[
M_{T_n} = \frac{1}{2} \cdot \left[ \sum_{i=n+1}^{N} C_i \cdot V_p \cdot \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} \cdot L \right) + F_{BV_n} \cdot \frac{L}{2} - A_n \cdot L \text{ if } n < N \\
\sum_{i=n}^{N} \left( P_{D_i} \cdot L \right) + F_{BV_n} \cdot \frac{L}{2} - A_n \cdot L \text{ otherwise}
\]  

(3.18)

where: \( n = \text{story}, \ i = \text{story or floor}, \ N = \text{total number of stories or floors}, \ C = \text{coefficients distributing plastic base shear to each floor elevation}, \ V_p = \)
plastic base shear, \( j = \) story, \( h = \) story height, \( P_D = \) gravity load tributary to braced frame columns, \( L = \) width of braced frame, \( F_{BV} = \) vertical component of combined brace forces, \( A_n = \) braced frame column axial force.

\[
V_n = \sum_{i=n}^{N} \left( \frac{C_i \cdot V_p}{2} \right) - \frac{F_{BH_n}}{2}
\]  

(3.19)

where: \( i = \) story or floor, \( N = \) total number of stories or floors, \( F_{bhET} = \) horizontal component of brace expected tension force, \( F_{bhEPB} = \) horizontal component of brace expected post-buckling strength, \( h = \) story height, \( M_{pc} = \) column plastic moment capacity, \( H = \) floor elevation, \( C = \) coefficients distributing plastic base shear to each floor elevation.

\[
M_{B_n} = M_{T_n} + V_n \cdot h_n
\]  

(3.20)

where: \( i = \) story or floor, \( N = \) total number of stories or floors, \( F_{bhET} = \) horizontal component of brace expected tension force, \( F_{bhEPB} = \) horizontal component of brace expected post-buckling strength, \( h = \) story height, \( M_{pc} = \) column plastic moment capacity, \( H = \) floor elevation, \( C = \) coefficients distributing plastic base shear to each floor elevation.

A design moment diagram is then constructed considering the maximum column moment in each story. Once design axial force and bending moment diagrams are obtained considering each story’s braces degraded (\( n_{PB} \)) to ET/EPB separately, maximum demands from all \( n_{PB} \) considerations are used to design the columns considering axial-moment interaction based on AISC 360 (2010).
If fixed-base braced frame columns are considered, the proposed improved column design procedure is an iterative process due to the plastic base shear being a function of the plastic moment at the base of the columns however the plastic bending capacity of the columns does not significantly affect the plastic base shear strength compared to the braces and thus few iterations (if any) are necessary. This design procedure yields a frame whose columns can adequately resist the inter-story shear demand imposed by non-uniform brace degradation occurring on any story without forming a panel mechanism (soft-story).

3.6.1 Drift Prediction for Improved Column Design

For a conservative calculation of the plastic base shear for the resulting uniform drift plastic mechanism, all tension and compression braces are assumed to be at $F_{ET}$ and $F_{EPB}$, respectively. This mechanism is illustrated in Figure 3.14. Note that an SRC (with pin-pin-ended links) is included in this illustration. This is because the same (conservative) plastic mechanism controls for braced frames with improved columns or existing frames rehabilitated using an SRC without yielding members. The base shear force for this conservative uniform drift plastic mechanism is given by Equation 3.21 and can be used to predict maximum drift of improved column design braced frames for a given earthquake hazard as discussed in Section 3.4.1.
\[
V_{p,R,UD} = \frac{\sum_{i=1}^{N} \left[ (F_{bhET_i} + F_{bhEPB}) \cdot h_i \right] + 2M_{pc_i}}{\sum_{i=1}^{N} (H_i \cdot C_i)}
\]

where: \( i = \) story or floor, \( N = \) total number of stories or floors, \( F_{bhET} = \) horizontal component of brace expected tension force, \( F_{bhEPB} = \) horizontal component of brace expected post-buckling strength, \( h = \) story height, \( M_{pc} = \) column plastic moment capacity, \( H = \) floor elevation, \( C = \) coefficients distributing plastic base shear to each floor elevation.

### 3.7 Summary

This section presented methods for calculating critical values describing the nonlinear, dynamic behavior of existing sub-standard and rehabilitated frames. Quantifying critical values such as plastic frame strength, frame lateral stiffness, peak internal member forces, and link deformations led to the development of simplified methods for predicting frame drift and forces for strength design. The simplified methods can be used in a design procedure to size rehabilitation components (SRC, SYL, RC) to achieve desirable seismic performance. These design procedures and predicted response quantities are used in Section 4 and compared with nonlinear transient seismic analysis results. Additionally, a design procedures for improved column design for new CBFs was proposed and will also be applied and evaluated in Sections 4 and 5.
Table 3.1: Prototype Gravity Framing and Equivalent Single Gravity Columns

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<th>Prototype Gravity Column Properties</th>
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### Table 3.2: Prototype Frame Critical Properties for SRC-Rehabilitation and Improved Column Design

| Frame | Story/Floor | $h_i$ (feet) | $F_{ET}$ (kips) | $F_{EB}$ (kips) | $F_{EPB}$ (kips) | $M_{pc}$ (kip-ft) | $M_{pb}$ (kip-ft) | $V_p$ (kips) | $V_p/W_{sl}$
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<td>507.5</td>
<td>270.1</td>
<td>81.0</td>
<td>1073.0</td>
<td>2763.8</td>
<td>2612.5</td>
<td></td>
</tr>
<tr>
<td>3NCBF</td>
<td>1</td>
<td>13</td>
<td>577.7</td>
<td>394.8</td>
<td>118.4</td>
<td>673.8</td>
<td>N/A</td>
<td>1902.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>13</td>
<td>627.3</td>
<td>395.6</td>
<td>118.7</td>
<td>673.8</td>
<td>N/A</td>
<td>2140.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>13</td>
<td>507.5</td>
<td>234.0</td>
<td>70.2</td>
<td>673.8</td>
<td>N/A</td>
<td>1902.1</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- All frames were thirty feet wide (column centerline-to-centerline).
- **a** Controlling plastic mechanism strength for existing frames: soft-story formation, $n_{PB} = 1$.
- **b** Ratio of controlling plastic mechanism strength to first mode effective weight.
- **c** Expected brace forces without considering expected pulldown or connection fracture behaviors.
### Table 3.3: Prototype SRC and Link Properties

<table>
<thead>
<tr>
<th>Frame</th>
<th>Reaction Column&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Section</th>
<th>(V_p) (kips)&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Length (in)</th>
<th>(I_{equiv}) (in&lt;sup&gt;4&lt;/sup&gt;)&lt;sup&gt;e&lt;/sup&gt;</th>
<th>SRC</th>
<th>Braces</th>
<th>Beams</th>
<th>Short Column&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Long Column&lt;sup&gt;a&lt;/sup&gt;</th>
<th>(k_{Rehab}/k_{0\text{EAf}})&lt;sup&gt;e&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SCBF</td>
<td>W14x342</td>
<td>W30x116</td>
<td>480.0</td>
<td>30.0</td>
<td>2363.0</td>
<td></td>
<td>W14x193</td>
<td>W14x193</td>
<td>W33x169</td>
<td>W14x109</td>
<td>1.458</td>
</tr>
<tr>
<td>6SCBF</td>
<td>W14x233</td>
<td>W14x233</td>
<td>202.0</td>
<td>33.0</td>
<td>351.0</td>
<td></td>
<td>W14x193</td>
<td>W14x193</td>
<td>W14x211</td>
<td>W14x176</td>
<td>1.143</td>
</tr>
<tr>
<td>3NCBF-WB</td>
<td>W14x145</td>
<td>W10x88</td>
<td>160.0</td>
<td>48.0</td>
<td>252.0</td>
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<td>W14x145</td>
<td>W14x145</td>
<td>W14x159</td>
<td>W14x74</td>
<td>1.238</td>
</tr>
<tr>
<td>3NCBF-BC</td>
<td>W14x455</td>
<td>W30x148</td>
<td>533.0</td>
<td>48.0</td>
<td>3164.0</td>
<td></td>
<td>W14x233</td>
<td>W14x257</td>
<td>W36x231</td>
<td>W14x159</td>
<td>1.723</td>
</tr>
</tbody>
</table>

**Notes:**

- **a** Reaction column and SRC columns are continuous members with the same sections for all stories.
- **b** Yield strength assigned to rigid-plastic shear spring element for FEA (Section 4).
- **c** Equivalent moment of inertia assigned to link beam elements for FEA to capture elastic shear deformations in flexure.
- **d** SRC inter-story shear demand calculated considering MCE hazard level and yielding links.
- **e** Ratio of calculated SDOF stiffness after rehabilitation (with SYL) to existing frame SDOF stiffness.

All elements were modeled (FEA) as linear-elastic with the exception of the rigid-plastic zero-length link shear spring.
Figure 3.1: 3SCBF and 3NCBF Prototype Braced Frames

NOTE: FOR 3SCBF IMPROVED COLUMN DESIGN, ALL COLUMN SECTIONS ARE W14x211.

TRIBUTARY MASS FOR EACH 3SCBF AND 3NCBF BRACED FRAME IS 1,687 KIP-S/IN PER FLOOR (AND ON ROOF).

TRIBUTARY P-Δ LOAD FOR EACH 3SCBF AND 3NCBF BRACED FRAME IS 919 KIPS PER FLOOR (FIRST AND SECOND) AND 863 KIPS ON THE ROOF.
Figure 3.2: 6SCBF Prototype Braced Frame

NOTE: FOR 6SCBF IMPROVED COLUMN DESIGN, LOWER COLUMN SECTIONS ARE W14x500, UPPER COLUMN SECTIONS ARE W14x311.
Plastic flexural hinge forms when:

$$F_{BV} = F_{PD} = \frac{4M_{pb}}{L}$$

Vertical resultant of combined brace force:

$$F_{BV} = (F_{TEPB} + F_{EPB}) \sin(\phi_{br})$$

Solving for $F_{TEPB}$:

$$F_{TEPB} = \frac{4M_{pb}}{L \sin(\phi_{br})} + F_{EPB}$$

Figure 3.3: Beam Pull-Down Free Body Diagram
3SCBF prototype frame model configuration naming convention: 3S_C#_R#_L#_G# (as needed).

Each configuration (12 total) subjected to MCE and NF ground motion suites.

**Figure 3.4: 3SCBF Configurations**
6SCBF prototype frame model configuration naming convention:  
6S_C#_R#_L#_G# (as needed).

Each configuration (15 total) subjected to MCE and NF ground motion suites.

Figure 3.5: 6SCBF Configurations
3NCBF prototype frame model configuration naming convention: $3N_R#_L#_G#$ (as needed).

Each configuration (9 total) subjected to DBE and MCE ground motion suites.

**Figure 3.6: 3NCBF Configurations**
Figure 3.7: Uniform Drift Plastic Mechanism

Brace Internal Work:

$$W_{Ib} = \sum_{i=1}^{N} \left[ \left( F_{bhET_i} + F_{bhEB_i} \right) \cdot \theta_t \cdot h_i \right]$$

Column Internal Work:

$$W_{Ic} = 2 M_{pc_1} \cdot \theta_t$$

External Work:

$$W_E = V_p \cdot \sum_{i=1}^{N} \left( \theta_t \cdot H_i \cdot C_i \right)$$

Existing frame uniform drift plastic mechanism strength:

$$W_E = W_{Ib} + W_{Ic}$$
Figure 3.8: Existing SCBF Soft-Story Plastic Mechanism

Brace internal work:
\[ W_{ib} = \left( F_{bhETn_{PB}} + F_{bhEPBn_{PB}} \right) \cdot \theta_t \cdot h_{n_{PB}} \]

Column internal work:
\[ W_{ic} = \begin{cases} 2M_{pc_3} \cdot \theta_t & \text{if } n_{PB} = 3 \\ 4M_{pc_{nPB}} \cdot \theta_t & \text{otherwise} \end{cases} \]

External work:
\[ W_E = v_{p} \cdot \theta_t \cdot h_{n_{PB}} \cdot \sum_{i=n_{PB}}^{N} C_i \]

Soft-story plastic mechanism strength:
\[ W_E = W_{ib} + W_{ic} \]
\[ v_{p} = \frac{n_{PB}}{h_{n_{PB}} \cdot \sum_{i=n_{PB}}^{N} C_i} \]

ET = Expected Tension  
EB = Expected Buckling  
EPB = Expected Post-Buckling
Figure 3.9: Existing NCBF-WB Soft-Story Plastic Mechanism
Figure 3.10: Existing NCBF-BC Soft-Story Plastic Mechanism

Column internal work:

\[ W_{ic} = \begin{cases} 
2 M_{pc} \cdot \theta_t & \text{if } n_{PB} = 3 \\
4 M_{pcn_{PB}} \cdot \theta_t & \text{otherwise} 
\end{cases} \]

External work:

\[ W_E = V_p \cdot \theta_t \cdot h_{nPB} \cdot \sum_{i=n_{PB}}^{N} C_i \]

Soft-story plastic mechanism strength:

\[ W_E = W_{ic} \]

\[ V_p = \begin{cases} 
2 M_{pc} & \text{if } n_{PB} = 3 \\
4 M_{pcn_{PB}} & \text{otherwise} 
\end{cases} \]
Figure 3.11: SRC-Rehabilitation Concept
### Link Rotation

<table>
<thead>
<tr>
<th>Stepping Truss</th>
<th>Hinged Column</th>
<th>Hinged Truss</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{p,\text{link}} = \frac{(L - e) \Delta_r}{b_{\text{SRC}}} )</td>
<td>( \gamma_{p,\text{link}} = \frac{\Delta_r}{h_{\text{SRC}}} )</td>
<td>( \gamma_{p,\text{link}} = \left(1 - \frac{2}{L}</td>
</tr>
</tbody>
</table><p>ight) \frac{\Delta_r}{h_{\text{SRC}}} ) |</p>

#### Notes

- Unsymmetric Hysteretic Behavior with 1-sided implementation of links
- Symmetric Global Hysteretic Behavior with 2-sided link implementation (not shown)
- Self-centering force through post-tensioning or Re-centering forces applied post-earthquake
- Requires a minimum restoring force of \( n_L \gamma_p V_p \) on leg attached to links and \( V_p e/2b_{\text{SRC}} \) on other leg
- Distance for link rotation will depend on connection detail used at base (shown for mechanical hinge or rocker bearing with center of rotation at center of SRC)
- Symmetric Global Hysteretic Behavior with center of rotation at center of SRC
- Small link rotation amplification with this configuration (smaller energy dissipation for given building drift)
- Smaller lever arm on restoring force requiring large restoring force
- Self-centering likely difficult and Re-centering post-earthquake possible through tensioned diagonal cables

### Figure 3.12: SRC Types

- A simple non-moment resisting base plate detail may be used as hinge support (same rotation as gravity column bases)
- Symmetric Global Hysteretic Behavior
- Self-centering may not be possible
- Re-centering post-earthquake possible through post-tensioning or jacking

---

Stiff Rocking Core

Hysteretic Energy Dissipating Elements (Link)
Figure 3.13: Equivalent Moment Frame Illustration for SRC Flexural Stiffness Calculation
Figure 3.14: Rehabilitated SCBF Uniform Drift Plastic Mechanism
Figure 3.15: Rehabilitated NCBF-WB Uniform Drift Plastic Mechanism
Figure 3.16: Rehabilitated NCBF-BC Uniform Drift Plastic Mechanism

Column internal work:
\[ W_{Ic} = 2M_{pc1} \cdot \theta_t \]

External work:
\[ W_E = V_p \sum_{i=1}^{N} (\theta_i \cdot H_i \cdot C_i) \]

Uniform drift plastic mechanism strength:
\[ V_p = \frac{2M_{pc1}}{N} \sum_{i=1}^{N} (H_i \cdot C_i) \]
Figure 3.17: Rehabilitated SCBF Plastic Mechanism with Yielding Links
Figure 3.18: Rehabilitated NCBF-WB Plastic Mechanism with Yielding Links
Figure 3.19: Rehabilitated NCBF-BC Plastic Mechanism with Yielding Links
Figure 3.20: Equal Energy Calculation to Obtain Required Rehabilitated Frame Plastic Mechanism Strength to Achieve Target Drift
- SRC braces are sized considering the differential inter-story shear ($\Delta V$) between plastic mechanism lateral demands and braced frame capacities separately considering each story at $n_{PB}$ (plus higher mode forces).
- Link strength ($V_{pl}$) and length ($e_L$) chosen to achieve target drift.
- RC is capacity designed for axial-moment interaction: $A = 3 \sum_{i=1}^{N} V_{pl} \cdot M = \sum_{i=1}^{N} V_{pl} e_L$.
- SRC columns are designed for axial-moment interaction considering the maximum demands from all combinations of expected SRC brace forces and link forces/moments.
- SRC beams are designed for maximum expected axial demand.

Figure 3.21: Braced Frame, Reaction Column, Yielding Links, and SRC Free Body Diagram
Figure 3.22: SRC-Rehab Design Procedure Flowchart
Figure 3.23: Improved Braced Frame Column Design Free Body Diagram

\[ M_T = \frac{1}{2} \sum_{i=n}^{N} \left[ C_i V_p \left( \sum_{j=n}^{i} h_j \right) + \sum_{j=n}^{i} P_{D,j} L_j + F_{BV,n} \frac{L}{2} - A_n L \right] \text{ if } n < N \]

\[ V_n := \sum_{i=n}^{N} \left( \frac{C_i V_p}{2} \right) - \frac{F_{BH,n}}{2} \]

\[ M_B = M_T + V_n h_n \]
Section 4: Seismic Performance Evaluation of Prototype Braced Frames using Nonlinear Dynamic Finite Element Analysis

A seismic performance evaluation of existing and rehabilitated buildings was performed to examine the fundamental behaviors discussed in the previous section and demonstrate the effectiveness of SRC rehabilitation and improved column design. Prototype buildings were selected and frames were designed to represent both modern and older low- and mid-rise steel braced frame construction as described in Section 3.1. Several configurations of each prototype frame were considered, and finite element models developed for each configuration were subjected to a series of ground motions. Full details on the prototype buildings, finite element models, numerical simulations, and results are included in this section.

4.1 Finite Element Models

Finite element models of the prototype building frames developed in OpenSees (McKenna and Fenves, 2004) by Sanchez-Zamora (2013) were adapted for seismic performance evaluation. These models included the prototype braced frame, hinged truss SRC with reaction column (for rehabilitated configurations), and gravity column containing all tributary mass and P-Δ loads. Several features were included in the finite element models to accurately capture the nonlinearity and degradation of strength and stiffness expected in the LFRS (primarily in the braces) during a seismic event. Details
of these models, including the features described below, are depicted in Figure 4.1 for a three-story frame with SRC.

### 4.1.1 Braced Frame

All prototype braced frames were thirty feet wide with story heights of thirteen feet, with the exception of the first story of the six-story frame which was eighteen feet tall. These frames featured continuous columns (splice between third and fourth stories for six-story frame) with fixed base connections. While fixed base connections are not typical in a braced frame design they were chosen for consistency with the extensive verification study performed by Sanchez-Zamora in which fixed-base LFRS columns were used. Additionally, Sabelli justified the use of fixed-base LFRS columns in his original prototype braced frame designs by reasoning that the base connection detail is approximately fixed due to the rigidity and anchorage requirements provided by the relatively large gusset plates for first story braces. The mid-span node in each braced frame beam was horizontally constrained to the mass-containing gravity column node at that elevation thus assuming a rigid in-plane movement provided by the floor diaphragm.

### Elements and Materials

All braced frame members were comprised of nonlinear force-based fiber beam-column elements with three integration points for braces and four integration points for beams and columns. The “Steel02” material built into OpenSees was utilized for all nonlinear members and features a gradual transition between linear-elastic and nonlinear behavior with kinematic and isotropic strain hardening, and a 1% kinematic strain hardening ratio was assigned. Recommended values of 20, 0.925, and 0.15 were assigned to the three material parameters that control the transition between elastic and
plastic behavior. Isotropic hardening parameters were assigned to increase the tension and compression yield envelopes by 0.05% of the yield strength after a plastic strain. A modulus of elasticity of 29,000 ksi was used for all members, and expected yield strengths \(= R_y F_y\) of 55, 64.4, and 54 ksi were specified for all beams/columns \(F_y = 50\) ksi, \(R_y = 1.1\), braces \(F_y = 46\) ksi, \(R_y = 1.4\), and gusset plate sections \(F_y = 36\) ksi, \(R_y = 1.5\), respectively. The cross-section fiber discretization for all wide flange shape members (beams and columns) consisted of eight rectangular fibers along the thickness of each flange and four rectangular fibers along the depth of the web. Similarly, HSS shape sections (for braces) were discretized into eight fibers along the thickness of the top and bottom walls (walls furthest from buckling axis) and four fibers for each side wall. These FEA mesh discretization—pictured in the top right corner of Figure 4.1—were selected and verified in previous studies (Sanchez-Zamora, 2013).

**Connections and Rigid Offsets**

Linear-elastic rigid end offsets were included at all braced frame connections to account for the rigidity provided by the various connecting members in these regions. Beam-to-column connections in the 3SCBF and 6SCBF frames were offset from the centerline intersection by thirty inches and modeled as pinned to represent a beam fillet welded to the column flange and corner gusset plate with a shear tab (moment release) in the beam at the end of the gusset, which is a typical connection detail in modern braced frame design. At the roof beam-to-column connections, offsets of seven inches—about half of the columns’ depth—were used because the absence of corner gusset plates in these regions allows for a simpler pinned connection detail at the face of the column flange. The 3NCBF models utilized a seven inch offset for all beam-to-column
connections because an angle section bolted to the corner gusset and beam and welded to the column face was identified by Sloat et al. (2013) as a typical connection detail in older braced frames. Sabelli used this type of connection detail in his original prototype frame designs and assumed all beam-to-column connections with a corner gusset plate to be fully fixed, which is likely not representative of their true behavior. For this study, all 3NCBF beam-to-column connections with a corner gusset plate (all floors except the roof) were modeled as partially fixed by the inclusion of a nonlinear zero-length fiber element representing the beam’s web area at the beam-to-column moment release. Partial fixity was utilized because the assumed beam-to-column connection detail has the potential to develop a non-negligible moment during beam bending due to the coupled bolt forces acting at larger distances relative to the SCBF connection detail, but this moment is not likely to approach the full capacity of the beam. The 3NCBF roof beam-to-column connection was modeled as pinned. For all prototype frame models, columns were modeled as rigid from the bottom flange of each connecting beam to the top of the corner gusset plate, and beams were rigid at mid-span along the entire length of the connecting gusset plate. Braces were modeled as rigid from their joint centerline intersections to the ends of the brace-to-gusset welds.

**Bracing Members**

Braces were modeled using fifteen nonlinear force-based beam-column elements per brace. An initial out-of-straightness of 0.5% of the brace’s effective length (as shown in Figure 4.1) was included to capture buckling behavior under compressive loads. When braces are subjected to repeated compression buckling and tension yielding, the accumulation of large inelastic strains can lead to low-cycle fatigue rupture. To model
this behavior, the “Fatigue material” in OpenSees was specified for all brace elements to include fatigue assessment of the Steel02 material model assigned to the braces and described previously. The “Fatigue material” uses a modified rainflow cycle counting algorithm to quantify accumulated damage using Miner’s Rule and if a fiber reaches the specified damage threshold, it returns a stress of effectively zero ($10^8$ times the stress calculated using the parent material) for any strain, eliminating its contribution to the model’s strength and stiffness. If a fiber reaches the point of fracture in compression, it retains the parent material’s stress-strain relationship until the compressive strain is relieved, at which point the material fails and zero stress is returned for the remainder of the simulation. This prevents braces from fracturing in compression, which is consistent with physical fracture phenomena. Defining low-cycle fatigue failure at the material level allows fibers in a section to fail individually (as opposed to a section-level fatigue definition, where an entire section fails at once), which is consistent with the gradual tearing behavior observed in experimental tests. Additionally it should be noted that while the model does capture member buckling, it does not capture the local buckling behavior of the bracing member walls which can significantly increase the localized plastic strains on the member that accelerate low-cycle fatigue. However, the parameters assigned to the fatigue material model were calibrated to experimental test results for bracing members similar to the sizes considered here and thus to some extent do account for the effects of local buckling on fatigue damage. A linear-elastic truss element with very small area relative to the HSS brace was included in parallel with each brace to diminish convergence issues due to the rapid changes in strength and stiffness caused by brace fracture.
For the 3NCBF, modeling of brace-to-gusset weld fracture—a purposely under-designed detail for this frame—was considered using the maximum strain option within the Fatigue material definition. The end elements in each brace had this option set to fail the material when a tensile strain corresponding to the weld’s design stress was exceeded. Note that this behavior is not related to fatigue, but allows for brittle fracture simulation of deficient welds. Inclusion of this brittle connection behavior led to the development of an additional 3NCBF prototype braced frame model, referred to as 3NCBF-BC (brittle connection), which was subjected to the same ground motions for each 3NCBF configuration detailed in Figure 3.6. The 3NCBF model without brittle connections is controlled by a beam pulldown plastic mechanism as the braces transition from expected to expected post-buckling strength and will thus be referred to as 3NCBF-WB (weak beam).

**Gusset Plates**

Gusset plate sections were modeled using nonlinear beam-column elements with rectangular cross-sections between the rigid end offsets and braces (see Figure 4.1) to capture the “folding” behavior along the yield line expected during brace buckling in well-designed concentrically braced frames. In reality, square HSS braces (such as those considered in this study) buckle out-of-plane because the gusset section’s out-of-plane flexural stiffness is small relative to its in-plane stiffness resulting in a larger effective compressive buckling length for the out-of-plane direction. For analytical modeling, each gusset plate section was rotated 90 degrees about the longitudinal axis of the brace to allow this folding action and brace buckling to occur in-plane with the rest of the
model, eliminating the need for—and computational cost of—including a third spatial dimension.

4.1.2 SRC, Reaction Column, and Links

**SRC**

For this study, the SRC consists of a hinged truss with continuous columns, a pinned base, and pin-connected beams and braces as shown in Figure 4.1. The hinged truss design represents a realistic design for the SRC as described in Section 3.4. A hinged truss SRC was also used during the experimental tests and thus provides consistency between analytical modeling and experimental testing (see Section 5.3.3.1). All SRC elements were linear-elastic as it is designed to remain elastic and inelastic demands were not observed in any analytical simulations or experimental tests. The section properties for the SRC members, reaction column, and links are shown in Table 3.3 and were designed according to the procedures detailed in Section 3.4.

**Reaction Column**

A reaction column between the braced frame and SRC was included in all rehabilitated model configurations (see Figure 4.1). This continuous pinned-base column was attached to the braced frame via rigid elastic truss elements. As described in Section 3.4.3.2, the reaction column must be adequately designed to remain elastic when subjected to the axial and bending demands imposed by the fix-fix-ended links, and its cross sectional area is a key parameter for calculating the rehabilitated frame’s lateral stiffness to achieve a target drift. The reaction column was modeled using linear-elastic beam-column elements and inelastic demands were not observed during any simulation.
Steel Yielding Links

The steel yielding link members connecting the SRC to the reaction column are expected to develop bi-linear displacement based force-deformation behavior in the transverse direction with kinematic and isotropic hardening similar to that of an eccentrically braced frame (EBF) link. Axial force interaction effects were not considered on the link element behavior in the transverse direction, and axial loads were resisted in a linear-elastic manner. The link sections were designed to provide a rehabilitated frame stiffness and strength to achieve a target drift while also transferring the interaction forces from the sub-standard frame as described in Section 3.4.3.1. The links were modeled using two linear-elastic beam-column elements with a zero-length nonlinear shear spring in between as illustrated in Figure 4.1. The linear-elastic beam elements were assigned properties to capture the elastic response of the link elements, while the rigid-plastic nonlinear shear spring properties were specified to match the shear force at yield of the fix-fix-ended link. The use of shear springs and linear elastic beam elements in this manner could be used to model any general force-deformation behavior representing shear yielding or flexural yielding link members. The “Steel01” material in OpenSees was utilized for the shear springs and features a bilinear stress-strain relationship with optional kinematic and isotropic hardening parameters. Preliminary predicted drift performance of the SCBF frames rehabilitated with full-height SRC, reaction column, and pin-pin-ended links under the MCE hazard level indicated that the 3SCBF and 6SCBF would achieve the life safety (1.5%) and collapse prevention (2%) performance objectives, respectively. Fix-fix ended links were designed according to the procedure in Section 3.4.3.1 to achieve the next performance objective for rehabilitative
design: immediate occupancy (0.5%) for the 3SCBF and life safety (1.5%) for the 6SCBF. Links for the 3NCBF-WB and 3NCBF-BC models were designed to achieve the collapse prevention performance objective for rehabilitative design because the predicted MCE drift response of these frames rehabilitated with SRC, reaction column, and pin-pin-ended links was greater than 2%. An equivalent moment of inertia was assigned to the elastic beam link elements to capture both the elastic bending and shear deformations since the element type only includes bending deformations however these short members would in reality have significant shear deformations.. The elastic stiffness of the nonlinear link shear spring was set to 1000 times the flexural stiffness of the spring-to-reaction column elastic beam element with fixed ends in double curvature to essentially eliminate elastic deformations from the spring element. A 1% kinematic strain hardening ratio was assigned with no isotropic hardening. While isotropic hardening is expected in an EBF-type link, this behavior is highly dependent on the specific link design and kinematic hardening is believed to reasonably capture a general hardening behavior in the range of plastic link deformations observed in this study. For pin-pin-ended link model configurations (discussed in Section 3.2), additional moment releases were added between the linear-elastic beam link elements and the SRC and reaction column, eliminating the contribution of the nonlinear link shear spring and effectively rendering the link as a linear-elastic truss element.

4.1.3 Gravity Column

Gravity framing was modeled using a single column comprised of continuous beam-column elements with cross-sectional properties (A, I, Z) equal to the sum of all tributary gravity columns. Tributary gravity column and equivalent single column
properties for all prototype frames are provided in Table 3.1. The gravity column also included all tributary inertia mass and gravity loads through concentrated mass and a vertical point force at each floor elevation. The gravity loads allow for P-Δ effects on the frames through a large deformation solution in the analyses. The gravity frame column was connected to the LFRS frame via linear-elastic truss elements with relatively large cross-sectional area to simply enforce equal horizontal deformation between the frame and gravity frame column. The gravity column elements were modeled differently depending on their assumed contribution in the lateral seismic resistance with the LFRS. For cases where the influence of continuous gravity columns on LFRS response was not considered, these members were defined as linear-elastic, axially rigid (large cross sectional area), and flexible (negligible moment of inertia). When gravity framing was assumed to contribute to the LFRS, nonlinear beam-column elements were assigned. An equivalent wide flange shape cross section with gross area, web area, moment of inertia, and plastic section modulus equal to the sum of those properties for all tributary gravity columns was determined using the procedure discussed in Section 5.4 and Appendix 3.1 to replicate the behavior of several columns using a single equivalent column. The nonlinear fiber gravity column elements used the same element and material definitions as the braced frame columns described in Section 4.1.1 but did not include rigid end offsets.

4.1.4 Model Dynamics and Solution Method

Model dynamics were handled by a subroutine script that cycled through various solution algorithms included in OpenSees to achieve convergence in every analysis time step. When a given technique failed to achieve convergence within a specified tolerance
after a prescribed number of iterations, the program moved to the next method with a relaxed tolerance and increased number of iterations. A norm displacement increment test was used to check for convergence for all solution algorithms. Convergence tolerance was set to 1.0e-8 inches after ten iterations by default, and the first solution algorithm attempted was the Newmark method with Newton-Raphson iteration in OpenSees with no optional parameters. If convergence was not achieved, 500 iterations were performed with the same convergence tolerance and solution algorithm. If a convergent solution was still not achieved, the convergence tolerance was relaxed by an order of magnitude and 500 iterations of the Newmark method with Newton-Raphson iteration and line search algorithm were performed. Failure to converge using this solution method prompted the program to relax the tolerance by another order of magnitude and attempt 500 iterations of the Newmark method with Newton-Raphson iteration algorithm with the “initial then current” option specified, which uses the initial stiffness at the first step and the current stiffness for subsequent steps. The final two methods attempted used a convergence tolerance of 1.0e-5 inches after 500 iterations of the Krylov-Newton algorithm (second-to-last algorithm attempted) and Broyden-Fletcher-Goldfarb-Shanno (BFGS) algorithm (last algorithm attempted). If all of these integration techniques failed then the analysis was aborted. This solution technique improved performance by allowing the simulations to focus computational cost on numerically difficult steps without sacrificing speed or accuracy at all other time steps. A time step of 0.0025 seconds was used for all simulations to increase accuracy and diminish convergence problems, and 2% Rayleigh damping in the first and third modes was specified. Commit (or tangent) stiffness was used to calculate the stiffness
proportional term for the calculation of the damping matrix to ensure that the stiffness matrix used for damping was constantly updated to account for changes in stiffness as the system degraded.

A study was performed using the 3S_C0_R0_G1 model configuration (un-rehabilitated 3SCBF prototype model with original braced frame columns and weak-axis gravity framing; see Figure 3.4) to verify the accuracy of solutions obtained using the iterative solution algorithm with relaxing convergence tolerance scheme described in the previous paragraph. Constant convergence tolerances of 1.0e-8 and 1.0e-5 inches—the smallest and largest values specified in the relaxing tolerance scheme—were assigned and the Newmark solution algorithm with Newton-Raphson iteration was utilized for all time steps. These models were subjected to the full set of MCE and NF ground motions (described in the following section) and maximum inter-story drift results were compared with those obtained using the full solution algorithm procedure. Results from this study are shown in Table 4.1. Maximum drift results using the constant convergence tolerance and single solution algorithm are identical to those obtained using the iterative solution algorithm scheme with relaxing convergence tolerance for all ground motions out to at least four decimal places, indicating that the previously described solution method produces accurate results. It should be noted that several simulations using a constant convergence tolerance of 1.0e-5 inches had to be repeated with a smaller time step to run to completion (end of the ground motion) or collapse (defined as 10% maximum inter-story drift), and the simulations with a constant convergence tolerance of 1.0e-8 inches required considerably more iterations (and, therefore, time) than those with the relaxing convergence tolerance and iterative solution algorithm. These facts demonstrate the
benefits of the chosen solution and convergence method: an accurate solution is obtained with minimal computational cost. Studies were performed using the other braced frame prototype models with similar results; maximum inter-story drifts were essentially identical regardless of the chosen solution algorithm, but the procedure used for the full analytical study was more computationally efficient.

4.2 Ground Motion Suites

Each finite element model configuration was subjected to multiple ground motions at different hazard levels. The ground motions considered were developed for Los Angeles in the SAC Steel Project and are described in Table 4.2. These ground motions are categorized into three suites of ten motions: A design-basis earthquake (DBE) suite in which ground motions are scaled to have a probability of exceedance of 10% in 50 years; a maximum-considered earthquake (MCE) suite with ground motions scaled to have a probability of exceedance of 2% in 50 years; and a near-fault (NF) suite consisting of DBE hazard level ground motions with characteristics specific to near-fault shaking. DBE and MCE ground motions are of interest because seismic performance objectives under these hazard levels are explicitly defined in ASCE 7 for new design and ASCE 41 for seismic rehabilitation. NF motions were examined because prior research has indicated that buildings may be more vulnerable to soft-story failure when subjected to near-fault shaking (Alavi and Krawinkler, 2004). Only horizontal shaking was considered in this study. Design short-period spectral acceleration values ($S_{DS}$) for the DBE, MCE, and NF suites assuming 5% damping were calculated as 1.1g, 1.6g, and 1.4g, respectively. Additionally, one-second period spectral acceleration ($S_{D1}$) values of 0.68g, 1.2g, and 1.6g were calculated for the three ground motion suites.
4.3 Results

4.3.1 Sample Analytical Results and Model Verification

Before examining the full set of analytical results for all prototype frames, configurations, and ground motions, detailed results for select cases and ground motions will be presented and discussed to verify the behavior of the analytical models and also to illustrate the critical response quantities for rehabilitative design. Results obtained from the 3S_C0_R0_G1 and 3S_C0_R3_L1_G1 models (see Figure 3.4) subjected to the LA27 ground motion (see Table 4.2) are presented to illustrate the effects of SRC rehabilitation on the modernly designed prototype frames during a median-representative ground motion that induces a soft story mechanism in the first story of the unrehabilitated frame. Results from the 3N_R0_G1 (“weak beam” version) model (see Figure 3.6) subjected to the LA29 ground motion are also presented to highlight key behavioral differences between the existing modern SCBF and pre-modern 3NCFB prototypes.

3SCBF, Original Braced Frame Columns, Without SRC, Weak-Axis Gravity Column

Detailed results for the 3S_C0_R0_G1 model configuration subjected to the LA27 ground motion are presented in Figures 4.2 through 4.5. This model included the 3SCBF prototype braced frame with original columns and a weak-axis continuous gravity column. Inter-story drift time histories are shown in Figure 4.2. The formation of a soft story mechanism is apparent from these drift results, with peak first story drifts exceeding the collapse prevention performance objective of 2% and the upper stories drifting less than 1%. Median maximum drift response of each story for all existing prototype frames
indicated that soft story formation—typically in the first story—was the typical plastic mechanism in all simulations where drift exceeded the desired performance objective. Inter-story shear time histories for the gravity column, braced frame columns, and braced frame braces are shown in Figure 4.3. The braced frame braces clearly contribute the majority of the total inter-story shear, followed by the braced frame columns and gravity column. Braces reach their expected horizontal force (horizontal component of expected tensile capacity plus expected buckling capacity; about 1075 kips) in the first and second stories during peak drifts. The gravity column inter-story shear typically acts opposite the total system shear because the P-Δ loads applied at each floor of the gravity column effectively push it further in the direction of drift, while the braced frame acts against the direction of drift.

Braced frame hysteretic behavior (separated into columns and braces) is shown in Figure 4.4. This figure illustrates the hysteretic behavior of both braces in each story and response of the columns throughout the simulation. Hysteretic curves for individual braces shown in Figure 4.5 again indicate brace yielding and a large drift concentration on the first story. Note that these plots depict the relationship between the horizontal component of each brace’s axial force and inter-story drift at that story. This relationship between member force and global response resembles the axial force-deformation curve for a single brace, indicating that global response is largely controlled by the braces. No brace fracture was observed during this simulation, however the combined column shear required to form a panel mechanism (4M_{pe}/h = 207 kips) was exceeded in the first story during peak drift which indicates complete soft-story formation in this braced frame.
3SCBF, Original Braced Frame Columns, SRC (Pin-Pin Links), Strong-Axis Gravity Column

The following sets of results were obtained using the 3SCBF model with the original braced frame columns, SRC with pin-pin-ended links, and a strong-axis gravity column. This model was subjected to the LA27 MCE ground motion, which was found to induce a soft-story mechanism in the first story of the un-rehabilitated 3SCBF model. Figure 4.6 shows inter-story drift time histories, which are nearly identical for all stories. This behavior is expected as the SRC enforces a uniform drift along the height of the braced frame. A maximum drift of about 1.1% was observed which is within the life safety performance objective. Inter-story shear time histories are presented in Figure 4.7 for the gravity column, braced frame, and SRC. The braced frame clearly contributes the majority of the total inter-story shear response, followed by the SRC and gravity column. During this simulation, peak SRC shear was about one fifth the maximum braced frame inter-story shear, both of which occurred on the first story. The peak gravity column shear was about one tenth of the maximum SRC shear. These relative contributions to total inter-story shear response are consistent with the distribution of lateral stiffness between each component.

Inter-story shear hysteretic behavior of the braced frame (separated into columns and braces) is shown in Figure 4.8. The braced frame braces provide the majority of the braced frame inter-story shear response, and the inter-story shear contributions of both the braced frame braces and columns act against the direction of drift. Degradation in the strength and stiffness of the braces was not observed because the uniform drift profile enforced by the SRC prevented a concentration of damage on a single story. Shear
demands on the braced frame columns reasonably agree with the AISC 341 design forces because all braces retain their expected strengths and the upper story columns do not have to account for the difference in strength between stories that is introduced when braces undergo non-uniform degradation.

**3NCBF-WB, Without SRC, Weak-Axis Gravity Column**

Results from the existing 3NCBF-WB with weak-axis gravity column subjected to the LA29 ground motion are shown in Figures 4.9 through 4.13. The median drift response for this frame in MCE is beyond the assumed collapse drift of 10% (see Figure 4.22) and thus the LA29 motion is instead used to illustrate behavior. Inter-story drift time histories in Figure 4.9 indicate the formation of a soft first story that exceeds the collapse prevention performance objective of 2%. Inter-story shear time history plots in Figure 4.10 show the concentration of demands on the first story columns as the first story braces degrade and eventually fracture. The braced frame hysteretic behavior plots in Figure 4.11 clearly illustrate the degradation of strength and stiffness in the first story braces until complete fracture of both braces eventually occurs, resulting in increased demand in the first story columns. Note that the combined horizontal force ($F_{BH}$ force) provided by the first story braces never reaches its expected maximum value (about 766 kips) despite large drifts because the vertical component of the brace forces yields the attached beam in flexure before the expected capacity of both braces can develop. This “beam pulldown” behavior is confirmed in Figure 4.12, which shows time histories of the bending moment in each beam. The first floor beam has a plastic moment capacity of about 18,720 kip-in, which is exceeded when the brace forces are sufficient to pull the beam down.
The difference in horizontal brace force provided by two adjacent stories ($\Delta F_{BH}$) was identified as a critical quantity for rehabilitative and improved column design (see Section 3.6). This quantity characterizes the non-uniform degradation occurring over the height of the LFRS which is not accounted for in current design specifications. A sudden increase in differential $F_{BH}$ force applies an impulse to the floor mass which is instantaneously resisted by its inertia. The impulsive load causes displacement in the floor mass which eventually transfers to the columns resulting in added shear forces. Inertial forces were ignored in the design procedures described in Section 3 by assuming the design inter-story shear force resulting from the maximum predicted $\Delta F_{BH}$ force occurs once the induced inertial force has been transferred entirely to the continuous LFRS columns and/or SRC. For the un-rehabilitated 3NCBF-WB prototype model currently being examined, the $\Delta F_{BH}$ force creates additional shear demand in the lower of two adjacent stories as it induces a displacement whose resulting shear force must be carried through the columns. Figure 4.13 depicts this effect by showing the difference in $F_{BH}$ forces on adjacent stories overlaid with the shear force in the braced frame columns of the lower of the two stories. The bottom plot illustrates how the difference in $F_{BH}$ force increases as the first story braces degrade and eventually fracture, and the majority of the shear demands on the first story braced frame columns comes from this difference as the simulation progresses. In the top plot, shear force in the second story braced frame columns does not correspond to the $\Delta F_{BH}$ force between the third and second story braces because the drifts on these stories are nearly identical throughout the simulation due to the formation of a soft first story mechanism, preventing the columns from deforming significantly over their height. This behavior emphasizes the importance of explicitly
designing the controlling plastic mechanism to predict drift and calculate inter-story shear demands for capacity design as discussed in Section 3.4.

These illustrative results are representative of all simulations performed for this study. All models are believed to be accurate because the response quantities of interest behave as expected and the solution algorithm and convergence tolerance scheme are adequate based on the convergence study discussed in Section 4.1.4. Further examination of analytical results will not include explicit time history or hysteretic response of these quantities; however the following presentation of peak results will suffice to characterize the critical behaviors for improved or rehabilitative design of all prototype frames considering multiple parameters.

4.3.2 3SCBF Full Analytical Results

Table 4.3 provides dynamic modal properties—including period, participation factor, modal mass, and mode shape—for the first three (horizontal) modes of all 3SCBF FEA model configurations. The first mode for the existing 3SCBF has a period of about 0.38 seconds which is unaffected by the inclusion of continuous gravity framing and/or full-height SRC with pin-pin-ended links because these pinned-base components do not contribute to lateral stiffness during a (nearly) uniform drift (first mode response). The models with improved braced frame columns or SRC with fix-fix-ended links have shorter first periods (0.36 and 0.3 seconds, respectively) due to the increased lateral stiffness provided by these components. All mode shapes are normalized by the mass matrix in OpenSees, resulting in modal masses of nearly one for all modes. The participation factors indicate that the first mode participates in dynamic response about 2.5 and 5 times more than the second and third modes, respectively. For the higher
modes, inclusion of continuous elastic bodies (SRC, gravity column) reduces the period because the shapes of these modes engage the inter-story stiffness of these pinned-base components which increases the stiffness of the system thus reducing the period.

Maximum peak inter-story drift results for the prototype 3SCBF models are presented in Tables 4.4 and 4.5 for the MCE and NF ground motion suites, respectively. Median drift response for the existing frame was beyond the collapse prevention performance objective for both ground motion suites, and collapse—designated as a maximum peak inter-story drift of 10% or higher—occurred during several MCE and NF simulations for all gravity column cases. Increasing the contribution of the gravity column (from none, to weak-axis, then strong-axis) worsened the median drift response for the MCE ground motion suite, which is somewhat counterintuitive however it is observed from the table that only a single ground motion exhibits this trend (LA33) which happens to be one of the median-representative values. Other configurations saw slight decreases in drift when a stronger gravity column was included (as expected). Overall, these results indicate that the existing prototype 3SCBF does not perform adequately under the MCE and NF hazard levels and would require redesign or rehabilitation.

Models featuring the improved braced frame column design sections performed significantly better than the existing prototype frame models. Median maximum peak inter-story drift achieved the collapse prevention performance objective (2%) for the MCE suite and the life safety objective (1.5%) for the NF suite for all gravity column cases. Fewer simulations reached collapse-level drifts for the improved column models relative to the existing frame models.
The inclusion of an SRC further decreased peak drift response. The life safety performance objective was achieved by the median response of all gravity column and braced frame-to-SRC link configurations to both ground motions suites considered. Note that median drift response of the improved column design configurations and SRC-rehabilitated configurations with pin-pin-ended links were similar because the design intent of these systems—to mitigate soft-story formation by enforcing a uniform drift without strengthening the system explicitly—is the same. Fix-fix-ended links reduced drift more than pin-pin-ended links due to the increased system strength and stiffness afforded by the fix-fix links.

These tabular results are presented graphically in Figure 4.14. The vertical axes on these plots correlates to the naming convention for each model configuration detailed in Figure 3.4. Model configurations are grouped into threes for each assumption of gravity frame contribution; starting from the top, the first three models are the existing 3SCBF prototype, followed by the models with improved braced frame column design, then the SRC-rehabilitated models with pin-pin-ended links, and finally the SRC-rehabilitated models with fix-fix-ended links. Individual ground motion drift results are plotted along the horizontal axis for each model configuration, and median and quartile values are represented with vertical lines to illustrate how these statistical descriptions of the distribution of results change for each model. Note that the dispersion of results, which can be observed qualitatively as the distance between the first and third quartile values, becomes less widely distributed moving from top to bottom on these plots. This indicates that there is less statistical uncertainty—and, therefore, more reliability—in the response of the 3SCBF prototype as increasingly rigorous improvement/rehabilitative
efforts are applied to the existing frame (improved column design, SRC-rehab with pin-pin links, then SRC-rehab with fix-fix links). This is important for any future reliability-based SRC-rehab design considerations. Figure 4.14 also contains a vertical line on each plot indicating the predicted maximum drifts for each model configuration obtained using the equal energy approach detailed in Section 3.4.1. Median drift response of the SRC-rehabilitated frames with fix-fix-ended links closely matched the predicted values for both ground motion suites. The predictions for SRC-rehab with pin-pin links and improved column design were slightly conservative based on median response, and the median response of the existing frame configurations exceeded predictions. Accurate prediction of a median response at high levels of drift is difficult because the response of a frame behaving plastically while undergoing strength and stiffness degradation is highly sensitive to the characteristics of each individual ground motion.

Maximum combined braced frame column shear demands are presented in Tables 4.6 and 4.7 for the MCE and NF ground motion suites. Note that these tables indicate maximum values of the summation of both column shears at all points throughout each simulation and each column is assumed to carry half of these maximum demands for comparison with predicted design values (see Section 3.6) and expected strengths (from AISC 340). The existing column 3SCBF model exceeds the column panel mechanism shear capacity (4M\(_{pe}/h\), noted at the bottom of the table) for most of the analyses indicating that a soft-story mechanism is forming during the analyses. During simulations that reached “collapse” (10% maximum inter-story drift or greater), strain hardening and unrealistic model deformations resulted in excessively large force response. Because of this, all maximum force response quantities presented in tables for
all prototype frames omit results obtained from collapse-level simulations but still include these large values in calculation of the median response. The improved column 3SCBF model did not exceed the expected column panel mechanism shear capacity with a median value from analysis of approximately 460 kips. The excess capacity is likely due to the fact that the column is designed for the moment and axial compression interaction and thus must have excess capacity with respect to the moment alone. The inclusion of an SRC (with original LFRS columns) reduced LFRS column shear demands, and the fix-fix-ended link configurations never exceeded the nominal combined LFRS column mechanism capacity.

Maximum combined braced frame column shear force for each model and ground motion is also shown in Figure 4.15. For the purposes of this figure, unreasonably large values extracted from collapse-level simulations were capped at a value slightly greater than the largest realistic value for any model. This allows the median and quartile calculations that describe the representative and distribution of response, respectively, to include all data points in a realistic fashion. These plots further illustrate the trends described in the previous paragraph; the median response of the existing frame includes the formation of a panel mechanism in the braced frame columns, while all other model configurations avoid soft story formation resulting in more desirable behavior.

Maximum inter-story shear demands on the SRC for rehabilitated 3SCBF model configurations are provided in Tables 4.8 and 4.9 and illustrated in Figure 4.16. Median values are less than the predicted values from Section 3.4.4 and inelastic demands on the SRC were not observed during any 3SCBF simulation. It was observed that the full expected SRC design shear forces did not develop in the majority of SRC-rehabilitated
model simulations because differential shear demand from braces is dependent on non-uniform degradation which is largely mitigated by the uniform drift profile enforced by the SRC. The SRC must be designed to resist differential inter-story shears to prevent the formation of a panel mechanism (soft story), but there is an optimal point between designing for the full expected forces (which develop when the frame forms a soft story mechanism) and those which will develop in the rehabilitated frame (undergoing uniform drift) that relies on quantifying the extent of non-uniform degradation in the braces as a function of the predicted drift for rehabilitative design. Some effort was made to formulate this critical relationship using results obtained in this study, but the variations in ground motion characteristics made regression analysis using maximum data points from various simulations meaningless thus preventing a means for determining more reasonable SRC design shear forces based on expected peak drift. It was found that brace degradation is load history dependent and can therefore not be fully characterized using only peak deformation. A study for obtaining this relationship is suggested for future work in Section 6.

Tables 4.10 and 4.11 and Figure 4.17 show the maximum difference in the combined horizontal force in the LFRS braces between adjacent stories (ΔF_{BH}), which was identified in Section 4.3.1 as a critical quantity for determining LFRS column demands and differential shear demands for rehabilitative design. Predicted design ΔF_{BH} forces—obtained by calculating the maximum difference between expected post-buckling and expected buckling brace forces in adjacent floors—were exceeded in most 3SCBF simulations. However, these large differences in inter-story shear force provided by braces on adjacent stories were not entirely reflected in the maximum LFRS column
shear demands (Tables 4.6 and 4.7) or maximum SRC inter-story shear demands (Tables 4.8 and 4.9) because the $\Delta F_{BH}$ force is not directly imparted onto the other LFRS members in the system. Initial resistance from the inertia of the floor mass cedes to the impulse caused by the $\Delta F_{BH}$ force, leading to displacement of the floor which deforms the continuous columns (and SRC, if included) to induce a restoring shear force that returns the mass to rest as described in Section 4.3.1. The continuous columns and SRC effectively feel the difference between the recorded maximum $\Delta F_{BH}$ force and the inertia of the floor mass once the mass has returned to rest (zero acceleration) and the displacement has reached its peak. Over this time, the input ground acceleration is changing and can either supplement or counteract the force which will eventually be imparted onto the structure, making accurate prediction of this effect difficult. Additionally, closer inspection of the instant at which the maximum differential $F_{BH}$ forces were recorded for each simulation revealed that some of these values occurred when adjacent stories were drifting in opposite directions, and the large $\Delta F_{BH}$ forces were the result of a sign difference instead of the effects of non-uniform degradation. Nevertheless, the maximum $\Delta F_{BH}$ quantity is still believed to adequately quantify the differential inter-story shear force resulting from non-uniform degradation of braces, and results from the “well-behaved” simulations agree reasonably with predicted values.

4.3.3 6SCBF Full Analytical Results

Table 4.12 contains elastic dynamic properties of the 6SCBF models. Trends similar to those discussed in Section 4.3.2 for the 3SCBF models are also present for this frame. The fundamental period—about 0.68 seconds with the original braced frame
columns—is essentially unaffected by the inclusion of continuous gravity framing and/or SRC with pin-pin-ended links. With improved braced frame columns or a full-height SRC with fix-fix-ended links (and original braced frame columns), the period of the first mode reduces to about 0.62 seconds. The higher modes are more affected by the inclusion of continuous gravity framing and/or SRC because these rocking components contribute to inter-story stiffness during non-uniform drift response, which is the case for all higher modes. All mode shapes are normalized by the mass matrix, and participation factors indicate that the first mode contributes more to the dynamic response than the second and third modes by factors of about 2.25 and 5.5, respectively.

Drift results for the prototype 6SCBF models are provided in Tables 4.13 (MCE) and 4.14 (NF). These results are also illustrated in Figure 4.18. The existing (original braced frame columns and no SRC) 6SCBF models, improved braced frame column models, and models with two-story SRC and pin-pin-ended links exceed the collapse prevention performance objective of 2% median maximum peak inter-story drift for both the MCE and NF suites. The partial height SRC prevented soft story formation in the bottom two stories of the six story frame, but placed additional demands on the weaker upper stories resulting in drift response comparable with the un-rehabilitated configurations. The improved column design configurations had smaller median drift response and only reached collapse-level drifts (10%) during one ground motion (NF19). For both ground motion suites, the full-height SRC model configurations achieved the collapse prevention performance objective with pin-pin-ended links, and the life safety performance objective was achieved with fix-pin-ended links. In Figure 4.18, the three model configurations at the top of the vertical axis of these plots correspond to the
existing frame (with three assumptions for the contribution of gravity framing). Moving
down the vertical axis, the models are presented in the following order (in groups of
three): improved braced frame column design, two-story SRC with pin-pin-ended links,
six-story SRC with pin-pin-ended links, and six-story SRC with fix-fix-ended links.
Median and quartile lines illustrate how the existing frame’s performance is improved by
each model configuration, as well as the increase in reliability afforded by the narrowed
dispersion of results (the distance between quartile lines) for improved and SRC-
rehabilitated model configurations. Predicted drifts were somewhat conservative for the
existing and pin-pin link full-height SRC configurations. Note that all predictions for the
two-story SRC model configurations are identical to those calculated with a full-height
SRC with pin-pin-ended links because these rehabilitated frames were expected to
behave similarly. However, these results indicate that the full-height SRC is much more
effective than a partial-height SRC for improved drift performance, and the median
response of the two-story SRC model configurations is similar to that of the existing
frame.

Maximum combined column shear demands for the 6SCBF model simulations are
presented in Tables 4.15 and 4.16 and Figure 4.19. The existing 6SCBF models
experienced median shear demands that exceeded the expected panel mechanism capacity
of the braced frame columns in the first story. For the improved column design
configurations, median shear demands on the braced frame columns slightly exceeded the
value required to form a panel mechanism on the first story without considering any
contribution from the gravity framing, but the median response of improved column
6SCBF models with a weak- or strong-axis gravity column did not exceed this value.
Median response with a partial-height SRC indicates that it does not adequately prevent soft story mechanism formation, while nearly all full-height SRC simulations effectively prevented this mechanism. As was the case for the 3SCBF results, all maximum force response quantities from 6SCBF simulations that reached collapse (10% maximum inter-story drift) have been omitted from tables and capped at realistic large values in figures.

Tables 4.17 and 4.18 and Figure 4.20 contain maximum SRC inter-story shear demands for all rehabilitated 6SCBF model simulations. Median results match predicted maximum SRC shear values reasonably well despite the fact that the same issue regarding quantification of non-uniform degradation discussed in Section 4.3.2 for the 3SCBF prototype also applies to this prototype frame. Maximum differences between combined horizontal LFRS brace forces on adjacent stories are presented in Tables 4.19 and 4.20 and Figure 4.21. Similar to the 3SCBF models, the 6SCBF simulations frequently exceeded the predicted design $\Delta F_{BH}$ force, but these demands were not entirely reflected by the maximum column shear (Tables 4.15 and 4.16 and Figure 4.19) and SRC inter-story shear demands (Tables 4.17 and 4.18 and Figure 4.20) due to the inertial forces and method of extraction, as described in Section 4.3.2.

### 4.3.4 3NCBF Full Analytical Results

Dynamic properties for the first three horizontal modes of the 3NCBF-WB FEA models are given in Table 4.21. The period of the first mode was about 0.45 seconds for the existing 3NCBF-WB models, indicating that it is less stiff than the 3SCBF since both prototype frames have identical tributary mass. All dynamic properties for the existing 3NCBF-BC models—provided in Table 4.22—are identical to those of the “weak beam” version because the elastic properties of the frames are identical. Similar to the 3SCBF
and 6SCBF models, the first mode properties are mostly unaffected by continuous gravity framing or a full-height SRC with pin-pin-ended links. Models with a full-height SRC with fix-fix-ended links had a first mode period of about 0.39 seconds for the “weak beam” models and 0.3 seconds for the “brittle connection” models. Note that separate SRCs were designed for the two versions of the 3NCBF because their controlling plastic mechanisms are different and require separate consideration for rehabilitative design. All mode shapes are mass-normalized. Participation factors indicate that the first mode contributes to elastic dynamic response more than the higher modes by factors of about 2.8 (second mode) and 7.75 (third mode), making the 3NCBF FEA models slightly more first mode dominant than the 3SCBF and 6SCBF models.

Prototype 3NCBF-WB maximum peak inter-story drift results are given in Tables 4.23 (DBE) and 4.24 (MCE) and illustrated in Figure 4.22. The median response of the existing (without SRC) prototype 3NCBF-WB exceeds the life safety performance objective (1.5%) under DBE loading but achieves the collapse prevention objective (2%) and does not reach collapse-level drift (10%) for any ground motion in the suite. LA1 and LA3 were found to be median-representative DBE ground motions, and median maximum peak inter-story drift decreased slightly as the gravity column contribution increased. This frame performed much better than predicted under the DBE hazard level. However, under MCE loading, most of the simulations reached collapse resulting in a collapse (10% drift) median response, which exceeds the predicted maximum drift, for all gravity column configurations. Similar to the SCBF results, the dispersion of drift response decreased with the inclusion of an SRC. Maximum peak inter-story drift results obtained using the 3NCBF-BC models are provided in Tables 4.25 (DBE) and 4.26.
(MCE) and Figure 4.23. As expected, the un-rehabilitated versions of this model reached larger peak drifts than the existing weak beam models and collapsed during several DBE and MCE ground motions. The median drift response of the existing 3NCBF-BC did not achieve the collapse prevention performance objective for either ground motion suite; however, it did perform better than the 3NCBF-WB models under the MCE hazard level with the inclusion of gravity framing contribution.

Inclusion of an SRC with pin-pin-ended links reduced the median 3NCBF-WB DBE drift to just below 1%, which is within the desired performance objective (life safety; 1.5%) for the DBE hazard level. The brittle connection version of the 3NCBF prototype rehabilitated with SRC and pin-pin-ended links did not achieve this performance objective for the DBE hazard level. Under MCE loading, the collapse prevention objective was not achieved using pin-pin links for either version of the rehabilitated prototype frame, with median maximum peak inter-story drifts of about 3% and 4% for the weak beam and brittle connection models, respectively. These values are much smaller than predicted for both versions of the model. With fix-fix-ended links, the weak beam models achieve median MCE response within the collapse prevention performance objective, while the brittle connection model achieved the life safety objective with fix-fix links. Median drifts exceeded their predicted values for both models under MCE loading. As was the case for the SCBF prototypes, the dispersion of maximum drift data is reduced with the inclusion of an SRC.

Peak braced frame column shear demands are provided in Tables 4.27 through 4.30 and Figures 4.24 and 4.25. Both versions of the existing 3NCBF model’s columns exceeded the expected shear capacity at panel mechanism formation during all DBE and
MCE ground motion simulations. When comparing median braced frame column shears with maximum inter-story drift, a relationship becomes evident; model configurations whose median column shear response exceeds the required force to form a panel mechanism do not achieve their desired drift performance objective, while those that prevent the soft story formation perform adequately. This frames the concept of SRC rehabilitation as a means for vertically redistributing differential inter-story shear forces to prevent panel mechanism formation resulting in desirable uniform drift response.

Tables 4.31 through 4.34 and Figures 4.26 and 4.27 contain the maximum inter-story shear demands on the SRC for all rehabilitated 3NCBF simulations. Median values for the rehabilitated 3NCBF-BC models compare reasonably to the design SRC inter-story shear values discussed in Section 3.4.4 because the expected demands on the SRC for this frame do not include the effects of degradation since all braces are expected to fracture at their connection points prior to developing their expected strength in tension and multiple cycles are required for degradation to occur. For the rehabilitated 3NCBF-WB models, SRC shear predictions were conservative for the same reasons related to degradation discussed in Section 4.3.2 for the 3SCBF prototype. The beam pulldown mechanism formed by the 3NCBF-WB prototype accelerates compression brace degradation because the downward vertical deformation at the beam’s centerline shortens the straight-line distance between endpoints of the compression brace, causing additional buckling and, therefore, accumulation of strain. This exacerbates the problem of quantifying non-uniform brace degradation discussed in Section 4.3.2, leading to conservative predictions for maximum SRC shear.
The maximum $\Delta F_{BH}$ forces from all 3NCBF simulations are given in Tables 4.35 through 4.38 and Figures 4.28 and 4.29. For the un-rehabilitated weak beam models, the predicted maximum $\Delta F_{BH}$ is exceeded in all simulations indicating that severe non-uniform brace degradation is occurring due to the beam pulldown effect described in the previous paragraph. Median $\Delta F_{BH}$ values from rehabilitated 3NCBF-WB simulations closely match the prediction, indicating that the method for predicting brace forces for this mechanism described in Section 3.3.3.1 is reasonable. For the brittle connection model, only a few outlier simulations reached $\Delta F_{BH}$ values larger than predicted. This indicates that the predicted maximum $\Delta F_{BH}$ force for this frame, which assumes one fractured tension brace and the rest of the braces at their expected strengths, is conservative. However, designing for the resulting inter-story shear demands based on this assumed mechanism produced an SRC whose median maximum shear demands closely matched their predicted values, verifying the assumption.

4.3.5 Comparison of Analytical Results to Predicted Values of Key Parameters for Rehabilitative Design

Several key parameters identified in Section 3 as critical for rehabilitative design were recorded during all analytical simulations for comparison with their predicted values. Table 4.39 summarizes these comparisons for all prototype frames and configurations. Predictions for the first period of all three story prototype frames were slightly longer than those obtained from the analysis models, however most configurations were within 10% of the predicted value. The 6SCBF prototype FEA models had slightly longer first periods than predicted. This is likely due to the increased
contribution of overturning flexibility in this frame relative to the three story frames. While this flexibility was approximately accounted for in the prediction, the true dynamic characteristics of the frame obtained from analysis of the FEA model do not necessarily match the assumptions discussed in Section 3. As was the case for the three-story frames, predicted first period values were usually within about 10% of those obtained from analysis.

Maximum inter-story drift was identified as the key parameter for assessing the global response of a frame to seismic loading. Accordingly, Table 4.39 includes median maximum drift response for all model configurations subjected to the two hazard level ground motion suites of interest. Performance objectives for rehabilitative design provided by ASCE are included for each hazard level, and the target drift for which each frame’s SRC, reaction column, and yielding links were designed to achieve (for the MCE hazard level) are also provided. As discussed in Section 4.3.2, predicting maximum drifts beyond the collapse prevention performance objective (2%) is difficult—especially for existing frames without SRC—due to the highly sensitive response behavior of frames near collapse. However, predicted drifts beyond the collapse prevention limit almost always correlate to median analysis response beyond 2% drift (and some are reasonably close), making the drift prediction calculation effective for determining if a frame will perform adequately under a given hazard. The 3SCBF models with improved braced frame column design performed better than predicted, partially due to the increased stiffness of the FEA model relative to the stiffness values calculated to predict the first period. All models with a full-height SRC and pin-pin-ended links performed significantly better than predicted, with median drifts from analysis that were about half
their predicted values. The most accurate drift predictions for each frame came from the SRC-rehabilitated configurations with fix-fix-ended links. This is likely due to the smaller levels of drift experienced by these models causing less brace degradation, as well as the fact that the SRC—the primary lateral force resisting component in a rehabilitated frame configuration—remained elastic during all simulations, resulting in more predictable response. Note that the predicted drifts for the rehabilitated SCBF models with fix-fix links exceed their target values. This is a result of the ongoing iterative process of refining the predictive calculation procedures, SRC, link, and reaction column design procedures, and performing and evaluating analysis results obtained using prospective rehabilitative designs. Once all predictive and design calculations were believed to be complete, a final rehabilitative design iteration to reduce the final predicted drift below the target value was not performed due to the required computational and post-processing time exceeding the time remaining before a deadline associated with this study. An additional rehabilitative design iteration could have also been performed on the 3NCBF specimens to optimize their predicted and analytical performance to match the target drift, but the proposed designs for these frames achieved the target drifts and additional refinement was deemed unnecessary.

For frames rehabilitated with an SRC, median maximum SRC inter-story shear values are compared with predictions used for design to evaluate the accuracy of the proposed SRC design procedure in Section 3.4. For most frames, there is a significant disparity between predicted SRC shears and those obtained from analysis, especially for pin-pin links. This disparity is believed to be related to the assumptions made to account for brace degradation, as discussed in Section 4.3.2. This belief is evidenced by the close
match between predicted and analytical SRC shears for the 3NCBF-BC prototype, whose braces fracture prior to the onset of degradation. Further efforts to quantify non-uniform degradation are recommended for future research in Section 6.
Table 4.1: Analytical Convergence Scheme Tolerance Study

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Note: Drifts capped at 10%
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### Table 4.4: 3SCBF Prototype MCE Inter-Story Drift Results

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**Notes:**
- Bold values are median representative.
- Drifts capped at 10%, assuming collapse occurs before this point.
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Notes: Bold values are median representative.
Drifts capped at 10%, assuming collapse occurs before this point.
| Hazard Level | Ground Motion | Gravity Column | Maximum Combined Shear Force (kips) |  |
|--------------|---------------|----------------|--------------------------------------|  |
|              |               | Without SRC     | With SRC, Original Columns            |  |
|              |               | Original Columns | Improved Columns | Pin-Pin Links | Fix-Fix Links |
| LA21         | None          | -              | -                                    | -   | 123.541 |
|              | Weak-Axis     | -              | -                                    | -   | 123.762 |
|              | Strong-Axis   | -              | -                                    | -   | 124.053 |
| LA23         | None          | 141.727        | 243.797                              | 116.114 | 42.352 |
|              | Weak-Axis     | 141.353        | 243.947                              | 115.934 | 42.593 |
|              | Strong-Axis   | 139.032        | 250.475                              | 114.308 | 43.120 |
| LA25         | None          | -              | 470.295                              | **209.438** | 132.175 |
|              | Weak-Axis     | -              | 476.185                              | **209.200** | 132.458 |
|              | Strong-Axis   | -              | 497.951                              | **209.899** | 132.728 |
| LA27         | None          | **253.675**    | 438.226                              | 163.785 | 117.996 |
|              | Weak-Axis     | **251.031**    | 438.475                              | 164.279 | 118.311 |
|              | Strong-Axis   | **240.182**    | 434.065                              | 165.745 | 118.881 |
| LA29         | None          | 173.764        | 451.621                              | 117.127 | 63.431 |
|              | Weak-Axis     | 173.854        | 455.205                              | 117.381 | 63.411 |
|              | Strong-Axis   | 174.341        | 470.264                              | 119.336 | 63.124 |
| LA31         | None          | -              | 650.378                              | **248.980** | 125.320 |
|              | Weak-Axis     | -              | 650.205                              | **248.205** | 125.198 |
|              | Strong-Axis   | -              | **551.811**                          | **253.913** | 126.780 |
| LA33         | None          | **248.479**    | 495.113                              | 167.347 | 113.786 |
|              | Weak-Axis     | **245.155**    | 482.644                              | 167.320 | 114.167 |
|              | Strong-Axis   | **234.704**    | 459.620                              | 166.439 | 115.301 |
| LA35         | None          | **248.479**    | 495.113                              | 167.347 | 113.786 |
|              | Weak-Axis     | **245.155**    | 482.644                              | 167.320 | 114.167 |
|              | Strong-Axis   | **234.704**    | 459.620                              | 166.439 | 115.301 |
| LA37         | None          | **237.242**    | 437.097                              | 158.042 | 62.317 |
|              | Weak-Axis     | **235.759**    | 438.097                              | 158.671 | 62.526 |
|              | Strong-Axis   | **227.567**    | 443.992                              | 160.810 | 63.282 |
| LA39         | None          | 178.531        | 327.165                              | 127.002 | 66.438 |
|              | Weak-Axis     | 177.131        | 327.051                              | 126.961 | 66.873 |
|              | Strong-Axis   | 168.471        | 326.066                              | 126.291 | 68.702 |
| Median       | None          | **251.077**    | 460.958                              | 165.566 | 115.891 |
|              | Weak-Axis     | **248.093**    | 465.695                              | 165.799 | 116.239 |
|              | Strong-Axis   | **237.443**    | 464.942                              | 166.092 | 117.091 |

Notes: Bold values exceed yield.
Shear to form Column Panel Mechanism (4M₂p/h): 207k (original columns), 550k (improved).
Results from simulations that reached collapse were omitted (still included in median calculation).
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Notes: Bold values exceed yield.

Shear to form Column Panel Mechanism (4Mpe/h): 207k (original columns), 550k (improved).

Results from simulations that reached collapse were omitted (still included in median calculation).
### Table 4.8: 3SCBF Prototype MCE Maximum SRC Inter-Story Shear

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Note: Bold values are median representative.
Table 4.9: 3SCBF Prototype NF Maximum SRC Inter-Story Shear

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Note: Bold values are median representative.
Table 4.10: 3SCBF Prototype MCE Maximum $\Delta F_{BH}$ Force between Adjacent Stories

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Notes: Bold values are median representative.

Maximum predicted difference: 727.8k (story 3 = EPB, story 2 = Exp.).

Results from simulations that reached collapse were omitted (still included in median calculation).
Table 4.11: 3SCBF Prototype NF Maximum $\Delta F_{BH}$ Force between Adjacent Stories

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Notes: Bold values are median representative.
Maximum predicted difference: 727.8k (story 3 = EPB, story 2 = Exp.).
Results from simulations that reached collapse were omitted (still included in median calculation).
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6SCBF Prototype Scale Analytical Model Dynamic Properties
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### Table 4.13: 6SCBF Prototype MCE Inter-Story Drift Results

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**Notes:**
- Bold values are median representative.
- Drifts capped at 10%, assuming collapse occurs before this point.
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Notes: Bold values exceed yield.
Shear to form Column Panel Mechanism ($4M_{pl}/h_1$): 398k (original), 885k (improved).

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Notes: Bold values exceed yield. Shear to form Column Panel Mechanism (4M_p, h_1): 398k (original), 885k (improved).
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Notes: Bold values are median representative.
Maximum predicted difference: 613.7k (story 5 = EPB, story 4 = Exp.).

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Notes: Bold values are median representative.

Maximum predicted difference: 613.7k (story 5 = EPB, story 4 = Exp.).
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**Notes:**
- Bold values are median representative.
- Drifts capped at 10%, assuming collapse occurs before this point.
## Table 4.24: 3NCBF-WB Prototype MCE Inter-Story Drift Results

### 3NCBF-WB Prototype Scale Analytical Model Drift Results

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**Notes:**
- Bold values are median representative.
- Drifts capped at 10%, assuming collapse occurs before this point.
Table 4.25: 3NCFB-BC Prototype DBE Inter-Story Drift Results

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Notes: Bold values are median representative.
Drifts capped at 10%, assuming collapse occurs before this point.
### Table 4.26: 3NCBF-BC Prototype MCE Inter-Story Drift Results

#### 3NCBF-BC Prototype Scale Analytical Model Drift Results

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**Notes:**
- Bold values are median representative.
- Drifts capped at 10%, assuming collapse occurs before this point.
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Notes: Bold values exceed yield.
Shear to form Column Panel Mechanism (4M[pe]/h): 207k.
Results from simulations that reached collapse were omitted (still included in median).
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Notes: Bold values exceed yield.
Shear to form Column Panel Mechanism ($4M_{pe}/h$): 207k.
Results from simulations that reached collapse were omitted (still included in median).
### Table 4.29: 3NCBF-BC Prototype DBE Max Combined Braced Frame Column Shear

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**Notes:**

- Bold values exceed yield.
- Shear to form Column Panel Mechanism (4M_{ep}/h): 207k.
- Results from simulations that reached collapse were omitted (still included in median).
### Table 4.30: 3NCBF-BC Prototype MCE Max Combined Braced Frame Column Shear

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**Notes:**
- Bold values exceed yield.
- Shear to form Column Panel Mechanism ($4M_{pe}/h$): 207k.
- Results from simulations that reached collapse were omitted (still included in median).
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Note: Bold values are median representative.
### Table 4.32: 3NCBF-WB Prototype MCE Maximum SRC Inter-Story Shear

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Note: Bold values are median representative.
| Hazard Level | Ground Motion | Maximum Interstory Shear (kips) | | | |
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|  |  |  |  |  | |
| LA1 | None | 498.2074 | 1160.8670 | |
|  | Weak-Axis | 495.2565 | 1163.1150 | |
|  | Strong-Axis | 479.8130 | 1163.4840 | |
| LA3 | None | 179.0750 | 962.7190 | |
|  | Weak-Axis | 178.9040 | 967.8390 | |
|  | Strong-Axis | 189.9770 | 952.0330 | |
| LA5 | None | 236.6769 | 690.4497 | |
|  | Weak-Axis | 237.5918 | 662.1351 | |
|  | Strong-Axis | 153.6260 | 728.1236 | |
| LA7 | None | 191.5485 | 977.4400 | |
|  | Weak-Axis | 186.3990 | 936.3070 | |
|  | Strong-Axis | 185.7730 | 983.4050 | |
| LA9 | None | 357.8450 | 1104.0270 | |
|  | Weak-Axis | 353.5150 | 1099.1380 | |
|  | Strong-Axis | 364.2100 | 1086.2070 | |
| DBE | LA11 | None | 554.3270 | 1179.5211 | |
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|  | Strong-Axis | 542.3280 | 1187.9290 | |
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| Median | None | 436.4227 | 1096.3350 | |
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Note: Bold values are median representative.
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Note: Bold values are median representative.
### Table 4.35: 3NCFB-WB Prototype DBE Max $\Delta F_{BH}$ Force between Adjacent Stories

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**Notes:**
- Bold values are median representative.
- Maximum predicted difference: 336.5k (story 3 = EPB, story 2 = Exp.).
- Results from simulations that reached collapse were omitted (still included in median).
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Notes: Bold values are median representative.
Maximum predicted difference: 336.5k (story 3 = EPB, story 2 = Exp.).
Results from simulations that reached collapse were omitted (still included in median).
**Table 4.37: 3NCBF-BC Prototype DBE Max $\Delta F_{BH}$ Force between Adjacent Stories**

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**Notes:**
- Bold values are median representative.
- Maximum predicted difference: 473.2k (story 2 = Exp., story 3 = fracture/EB).
- Results from simulations that reached collapse were omitted (still included in median).
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<th>Maximum SRC Interstory Shear (kips)</th>
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Notes: Average of median analysis results obtained using all three gravity column assumptions shown in this table.
All SRC/RC/links (fix-fix) designed to achieve target maximum interstory drift at MCE hazard level (in bold).
Figure 4.1: Finite Element Model Details

- **Node with Mass and Load**
- **B.F. Cl. N.ods Horizontally Constrained to Mass**
- **Elastic, Flexible, and Axially Rigid for No G.C. Cases**
- **Gusset Elements**
- **Moment Releases (Pinned Connections)**
- **Fifteen F.B.C. Per Brace**
- **“Ghost” Truss Brace**
- **Rigid Elastic Offsets**
- **Rigid**
- **Pinned Base**
- **Gravity Column**
- **Braced Frame**
- **RC Links**
- **SRC**

**Notes:** All identified details are typical of similar members/symbols. All pin-pin-ended members are truss elements. All other members are nonlinear force-based beam-column elements unless otherwise noted.
Figure 4.2: Existing 3SCBF Prototype with Weak-Axis Gravity Column Inter-Story Drift Time Histories
Figure 4.3: Existing 3SCBF Prototype with Weak-Axis Gravity Column Inter-Story Shear Time Histories
Figure 4.4: Existing 3SCBF Prototype with Weak-Axis Gravity Column Braced Frame Hysteretic Behavior
**Figure 4.5:** Existing 3SCBF Prototype with Weak-Axis Gravity Column Brace Hysteretic Behavior
Figure 4.6: 3SCBF Prototype with Original Braced Frame Columns and Strong-Axis Gravity Column Rehabilitated with SRC and Pin-Pin Links Column Inter-Story Drift Time Histories
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Figure 4.13: Existing 3NCBF-WB Prototype with Weak-Axis Gravity Column Brace Force Difference and Column Shear Demands
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Figure 4.15: 3SCBF Prototype Maximum Peak Combined Braced Frame Column Shear Forces from Analysis
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Figure 4.28: 3NCFB-WB Prototype Maximum Peak $\Delta F_{BH}$ Force from Analysis
Figure 4.29: 3NCBF-BC Prototype Maximum Peak $\Delta F_{BH}$ Force from Analysis
Section 5: Hybrid Experimental Testing

An experimental testing program was developed to investigate the fundamental behavior of the stiff rocking core (SRC) rehabilitation technique proposed as part of this research. Hybrid simulation was used to physically test only the critical structural components (LFRS, SRC, links) at slow loading rates (pseudo-dynamic testing) while the building dynamic inertial forces, P-Δ effects, and influence of gravity columns were captured in a computational model. The prototype buildings considered for the experimental tests are described in Section 3.1 and are based on the SAC building criteria (Gupta and Krawinkler, 1999) and the braced frame buildings from Sabelli (2001). Three 3-story specimen frames were tested: SCBF with improved column design (3SCBF), a non-seismic detailed CBF with 3-story SRC (3NCBF), and a 6-story (top 3 stories of LFRS modeled analytically) SCBF with 2-story SRC (6SCBF). Similitude scaling was applied to reduce the prototype frames to approximately 1/3 their original size for experimental testing. This scale was chosen based on the limiting actuator force capacity (220 kip) and to match the 2 foot on-center connection points on the lab strong wall. Experimental testing for this research was conducted in the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo (UB).
5.1 General Setup

Several aspects of the experimental setup were common to all tests. The general test setup is shown in Figure 5.1. Three 220-kip capacity MTS actuators—one attached at each floor elevation—were mounted on a reaction wall and used to apply loading based on the actuator force and displacement feedback with the computational model. Each actuator had +/-20 inches of stroke and a built-in linear variable differential transformer (LVDT) and load cell for measuring displacement and load, respectively. Test specimen frames and the SRC were installed on a foundation beam attached to the laboratory strong floor. A lateral bracing frame straddled the foundation beam, specimen frame, and SRC, and used bearing plates adhered with low-friction PTFE material to laterally brace the specimen frame beams at 3-foot spacing, as well as the SRC columns at each floor elevation. The lateral bracing system was designed based on Appendix 6 of the AISC specifications (AISC, 2011) and in accordance with the bracing requirements on a SCBF in the AISC seismic provisions (AISC, 2010). The lateral bracing was designed using modern specifications and was intended to prevent lateral instabilities.

Instrumentation for all tests included strain gauges, string potentiometers, LVDTs, and light-emitting diodes (LEDs). All instruments were wired into the SEES Laboratory’s Pacific Instruments data acquisition (DAQ) system with the exception of the LEDs, whose positions were tracked and recorded by a Krypton K600 coordinate positioning system. Several video cameras were placed on and around each specimen, and a still camera was used to take photographs before, during, and after testing.
5.2 Similitude Scaling and Model Scale Specimen Selection

Similitude scaling was used to reduce the size of test frames and associated setup while still replicating the behavior of the full-scale prototype structure. A length scale factor ($\lambda_L$) of 3.25 was chosen. This scale was limited by the actuator force capacity (220 kip) and to match the strong wall hole spacing (2 feet on-center). This necessitated force and mass scale factors ($\lambda_F$ and $\lambda_M$, respectively) of 10.6, or the length scale factor squared. Simulation time was reduced by a factor ($\lambda_T$) of 1.80—the square root of the length factor—because acceleration cannot be scaled when testing a physical specimen subjected to gravitational acceleration (although this is not critical for these tests since nearly all mass is simulated in the computational model). The analytical sub-structure was also scaled for the hybrid test as described in Section 5.4. Table 5.1 lists all of the important scale factors, and Tables 5.2 through 5.4 show the critical scaled quantities for each specimen.

The 3SCBF and 6SCBF specimens represented scaled models of the prototype braced frames discussed in Section 3.1. The critical quantities scaled for testing included: brace member buckling and tensile yield strength, tube wall width and b/t ratio, and the beam and column strong-axis moment of inertia and plastic section modulus. Since the bracing members are the primary structural LFRS components, appropriate scaling of these members was critical to get model-scale behavior similar to that observed at prototype scale including the effects of brace yielding and buckling accounting for post-buckling strength degradation due to hinge formation, local buckling, and low-cycle fatigue. While it is impossible to scale members to exactly achieve highly nonlinear behavior, the chosen scaling parameters are believed to be correlated with these
behaviors. Since the beams and columns were expected to undergo elastic or elastic-plastic behavior primarily in flexure, strong-axis moment of inertia and plastic section modulus were chosen as the critical scaling parameters. Some consideration was also given to maintaining the b/t and h/t_w ratios of these members. Ultimately, the beams and column designs were controlled by the capacity requirements based on expected brace forces per AISC 341.

5.3 Test Specimen Frames

Three specimen frames and a stiff rocking core were designed and fabricated for testing (as shown in Figures 5.2 through 5.5). After the primary model members were selected based on similitude scaling laws, specimen connections were designed and detailed based on bracing member capacities. For the 3SCBF and 6SCBF specimens, connections were designed based on the expected brace forces using capacity design principles of AISC seismic specifications (AISC, 2010). Full structural drawing sets are provided in Appendix 4.

5.3.1 3SCBF and 6SCBF Connection Design

The load and resistance factor design (LRFD) procedures in AISC 360 (AISC, 2010) were used for all connection strength designs. The expected brace axial strength at tension yield and post-buckling strength were taken respectively as 1.1R_yF_yA_g and 0.3(1.14F_{cre}A_g). At the brace to gusset connections, a free edge-to-edge gusset fold line offset from the brace end by a distance equal to twice the thickness of the gusset was
included. Both the 3SCBF and 6SCBF specimens were three-story frames, but the 6SCBF specimen was designed as the bottom three stories of the six-story prototype building previously described and, therefore, had slightly larger sections and required stronger connection designs overall.

5.3.2 3NCBF Connection Design

The 3NCBF specimen was designed as a non-ductile braced frame representative of older (pre-1988) construction found on the west coast of the United States. Lateral design forces for the 3NCBF specimen were calculated using a 1959 SEAOC procedure. This procedure yielded a design base shear that was about 80% of the base shear calculated using the ASCE 7 (ASCE, 2013) equivalent lateral force procedure (ELFP). Both procedures used equivalent factors for site soil characteristics (1.0), importance (1.0), and response modification (6.0). The seismic response coefficient obtained using the ASCE 7 ELFP was controlled by the short period design spectral acceleration and thus was not dependent on the structure’s fundamental vibrational period, while the SEAOC procedure includes an estimation of this property. The SEAOC procedure used a slightly smaller factor for site seismicity (1.5 compared to 1.61), and this difference coupled with the inclusion of the structure’s period in the calculation resulted in a reduced design base shear using this procedure. Full details of both procedures are provided in Appendix 1. Braces were selected from a static analysis of the frame using these equivalent lateral forces. Beam sections were chosen to achieve a target deficient demand-to-capacity ratio (DCR) in flexure based on a survey of older braced frames.
A survey of pre-1988 construction in the western United States conducted by Sloat et al. (2013) identified and quantified commonly under-designed limit states. Four connection details were found to be deficient in at least 50% of the buildings surveyed: brace net section fracture, gusset Whitmore section yielding, beam-gusset weld fracture, and brace-gusset weld fracture. Beams were also found to be severely under-designed (DCR > 1.5) for flexure in 75% of buildings. These five deficient details were selected for incorporation into the 3NCBF specimen, and design DCR values were calculated as a weighted average of the survey statistics. The frame was designed to match the design DCR values for each limit state as closely as possible. Table 5.5 shows the target and final DCR values used in the 3NCBF design.

5.3.3 SRC and Link Design

5.3.3.1 SRC Design

A single SRC design was used for testing both the 3NCBF and 6SCBF specimens in order to reduce the experimental setup costs. A hinged truss SRC concept was utilized in the experimental tests as seen in Figure 5.5. Different SRC concepts were considered that included the hinged truss, a post-tensioned rocking braced frame, and a single pin supported column. The hinged truss was believed to be an effective form for the SRC rehabilitation technique for steel frames. The other SRC concepts may work and be more effective for rehabilitation of certain building types however, for the braced frames considered here, the rocking braced frame was more mechanically complex, would
impose a large foundation overturning moment compared to the hinged truss, and would create non-symmetric hysteretic behavior for a single sided link implementation (considered for the experiments). A single pin connected steel, concrete, or composite column may work for relatively light frame construction (such as wood frame buildings) or for steel frames with relatively minor deficiencies.

Since the same SRC was used for both the 3NCBF and 6SCBF specimen tests, the envelope rehabilitative design for the two frame specimens was used in the tests. The SRC used was three stories in height and the 3NCBF specimen used steel yielding link elements (described in Section 5.3.3.2) connected between the SRC and 3NCBF frame at all three floors as shown in Figure 5.6 with a moment-resisting end connection at the SRC side. The 6SCBF specimen simply used a pin-pin ended member between the SRC and 6SCBF frame on the first two floors and thus the upper third of the SRC was not needed.

Design of the SRC members is controlled by (i) the re-distribution forces from the braced frame, (ii) the forces to develop the plastic capacity of the steel yielding link members (only applicable for the 3NCBF specimen), and (iii) higher mode forces resisted by the SRC. Ultimately, the SRC was designed based the observed demands from nonlinear time history analyses of each specimen however some estimates were made as to the contribution of each force and is generally described here. The re-distribution forces depend on differences in the nominal inter-story shear demand and capacities and the extent of non-uniform inter-story hysteretic cyclic degradation that occurs. The nominal differences between the inter-story shear demand and capacities can be estimated and based on the expected braces strengths in the frame. The re-distribution
forces resulting from the non-uniform cyclic degradation is difficult to predict (and is not currently considered in code-prescribed seismic design provisions of steel braced frames). The plastic capacity forces from the link elements could be determined (as discussed in Section 5.3.3.2) and applied to the right column of the SRC. The higher mode forces applied to the SRC could be estimated through calculation of the equivalent elastic modal force vectors and adding the SRSS combination of the higher mode forces to the frame re-distribution forces and link plastic capacity forces. This is an unconventional combination of forces for multi-story buildings and results due to the stiff continuous elastic body along the building height.

Once the force demands on the SRC frame were determined, the SRC was designed based on an elastic analysis model of the SRC and applying the uniform force method for concentrically braced frames to estimate the distribution of connection forces. The right column of the SRC however is also subjected to large bending and shear forces when a steel yielding link element is attached and was appropriately designed as a beam-column. The AISC specifications (AISC, 2011) are utilized for the SRC design since it is designed to remain elastic. Ultimately, the experimental SRC was overdesigned by at least 50% to ensure the SRC was not damaged during testing.

5.3.3.2 Link Design

The steel yielding link members used during the 3NCBF specimen tests consisted of two rectangular HSS members that were designed to yield in flexure while also transferring the re-distribution forces between the frame and SRC. The members were similar to the Double-HSS Links proposed from eccentrically braced frames described in Price and Chao (2014). The links were detailed with a moment-resisting connection on
the SRC side and a single pin detail on the existing frame side as seen in the drawings in Appendix 4. The moment-resisting side used a WT section bolted to the right column of the SRC and a combined fillet and flare-bevel groove welds between the WT and each HSS member. The connection to the existing frame was detailed with a single pin in order to limit the demands that would be placed on the existing sub-standard framing and hence transferred only the horizontal re-distribution force, vertical plastic shear capacity of the link, and an eccentric moment to the frame column. These demands however did require some strengthening of the existing frame column in order to increase its axial compressive buckling capacity.

Each link was 13 inches long from the end of the welds on the moment-resisting side to the pin connection. This link length results in primarily flexural plastic behavior. However, unlike W-shape flexural yielding links used in eccentrically braced frames which must be designed for a maximum rotation angle of 0.02 rad., the double-HSS links have been shown (Price and Chao, 2014) to easily reach 0.08 rad. and were designed for this level of link rotation for the 3NCBF specimen. Each link was also designed to transfer a re-distribution force of approximately 100 kip (axial force on link) and had a nominal plastic shear capacity of about 40 kips. Other details of the link members’ connections were designed using elastic steel design methods to resist the plastic capacity of the links in addition to simultaneous application of the peak re-distribution force.

5.4 Hybrid Analytical Sub-Structuring

Seismic testing was performed using hybrid simulation at slow loading rates, also known as pseudo-dynamic testing. Hybrid testing techniques couple experimental
specimens with computational models through feedback control. In this application, slow hybrid testing techniques were utilized since the response of the experimental setup was essentially rate-independent and the dynamic inertial effects were modeled in a computer. The computational models for each hybrid test are illustrated in Figure 5.6. The hybrid tests are programmed in OpenSees (McKenna and Fenves, 2004) and combined with the experimental substructures through OpenFresco (Schellenberg et al., 2009). All computational models contained a single column comprised of elastic beam elements to represent all gravity columns tributary to the frame considered based on the prototype buildings from Sabelli (2001). The gravity column included all tributary building mass and gravity loads (for P-Δ effects), and OpenSees was used for the analytical substructure. The sum of gross section areas and strong-axis moments of inertia for gravity columns tributary to the frame considered were assigned to each element in the analytical gravity column.

For the 6SCBF specimen, the top three stories of the LFRS were modeled analytically in addition to the gravity framing. This model included all features present in the full analytical model, including brace buckling and fracture simulation. Beam and column elements were scaled to produce equivalent cross-sectional properties including: gross section area (axial rigidity), web area (shear capacity), strong-axis moment of inertia (elastic flexural rigidity), and strong-axis plastic section modulus (plastic flexural capacity). Prototype cross section properties were scaled down using the appropriate power of the length scale factor and an iterative calculation was performed to determine the depth, web thickness, and flange width and thickness of an equivalent W-section that produced these desired properties using a fiber beam element. Similarly, brace members
were scaled to produce equivalent square HSS cross sections with desired gross section area (axial rigidity), moment of inertia (elastic flexural rigidity), and radius of gyration (compression capacity). Equivalent cross section calculation details are provided in Appendix 3.

Several options were considered to more accurately represent the boundary conditions present at the interface between the physical and analytical LFRS frames (between the 3rd and 4th story) for the 6SCBF specimen, including two “overlapping” methods and a simple shear connection approach. In the overlapping methods, the physical substructure boundary overlapped the numerical substructure such that elements and joints were present in both, then instrumentation feedback was used to return the expected forces of the numerical model. In total, three different approaches were considered for experimental testing: i) no overlapping by assuming pure shear building type behavior, ii) overlapping of the second floor beam, iii) overlapping of the entire third story. A shear building type behavior was found to be inaccurate, and overlapping of the entire third story required measurement of brace axial forces from strain gauges which were highly susceptible to breaking due to high inelasticity. For these reasons, the second approach was selected in which the first and second story actuator forces were fed back to the numerical model.

At each step in the simulated ground motion, a target displacement command was sent to the actuators according to the computational model’s response. Forces from the physical frame measured by each actuator load cell were fed back into the computational model for use in the next computational time step. The computational models used an implicit time integration algorithm with a fixed number of numerical iterations in an
effort to achieve convergence and a prescribed time delay to ensure smooth, continuous motion of the actuators. The time step and number of iterations used for the tests was determined prior to testing considering a sub-structured analytical model to assess numerical divergence or inaccuracy by measuring numerical errors and load imbalances in the model. A diagram of the hybrid feedback control communication setup is shown in Figure 5.7.

Each analytical model included a damping matrix proportional to the computationally-assigned mass matrix and the initial stiffness matrix of the LFRS frame. The initial stiffness matrix must be used in the hybrid testing method since an updated matrix cannot be calculated during testing. The influence of this approximation on the global frame response was evaluated prior to experimental testing through analytical sub-structuring comparing the response with damping based on a constant initial stiffness with analyses using an updated (commit) stiffness matrix in OpenSees. For the frames and response levels considered in these tests, minor differences were found due to the damping model simplification. However, for frames behaving near collapse level, the damping model simplification was observed to induce significant differences resulting from the artificially high damping forces using the initial stiffness matrix. For the 3SCBF tests, the stiffness matrix calculated from the stiffness test performed on the physical frame was used because it reasonably matched the analytical stiffness matrix of this specimen. Natural frequencies were reduced by a factor of 1.2 for the second ground motion to account for the decreased stiffness of the frame due to damage from the first ground motion. Two percent critical viscous damping in the first and second modes was used for the 3SCBF specimen. The 3NCBF and 6SCBF specimens both used two percent
critical viscous damping in the first and third modes and stiffness matrices obtained from
the analytical models. Analytical stiffness matrices were chosen because the increased
stiffness provided by the SRC made accurate calculation of a physical stiffness matrix
difficult at the low levels of displacement required to avoid damaging the specimen
frames. A modal analysis performed after simulating the first ground motion yielded
decreased (damaged) natural frequencies for use during hybrid testing of the second
ground motion. Details of the damping assumptions used for each test are provided in
Table 5.6.

5.5 Testing Program

The general approach for testing each specimen was similar and included a series
of preliminary low-amplitude tests followed by full amplitude hybrid seismic tests and
then quasi-static cyclic testing of the frames to failure. The sequence of tests performed
on each specimen is provided in Table 5.7.

Preliminary tests performed for all specimen frames included a stiffness test,
white noise excitation, and low-amplitude ground motions. For the stiffness test, a small
displacement was imposed in each actuator one-at-a-time while the other two were held
in place. Measured forces in the actuators were recorded and used to calculate the elastic
stiffness matrix of each 3DOF specimen which was used for calculating the damping
parameters assigned in the analytical model. The white noise excitation tests sent a
random, low-amplitude displacement signal to each actuator to check possible coupling
between the actuators due to the high stiffness of the test specimens. Two low-amplitude
ground motion tests were performed for each specimen. First, a test was performed
without using feedback forces from the physical frame to check communication between the analytical model and controller. The second low-level amplitude ground motion test used feedback from the physical specimen to ensure all hybrid feedback control procedures worked correctly. The low-amplitude ground motions were simply the full-scale motions planned for the specimen scaled to 8% of their amplitude. For all preliminary tests, displacement limits were set on each actuator to avoid damaging the specimen before full-amplitude seismic testing.

Each specimen frame was subjected to two full-amplitude ground motions back-to-back. The full-amplitude motions were the ground motions from the seismic hazard level of interest, either 10% in 50 year (DBE) or 2% in 50 year (MCE) level hazard depending on the specimen as described below. Ground motions were selected from the SAC ground motion bins for the LA site as described in Section 4.2. The purpose of testing two motions back-to-back was to simulate an aftershock event or follow-up earthquake occurring before any repairs could be made to the damaged building. While this is not a very likely scenario nor considered in typical design, it was anticipated (and observed) that the rehabilitated specimens would have significant remaining strength and ductility after one ground motion, and extracting additional test data on each specimen was desirable. The ground motions selected for the experimental tests were based on the analytically predicted maximum peak inter-story drift results discussed in Section 4.3 and ground motions representative of the median response both before and after rehabilitation were selected in most cases. Note that median representative ground motions were selected using results obtained from preliminary finite element models and are not necessarily reflective of the results presented in Section 4.3.
The 3SCBF specimen was subjected to the LA33 and LA27 MCE motions in succession. Only MCE level motions were considered for this modernly designed frame since performance of the existing frame under the DBE hazard was acceptable. The motions used for the 3NCBF specimen were LA11 (DBE) and LA37 (MCE). This older non-ductile frame exceeded the collapse prevention drift limits for half of the DBE motions before rehabilitation and thus was tested under both DBE and MCE level motions. An additional 3NCBF seismic test was added because the frame sustained relatively minor damage during the back-to-back ground motions. For this test, the LA27 motion, which was median-representative for this frame, was scaled in amplitude by a factor of 1.25. This test had to be aborted shortly into the ground motion because the top of the east column at connection of the 3rd floor actuator started twisting out-of-plane and threatened to damage the lateral bracing frame and actuator swivel head. LA27 (MCE) and NF09 (near-fault) were selected for the 6SCBF test. The LA27 record represented median response for this specimen and the NF motion was used to investigate the drift and load redistribution response of the specimen and SRC using a ground motion with characteristics of near-fault shaking.

Following seismic testing, the 3SCBF and 6SCBF specimens were subjected to quasi-static cyclic testing until all braces were fractured, and then a single cycle with all braces fractured was run to extract bare (damaged) frame hysteretic response. A uniform drift in both directions with a saw tooth displacement time history was imposed by the actuators for these tests. The quasi-static displacement load history began with a cycle at the largest drift experienced during the hybrid seismic tests. The 3SCBF specimen was cycled at a constant 4% drift to failure. It was decided not to continually increase drift
levels during this cyclic test to avoid damage to the test setup since this was the first test and remaining tests needed to be completed with an approaching deadline. Twisting in the 3NCBF east column and actuator prevented any cyclic testing, but a “pull back” test was performed, with each actuator retracting to create a uniform drift. Cyclic testing for the 6SCBF specimen started at 4% drift and increased by 1% with each cycle. Test results are discussed in Section 5.7.

Once all tests were complete, several material coupons were cut from each damaged specimen frame for tension testing. Details and results from these tests are included in Appendix 5.

5.6 Instrumentation

Instrumentation plans were developed to measure all physical quantities of interest, including:

- inter-story shear and drift
- shear force, axial force, and bending moment along all beams, columns, and links
- axial force, axial deformation, and out-of-plane bending moment and deformation of specimen frame braces
- link element angle of rotation
- horizontal sliding of base plates and the foundation beam

Only axial force was of interest in the SRC beams and braces, but the columns were instrumented to attain axial and shear forces and bending moments. The built-in LVDT and load cell in each actuator were included in the instrumentation plan and used for
hybrid feedback control. Final instrumentation layouts are shown in Figures 5.8 through 5.11. Instrumentation inventory and wiring are provided in Tables 5.8 through 5.10.

5.6.1 Strain Gauges and Calculation of Internal Forces

At least three strain gauges per clear span in all beams and columns were required to measure shear and axial forces and bending moment as shown in Appendix 6.1. Readings from two gauges at a cross section were averaged to determine strain due to axial force. The linear strain gradient at both cross sections was determined to calculate the bending moment at two points along each clear span. The slope between these two points on the bending moment diagram is, by definition, the shear force. Internal forces at points of measured strain were extrapolated across each clear span to produce continuous internal force and bending diagrams along the length of each member. Axial force and bending moment in the specimen braces were similarly calculated using two strain gauges at a clear span cross section. All strain gauges were spaced at least a section depth away from connections to avoid stress concentrations and inelasticity. The locations of measured strain on any particular clear span were kept at the largest possible distance apart to calculate a reliable moment gradient to obtain shear force. Installing the required number of strain gauges to obtain all desired physical quantities with adequate spacing was not possible in the first story of the left (western) SRC column and the first floor beam of the 6SCBF specimen. Only two gauges at the same cross section were installed in these spans, and the lack of a third strain gauge was accounted for by assuming a true pin (zero moment) at the SRC rocker bearing and 6SCBF specimen first floor beam shear tabs.
5.6.2 Krypton Coordinate Measurement and Calculation of Deformations

A Krypton K600 high performance dynamic mobile coordinate measurement machine was used to track and record the three-dimensional positions of LEDs placed on specimen frame braces and link elements (see Figures 5.8 to 5.11). Two LEDs were placed on each western brace in the specimen frames: one near the end and one near mid-span. To calculate brace axial deformation, the average axial strain between these two LEDs—the change in distance between LEDs divided by their original spacing—was multiplied by total brace length. Peak out-of-plane deformations due to plastic buckling were calculated by assuming each brace forms plastic hinges in the center and at the ends (gussets) when buckled. Using this assumed “V” buckled shape, out-of-plane slope between the two LEDs was calculated and extrapolated to estimate out-of-plane deformation at the middle of the brace. Details of these calculations can be found in Appendix 6.2. Severe local buckling occurs near the kink in a buckled brace, so LEDs were not placed directly in the middle of braces because they are prone to fall off when adhered to a surface that undergoes large deformations.

Three LEDs were installed on each yielding link element to calculate rotation angle. An LED at the top and bottom of each link at the fixed end (SRC side) were used to calculate rotation, and one LED above the pin on the specimen frame side was used to calculate vertical deformations relative to the fixed end. Link rotation angle was calculated considering both fixed end rotation and relative vertical deformation across the element as shown in Appendix 6.3.

The availability of the Krypton camera was uncertain prior to the 3SCBF tests. To account for this, a modified instrumentation plan was developed. Two out-of-plane
string pots were attached to each first story brace where the Krypton LEDs were originally planned. An axially-oriented string pot was also added to each first story brace. The Krypton camera was available and operational for all tests, but these additional string pots were kept for the 3SCBF test to provide some instrument redundancy.

The base plates of each specimen column were instrumented with an LVDT to monitor any potential horizontal sliding relative to the foundation beam. A string pot measured displacement of the foundation beam relative to the strong floor. Negligible sliding of the column bases and foundation beam was observed in the 3SCBF and 6SCBF tests. Some limited base sliding was observed during the 3NCBF hybrid tests and this sliding was extracted from the inter-story displacement calculations.

5.7 Results

5.7.1 Corrections made to Measured Experimental Quantities

Several steps were taken to correct any errant instrumentation readings observed in test data. The Krypton system recorded a large constant value whenever the line of sight between an LED and the camera was obscured. These spikes in the data were corrected using linear interpolation between the nearest reliable data points. Line of sight issues often occurred during large deformations and the true peak Krypton readings could not be recovered using linear interpolation. Additionally, several LEDs fell off during testing, resulting in an unrecoverable loss of data for those LEDs for the remainder of the test. These issues limited the usefulness of the Krypton data set.
Strain gauge data issues included inelastic strain readings with residual strains and data loss due to broken gauges. To correct for these, inelastic strains were used to calculate stress assuming a bilinear stress-strain relationship. These readings were then replaced by a strain value that produces the same stress assuming a linear-elastic stress-strain relationship. This correction was necessary because all stress calculations from strain readings in the main post processing script assumed a linear elastic stress-strain relationship. Inelastic strains lead to residual strains after unloading, but the residual stresses—which are not physically present—calculated from these readings were relatively small for all members (with the exception of braces, which will be discussed in further detail), therefore no correction was made to remove the resulting fictional forces. Any internal member forces and moments that could not be calculated directly due to broken strain gauges were obtained from equilibrium expressions at frame joints where all other connecting member forces and moments were known.

Highly inelastic strains with large residual strains were measured in the specimen frame braces for all tests. Additionally, several strain gauges installed on braces broke during testing because of the high levels of strain. These issues prevented the calculation of brace forces directly from strain gauge data. Brace forces were calculated using moment equilibrium about the beam-to-column centerline intersections of a section cut portion of the test setup. This procedure is illustrated in Figure 5.12. The illustration also includes an explanation of the resultant horizontal “$F_{BH}$” and vertical “$F_{BV}$” forces from each pair of braces. The $F_{BH}$ force was identified as a response quantity of interest because it is the combined lateral force provided by the primary lateral force resisting members—the braces—on each story. The $F_{BV}$ force is the out-of-balance vertical force
from the braces that can pull the connected beam down if it is sufficient to form a plastic hinge at the center of the beam.

### 5.7.2 3SCBF Results

A timeline of important frame and member response quantities at various points of interest throughout testing of the 3SCBF specimen is provided in Tables 5.11 and 5.12. These tables show the lateral force distribution between braces and columns as the braces’ strength and stiffness degrade and fracture eventually occurs.

Figure 5.13 shows the global specimen response to the first hybrid ground motion test (LA33), including inter-story drift time histories, inter-story shear vs drift hysteretic curves, and base shear vs roof drift hysteresis. A maximum drift of just over 2%—the collapse prevention performance objective—was observed in the first and second stories, while the maximum drift in the third story was about 1%. All quantities presented in this figure were calculated using forces and deformations recorded by the actuators. Displacement readings from the actuators include connection slop from bolt holes, and the effect of this can be seen in the full-page first story hysteresis plot provided in Figure 5.14. The elastic lateral stiffness of the braces frame was observed at very low levels of drift where connection forces were below that to cause slipping. Bolted connections were only specified as snug-tight however some pre-tension is still present. Once bolt slipping occurred, a reduced stiffness was observed (53.8 k/in) which is greater than the effective stiffness of the two cantilever columns but significantly less than the expected frame stiffness. Slippage in the bolted connections then occurs until the full frame stiffness re-engages. This effect is a consequence of scaling, as fabrication tolerances could not be scaled and the deformations required to engage all connections became significant.
relative to the size of the frame. The frame stiffness degradation after repeated drift cycles is also due to accumulated damage in the braces resulting from compressive buckling and tensile yielding cycles.

Figure 5.15 shows time histories of all brace axial forces, as well as the combined $F_{BH}$ force in each story. Reference lines indicating the nominal (AISC, 2011) and expected tensile and compressive capacities (AISC, 2010) in each brace are included on the axial force plots. The $F_{BH}$ plots include reference lines showing the lower bound (expected tensile plus expected post-buckled strength), nominal (nominal tensile capacity plus nominal compressive), and upper bound (expected tensile plus expected compressive capacities) strength of the combined horizontal brace forces in each story. Brace axial force vs inter-story drift hysteresis plots are shown in Figure 5.16. The decrease in strength and stiffness of each brace over repeated drift cycles corresponds to the similar degradation observed in the inter-story shear hysteresis plots in Figure 5.13. Note that the hysteretic curves for the western braces appear mirrored about the y-axis of these plots because positive actuator displacements (and, therefore, drifts) pushed the specimen frame to the west, subjecting the western braces to compressive forces, which are negative according to the assumed sign convention.

Hysteretic behavior of the combined brace forces ($F_{BH}$ and $F_{BV}$) and sum of column shear forces are shown in Figure 5.17. The relative contributions of the braces and columns to the total lateral force response of the frame can be seen in these plots. At low levels of drift (<1%), the braces carry most of the lateral shear. At peak drift, braces and columns both contribute significant lateral forces in the first and second stories. The column shear sum plots also include reference lines noting the design column shear force.
sum based on AISC 341-10 which considers the envelope of analyses including brace strength at expected tensile and either expected compressive or expected post-buckled compressive strength. It is interesting to note that the column shear demands are near the design shear at the life safety drift levels (1.5%) however they significantly exceed the design shear at the peak drifts during the MCE level test. Furthermore, reference lines are included on the column shear plots indicating the column shear forces required to form an inter-story plastic panel mechanism (flexural hinging at the top and bottom of columns). It is observed that the column shear demands approach this limit on the first story. The $F_{BV}$ brace force hysteresis plots include reference lines indicating the magnitude of a transverse point load required to form a plastic hinge at mid-span of the beam to which each pair of braces connects. This combined vertical brace force was only exceeded by the second story braces during this test.

Figures 5.18 and 5.19 show the shear force and maximum bending moment time histories, respectively, in each story of each column. Shear force plots include nominal shear capacity reference lines, and maximum moment plots show the plastic moment capacity as a reference. To determine the maximum column bending moment in each story, moments calculated at points of measured strain were projected using the shear force to each adjacent floor centerline elevation. The maximum was taken as the larger of the moments projected to either end of a given story and capped at $M_p$ to prevent a calculated moment from exceeding the section’s plastic bending capacity whenever a plastic hinge formed. In general, larger shear forces and bending moments were observed in the east column for all tests because it was directly attached to the actuators and all applied forces had to transfer through it before being distributed to the rest of the
Plastic hinging was observed in the first story of both columns throughout the test, as well as the third story of the east column during peak drift.

A similar set of results for the second hybrid ground motion test (LA27) are presented in Figures 5.20 through 5.25. Inter-story drift of about 3% was observed in the first and third stories, with a peak drift over 4% occurring in the second story. Hysteretic curves show further degradation of strength and stiffness in all lateral force resisting members. Complete fracture occurred in the eastern second story brace as the frame approached peak drift. Note that a small axial force is still calculated in this brace after fracture because the imbalances in force and moment about a section cut of the specimen frame are accounted for by this brace force when equilibrium is applied. The hysteretic curves in Figure 5.23 show a sharp decrease in the second story’s $F_{BH}$ force after the brace in tension fractures, and significantly larger lateral forces in this story are transferred through shear in the columns as evident in the column shear sum (Figure 5.23) plot that shows a more rapid increase in column shear per unit drift beyond 3.5%.

Figure 5.26 shows how the hybrid experimental drift results compare with the analytical predictions for the 3SCBF specimen. Analytical results were produced using an experimental-scale 3SCBF OpenSees model similar to the models described in Section 4.1. This model included all features present in the prototype-scale model, but the rigid end offsets were removed to better match the natural periods of the experimental specimen. Note that the two sets of results are slightly out of phase due to slowdown in the actuators during experimental testing. This slowdown could not be fully corrected for because the dataset monitoring the actuators’ state (prediction, correction, or slowdown) had to be down-sampled to avoid reaching a file size limit that would have prevented this
data from being acquired throughout the entire duration of hybrid testing. Despite this shortcoming, reasonable agreement between analytical and hybrid experimental drift results can be observed for both ground motions. The physical frame’s reduced stiffness relative to the analytical model resulted in higher drifts than were predicted during the first ground motion. As testing progressed, more brace fracture occurred in the analytical model than the physical frame, reducing its stiffness. As a result, the peak and residual drifts observed during hybrid testing were smaller than those predicted by the OpenSees model during the LA27 ground motion.

Changes in the distribution of lateral forces are evident in the results of the cyclic test to failure (Figures 5.27 to 5.32), as braces continue to fracture and place increased demand on the columns. All three western braces fractured nearly simultaneously during the first negative drift cycle (frame drifting east with western braces in tension). The first story and base shear hysteretic behavior shown in Figure 5.27 includes the column base flexural hinging which has nominal plastic strength assuming column cantilever action of about 31 kips per column (62 kips total for the first story). The reversed cyclic flexural yielding of the column is also evident in the column moment history in Figure 5.32. After the eastern third story brace fractured (around 240 seconds), the frame’s response changed suddenly, with observable jumps in internal force and moment for several members at this time. The western first story brace force calculated from equilibrium became unrealistically large immediately following fracture of the eastern third story brace due to an unknown redistribution of internal member forces and moments. This new load path was not accounted for in the equilibrium calculation used to obtain the brace force, so the unbalanced forces and moments necessitated a large axial force in the
fractured brace to satisfy equilibrium. The eastern brace on the first story was the last brace remaining until it fractured during the fourth positive (frame drifting west with eastern braces in tension) drift cycle. The contribution of the eastern first story brace until fracture is seen in Figure 5.27 first story hysteretic plot and base shear hysteresis that includes the column base hysteresis and final degradation of the braces.

5.7.3 3NCBF Results

Figures 5.33 through 5.52 contain results from the 3NCBF tests. Figure 5.33 shows global response quantities during the LA11 (DBE) hybrid test for the combined specimen frame and SRC system obtained from actuator forces and displacements. Drifts remained under 1% for all stories during this test, with an apparent concentration of drift in the first story. As mentioned in Section 5.6.2, some base plate sliding was observed during hybrid testing of this frame, and this quantity was subtracted out from the first floor actuator displacement reading to eliminate its effect on drift calculations. The 3NCBF specimen had to be fabricated in the laboratory due to a miscommunication with the steel fabricator, likely resulting in additional connection slop relative to the 3SCBF specimen (which was delivered fully fabricated). While this drift concentration is uncharacteristic of a frame rehabilitated using an SRC, all stories remained within the life safety performance objective (1.5% drift) for the duration of the test.

Figure 5.34 separates the global inter-story shear response into relative contributions of the SRC and specimen frame. The SRC and braced frame each contributed roughly equal inter-story shear forces along the height of the system, indicating that the SRC and links were fully engaged and redistributing lateral forces to mitigate soft-story formation. Figure 5.35 shows time histories of axial and shear forces.
in each link. Flexural yielding did not occur in any link according to the shear force plots. Link rotations could not be calculated for this test due to a loss in Krypton data.

Figure 5.36 shows the hysteretic behavior of the specimen frame inter-story shear. Some flattening of this curve can be seen on the first story plot near zero drift, indicating that connection slop prevented the full stiffness of the frame from activating until all connections engaged. Figures 5.37 through 5.41 further separate the total specimen frame response into contributions from the braces and columns. All brace forces were calculated directly from strain gauge data rather than the equilibrium procedure described in Section 5.7.1 because all gauges remained operational throughout the duration of testing and limited inelasticity was observed in their readings. Brace hysteretic curves in Figure 5.38 show limited degradation of brace strength and stiffness during this test due to the low levels of drift and, therefore, small amount of accumulated damage in any given brace. Figure 5.39 indicates that the columns exceeded the AISC 341 design shear force in the first story, but a panel mechanism did not form due to the contribution of the SRC. Figures 5.40 and 5.41 confirm that the braced frame columns did not pick up significant lateral force during this test.

A similar set of results for the LA37 (MCE) test are presented in Figures 5.42 through 5.50. Drift concentration again occurred in the first story, but all stories remained within the collapse prevention performance objective (2% drift) for the duration of this test. The SRC frequently contributed more inter-story shear than the specimen frame during this test due to an accumulation of damage in the braces, which results in a loss of strength and stiffness and causes the undamaged SRC to attract more force. Figure 5.44 shows the response of the links during this test, including their shear-rotation
hysteretic behavior. The first floor link may have yielded during the peak drift in this test, and a small amount of whitewash cracking was observed on this link after hybrid testing. Note that the direction of the hysteretic curves is due to the assumed sign convention, with positive rotations taken as positive curvature (positive moment at the fixed end) which require a negative shear force to return the moment diagram to zero at the pin connection. The specimen frame columns (Figure 5.49) experienced larger shear forces and moments than during the LA11 test, but flexural yield was still not observed. The AISC 341 design column shear force was significantly exceeded in the first story specimen frame columns.

Figure 5.51 shows how the analytical predictions compare to hybrid experimental results for the 3NCBF specimen. Analytical results were produced using an experimental-scale 3NCBF OpenSees model similar to the models described in Section 4.1. This model included all features present in the prototype-scale model, but the rigid end offsets were removed to better match the natural periods of the experimental specimen. The LA11 results are slightly out of phase due to actuator slowdown that occurred during the first large drift cycle. Analytical and experimental results were lined up to match the time at which the largest peak drift occurs (around four seconds into the simulated motion), and slowdown effectively expanded the time axis for the hybrid experimental data set prior to this point, causing the analytical prediction to lag behind experimental results prior to the peak drift. The first story experienced slightly larger than predicted drifts during both ground motions, while the second story underwent less drift than expected. In the third story, some higher mode response was observed analytically that is not present in the experimental results. This makes a direct
comparison difficult, but the magnitudes of peak experimental drifts in this story are similar to the analytical predictions. These plots show reasonable agreement between analytical predictions and experimental results, indicating that the analytical model accurately captures the behavior of the physical specimen.

Global response results for the 3NCBF pullback test are shown in Figure 5.52. The sudden drop in inter-story shear force observed in the third story at about -1.5% drift was due to fracture of the connection between the third floor beam and east column in the specimen frame. A bolt bearing and tear out failure occurred in the second floor beam’s eastern stub at about -2.3% drift, resulting in a significant decrease in strength in the second story. No brace fracture occurred during any 3NCBF specimen tests.

5.7.4 6SCBF Results

6SCBF test results are shown in Figures 5.53 to 5.76. Figure 5.53 shows that the first two stories—which were connected to the SRC by pin-pin ended links—drifted nearly uniformly together for the duration of the LA27 hybrid test. Note that the 6SCBF specimen was not connected to the SRC at the third floor. Peak drifts in these two stories exceeded the collapse prevention performance objective (2% drift) because the pin-pin ended links are not able to reduce overall drift by dissipating energy through yielding, limiting the function of the SRC to simply enforcing a uniform drift. Drifts in the third story remained under 2% for the duration of this test.

The relative contributions of the SRC and specimen frame to total inter-story shear are shown in Figure 5.54. Inter-story shear in the SRC was small relative to that in the specimen frame for this test, indicating that the SRC was not significantly engaged. Braced frame hysteretic behavior is shown in Figure 5.55, and the degradation of strength
and stiffness after repeated cycles can be observed. Figure 5.56 shows axial and $F_{BH}$ forces in the braces. For this specimen, the braces (particularly in the first story) were stronger than expected and exceeded the maximum expected force in both tension and compression at several points throughout testing. Brace axial force hysteresis plots in Figure 5.57 show degradation in strength and stiffness that corresponds to the hysteretic degradation for the entire frame observed in Figure 5.55. The brace axial force hysteresis plots also show the first story braces hardening during peaks in tension, resulting in higher than expected forces in these braces. No brace fracture was observed during this test.

Relative contributions of the specimen frame braces and columns can be seen in the hysteretic plots in Figure 5.58. Note that the AISC 341 design column shears were exceeded in each story during this test. These design values severely underestimated the shear demands on the first and second story columns because these two stories used the same brace sections, resulting in no difference between their lateral force capacities. When performing the analysis to determine design column shears per AISC 341, the columns simply have to transfer the interaction force between these two stories and do not have to account for a difference in lateral force capacity if the braces are identical. In experimental testing, differences in the rates at which each story’s braces degraded introduced a difference in lateral force capacity that the columns had to account for, resulting in column shear demands that significantly exceeded their design forces. Column shear force and maximum moment time histories are shown in Figures 5.59 and 5.60, respectively. The shear plots again indicate significant demands placed on the columns and the moment time histories suggest the formation of a two-story panel.
mechanism, with flexural hinging occurring at the base and second floor elevations of the columns during peak drifts.

Results from the NF09 (near-fault) hybrid test are shown in Figures 5.61 though 5.68. Note that this test was aborted after the largest pulse in the ground motion due to an unexpected hydraulic supply shutoff in the laboratory. Similar trends in results can be observed from this test as in the LA27 test. Drift was uniform but exceeded 2% in the first and second stories. The SRC contributed more to the global response during this test than for the LA27 test, which is expected due to the higher level of drift and reduced strength and stiffness in the braced frame due to accumulated damage in the braces. The western brace in the first story of the specimen frame exceeded the maximum expected tensile strength during the largest negative drift peak, and fracture was observed in this brace near this point. Fracture was also observed in the second story west brace near the peak negative drift. Column shears again exceeded their AISC 341 design forces, and the formation of a two-story panel mechanism was observed.

A comparison between analytically predicted drifts and hybrid experimental results for the 6SCBF specimen is shown in Figure 5.69. Analytical results were produced using an experimental-scale 6SCBF OpenSees model similar to the models described in Section 4.1. This model included all features present in the prototype-scale model, but the rigid end offsets were removed to better match the natural periods of the experimental specimen. This specimen matched predicted drift values better than any other, verifying the accuracy of the analytical model.

Results from the cyclic test are shown in Figures 5.70 to 5.76. The drift time history plots on Figure 5.70 begin with the frame being straightened back to zero drift.
from where the NF09 test was aborted. The second story eastern brace fractured during the first positive drift excursion, and the western third story brace fractured during the following drift in the negative direction. The eastern braces in the first and third stories fractured during the second positive drift excursion, and the negative drift pull was not performed because all braces had already fractured. The bare frame was cycled at 4% drift to conclude the cyclic test. Global inter-story shear hysteretic plots clearly show a reduction in strength and stiffness as more braces fracture.

The reduced strength of the braced frame specimen can clearly be observed in Figure 5.71 in the braced frame inter-story shear time history plots. Successive peaks get smaller as the frame loses strength, and a corresponding increase in inter-story shear can be observed in the SRC time history plots. Figures 5.73 and 5.74 show how the braced frame columns pick up additional shear as the braces fracture, eventually simply yielding the columns in flexure acting as cantilever elements.
Table 5.1: Similitude Scaling Factors

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<th>Scaling Quantity</th>
<th>Dimensional Scale Requirements</th>
<th>Required Scale Factor</th>
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<td>*Length, L</td>
<td>$\lambda_L = 3.25$</td>
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<td>Area, A</td>
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<tr>
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</tr>
<tr>
<td>Mass, m</td>
<td>$\lambda_m = \lambda_E/\lambda_a$</td>
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</tr>
</tbody>
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*Geometric length scale limited by actuator force

$^\wedge$Acceleration scale factors equal to one due to model tested in 1-g acceleration field

$^\circ$Elastic modulus scale factor equal to one because steel used for both prototype and model
Table 5.2: 3SCBF Member Section Scaling

## Beams

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<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
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<td>Iₓ (in⁴)</td>
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## Columns

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<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
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<td>Zₐ (in³)</td>
<td>Iₓ (in⁴)</td>
</tr>
<tr>
<td>1-3</td>
<td>W14x211</td>
<td>390.0</td>
<td>2660.0</td>
</tr>
</tbody>
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## Braces

<table>
<thead>
<tr>
<th>Story</th>
<th>Full-Scale Sections</th>
<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section</td>
<td>b/t, rᵧ (in)</td>
<td>Aᵧ (in²)</td>
</tr>
<tr>
<td>1, 2</td>
<td>HSS8x8x1/2</td>
<td>14.20, 3.04</td>
<td>13.50</td>
</tr>
<tr>
<td>3</td>
<td>HSS6x6x3/8</td>
<td>14.20, 2.28</td>
<td>7.58</td>
</tr>
<tr>
<td>Floor</td>
<td>Beams</td>
<td>Full-Scale Sections</td>
<td>Scaled Properties</td>
</tr>
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<td>-------</td>
<td>-------</td>
<td>---------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Section</td>
<td>Z&lt;sub&gt;x&lt;/sub&gt; (in&lt;sup&gt;3&lt;/sup&gt;)</td>
</tr>
<tr>
<td>1, 3</td>
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<td>W33x118</td>
<td>415.0</td>
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<td>2</td>
<td></td>
<td>W33x130</td>
<td>467.0</td>
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<table>
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<th>Columns</th>
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<th>Full-Scale Sections</th>
<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Section</td>
<td>Z&lt;sub&gt;x&lt;/sub&gt; (in&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>I&lt;sub&gt;x&lt;/sub&gt; (in&lt;sup&gt;4&lt;/sup&gt;)</td>
</tr>
<tr>
<td>1-3</td>
<td></td>
<td>W12x96</td>
<td>147.0</td>
<td>833.0</td>
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<table>
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<tr>
<th>Braces</th>
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<th>Full-Scale Sections</th>
<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Section</td>
<td>b/t</td>
<td>r&lt;sub&gt;y&lt;/sub&gt; (in)</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td>HSS7x7x3/8</td>
<td>17.10</td>
<td>2.69</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>HSS6x6x1/2</td>
<td>9.90</td>
<td>2.23</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>HSS5x5x1/2</td>
<td>7.75</td>
<td>1.82</td>
</tr>
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</table>
Table 5.4: 6SCBF Member Section Scaling

<table>
<thead>
<tr>
<th>Floor</th>
<th>Full-Scale Sections</th>
<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section</td>
<td>$Z_x$ (in(^3))</td>
<td>$I_x$ (in(^4))</td>
</tr>
<tr>
<td>1</td>
<td>W44x230</td>
<td>1100.0</td>
<td>20800.0</td>
</tr>
<tr>
<td>2, 3</td>
<td>W30x211</td>
<td>751.0</td>
<td>10300.0</td>
</tr>
<tr>
<td>4</td>
<td>W30x211</td>
<td>751.0</td>
<td>10300.0</td>
</tr>
<tr>
<td>5</td>
<td>W27x178</td>
<td>570.0</td>
<td>7020.0</td>
</tr>
<tr>
<td>6</td>
<td>W27x178</td>
<td>570.0</td>
<td>7020.0</td>
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</table>

<table>
<thead>
<tr>
<th>Story</th>
<th>Full-Scale Sections (Improved)</th>
<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
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<tbody>
<tr>
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<td>Section</td>
<td>$Z_x$ (in(^3))</td>
<td>$I_x$ (in(^4))</td>
</tr>
<tr>
<td>1-3</td>
<td>W14x211</td>
<td>390.0</td>
<td>2660.0</td>
</tr>
<tr>
<td>4-6</td>
<td>W14x132</td>
<td>234.0</td>
<td>1530.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Story</th>
<th>Full-Scale Sections</th>
<th>Scaled Properties</th>
<th>Experimental-Scale Sections</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>Section</td>
<td>$b/t$</td>
<td>$r_y$ (in)</td>
</tr>
<tr>
<td>1</td>
<td>HSS10x10x1/2</td>
<td>18.50</td>
<td>3.86</td>
</tr>
<tr>
<td>2, 3</td>
<td>HSS8x8x1/2</td>
<td>14.20</td>
<td>3.04</td>
</tr>
<tr>
<td>4</td>
<td>HSS8x8x1/2</td>
<td>14.20</td>
<td>3.04</td>
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<tr>
<td>5</td>
<td>HSS6x6x1/2</td>
<td>9.90</td>
<td>2.23</td>
</tr>
<tr>
<td>6</td>
<td>HSS5x5x1/2</td>
<td>7.75</td>
<td>1.82</td>
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</table>

* (equiv) section is used in hybrid computational model with cross section dimensions exactly scaled
Table 5.5: Design to Capacity Ratios for 3NCBF Specimen Design

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Survey Resultsa</th>
<th>3NCBF Design</th>
<th></th>
<th></th>
</tr>
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<tbody>
<tr>
<td></td>
<td>DCR Bin Average</td>
<td>% of Frames in Bin</td>
<td>Design DCR</td>
<td>Member</td>
</tr>
<tr>
<td>Brace Net Section Fracture</td>
<td>1.1</td>
<td>65.0%</td>
<td>1.153</td>
<td>Top Brace</td>
</tr>
<tr>
<td></td>
<td>1.35</td>
<td>17.5%</td>
<td></td>
<td>Middle Brace</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>0.0%</td>
<td></td>
<td>Bottom Brace</td>
</tr>
<tr>
<td>Gusset Whitmore Yielding*</td>
<td>1.1</td>
<td>10.0%</td>
<td>1.483</td>
<td>Top Gusset Plates</td>
</tr>
<tr>
<td></td>
<td>1.35</td>
<td>15.0%</td>
<td></td>
<td>Middle Gusset Plates</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>50.0%</td>
<td></td>
<td>Bottom Gusset Plates</td>
</tr>
<tr>
<td>Beam-Gusset Weld Fracture*</td>
<td>1.1</td>
<td>25.0%</td>
<td>1.313</td>
<td>Top Beam/Gusset</td>
</tr>
<tr>
<td></td>
<td>1.35</td>
<td>7.5%</td>
<td></td>
<td>Middle Beam/Gusset</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>17.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brace-Gusset Weld Fracture</td>
<td>1.1</td>
<td>7.5%</td>
<td>1.350</td>
<td>Top Brace/Gusset</td>
</tr>
<tr>
<td></td>
<td>1.35</td>
<td>35.0%</td>
<td></td>
<td>Middle Brace/Gusset</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>7.5%</td>
<td></td>
<td>Bottom Brace/Gusset</td>
</tr>
<tr>
<td>Beam Bending</td>
<td>1.1</td>
<td>0.0%</td>
<td>1.600</td>
<td>Top Beam</td>
</tr>
<tr>
<td></td>
<td>1.35</td>
<td>0.0%</td>
<td></td>
<td>Middle Beam</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>75.0%</td>
<td></td>
<td>Bottom Beam</td>
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</table>

*a: based on Sloat et. al (2013)
*Capacity of detail governed by geometry of gusset imposed by brace-gusset weld length DCR design
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Modes</th>
<th>Rayleigh Damping</th>
<th>Ground Motion</th>
<th>Natural Frequencies (hz)</th>
</tr>
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<tr>
<td></td>
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<td>Mass</td>
<td>Stiffness</td>
<td>f₁</td>
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<tr>
<td>3SCBF</td>
<td>2% 1st and 2nd</td>
<td>Analytical</td>
<td>Initial Physical</td>
<td>LA33</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>LA27</td>
</tr>
<tr>
<td>3NCBF</td>
<td>2% 1st and 3rd</td>
<td>Analytical</td>
<td>Initial Analytical</td>
<td>LA11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>LA37</td>
</tr>
<tr>
<td>6SCBF</td>
<td>2% 1st and 3rd</td>
<td>Analytical</td>
<td>Initial Analytical</td>
<td>LA27</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NF109</td>
</tr>
</tbody>
</table>
Table 5.7: Experimental Testing Program

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Setup SRC</th>
<th>Links</th>
<th>Date</th>
<th>Name</th>
<th>Test Description</th>
<th>Analytical Sub-Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SCBF</td>
<td>None</td>
<td>N/A</td>
<td>7/7/2014</td>
<td>Stiffness Test</td>
<td>Displacement limit = 0.05&quot;</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7/8/2014</td>
<td>White Noise Excitation</td>
<td>Displacement limit = 0.05&quot;</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7/15/2014</td>
<td>Low-Amplitude Seismic</td>
<td>8% amplitude LA33</td>
<td>Gravity frame, no feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7/16/2014</td>
<td>Low-Amplitude Seismic</td>
<td>8% amplitude LA33</td>
<td>Gravity frame, with feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7/29/2014</td>
<td>Back-to-Back Full Seismic</td>
<td>100% amplitude LA33-LA27</td>
<td>Gravity frame, with feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7/30/2014</td>
<td>Cyclic to Failure</td>
<td>Constant uniform 4% drift</td>
<td>None</td>
</tr>
<tr>
<td>3NCBF</td>
<td>3-story</td>
<td>Fix-Pin</td>
<td>9/12/2014</td>
<td>Stiffness Test</td>
<td>Displacement limit = 0.05&quot;</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>α = 0.40</td>
<td></td>
<td>9/12/2014</td>
<td>White Noise Excitation</td>
<td>Displacement limit = 0.05&quot;</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9/15/2014</td>
<td>Low-Amplitude Seismic</td>
<td>8% amplitude LA11</td>
<td>Gravity frame, no feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9/16/2014</td>
<td>Back-to-Back Full Seismic</td>
<td>100% amplitude LA11-LA37</td>
<td>Gravity frame, with feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9/18/2014</td>
<td>Amplified Seismic</td>
<td>125% amplitude LA27</td>
<td>Gravity frame, with feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9/18/2014</td>
<td>Pullback</td>
<td>Actuators retracted w/ uniform drift profile</td>
<td>None</td>
</tr>
<tr>
<td>6SCBF</td>
<td>2-story</td>
<td>Pin-Pin</td>
<td>9/24/2014</td>
<td>Stiffness Test</td>
<td>Displacement limit = 0.05&quot;</td>
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<td>α = 0.20</td>
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<td>Displacement limit = 0.05&quot;</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>9/24/2014</td>
<td>Low-Amplitude Seismic</td>
<td>8% amplitude LA27</td>
<td>Gravity frame, upper 3 stories of 6SCBF, no feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9/24/2014</td>
<td>Low-Amplitude Seismic</td>
<td>8% amplitude LA27</td>
<td>Gravity frame, upper 3 stories of 6SCBF, with feedback</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9/25/2014</td>
<td>Back-to-Back Full Seismic</td>
<td>100% amplitude LA27-NF109</td>
<td>Gravity frame, upper 3 stories of 6SCBF, with feedback</td>
</tr>
<tr>
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<td></td>
<td>9/26/2014</td>
<td>Cyclic to Failure</td>
<td>Increasing uniform drift</td>
<td>None</td>
</tr>
</tbody>
</table>
### Table 5.8: Experimental Testing Instrumentation Inventory

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<thead>
<tr>
<th>Count</th>
<th>Name</th>
<th>Description and Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>GLW3T</td>
<td>strain gauge; west link; 3rd floor; on top of member</td>
</tr>
<tr>
<td>2</td>
<td>GLW3B</td>
<td>strain gauge; west link; 3rd floor; on bottom of member</td>
</tr>
<tr>
<td>3</td>
<td>GBW3T</td>
<td>strain gauge; west half of beam; 3rd floor; on top of member</td>
</tr>
<tr>
<td>4</td>
<td>GBW3BW</td>
<td>strain gauge; west half of beam; 3rd floor; on bottom of member; west of similar gauge</td>
</tr>
<tr>
<td>5</td>
<td>GBW3BE</td>
<td>strain gauge; west half of beam; 3rd floor; on bottom of member; east of similar gauge</td>
</tr>
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<td>6</td>
<td>GBE3T</td>
<td>strain gauge; east half of beam; 3rd floor; on top of member</td>
</tr>
<tr>
<td>7</td>
<td>GBE3BE</td>
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</tr>
<tr>
<td>8</td>
<td>GBE3BW</td>
<td>strain gauge; east half of beam; 3rd floor; on bottom of member; west of similar gauge</td>
</tr>
<tr>
<td>9</td>
<td>GCW3WA</td>
<td>strain gauge; west column; 3rd storey; on west side of member; above similar gauge</td>
</tr>
<tr>
<td>10</td>
<td>GCW3EA</td>
<td>strain gauge; west column; 3rd storey; on east side of member; above similar gauge</td>
</tr>
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<td>11</td>
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<tr>
<td>12</td>
<td>GCW3EB</td>
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</tr>
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<td>13</td>
<td>GCE3WA</td>
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<td>GCE3EA</td>
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<td>15</td>
<td>GCE3WB</td>
<td>strain gauge; east column; 3rd storey; on west side of member; below similar gauge</td>
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<tr>
<td>16</td>
<td>GCE3EB</td>
<td>strain gauge; east column; 3rd storey; on east side of member; below similar gauge</td>
</tr>
<tr>
<td>17</td>
<td>GDW3N</td>
<td>strain gauge; west diagonal; 3rd storey; on north side of member</td>
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<tr>
<td>18</td>
<td>GDW3S</td>
<td>strain gauge; west diagonal; 3rd storey; on south side of member</td>
</tr>
<tr>
<td>19</td>
<td>GDE3N</td>
<td>strain gauge; east diagonal; 3rd storey; on north side of member</td>
</tr>
<tr>
<td>20</td>
<td>GDE3S</td>
<td>strain gauge; east diagonal; 3rd storey; on south side of member</td>
</tr>
<tr>
<td>21</td>
<td>GLW2T</td>
<td>strain gauge; west link; 2nd floor; on top of member</td>
</tr>
<tr>
<td>22</td>
<td>GLW2B</td>
<td>strain gauge; west link; 2nd floor; on bottom of member</td>
</tr>
<tr>
<td>23</td>
<td>GBW2T</td>
<td>strain gauge; west half of beam; 2nd floor; on top of member</td>
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<tr>
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<td>GBW2BW</td>
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</tr>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
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</tr>
<tr>
<td>27</td>
<td>GBE2BE</td>
<td>strain gauge; east half of beam; 2nd floor; on bottom of member; east of similar gauge</td>
</tr>
<tr>
<td>28</td>
<td>GBE2BW</td>
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</tr>
<tr>
<td>29</td>
<td>GCW2WA</td>
<td>strain gauge; west column; 2nd storey; on west side of member; above similar gauge</td>
</tr>
<tr>
<td>30</td>
<td>GCW2EA</td>
<td>strain gauge; west column; 2nd storey; on east side of member; above similar gauge</td>
</tr>
<tr>
<td>31</td>
<td>GCW2WB</td>
<td>strain gauge; west column; 2nd storey; on west side of member; below similar gauge</td>
</tr>
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<td>32</td>
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<td>strain gauge; west diagonal; 2nd storey; on south side of member</td>
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<tr>
<td>39</td>
<td>GDE2N</td>
<td>strain gauge; east diagonal; 2nd storey; on north side of member</td>
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<td>SG37</td>
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<td>SG38</td>
<td>strain gauge; SRC west column; 1st storey; on east side of member</td>
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<td>SG39</td>
<td>strain gauge; SRC west column; 2nd storey; on west side of member; below similar gauge</td>
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<td>SG41</td>
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<td>strain gauge; SRC west column; 2nd storey; on east side of member; above similar gauge</td>
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<td>SG44</td>
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<td>SG54</td>
<td>strain gauge; SRC east column; 3rd storey; on east side of member; above similar gauge</td>
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<td>strain gauge; SRC beam; 1st floor; in middle of member; on web</td>
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<td>SG56</td>
<td>strain gauge; SRC beam; 2nd floor; in middle of member; on web</td>
</tr>
<tr>
<td>79</td>
<td>SG57</td>
<td>strain gauge; SRC beam; 3rd floor; in middle of member; on web</td>
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<td>SG58</td>
<td>strain gauge; SRC brace; 1st storey; near middle of member; at centroid</td>
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<td>SG59</td>
<td>strain gauge; SRC brace; 2nd storey; near middle of member; at centroid</td>
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<tr>
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<td>SG60</td>
<td>strain gauge; SRC brace; 3rd storey; near middle of member; at centroid</td>
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<tr>
<td>83</td>
<td>KLW3WA(X,Y,Z)</td>
<td>krypton LED; west link; 3rd floor; on west side of member; above similar LED; x,y,z channels</td>
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<tr>
<td>84</td>
<td>KLW3WB(X,Y,Z)</td>
<td>krypton LED; west link; 3rd floor; on west side of member; below similar LED; x,y,z channels</td>
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<tr>
<td>85</td>
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<td>krypton LED; west link; 3rd floor; on east side of member; x,y,z channels</td>
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<tr>
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<td>87</td>
<td>KDW3M(X,Y,Z)</td>
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<td>KDW2E(X,Y,Z)</td>
<td>krypton LED; west diagonal; 2nd storey; on east side of member; x,y,z channels</td>
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<td>KLW1WA(X,Y,Z)</td>
<td>krypton LED; west link; 1st floor; on west side of member; above similar LED; x,y,z channels</td>
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<td>KLW1WB(X,Y,Z)</td>
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<td>KDW1E(X,Y,Z)</td>
<td>krypton LED; west diagonal; 1st storey; on east side of member; x,y,z channels</td>
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<td>KDW1M(X,Y,Z)</td>
<td>krypton LED; west diagonal; 1st storey; in middle of member; x,y,z channels</td>
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<tr>
<td>98</td>
<td>SP17</td>
<td>string pot; foundation beam top flange; relative to strong floor</td>
</tr>
<tr>
<td>99</td>
<td>SPAW</td>
<td>string pot; west brace storey 1; oriented along longitudinal axis of member</td>
</tr>
<tr>
<td>100</td>
<td>SPMW</td>
<td>string pot; west brace storey 1; near middle of member, oriented out-of-plane</td>
</tr>
<tr>
<td>101</td>
<td>SPEW</td>
<td>string pot; west brace storey 1; near end of member, oriented out-of-plane</td>
</tr>
<tr>
<td>102</td>
<td>SPAE</td>
<td>string pot; east brace storey 1; oriented along longitudinal axis of member</td>
</tr>
<tr>
<td>103</td>
<td>SPME</td>
<td>string pot; east brace storey 1; near middle of member, oriented out-of-plane</td>
</tr>
<tr>
<td>104</td>
<td>SPEE</td>
<td>string pot; east brace storey 1; near end of member, oriented out-of-plane</td>
</tr>
<tr>
<td>105</td>
<td>D1</td>
<td>LVDT; foundation beam top flange/braced frame west column base plate</td>
</tr>
<tr>
<td>106</td>
<td>D2</td>
<td>LVDT; foundation beam top flange/braced frame east column base plate</td>
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Table 5.9: Pacific DAQ Channel Wiring

<table>
<thead>
<tr>
<th>Channel</th>
<th>Name</th>
<th>Type</th>
<th>Location</th>
<th>Units</th>
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<td>1</td>
<td>T</td>
<td>Time</td>
<td>N/A</td>
<td>seconds</td>
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<tr>
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<td>A3F</td>
<td>Load Cell</td>
<td>3rd floor actuator</td>
<td>kips</td>
</tr>
<tr>
<td>3</td>
<td>A3D</td>
<td>LVDT</td>
<td>3rd floor actuator</td>
<td>inches</td>
</tr>
<tr>
<td>4</td>
<td>A2F</td>
<td>Load Cell</td>
<td>2nd floor actuator</td>
<td>kips</td>
</tr>
<tr>
<td>5</td>
<td>A2D</td>
<td>LVDT</td>
<td>2nd floor actuator</td>
<td>inches</td>
</tr>
<tr>
<td>6</td>
<td>A1F</td>
<td>Load Cell</td>
<td>1st floor actuator</td>
<td>kips</td>
</tr>
<tr>
<td>7</td>
<td>A1D</td>
<td>LVDT</td>
<td>1st floor actuator</td>
<td>inches</td>
</tr>
<tr>
<td>8</td>
<td>GLW3T</td>
<td>Strain Gauge</td>
<td>link; 3rd floor; on top of member</td>
<td>μstrain</td>
</tr>
<tr>
<td>9</td>
<td>GLW3B</td>
<td>Strain Gauge</td>
<td>link; 3rd floor; on bottom of member</td>
<td>μstrain</td>
</tr>
<tr>
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<td>Strain Gauge</td>
<td>west half of beam; 3rd floor; on top of member; west of similar gauge</td>
<td>μstrain</td>
</tr>
<tr>
<td>11</td>
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<td>Strain Gauge</td>
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<td>μstrain</td>
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<td>Strain Gauge</td>
<td>west half of beam; 3rd floor; on bottom of member; east of similar gauge</td>
<td>μstrain</td>
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<td>Strain Gauge</td>
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<td>μstrain</td>
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<td>Strain Gauge</td>
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<tr>
<td>16</td>
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<td>μstrain</td>
</tr>
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<td>μstrain</td>
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<td>Strain Gauge</td>
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<td>μstrain</td>
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<td>Strain Gauge</td>
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<td>μstrain</td>
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<td>μstrain</td>
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<tr>
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<td>west diagonal; 3rd storey; on south side of member</td>
<td>μstrain</td>
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<tr>
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<tr>
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<td>GLW2T</td>
<td>Strain Gauge</td>
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<tr>
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<td>GLW2B</td>
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<td>µstrain</td>
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<td>Strain Gauge</td>
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<td>Strain Gauge</td>
<td>west column; 2nd storey; on west side of member; above similar gauge</td>
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*Not used for 6SCBF tests - inadequate space for 3 gauges on each clear span in 1st floor beam
*Not used for 3SCBF tests - no SRC or links
^Only used for 3SCBF tests - backup instrumentation plan in case Krypton system was unavailable
<table>
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<td>Notes</td>
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*Not used for 3SCBF tests - no SRC or links*
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### Table 5.12: 3SCBF Cyclic Test Timeline of Critical Events

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<th>East Column</th>
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**Notes:** Point of fracture could not be identified for the eastern first story brace (last brace to fracture, during cyclic test).
Figure 5.1: General Test Setup
Figure 5.2: 3SCBF Elevation View
Figure 5.3: 3NCBF Elevation View
Figure 5.5: SRC and Links Elevation View
Figure 5.6: Hybrid Analytical Sub-Structuring
Figure 5.7: Hybrid Feedback Control Communication Setup
Figure 5.8: 3SCBF Instrumentation
Figure 5.9: 3NCF Instrumentation
Figure 5.10: 6SCBF Instrumentation
Figure 5.11: SRC and Link Instrumentation

NOTE: PIN-PIN LINKS FOR 65C3B TESTS INSTRUMENTED THE SAME AS FIX-PIN LINKS PICTURED HERE
Figure 5.12: Section Cut Equilibrium and Resultant Brace Force Illustration

\[ F_{bi} = H_{we} + H_{we} = (A_{we} - A_{we}) \cos(\theta) \]
\[ F_{bv} = -(V_{we} + V_{we}) = -(A_{we} + A_{we}) \sin(\theta) \]

Notes: All forces shown with positive internal force sign convention.
Equilibrium calculations for 3NCBF and 6SCBF specimens include SRC forces and moments.
Figure 5.13: 3SCBF LA33 Hybrid Test Global Response
Figure 5.14: 3SCBF LA33 Hybrid Test Illustrative First Story Hysteresis
Figure 5.15: 3SCBF LA33 Hybrid Test Brace Force Time Histories
Figure 5.16: 3SCBF LA33 Hybrid Test Brace Axial Force Hysteretic Behavior
Figure 5.17: 3SCBF LA33 Hybrid Test Brace and Column Resultant Force Hysteretic Behavior
Figure 5.18: 3SCBF LA33 Hybrid Test Column Shear Force Time Histories
Figure 5.19: 3SCBF LA33 Hybrid Test Column Maximum Moment Time Histories
Figure 5.20: 3SCBF LA27 Hybrid Test Global Response
Figure 5.21: 3SCBF LA27 Hybrid Test Brace Force Time Histories
Figure 5.22: 3SCBF LA27 Hybrid Test Brace Axial Force Hysteretic Behavior
Figure 5.23: 3SCBF LA27 Hybrid Test Brace and Column Resultant Force Hysteretic Behavior
Figure 5.24: 3SCBF LA27 Hybrid Test Column Shear Force Time Histories
Figure 5.25: 3SCBF LA27 Hybrid Test Column Maximum Moment Time Histories
Figure 5.26: 3SCBF Comparison of Hybrid Experimental and Analytical Drifts
Figure 5.27: 3SCBF Cyclic Test Global Response
*Note: Axial force calculation in western first story brace became unreliable after $t=240s$ due to instrumentation error

Figure 5.28: 3SCBF Cyclic Test Brace Force Time Histories
*Note: Axial force calculation in western first story brace became unreliable after $t=240s$ due to instrumentation error

**Figure 5.29**: 3SCBF Cyclic Test Brace Axial Force Hysteretic Behavior
*Note: Axial force calculation in western first story brace became unreliable after t=240s due to instrumentation error

**Figure 5.30:** 3SCBF Cyclic Test Brace and Column Resultant Force Hysteretic Behavior
Figure 5.31: 3SCBF Cyclic Test Column Shear Force Time Histories
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Figure 5.34: 3NCFB LA11 Hybrid Test Braced Frame and SRC Inter-story Shear Time Histories
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Figure 5.36: 3NCBF LA11 Hybrid Test Braced Frame Inter-story Shear Hysteretic Behavior
Figure 5.37: 3NCBF LA11 Hybrid Brace Force Time Histories
Figure 5.38: 3NCBF LA11 Hybrid Test Brace Axial Force Hysteretic Behavior
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Figure 5.40: 3NCBF LA11 Hybrid Test Column Shear Force Time Histories
Figure 5.41: 3NCBF LA11 Hybrid Test Column Maximum Moment Time Histories
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Figure 5.43: 3NCBF LA37 Hybrid Test Braced Frame and SRC Inter-story Shear Time Histories
Figure 5.44: 3NCBF LA37 Hybrid Test Link Axial and Shear Force Time Histories and Shear-Rotation Hysteresis Behavior
Figure 5.45: 3NCBF LA37 Hybrid Test Braced Frame Inter-story Shear Hysteretic Behavior
Figure 5.46: 3NCBF LA37 Hybrid Test Brace Force Time Histories
Figure 5.47: 3NCBF LA37 Hybrid Test Brace Axial Force Hysteretic Behavior
Figure 5.48: 3NCBF LA37 Hybrid Test Brace and Column Resultant Force Hysteretic Behavior
Figure 5.49: 3NCBF LA37 Hybrid Test Column Shear Force Time Histories
Figure 5.50: 3NCBF LA37 Hybrid Test Column Maximum Moment Time Histories
Figure 5.51: 3NCBF Hybrid Experimental Drift Compared to Analytical Predictions
Figure 5.52: 3NCBF Pullback Test Global Response
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Figure 5.54: 6SCBF LA27 Hybrid Test Braced Frame and SRC Inter-story Shear Time Histories
Figure 5.55: 6SCBF LA27 Hybrid Test Braced Frame Inter-story Shear Hysteretic Behavior
Figure 5.56: 6SCBF LA27 Hybrid Test Brace Force Time Histories
Figure 5.57: 6SCBF LA27 Hybrid Test Brace Axial Force Hysteretic Behavior
Figure 5.58: 6SCBF LA27 Hybrid Test Brace and Column Resultant Force Hysteretic Behavior
Figure 5.59: 6SCBF LA27 Hybrid Test Column Shear Force Time Histories
Figure 5.60: 6SCBF LA27 Hybrid Test Column Maximum Moment Time Histories
Note: An unexpected hydraulic shutoff occurred after about 2 seconds (simulation time), ending this test shortly after the largest pulse in the ground motion.

**Figure 5.61:** 6SCBF NF09 Hybrid Test Global Response
Figure 5.62: 6SCBF NF09 Hybrid Test Braced Frame and SRC Inter-story Shear Time Histories
Figure 5.63: 6SCBF NF09 Hybrid Test Braced Frame Inter-story Shear Hysteretic Behavior
Figure 5.64: 6SCBF NF09 Hybrid Test Brace Force Time Histories
Figure 5.65: 6SCBF NF09 Hybrid Test Brace Axial Force Hysteretic Behavior
Figure 5.66: 6SCBF NF09 Hybrid Test Brace and Column Resultant Force Hysteretic Behavior
Figure 5.67: 6SCBF NF09 Hybrid Test Column Shear Force Time Histories
Figure 5.68: 6SCBF NF09 Hybrid Test Column Maximum Moment Time Histories
Figure 5.69: 6SCBF Hybrid Experimental Drift Compared to Analytical Predictions
Figure 5.70: 6SCBF Cyclic Test Global Response
Figure 5.71: 6SCBF Cyclic Test Braced Frame and SRC Inter-story Shear Time Histories
Note: Spikes in third story plot due to a column strain gauge sporadically shorting out during this test

**Figure 5.72**: 6SCBF Cyclic Test Braced Frame Inter-story Shear Hysteretic Behavior
Note: Spikes in third story plot due to a column strain gauge sporadically shorting out during this test (brace forces obtained from equilibrium including columns)

**Figure 5.73:** 6SCBF Cyclic Test Braced Frame Lateral Force Time Histories
Note: Spikes in third story plot due to a column strain gauge sporadically shorting out during this test (brace forces obtained from equilibrium including columns)

**Figure 5.74:** 6SCBF Cyclic Test Braced Frame Lateral Force Hysteretic Behavior
Figure 5.75: 6SCBF Cyclic Test Column Shear Force Time Histories

Note: Spikes in third story plot due to a column strain gauge sporadically shorting out during this test.
Note: Spikes in third story plot due to a column strain gauge sporadically shorting out during this test

**Figure 5.76**: 6SCBF Cyclic Test Column Maximum Moment Time Histories
Section 6: Conclusions and Recommendations for Future Research

In this study, the behavior of existing low- to mid-rise steel concentrically braced frames (CBFs) subjected to seismic hazards was investigated, and a design methodology for rehabilitation using a stiff rocking core (SRC) was proposed and evaluated both analytically and experimentally. A review of CBFs designed prior to modern seismic specifications revealed deficient frame member design and detailing considerations resulting in capacity irregularities along the height of a structure which can lead to soft-story formation during a seismic event. Additionally, modern seismic CBF design was found to neglect consideration of non-uniform degradation of braces along the structure’s height which could potentially lead to soft-story failure. Quantification of these behavioral phenomena using the principles of structural mechanics and dynamics led to the development of a rehabilitative design methodology which uses a stiff rocking core to redistribute lateral forces along the structure’s height to enforce a nearly uniform drift profile and improve seismic performance. This method can also incorporate steel yielding link (SYL) elements to further improve seismic performance through increased building lateral strength and stiffness. Additionally, an improved method for designing CBF columns for new frame design to resist the maximum expected demands including the effects of non-uniform brace degradation was proposed. A parametric nonlinear dynamic finite element analysis study was performed to assess the seismic performance of a set of representative prototype CBFs including the proposed SRC-rehabilitation and improved CBF column design methods along with evaluation of the existing CBFs.
Large-scale hybrid experimental testing was performed to verify these analytical models and to investigate some of the practical challenges associated with SRC-rehabilitation.

6.1 Conclusions

The following conclusions were made based on the results of this study:

- Fundamental nonlinear, dynamic behavior of existing and rehabilitated CBFs was developed to understand and identify the critical behaviors that affect seismic response including system drift and distribution of internal forces for strength design (see Section 3). In modern CBFs, non-uniform degradation of braces was found to impart excessive shear and bending demands to the columns with the potential to cause a panel yielding mechanism (soft-story). Older CBFs were found to exhibit undesirable, non-ductile behaviors during seismic response that can result in soft-story formation.

- The proposed method for designing the SRC, SYL, and reaction column (RC) to achieve a target maximum inter-story drift using fundamental principles of earthquake engineering (i.e. the equal energy rule) is effective compared to the results of nonlinear transient seismic analysis (See Table 4.39).

- The proposed SRC-rehabilitation design methodology accurately predicts the maximum expected drift response when the structure is expected to remain within the rehabilitated drift performance objectives (2% maximum inter-story drift for collapse prevention). For systems responding beyond this level of drift (i.e. existing sub-standard frames), large deformation effects and the highly-damaged
system’s sensitivity to variations in ground motion input make accurate drift prediction difficult.

- Pin-pin-ended (non-yielding) braced frame-to-SRC links are likely adequate for newer CBFs, which exhibit ductile failure modes, to achieve the desired rehabilitated drift performance objective (see Figures 4.14 and 4.18) however yielding links may be required for older CBFs (Figures 4.22 and 4.23), which exhibit non-ductile brittle failure modes, in order to provide the necessary lateral strength and stiffness.

- Inter-story shear demands for SRC design were found to be conservative (Table 4.39 and Figures 4.16, 4.20, 4.26, and 4.27), likely due to the fact that the design procedure assumes that the compression brace will fully degrade to its expected post-buckling strength. For the SRC-rehabilitated 3NCBF-BC (3-story frame with brittle connections), the predicted SRC values closely match the median FEA results because all braces in this frame were expected (and observed) to fracture. This indicates that a more detailed consideration of expected brace behavior is required to accurately predict the maximum expected SRC inter-story shear force for other frame types. However, the SRC must remain elastic to function properly and perform reliably therefore reasonably conservative forces are desirable for design.

- The proposed improved column design procedure for new seismic CBFs is effective for achieving the desired performance objective in low-rise structures (3SCBF). However, larger braces are likely needed in addition to improved columns for mid-rise (6SCBF) structures (Table 4.39 and Figures 4.14 and 4.18)
as the building exceeded the target drift. The larger drifts in the mid-rise frame however were not a result of significant concentrated drift but rather a lack of overall frame strength and stiffness indicating the need for larger braces however the improved columns prevented inter-story panel mechanisms in the columns (see Figures 4.15 and 4.19) thus preventing soft-story formation.

- SRC rehabilitation and improved SCBF column design show enhanced seismic behavior with reduced variability in response compared to existing frame response (evident from reduced 3rd quartile to median response lines from analysis results) which is important for reliability of the seismic design (Figures 4.14, 4.18, 4.22, and 4.23).

- Large-scale hybrid experimental test results verified the accuracy of the drift response (global behavior) observed in FEA simulations (Figures 5.26, 5.51, and 5.69). Additionally, the combined lateral forces in braced frame braces, columns, and the SRC during experimental testing reasonably agree with those observed in FEA simulations and calculated using structural mechanics. As such, the proposed design methodologies and finite element models are believed to capture the true distribution of lateral demands during both soft-story response and after SRC rehabilitation.

### 6.2 Recommendations for Future Research

During the course of this study, several issues were identified which warrant further investigation in order to fully understand the soft-story failure phenomenon in steel CBFs, the SRC-rehabilitation technique, and improved CBF column design.
The following topics are recommended for future research into the
concepts discussed in this study:

- The low-cycle fatigue material model utilized by the finite element models in this study was calibrated to particular HSS sections and is not necessarily applicable to all HSS sections. The frame-level analytical models considered here utilize nonlinear fiber beam elements which are not capable of capturing the local buckling phenomenon which significantly impacts low-cycle fatigue resistance due to the large plastic strains at the brace hinging region. The HSS sections chosen for the prototype frames in this study are believed to be accurately represented by this low-cycle fatigue material model through empirically derived factors. However, additional data is needed to accurately capture the expected low-cycle fatigue behavior of any brace member. Higher resolution analytical models of the bracing members using shell and/or solid elements that are capable of capturing local buckling are desirable. Incorporating such a brace model into nonlinear transient seismic analysis of frames may not be computationally practical today.

- Some effort was made to quantify the extent of non-uniform CBF brace degradation as a function of maximum drift using results from the FEA investigation (Section 4). This proved difficult due to the complexity of the finite element models utilized in this study. This phenomenon is likely a function of both global and local slenderness of the braces, as well as the deformation amplitude and number of plastic cycles to which each brace is subjected. A study focused on quantifying non-uniform degradation would allow for more realistic
maximum inter-story shear predictions for strength design of the SRC and improved CBF column design. This might be done using models similar to that presented here but using nonlinear static cyclic analysis to extract the differential shear. However, higher resolution models described previously might be warranted for any general bracing members.

- The prototypes considered here are reasonably representative of a significant portion of existing and new construction of CBFs. However, a more comprehensive parametric study of the SRC-rehabilitation technique applied to various building types—including different braced frame LFRS configurations, structures with moment frame LFRSs, and buildings with varying dimensions—is needed to fully characterize its effectiveness and to establish a statistical measure of reliability. This study might include incremental dynamic analysis and a more generalized parametric FEA model.

- The hinged truss SRC considered in this study could potentially offer re-centering capabilities after an earthquake by means of hydraulic jacking or cable tensioning forces applied to the SRC. A more detailed study of re-centering using an SRC is needed to confirm that the design methodology proposed in this study results in an SRC with adequate capacity to resist all re-centering demands.
APPENDIX 1: EQUIVALENT LATERAL FORCE PROCEDURE CALCULATIONS

ELFI

\[ S_k := 1.6 \]
\[ S_I := 1.1' \]
\[ F_a := 1.0 \]
\[ F_v := 1.0 \]
\[ S_{DS} := 0.67F_aS_s = 1.079 \]
\[ S_{D1} := 0.67F_vS_I = 0.797 \]
\[ C_t := 0.0 \]
\[ x := 0.7' \]
\[ h_n := 39 \]
\[ T := C_t h_n^x = 0.312 \]
\[ R := 6 \]
\[ I := 1.0 \]
\[ C_s := \min \left[ \frac{S_{DS}}{R}, \frac{S_{D1}}{I} \right] \]
\[ C_s = 0.18 \]

From ASCE 7
From seismicity maps (Los Angeles)
From seismicity maps (Los Angeles)
Assume site class B
Assume site class B
Short period design spectral acceleration
1 second design spectral acceleration
From ASCE 7-05 Table 12.8-2
From ASCE 7-05 Table 12.8-2
Frame height in feet
Fundamental period of vibration (estimate)
Response modification factor
Importance factor
Seismic response coefficient
This number is multiplied by the seismic weight of the frame to obtain the design base shear.

Historic Procedure

\[ Z := 1.5 \]
\[ S := 1.0 \]
\[ H := 39 \]
\[ D := 30 \]
\[ T := \frac{0.05H}{\sqrt{D}} = 0.356 \]
\[ C := \frac{1}{15\sqrt{T}} = 0.112 \]
\[ I := 1.0 \]
\[ R := 6 \]
\[ K := \frac{5.1}{R} = 0.85 \]
\[ Z-I-K-C\cdot S = 0.142 \]

Comparison

\[ \frac{Z-I-K-C\cdot S}{C_s} = 0.792 \]

From SEAOC 1959
Zone factor (Los Angeles)
Soil factor
Frame height in feet
Frame width in feet
Fundamental period of vibration (estimate)
Seismic coefficient
Importance factor
Response modification factor
This number is multiplied by the seismic weight of the frame to obtain the design base shear.
Ratio of design base shears obtained using the 1959 SEAOC procedure and the modern equivalent lateral force procedure.
APPENDIX 2: BRACED FRAME CALCULATIONS FOR REHABILITATIVE DESIGN

This appendix contains full calculation procedures for rehabilitative braced frame design discussed in Section 3. Numerical values for each frame correspond to the prototype braced frame designs developed for finite element analysis discussed in Section 4. The calculation procedures are presented in separate sub-sections of this appendix and are organized as follows:

A2.1 3SCBF Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift
A2.2 6SCBF Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift
A2.3 3NCBF-WB Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift
A2.4 3NCBF-BC Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift
A2.5 3SCBF SRC Design Inter-Story Shear Force
A2.6 6SCBF SRC Design Inter-Story Shear Force
A2.7 3NCBF-WB SRC Design Inter-Story Shear Force
A2.8 3NCBF-BC SRC Design Inter-Story Shear Force
A2.9 3SCBF Improved Braced Frame Column Design Procedure
A2.10 6SCBF Improved Braced Frame Column Design Procedure

A2-1
A2.1 3SCBF Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift

3SCBF SRC-Rehab Frame Stiffness and Drift Prediction Calculations

OBJECTIVE
The objective of this calculation is to develop an approximate approach for determining the added stiffness provided by the SRC, Links, and Reaction Column. Knowing the rehabilitated lateral stiffness of the frame allows for calculation of the rehabilitated period of vibration which, in turn, allows for prediction of the first mode lateral displacement of the nonlinear system through application of the equal displacement or equal energy rule. Given a target drift criteria for each hazard level, the critical rehabilitation parameters for achieving the necessary frame stiffness and strength can be sized.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration are used here for example calculations.

METHODOLOGY
The derived lateral stiffness of the frame includes the shear and overturning flexibility of the RC, links (including both shear and flexural deformations since they are short and likely shear controlled), and SRC considered as an equivalent moment frame. The existing BF stiffness is added to the SRC/Links/RC stiffness since the two systems act in parallel. The existing BF stiffness includes both shear and overturning deformations.

RESULTS/CONCLUSIONS
- Existing BF stiffness and period match reasonably well with the analytical models.
- The added stiffness of the SRC/Links/RC is very dependent on the SRC width (bSRC), reaction column axial stiffness, and the link stiffness.
- The predicted drift is based on equal energy and equal displacements rules however all cases for the 3SCBF fall within the equal energy range.
- The predicted drifts for the Existing frame (~3%), Rehabilitated without Links (~2%), and Rehabilitated with links (~0.6%) seems reasonable compared with results of NLTHA.
**Input**

Earthquake Hazard  
Haz := "MCE"

Braced Frame Columns  
BFcols := "Original"

Stories, Floors, Modes  
N := 3  
i := 1, 2.. N  
m := 1, 2.. N

Material Properties  
\( E_s := 29000 \text{ksi} \)  
\( \nu_s := 0.30 \)  
\( G_s := \frac{E_s}{2(1 + \nu_s)} = 1.115 \times 10^4 \text{ksi} \)

\( F_{ye} := 55 \text{ksi} \)

Braced Frame Geometry  
\( h_i := \)  
\( L_{be_i} := \)  
\( C_i := \)

| \(13\text{ft} \) | \(25\text{ft} \) | \( \frac{1}{6} \) |
| \(13\text{ft} \) | \(25\text{ft} \) | \( \frac{1}{3} \) |
| \(13\text{ft} \) | \(28.75\text{ft} \) | \( \frac{1}{2} \) |

\( L := 30\text{ft} \)

\( L_{b_1} := \sqrt{\left(\frac{L}{2}\right)^2 + \left(h_j\right)^2} \)

\( \theta_{b_1} := \arctan\left(\frac{(2 \cdot h_k)}{L}\right) \)

\( H_i := \sum_{j=1}^{i} h_j \)

\( H = \begin{pmatrix} 13 \\ 26 \text{ ft} \\ 39 \end{pmatrix} \)

\( L_{b_1} = \begin{pmatrix} 19.849 \text{ ft} \\ 19.849 \\ 19.849 \end{pmatrix} \)

\( \theta_b = \begin{pmatrix} 40.914 \text{ deg} \\ 40.914 \end{pmatrix} \)
Mass, Weight, Gravity Loads

\[
m_i := \frac{6.7 \text{kip sec}^2}{\text{in}^2} \sum_{i=1}^{N} m_i \cdot g = 1.94 \times 10^3 \text{-kip}
\]

\[
W_i := m_i \cdot g
\]

\[
P_{Di} := \begin{cases} 82 \text{kip} \\ 82 \text{kip} \\ 77 \text{kip} \end{cases}
\]

Braced Frame Braces

\[
A_b := \begin{cases} 1.35 \text{in}^2 \\ 1.35 \text{in}^2 \\ 7.58 \text{in}^2 \end{cases}
\]

\[
F_{bET_i} := \begin{cases} 869.4 \text{kip} \\ 869.4 \text{kip} \\ 488.15 \text{kip} \end{cases}
\]

\[
F_{bEC_i} := \begin{cases} 693.54 \text{kip} \\ 693.54 \text{kip} \\ 372.4 \text{kip} \end{cases}
\]

\[
F_{bEPB_i} := \begin{cases} 208.06 \text{kip} \\ 208.06 \text{kip} \\ 111.72 \text{kip} \end{cases}
\]

Existing BF story with braces at "post-buckling" strength

\[
F_{BH_i} := \begin{cases} (F_{bET_i} + F_{bEPB_i} \cdot \cos(\theta_{b_i})) \text{ if } i = nPB \\ (F_{bET_i} + F_{bEC_i} \cdot \cos(\theta_{b_i})) \text{ otherwise} \end{cases}
\]

\[
F_{BH_i} = \begin{cases} 814.225 \text{kip} \\ 1.181 \times 10^3 \text{kip} \end{cases}
\]

Braced Frame Beams and Columns

\[
Z_b := \begin{cases} 751 \text{in}^3 \\ 751 \text{in}^3 \\ 442 \text{in}^3 \end{cases}
\]

\[
Z_c := \begin{cases} 390 \text{in}^3 \\ 147 \text{in}^3 \end{cases}
\]

\[
M_{pc} := Z_c \cdot F_{ye}
\]

\[
I_{O_1} := 2 \left[ A_i \cdot \left( \frac{L}{2} \right)^2 \right]
\]

\[
M_{pc} = 673.75 \text{kip ft}
\]

\[
I_{O_1} = 88.125 \text{ ft}^4
\]
SRC, Links, Reaction Column

\[ b_{SRC} = 12 \text{ft} \]

Average area and moment of inertia of SRC columns

\[ A_{SC} = 40.75 \text{in}^2 \quad I_{SC} = 5265 \text{in}^4 \]

\[ A_d = \begin{bmatrix}
56.8 \text{in}^2 \\
68.5 \text{in}^2 \\
46.7 \text{in}^2 \\
56.8 \text{in}^2 \\
35.3 \text{in}^2 \\
26.5 \text{in}^2
\end{bmatrix} \quad A_{SB} = \begin{bmatrix}
56.8 \text{in}^2 \\
68.5 \text{in}^2 \\
46.7 \text{in}^2 \\
56.8 \text{in}^2 \\
35.3 \text{in}^2 \\
26.5 \text{in}^2
\end{bmatrix} \]

\[ \theta_d = \frac{h_i}{b_{SRC}} \]

\[ \theta_d = \begin{bmatrix}
47.291 \\
47.291 \cdot \text{deg}
\end{bmatrix} \]

Reaction Column Properties

\[ A_{RC} = 101 \text{in}^2 \quad I_{RC} = 4900 \text{in}^4 \]

Link Properties (see link design approach below)

Relationship between drift angle and link rotation angle

\[ \theta_L = \left( 1 + \frac{b_{SRC}}{2c_L} \right) \theta_d \]

- Use to size link length to maintain 0.08 rad link rotation (assuming shear link) for target drift angle

\[ c_L = 30 \text{in} \]

\[ I_L = 4930 \text{in}^4 \]

\[ A_v = 16.95 \text{in}^2 \]

\[ V_{pL; o} = 480 \text{kip} \]

Mode shapes and periods from analysis

Existing

\[ \phi_{1Ex} = \begin{bmatrix} 0.1827 \\ 0.3992 \\ 0.6351 \end{bmatrix} \quad \phi_{2Ex} = \begin{bmatrix} 0.469 \\ -0.4172 \\ 0.1362 \end{bmatrix} \quad \phi_{3Ex} = \begin{bmatrix} 0.5847 \\ -0.4843 \\ 0.0897 \end{bmatrix} \quad T_{Ex} = \begin{bmatrix} 0.3824 \\ 0.1517 \text{ s} \end{bmatrix} \]

Improved BF Columns

\[ \phi_{1Im} = \begin{bmatrix} 0.1837 \\ 0.4055 \\ 0.6307 \end{bmatrix} \quad \phi_{2Im} = \begin{bmatrix} 0.444 \\ -0.4278 \\ 0.1227 \end{bmatrix} \quad \phi_{3Im} = \begin{bmatrix} 0.6033 \\ -0.4642 \\ 0.1436 \end{bmatrix} \quad T_{Im} = \begin{bmatrix} 0.3628 \\ 0.0818 \end{bmatrix} \]

Existing w/SRC, RC

\[ \phi_{1R} = \begin{bmatrix} 0.1872 \\ 0.4031 \\ 0.6313 \end{bmatrix} \quad \phi_{2R} = \begin{bmatrix} 0.483 \\ -0.4192 \\ 0.1473 \end{bmatrix} \quad \phi_{3R} = \begin{bmatrix} 0.5718 \\ -0.4963 \\ 0.1202 \end{bmatrix} \quad T_{R} = \begin{bmatrix} 0.3822 \\ 0.0762 \end{bmatrix} \]

Existing w/SRC, RC, Links

\[ \phi_{1RL} = \begin{bmatrix} 0.1633 \\ 0.3932 \\ 0.644 \end{bmatrix} \quad \phi_{2RL} = \begin{bmatrix} 0.5128 \\ -0.3897 \\ 0.1712 \end{bmatrix} \quad \phi_{3RL} = \begin{bmatrix} 0.5528 \\ -0.5099 \\ 0.1148 \end{bmatrix} \quad T_{RL} = \begin{bmatrix} 0.2952 \\ 0.0748 \end{bmatrix} \]
Plastic Base Shear Capacity of Existing Frame and SRC with Links

- used later in calculation when applying equal energy approximation for inelastic system displacement calcs
- mixed mechanism formulation for determining plastic base shear capacity is considered here

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.

\[ F_{bTEB} := \min \left( F_{bET_1}, F_{bEC_i}, \frac{4Zb_iF_{ye}}{L_{bc}\sin(\theta_{b_i})} \right) F_{bTEB} = \begin{cases} 869.4 & \text{kip} \\ 488.15 \end{cases} \]

Mech\_\text{EB} := \begin{cases} \text{"Brace" if } F_{bTEB} = F_{bET_1} & \text{Mech\_EB = "Brace"} \\ \text{"Beam" otherwise} & \text{Mech\_EB = "Brace"} \end{cases}

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.

\[ F_{bTEPB} := \min \left( F_{bET_1}, F_{bEPB_i} + \frac{4Zb_iF_{ye}}{L_{bc}\sin(\theta_{b_i})} \right) F_{bTEPB} = \begin{cases} 869.4 & \text{kip} \\ 488.15 \end{cases} \]

Mech\_\text{EPB} := \begin{cases} \text{"Brace" if } F_{bTEPB} = F_{bEPB_i} & \text{Mech\_EPB = "Brace"} \\ \text{"Beam" otherwise} & \text{Mech\_EPB = "Brace"} \end{cases}

Existing frame controlling plastic mechanism: considers a panel mechanism and braces at EPB within the nPB story and all other stories elastic

Brace Internal Work
\[ W_{lb,Ex} := \left( F_{bTEPB_{nPB}} + F_{bEPB_{nPB}} \right) \cos(\theta_{h_{nPB}}, h_{nPB}) \]

Column Internal Work
\[ W_{lc,Ex} := \begin{cases} 2M_{pc} & \text{if } n_{PB} = N \\ 4M_{pc} & \text{otherwise} \end{cases} \]

External Work
\[ W_{E,Ex} := h_{nPB} \sum_{i=1}^{N} C_i \]

Internal Work = External Work
\[ V_{p,Ex} := \frac{W_{lb,Ex} + W_{lc,Ex}}{W_{E,Ex}} \]

Plastic base shear of frame (existing frame)
\[ V_{p,Ex} = 1.022 \times 10^3 \text{ kip} \]
Rehabilitated Frame controlling plastic mechanism (without Links): considers all braces at expected tensile and expected post-buckling strengths and flexural yielding at base of columns

Brace Internal Work

$$W_{ib,R} := \sum_{i=1}^{N} \left[ \left( F_{bET_i} + F_{bEPB_i} \right) \cdot \cos(\theta_{b_i}) \cdot h_i \right]$$

Column Internal Work

$$W_{ic,R} := 2 \cdot M_{pc}$$

External Work

$$W_{E,R} := \sum_{i=1}^{N} \left( H_i \cdot C_i \right)$$

Internal Work = External Work

$$V_{p,Rehab} := \frac{W_{ib,R} + W_{ic,R}}{W_{E,R}}$$

Plastic base shear of frame (rehabilitated without Links)

$$V_{p,Rehab} = 936.608 \text{ kip}$$

Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected post-buckling strengths, flexural yielding at base of columns, and yielding links with strength $V_{p,Lo}$ at each floor elevation

Brace Internal Work

$$W_{ib,RL} := \sum_{i=1}^{N} \left[ \left( F_{bET_i} + F_{bEPB_i} \right) \cdot \cos(\theta_{b_i}) \cdot h_i \right]$$

Column Internal Work

$$W_{ic,RL} := 2 \cdot M_{pc}$$

Link Internal Work

$$W_{II,RL} := N \cdot V_{p,Lo} \cdot \left( h_{SRC} + c_L \right)$$

External Work

$$W_{E,RL} := \sum_{i=1}^{N} \left( H_i \cdot C_i \right)$$

Internal Work = External Work

$$V_{p,Rehab,RL} := \frac{W_{ib,RL} + W_{ic,RL} + W_{II,RL}}{W_{E,RL}}$$

Plastic base shear of frame (rehabilitated with Links)

$$V_{p,Rehab,RL} = 1.625 \times 10^3 \text{ kip}$$
Existing Building Stiffness

Shear stiffness of each story of BF (note 0.65 factor for rigid offsets of braces)

\[ k_{ExF,s_i} := 2 \frac{E_x A_{p_i}}{\sqrt{0.65 \left( \frac{L_i}{2} \right)^2 + h_i^2}} \left( \cos(\theta_{ij}) \right)^2 \]

\[ k_{ExF,s} = \begin{cases} 2.328 \times 10^3 \text{ kip} \\ 1.307 \times 10^3 \text{ kip} \end{cases} \]

Inter-story shear force and overturning moment on BF

\[ V_{BF_i} := \sum_{j=1}^{N} C_j \]

\[ V_{BF} = \begin{bmatrix} 1 \\ 0.833 \\ 0.5 \end{bmatrix} \]

\[ M_{BF_i} := \sum_{j=1}^{N} (C_j h_j) \]

\[ M_{BF} = \begin{bmatrix} 13 \\ 10.833 \text{ ft} \\ 6.5 \end{bmatrix} \]

Overtwring or "flexural" stiffness of each story of BF using moment-area method to calculate deflections under unit lateral forces

\[ k_{ExF,f_i} := \frac{E_x I_{o_i}}{\frac{1}{2}M_{BF_i} h_i + \frac{2}{3} h_i} \]

\[ k_{ExF,f} = \begin{cases} 4.188 \times 10^4 \text{ kip} \\ 5.025 \times 10^4 \text{ kip} \\ 8.375 \times 10^4 \end{cases} \]

Stiffness of existing braced frame (series spring stiffnesses of shear and flexural frame stiffnesses)

\[ k_{ExF,i} = \frac{1}{\sum_{i=1}^{N} \left( H_i m_{f_i} \phi_{1Exi} \right)} \]

Effective first mode height

\[ h_{mEX} := \frac{1}{\sum_{i=1}^{N} \left( m_{f_i} \phi_{1Exi} \right)} \]

\[ f_{hEx} = \frac{h_3}{h_{mEX}} = 1.265 \]

Effective lateral stiffness of BF (derived based on calculating lateral deformation at roof due to unit base shear force)

\[ k_{oExF} = \frac{f_{hEx}}{7} \sum_{i=1}^{N} k_{ExF,i} = 1.033 \times 10^3 \text{ kip} \]

\[ k_{oExF} = \begin{cases} 4.076 \times 10^3 \text{ kip} \\ 1.033 \times 10^3 \text{ kip} \\ 1.033 \times 10^3 \text{ kip} \end{cases} \]

Effective first mode weight

\[ W_{1Ex} = \frac{(1.607 \times 10^3 \text{ kip}) \sum_{i=1}^{N} W_{fi} \left( \phi_{1Exi} \right)^2}{\sum_{i=1}^{N} \left[ W_{fi} \left( \phi_{1Exi} \right)^2 \right]} \]

\[ W_{1Ex} = \begin{cases} 1.607 \times 10^3 \text{ kip} \\ 1.607 \times 10^3 \text{ kip} \\ 1.607 \times 10^3 \text{ kip} \end{cases} \]

Existing building first mode period

\[ T_{Pred,Ex} = \frac{W_{1Ex}}{g k_{oExF}} \]

\[ T_{Pred,Ex} = 0.399 \text{ s} \]
Stiffness of improved braced frame

Effective first mode height
\[ h_{lm} = \frac{\sum_{i=1}^{N} \left( H_i m_f \phi_{1 lm_i} \right)}{\sum_{i=1}^{N} \left( m_f \phi_{1 lm_i} \right)} \]

Effective lateral stiffness of BF (derived based on calculating lateral deformation at roof due to unit base shear force)
\[ k_{0lmF} = \frac{f_{lm}}{7} \left( \sum_{i=1}^{N} k_{Ex_i} \right) = 1.036 \times 10^3 \text{ kip/in} \]

Effective first mode weight
\[ w_{1lm} = \frac{\sum_{i=1}^{N} \left[ W_{fi} \phi_{1 lm_i} \right]^2}{\sum_{i=1}^{N} \left[ W_{fi} \phi_{1 lm_i} \right]^2} \]

Improved building first mode period
\[ T_{Pred.lm} = 2\pi \sqrt{\frac{w_{1lm}}{g k_{0lmF}}} \]

BFcols = "Original"
\[ f_{lm} = \frac{H_3}{h_{mlm}} = 1.268 \]
\[ W_{1lm} = 1.615 \times 10^3 \text{ kip} \]
\[ \sum_{i} w_{1lm} = 0.832 \]
\[ T_{Pred.lm} = 0.399 \text{ s} \]
Existing Period Calculation using 3DOF Matrix (simply for comparison with SDOF approximation presented previously)

\[
K_{EXF} := \begin{pmatrix}
    k_{EF1} + k_{EF2} & -k_{EF2} & 0 \\
    -k_{EF2} & k_{EF2} + k_{EF3} & -k_{EF3} \\
    0 & -k_{EF3} & k_{EF3}
\end{pmatrix}
\]

\[
m_m := \begin{pmatrix}
    1.675 & 0 & 0 \\
    0 & 1.675 & 0 \\
    0 & 0 & 1.675
\end{pmatrix} \text{kip-sec}^2 \text{in}^{-1}
\]

\[
\omega_{EX} := \sqrt{\text{eigenvals}\left(K_{EXF}m_m^{-1}\right)}
\]

\[
f_{EX} := \frac{\omega_{EX}}{2\pi} = \begin{pmatrix}
    2.474 \\
    6.083 \\
    9.822
\end{pmatrix} \frac{1}{s}
\]

\[
T_{EX} = \frac{1}{f_{EX}} = \begin{pmatrix}
    0.404 \\
    0.164 \\
    0.102
\end{pmatrix}\text{ s}
\]

\[
\text{eigVec}_{EX} := \text{eigenvectors}\left(K_{EXF}m_m^{-1}\right)
\]

\[
\phi_1 := \text{submatrix}\left(\text{eigVec}_{EX} \cdot 1, 3, 1, 1\right)
\]

\[
T_{EX2} := 2\pi \sqrt{\frac{\phi_1^T m_m \phi_1}{\phi_1^T K_{EXF} \phi_1}} = 0.404 \text{ s}
\]

\[
\omega_{EX} = \begin{pmatrix}
    15.546 \\
    38.218 \\
    61.714
\end{pmatrix} \frac{\text{rad}}{\text{sec}}
\]

\[
f_{EX} = \begin{pmatrix}
    2.474 \\
    6.083 \\
    9.822
\end{pmatrix} \frac{1}{s}
\]

\[
T_{EX} = \begin{pmatrix}
    0.404 \\
    0.164 \\
    0.102
\end{pmatrix}\text{ s}
\]

\[
\text{eigVec}_{EX} = \begin{pmatrix}
    0.298 & 0.6 & -0.742 \\
    -0.54 & 0.535 & 0.65 \\
    -0.787 & -0.594 & 0.164
\end{pmatrix}
\]

\[
\phi_1 = \begin{pmatrix}
    -0.298 \\
    -0.54 \\
    -0.787
\end{pmatrix}
\]

\[T_{EX2} = 0.404 \text{ s}\]
Rehabilitated stiffness calculation that includes link and RC flexibility

Factor for correction of flexural beam stiffness to include shear flexibility

\[ C_1 = \frac{12 E_s I_L}{G_s A_v \epsilon_L^2} \]

\[ C_1 = 10.083 \]

\[ A_{eq} = \frac{N \sum h_i}{b_{SRC}^2 A_{d_i} + \frac{b_{SRC}^2 (h_i^2)}{3} A_{SB_i} + \frac{2 (h_i)^3}{3 A_{SC}^2 b_{SRC}^2}} \]

\[ A_{eq} = \begin{pmatrix} 106.189 \ 91.099 \ 65.297 \end{pmatrix} \text{ in}^2 \]

b_{effSRC} := \frac{1}{2} b_{SRC}

\[ x_{NA_1} := \frac{A_{RC} (b_{effSRC} + c_L)}{A_{RC} + A_{eq_i}} \]

\[ x_{NA} = \begin{pmatrix} 49.723 \ 53.629 \ 61.949 \end{pmatrix} \text{ in} \]

\[ I_{of_1} := A_{eq_i} x_{NA_i}^2 + A_{RC} (c_L + b_{effSRC} - x_{NA_i})^2 \]

\[ I_{of} = \begin{pmatrix} 5.386 \times 10^5 \ 4.983 \times 10^5 \ 4.126 \times 10^5 \end{pmatrix} \text{ in}^4 \]

\[ \Delta_B := 6.27 \left( \frac{h_i}{E_s I_{of_1}} \right)^3 = 1.524 \times 10^{-3} \text{ in/kip} \]

\[ k_s := \frac{\frac{3 E_s I_{RC}}{(h_1)^3} + \frac{3 E_s I_{SC}}{(h_1)^3} + \frac{E_s A_d_1}{\sqrt{b_{SRC}^2 + (h_1)^2}} (\cos(\theta_{d_1}))^2}{12 E_s I_{RC}} \]

\[ k_s = 3.802 \times 10^3 \text{ kip/in} \]

\[ k_s := \frac{\frac{12 E_s I_{RC}}{(h_2)^3} + \frac{E_s A_d_2}{\sqrt{b_{SRC}^2 + (h_2)^2}} (\cos(\theta_{d_2}))^2}{12 E_s I_{RC}} \]

\[ k_s = 3.867 \times 10^3 \text{ kip/in} \]

\[ k_s := \frac{\frac{12 E_s I_{RC}}{(h_3)^3} + \frac{E_s A_d_3}{\sqrt{b_{SRC}^2 + (h_3)^2}} (\cos(\theta_{d_3}))^2}{12 E_s I_{RC}} \]

\[ k_s = 3.15 \times 10^3 \text{ kip/in} \]

\[ \Delta_S := \frac{1}{k_1} + \frac{3}{6} k_2 + \frac{3}{6} k_3 + \frac{N b_{SRC}^2}{\frac{12 E_s I_L}{\epsilon_L^3} \left( \frac{1}{1 + C_1} \right)} \]

\[ \Delta_S = 1.115 \times 10^{-3} \text{ in/kip} \]
Effective first mode height \( h_{\text{mRL}} \)
\[
\frac{\sum_{i=1}^{N} \left( H_{i} m_{f_{i}} \phi_{1\text{RL}_{i}} \right)}{\sum_{i=1}^{N} \left( m_{f_{i}} \phi_{1\text{RL}_{i}} \right)}
\]

Effective first mode weight \( W_{1\text{RL}} \)
\[
\frac{\sum_{i=1}^{N} \left[ W_{f_{i}} \left( \phi_{1\text{RL}} \right) \right]^2}{\sum_{i=1}^{N} \left[ W_{f_{i}} \left( \phi_{1\text{RL}} \right) \right]^2}
\]

\[
\Delta_B \quad \Delta_S 
\]
\[
\frac{\Delta_B}{\Delta_B + \Delta_S} = 0.578
\]
\[
\frac{\Delta_S}{\Delta_B + \Delta_S} = 0.422
\]

\[
k_{\text{SRC}} = \frac{h_{\text{hRL}}}{\Delta_B + \Delta_S} = 473.591 \text{ kip/in}
\]

\[
k_{\text{Rehab}} = k_{0\text{ExF}} + k_{\text{SRC}}
\]

\[
k_{\text{Rehab}} = 1.507 \times 10^3 \text{ kip/in}
\]

\[
k_{0\text{ExF}} = \frac{k_{\text{SRC}}}{k_{\text{Rehab}}} = 0.686
\]
\[
k_{\text{Rehab}} = 0.314
\]

\[
T_{\text{Pred.Rehab}} = 2 \cdot \pi \sqrt{\frac{W_{1\text{RL}}}{\ell f_{\text{Rehab}}}}
\]

\[
T_{\text{Pred.Rehab}} = 0.326 \text{s}
\]
Effective Modal weights

\[
\begin{align*}
W_{E1} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{1E} \right) \right]^2 \\
W_{E2} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{2E} \right) \right]^2 \\
W_{E3} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{3E} \right) \right]^2 \\
W_{l1} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{1l} \right) \right]^2 \\
W_{l2} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{2l} \right) \right]^2 \\
W_{l3} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{3l} \right) \right]^2 \\
W_{R1} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{1r} \right) \right]^2 \\
W_{R2} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{2r} \right) \right]^2 \\
W_{R3} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{3r} \right) \right]^2 \\
W_{RL1} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{1rl} \right) \right]^2 \\
W_{RL2} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{2rl} \right) \right]^2 \\
W_{RL3} &= \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{3rl} \right) \right]^2
\end{align*}
\]

\[
\begin{align*}
W_{E1} &= 1.607 \times 10^3 \text{kip} \\
W_{E2} &= 272.377 \text{kip} \\
W_{E3} &= 60.846 \text{kip} \\
W_{l1} &= 1.615 \times 10^3 \text{kip} \\
W_{l2} &= 250.571 \text{kip} \\
W_{l3} &= 74.556 \text{kip} \\
W_{R1} &= 1.564 \times 10^3 \text{kip} \\
W_{R2} &= 266.957 \text{kip} \\
W_{R3} &= 53.956 \text{kip} \\
W_{RL1} &= 1.564 \times 10^3 \text{kip} \\
W_{RL2} &= 326.484 \text{kip} \\
W_{RL3} &= 49.83 \text{kip}
\end{align*}
\]

\[
\begin{align*}
\frac{W_{E1}}{\sum_{i} W_{f_i}} &= 0.828 \\
\frac{W_{l1}}{\sum_{i} W_{f_i}} &= 0.832 \\
\frac{W_{R1}}{\sum_{i} W_{f_i}} &= 0.806 \\
\frac{W_{RL1}}{\sum_{i} W_{f_i}} &= 0.806
\end{align*}
\]
Modal Participation
Factors

\[
\Gamma_{E1} := \frac{W_{E1}}{\sum_{i=1}^{N} W_{f1}(\phi_{1E1})_i} \quad \Gamma_{I1} := \frac{W_{I1}}{\sum_{i=1}^{N} W_{f1}(\phi_{1I1})_i} \quad \Gamma_{R1} := \frac{W_{R1}}{\sum_{i=1}^{N} W_{f1}(\phi_{1R1})_i} \quad \Gamma_{RL1} := \frac{W_{RL1}}{\sum_{i=1}^{N} W_{f1}(\phi_{1RL1})_i}
\]

\[
\Gamma_{E2} := \frac{W_{E2}}{\sum_{i=1}^{N} W_{f1}(\phi_{2E2})_i} \quad \Gamma_{I2} := \frac{W_{I2}}{\sum_{i=1}^{N} W_{f1}(\phi_{2I2})_i} \quad \Gamma_{R2} := \frac{W_{R2}}{\sum_{i=1}^{N} W_{f1}(\phi_{2R2})_i} \quad \Gamma_{RL2} := \frac{W_{RL2}}{\sum_{i=1}^{N} W_{f1}(\phi_{2RL2})_i}
\]

\[
\Gamma_{E3} := \frac{W_{E3}}{\sum_{i=1}^{N} W_{f1}(\phi_{3E3})_i} \quad \Gamma_{I3} := \frac{W_{I3}}{\sum_{i=1}^{N} W_{f1}(\phi_{3I3})_i} \quad \Gamma_{R3} := \frac{W_{R3}}{\sum_{i=1}^{N} W_{f1}(\phi_{3R3})_i} \quad \Gamma_{RL3} := \frac{W_{RL3}}{\sum_{i=1}^{N} W_{f1}(\phi_{3RL3})_i}
\]

Spectral acceleration values

\[
S_1 := \begin{cases} 1.2g & \text{if Haz = "MCE"} \\ 1.6g & \text{if Haz = "NF"} \end{cases} \quad S_1 = 1.2g
\]

\[
S_s := \begin{cases} 1.6g & \text{if Haz = "MCE"} \\ 1.4g & \text{if Haz = "NF"} \end{cases} \quad S_s = 1.6g
\]

\[
T_s := \frac{S_1}{S_s} \text{sec} = 0.75s
\]

\[
T_0 := 0.2T_s = 0.15s
\]

Coefficient to adjust 5% damped spectral acceleration values to 2% damped with T > To (ASCE 7-10)

\[
B_{d,To} := 0.8
\]

\[
B_{d,E1} := 1.0 - \frac{1.0 - B_{d,To}}{T_s}T_{E1} \quad B_{d,E1} = \begin{cases} 0.898 & \text{if Haz = "MCE"} \\ 0.96 & \text{if Haz = "NF"} \end{cases}
\]

\[
B_{d,I1} := 1.0 - \frac{1.0 - B_{d,To}}{T_s}T_{I1} \quad B_{d,I1} = \begin{cases} 0.903 & \text{if Haz = "MCE"} \\ 0.962 & \text{if Haz = "NF"} \end{cases}
\]

\[
B_{d,R1} := 1.0 - \frac{1.0 - B_{d,To}}{T_s}T_{R1} \quad B_{d,R1} = \begin{cases} 0.898 & \text{if Haz = "MCE"} \\ 0.968 & \text{if Haz = "NF"} \end{cases}
\]

\[
B_{d,RL1} := 1.0 - \frac{1.0 - B_{d,To}}{T_s}T_{RL1} \quad B_{d,RL1} = \begin{cases} 0.921 & \text{if Haz = "MCE"} \\ 0.969 & \text{if Haz = "NF"} \end{cases}
\]
Existing Building Drift (Soft Story)

\[
\text{SiteClass := "B"}
\]

\[
T_{\text{Ex}} = \begin{pmatrix} 0.38 \\ 0.15 \\ 0.09 \end{pmatrix}
\]

\[
V_{p, \text{Ex}} = 1.022 \times 10^3 \text{ kip}
\]

define a dummy stiffness matrix that has a soft first story to produce orthogonal mode shapes and calculate acceptable "plastic" mode shapes and other modal properties

\[
k_p := \begin{pmatrix} k_{p_1} + k_{p_2} & -k_{p_2} & 0 \\ -k_{p_2} & k_{p_2} + k_{p_3} & -k_{p_3} \\ 0 & -k_{p_3} & k_{p_3} \end{pmatrix} \quad k_p = \begin{pmatrix} 1 \times 10^3 & -1 \times 10^3 & 0 \\ -1 \times 10^3 & 2 \times 10^3 & -1 \times 10^3 \\ 0 & -1 \times 10^3 & 1 \times 10^3 \end{pmatrix} \text{kip} \text{ in}
\]

\[
\omega_p := \sqrt{\text{eigvals}(k_p m_m^{-1})}
\]

\[
\omega_p = \begin{pmatrix} 0.045 \\ 24.434 \end{pmatrix} \text{ rad} \text{ sec}^{-1}
\]

\[
f_p := \frac{\omega_p}{2\pi} = \begin{pmatrix} 7.1 \times 10^{-3} \\ 3.889 \end{pmatrix} \text{s}
\]

\[
f_p = \begin{pmatrix} 7.1 \times 10^{-3} \\ 3.889 \end{pmatrix} \text{s}
\]

\[
T_p := \frac{1}{f_p} = \begin{pmatrix} 140.847 \\ 0.257 \end{pmatrix} \text{s}
\]

\[
T_p = \begin{pmatrix} 140.847 \\ 0.257 \end{pmatrix} \text{s}
\]

\[
\phi_p := \text{eigenvs}(k_p m_m^{-1})
\]

\[
\phi_p = \begin{pmatrix} -0.577 & 0.707 & -0.408 \\ -0.577 & 3.536 \times 10^{-6} & 0.816 \\ -0.577 & -0.707 & -0.408 \end{pmatrix}
\]

\[
T_{p,m} := 2\pi \sqrt{k_p \phi_p m_m \phi_p^T \phi_p} = \ldots
\]

\[
T_{p,m} = \begin{pmatrix} 140.847 \\ 0.257 \end{pmatrix} \text{s}
\]
\[
W_{p_m} := \sum_{i=1}^{N} \left[ W_{f_i} \left( \frac{\phi_p^{(m)}}{\gamma_i} \right)^2 \right]
\]
\[
W_p := \begin{pmatrix}
1.94 \times 10^3 \\
3.233 \times 10^{-8} \\
1.198 \times 10^{-9}
\end{pmatrix} \text{kip}
\]
\[
\Gamma_{p_m} := \frac{W_{E_{m_p}}}{\sum_{i=1}^{N} \left[ W_{f_i} \left( \frac{\phi_p^{(m)}}{\gamma_i} \right)^2 \right]}
\]
\[
\Gamma_p := \begin{pmatrix}
1.667 \times 10^{-11} \\
-1.435
\end{pmatrix}
\]
\[
S_a := \frac{S_a}{B_d.E_{x_i}} \begin{cases}
0.4 + 0.6 \frac{T_{E_{x_i}}}{T_0} & \text{if } T_{E_{x_i}} < T_0 \\
S_a & \text{if } T_0 < T_{E_{x_i}} < T_s \\
S_1 \cdot \sec \frac{T_{E_{x_i}} - B_d.T_0}{T_{E_{x_i}}} & \text{otherwise}
\end{cases}
\]
\[
\begin{align*}
S_a &= \begin{pmatrix} 1.782 \\ 1.667 \cdot g \\ 1.244 \end{pmatrix} \\
S_{d_1} &= \frac{W_{p_1} S_{d_1}}{g k_{oExF}} \begin{pmatrix} 2.064 \times 10^{-2} \\ 2.064 \times 10^{-2} \end{pmatrix}
\end{align*}
\]
\[
\begin{align*}
S_{d_1} &= 6.154 \text{-in} \\
n_{PB} &= 1
\end{align*}
\]
\[
S_{d_2} := \frac{T_{P_2}}{4\pi^2} \cdot S_a \\
S_{d_3} := \frac{T_{P_3}}{4\pi^2} \cdot S_a
\]
\[
\begin{align*}
S_{d_2} &= 1.078 \text{-in} \\
S_{d_3} &= 0.268 \text{-in}
\end{align*}
\]
\[
S_{d,SRSS} := \sqrt{S_{d_1}^2 + S_{d_2}^2 + S_{d_3}^2}
\]
\[
S_{d,SRSS} = 6.254 \text{-in} \\
S_{u_{max,e}} = \frac{S_{d_1}}{S_{d,SRSS}} = 0.984
\]
\[
\phi_p^{(1)} := \phi_p^{(1)}
\]
\[
\begin{align*}
\phi^{(1)}_{p_1} := \phi^{(1)}_p \\
u_{max,e} := \Gamma_{p_1} \phi^{(1)}_{p_1} S_{d,SRSS}
\end{align*}
\]
\[
\begin{pmatrix}
5.179 \\
5.179 \\
5.179
\end{pmatrix} \text{-in}
\]
\[ \theta_{\text{existing}}_i = \begin{cases} u_{\text{max,}e_i} & \text{if } i = 1 \\ \frac{u_{\text{max,}e_i} - u_{\text{max,}e_{i-1}}}{h_i} & \text{otherwise} \end{cases} \]

\[ \theta_{\text{existing}} = \begin{pmatrix} 3.32 \\ 2.213 \times 10^{-5} \\ 1.107 \times 10^{-5} \end{pmatrix} \%

\text{PerfCat} = \begin{cases} \text{"Collapse" if } \max(\theta_{\text{existing}}) > 2.0\% \\ \text{"CP" if } 2.0\% > \max(\theta_{\text{existing}}) > 1.5\% \\ \text{"LS" if } 1.5\% > \max(\theta_{\text{existing}}) > 0.5\% \\ \text{"IO" if } 0.5\% > \max(\theta_{\text{existing}}) \end{cases}

\text{Haz} = \text{"MCE"}

\text{BFcols} = \text{"Original"}

**Improved Building Drift (Equal Disp and Equal Energy Calcs)**

First mode displacement applying equal energy or equal displacement rule

\[ S_a_i = \frac{S_s}{B_d.I_m_i} \begin{cases} 0.4 + 0.6 \frac{T_{I_m_i}}{T_0} & \text{if } T_{I_m_i} < T_0 \\ \frac{T_{I_m_i}}{T_0} & \text{if } T_0 < T_{I_m_i} < T_s \\ S_1 \text{-sec} & \text{otherwise} \end{cases} \]

\[ S_{d.I_m_1} = \frac{V_{p,Rchab}^2}{2k_{oImF}} + \frac{W_{I_m_1}}{k_{oImF}} \]

\[ S_{d.I_m_1} = 8.888 \text{ in} \]

Higher modes

\[ S_{d.I_m_2} = \frac{T_{I_m_2}}{4\pi^2} S_{a_2} \quad S_{d.I_m_3} = \frac{T_{I_m_3}}{4\pi^2} S_{a_3} \quad S_{d.I_m_2} = 0.327 \text{ in} \quad S_{d.I_m_3} = 0.078 \text{ in} \]

\[ S_{d.I_m.SRSS} = \sqrt{S_{d.I_m_1}^2 + S_{d.I_m_2}^2 + S_{d.I_m_3}^2} \]

\[ S_{d.I_m.SRSS} = 8.894 \text{ in} \]

\[ u_{\text{max,Im}} = \frac{3.345}{7.383} S_{d.I_m.SRSS} \]

\[ u_{\text{max,Im}} = \begin{pmatrix} 3.345 \\ 7.383 \\ 11.483 \end{pmatrix} \%

\[ S_{d.I_m} = \begin{pmatrix} 0.999 \\ \text{2.144} \\ \text{2.589} \end{pmatrix} \%

\text{PerfCat} = \begin{cases} \text{"Collapse" if } \max(\theta_{\text{Im}}) > 2.0\% \\ \text{"CP" if } 2.0\% > \max(\theta_{\text{Im}}) > 1.5\% \\ \text{"LS" if } 1.5\% > \max(\theta_{\text{Im}}) > 0.5\% \\ \text{"IO" if } 0.5\% > \max(\theta_{\text{Im}}) \end{cases}

\text{Haz} = \text{"MCE"}

\text{BFcols} = \text{"Original"}

A2.1-16
Rehabilitated Building Drift (Equal Disp and Equal Energy Calcs)

First check if added stiffness from SRCRehab needed or pin-pin links can be used

\[ S_{d1} := \begin{cases} S_k \left( 0.4 + 0.6 \frac{T_{R1}}{T_0} \right) & \text{if } T_{R1} < T_0 \\ \frac{S_k}{B_{d,R1}} & \text{if } T_0 < T_{R1} < T_s \\ \frac{S_k}{T_{R1}B_{d,To}} & \text{otherwise} \end{cases} \]

\[ S_R = \begin{cases} 0.382 \text{ s} & \text{if } T_{R1} < T_0 \\ 0.12 \text{ s} & \text{if } T_0 < T_{R1} < T_s \\ 0.076 \text{ s} & \text{otherwise} \end{cases} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,R1} := \begin{cases} \frac{V_{p,Rehab}}{2k_{oExF}} + \frac{WR_1^2}{g} \frac{S_{a1}}{k_{oExF}} & \text{if } T_{R1} < T_0 \\ \frac{WR_1^2}{g} \frac{S_{a1}}{k_{oExF}} & \text{otherwise} \end{cases} \]

\[ S_{d,R1} = 8.473 \text{ in} \]

Higher modes

\[ S_{d,R2} := \frac{\left( T_{R2} \frac{2}{\pi} S_{a2} \right)^2}{4\pi^2} \quad S_{d,R3} := \frac{\left( T_{R3} \frac{2}{\pi} S_{a3} \right)^2}{4\pi^2} \quad S_{d,R2} = 0.206 \text{ in} \quad S_{d,R3} = 0.065 \text{ in} \]

\[ S_{d,R,SRSS} := \sqrt{\left( S_{d,R1} \right)^2 + \left( S_{d,R2} \right)^2 + \left( S_{d,R3} \right)^2} \quad S_{d,R,SRSS} = 8.476 \text{ in} \quad \frac{S_{d,R1}}{S_{d,R,SRSS}} = 1 \]

\[ u_{max,R} := \Gamma_{R1} \phi_{R1} S_{d,R,SRSS} \quad u_{max,R} = \begin{cases} 3.141 \text{ in} & \text{if } i = 1 \\ 6.763 \text{ in} & \text{otherwise} \end{cases} \]

\[ \theta_{Rehab,i} := \begin{cases} u_{max,R_i} & \text{if } i = 1 \\ u_{max,R_i} - u_{max,R_{i-1}} & \text{otherwise} \end{cases} \]

\[ \theta_{Rehab} = \begin{cases} 2.013 \text{ %} & \text{if } 2.0% > \max(\theta_{Rehab}) > 1.5% \\ 2.322 \text{ %} & \text{if } 1.5% > \max(\theta_{Rehab}) > 0.5% \\ 2.454 \text{ %} & \text{otherwise} \end{cases} \]

PerfCat := "Collapse" if \( \max(\theta_{Rehab}) > 2.0% \\
"CP" if \( 2.0% > \max(\theta_{Rehab}) > 1.5% \\
"LS" if \( 1.5% > \max(\theta_{Rehab}) > 0.5% \\
"IO" if \( 0.5% > \max(\theta_{Rehab})

Haz = "MCE"  
BFCols = "Original"
Rehabilitated Building Drift with Links (Equal Disp and Equal Energy Calcs)

\[ S_{d} := \begin{cases} \frac{S_{s}}{B_{d,Rl}} & 0.4 + 0.6 \frac{T_{RL}}{T_{0}} \quad \text{if } T_{RL} < T_{0} \quad T_{RL} = \left( \begin{array}{c} 0.295 \\ 0.115 \quad \text{s} \\ 0.075 \quad \text{sec} \end{array} \right) \\ \frac{S_{s}}{B_{d,Rl}} & \text{if } T_{0} < T_{RL} < T_{s} \\ \frac{S_{s}}{B_{d,T0}} & \text{otherwise} \end{cases} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,RL1} = \begin{cases} \frac{V_{p,Rehab,L}^{2}}{2k_{Rehab}} + \frac{W_{RL1}^{2}}{g} - \frac{k_{Rehab}}{2 \sqrt{V_{p,Rehab,L}^{2} + W_{RL1}^{2}} - g} \quad \text{if } T_{RL} < \frac{S_{1}}{S_{s}} \\ \frac{W_{RL1}^{2}}{g} \quad \text{otherwise} \end{cases} \]

Higher modes

\[ S_{d,RL2} = \frac{\left( T_{RL2} \right)^{2} S_{d,RL1}}{4\pi^{2}} \quad S_{d,RL3} = \frac{\left( T_{RL3} \right)^{2} S_{d,RL1}}{4\pi^{2}} \quad S_{d,RL2} = 0.183 \text{-in} \quad S_{d,RL3} = 0.062 \text{-in} \]

\[ S_{d,RL,SRSS} = \sqrt{S_{d,RL1}^{2} + S_{d,RL2}^{2} + S_{d,RL3}^{2}} = 2.054 \text{-in} \]

\[ u_{\text{max,RL}} = \Gamma_{RL} \cdot \phi_{RL} \cdot S_{d,RL,SRSS} \quad u_{\text{max,RL}} = \left( \begin{array}{c} 0.676 \\ 1.627 \quad \text{in} \\ 2.665 \quad \text{in} \end{array} \right) \]

\[ \theta_{Rehab,L} = \begin{cases} \frac{u_{\text{max,RL}} - u_{\text{max,RL}}}{b_{i}} & \text{if } i = 1 \\ 0.61 \quad \% \end{cases} \]

Haz = "MCE"

BFCols = "Original"

PerfCat := "Collapse" if max(\(\theta_{\text{Rehab,L}}\)) > 2.0%

"CP" if 2.0% > max(\(\theta_{\text{Rehab,L}}\)) > 1.5%

"LS" if 1.5% > max(\(\theta_{\text{Rehab,L}}\)) > 0.5%

"IO" if 0.5% > max(\(\theta_{\text{Rehab,L}}\))

A2.1-18
Summary of Critical Values

Existing

\[ T_{\text{Ex}} = \begin{pmatrix} 0.382 \\ 0.152 \\ 0.09 \end{pmatrix} \text{s} \quad k_{\text{Ex}} = 1.033 \times 10^3 \text{ kip/in} \quad V_{p,\text{Ex}} = 1.022 \times 10^3 \text{ kip} \]

\[ \theta_{\text{Ex,MCE}} = \begin{pmatrix} 3.32 \\ 2.213 \times 10^{-5} \% \end{pmatrix} \]

Improved

\[ T_{\text{Im}} = \begin{pmatrix} 0.363 \\ 0.144 \\ 0.082 \end{pmatrix} \text{s} \quad k_{\text{Im}} := 1.059 \times 10^3 \text{ kip/in} \quad V_{p,\text{Im}} := 936.608 \text{ kip} \quad \theta_{\text{Im,MCE}} := \begin{pmatrix} 1.96 \\ 2.366 \% \end{pmatrix} \]

Existing with SRC, Pin-Pin Links

\[ T_{\text{R}} = \begin{pmatrix} 0.382 \\ 0.12 \\ 0.076 \end{pmatrix} \text{s} \quad k_{\text{RF}} := 1.033 \times 10^3 \text{ kip/in} \quad V_{p,\text{Rehab}} = 936.608 \text{ kip} \quad \theta_{\text{R,MCE}} := \begin{pmatrix} 2.013 \\ 2.322 \% \end{pmatrix} \]

Existing with SRC, Fix-Fix Links

\[ T_{\text{RL}} = \begin{pmatrix} 0.295 \\ 0.115 \\ 0.075 \end{pmatrix} \text{s} \quad k_{\text{Rehab}} = 1.507 \times 10^3 \text{ kip/in} \quad V_{p,\text{Rehab,L}} = 1.625 \times 10^3 \text{ kip} \]

\[ \theta_{\text{RL,MCE}} := \begin{pmatrix} 0.433 \\ 0.61 \% \end{pmatrix} \]

A2.1-19
Design Approach for Links and Reaction Column to Achieve Target Drift

Note: Final selected link and reaction column sections used in stiffness and drift calculations above.

existing frame plastic base shear strength considering soft-story mechanism

\[ V_{p,Ex} = 1.022 \times 10^3 \text{kip} \]

frame plastic base shear strength considering all story mechanism with comp. brace at EPB, Rehabilitated w/ pin-pin connected SRC

\[ V_{p,Rehab} = 936.608 \text{kip} \]

Predicted story drift of existing sub-standard frame

\[ \theta_{\text{existing}} = \begin{pmatrix} 3.32 \\ 2.213 \times 10^{-5} \\ 1.107 \times 10^{-5} \end{pmatrix} \%
\]

Predicted story drift with pin-pin SRC

\[ \theta_{\text{Rehab}} = \begin{pmatrix} 2.013 \\ 2.322 \% \\ 2.454 \end{pmatrix} \%
\]

\[ \frac{k_{\text{Rehab}}}{k_{oExF}} = 1.458 \]

Target interstory drift for rehabilitated building

\[ \theta_I := 0.5\% \]

Maximum allowable link plastic rotation

\[ \gamma_L := 0.08 \]

Lateral stiffness increase factor

- iterated design variable since both lateral strength and stiffness important for calculating drift for frame in equal energy region

\[ \omega_k := 1.125 \ldots 5 \]
Required frame plastic base shear strength to achieve target rotation (assumption equal energy, function of rehabilitated stiffness, see derivation)

\[ V_{pl} (\omega_k) := \frac{2}{3} \omega_k \theta_{12} k H N k_{oExF} - \frac{4}{9} \left( \frac{H N}{H N + k_{oExF}} - \omega_k k_{oExF} \right)^2 \]

Required plastic shear strength of each link

\[ V_{pl,L} (\omega_k) = \sum_{i} \left( \frac{C_i H_i}{N (b_{SRC} + c_L)} \right) (V_{pl} (\omega_k) - V_{p,Rehak}) \]

For design of the link, the existing all story mechanism frame strength with braces at EPB are conservatively assumed. The brace strength is likely dependent on the magnitude of the target drift.

\[ \omega_k = \left[ \begin{array}{c} 1 \\ 1.25 \\ 1.5 \\ 1.75 \\ 2 \\ 2.25 \\ 2.5 \\ 2.75 \\ 3 \\ 3.25 \\ 3.5 \\ 3.75 \\ 4 \\ 4.25 \\ 4.5 \\ \vdots \end{array} \right] \quad \begin{array}{c} 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 11 \\ 12 \\ 13 \\ 14 \\ 15 \end{array} \]

Range of solutions for VpL with each frame stiffness factor

Note that only positive, real values are physically possible.

Ideally, the smallest value of VpL is ideal since it reduces the RC, SRC, and foundation forces that need to be capacity designed for however such a small link may not achieve the stiffness needed or the axial capacity of a small link may not be adequate to transfer the interaction force. There, a few iterations trying the smallest VpL and checking the stiffness is needed (see below).

\[ V_{pl,L} (\omega_k) = \left[ \begin{array}{c} 470.934+1.869 \times 10^3 \\ 751.945+1.668 \times 10^3 \\ 1033+1.383 \times 10^3 \\ 1.314+10^3-941.744 \\ 1.049 \times 10^3 \\ 595.333 \\ 384.866 \\ 247.401 \\ 147.054 \\ 69.299 \\ 6.702 \\ -45.072 \\ -88.772 \\ -126.249 \\ -158.806 \\ -187.392 \\ -212.72 \end{array} \right] \text{kip} \]

trial stiffness factor \( \omega_{k,t} = 2.5 \)

corresponding VpL with stiffness factor to achieve target drift

\[ V_{pl,L} (\omega_{k,t}) = 384.866 \text{kip} \]
Link Selection and Checks (designed like an EBF shear link)

maximum axial force on link from SRC design calculation incl. higher mode effects and plastic interaction forces

\[ A_{L_{\text{max}}} := 764.246 \text{kip} \]

Required "squash" load of link to limit axial demand to 1/2 of yield strength

\[ P_y,_{\text{req}} := 2 \cdot A_{L_{\text{max}}} = 1.528 \times 10^3 \text{ kip} \]

Selected link section

\[ \text{LinkSection} := \"W30x116\" \]

\[ d_{L} := 30 \text{ in} \]

Link plastic shear strength

\[ V_{pL} := 480 \text{ kip} \]

\[ M_{pL} := 18900 \text{ kip-in} \]

\[ \Omega_{L} := 1.5 \]

Yield strength of section

\[ P_y := 1710 \text{ kip} \]

\[ \text{LinkConn} := \"fixfix\" \]

adjust link IXx since stiffness calculation below derived based on fix-fix links

\[ I_{L} := \begin{cases} 4930 \text{ in}^4 & \text{if } \text{LinkConn} = \"fixfix\" \\ 4 & \text{if } \text{LinkConn} = \"fixpin\" \end{cases} = 4.93 \times 10^3 \text{ in}^4 \]

\[ F_y := 50 \text{kpsi} \]

\[ A_y := \frac{V_{pL}}{0.6 F_y} = 16 \text{ in}^2 \]

Required link length to limit link rotation

\[ \epsilon_{L} := \max \left\{ \frac{b \text{SRC} \cdot \theta_t}{d_{L} \left( \gamma_{L} - \theta_t \right)} \right\} = 30 \text{ in} \]

\[ \text{LinkClass} := \begin{cases} \"Short Link\" & \text{if } \epsilon_{L} < \frac{1.6 M_{pL}}{V_{pL}} \\ \"Long Link\" & \text{if } \epsilon_{L} > \frac{2.6 M_{pL}}{V_{pL}} \\ \"Intermediate Link\" & \text{otherwise} \end{cases} \]

expected link plastic rotation with selected link length

\[ \gamma_{L} := \left( 1 + \frac{b \text{SRC}}{\epsilon_{L}} \right) \theta_t = 0.029 \]
target SRC/link/RC added stiffness

\[ k_{\text{SRC,t}} := \left( \omega_{k,t} - 1 \right) k_{\text{oExF}} = 1.55 \times 10^3 \text{kip/in} \]

Approximate Stiffness Calculation only including flexibility of RC

\[ k_{\text{SRC}} = \frac{3}{2} \left( \frac{E_s \lambda_{\text{ARC}}}{h_1} \right) \left( b_{\text{SRC}} + e_L \right)^2 \left( H_N \sum_i \left( C_i H_i \right) \right) \]

Required RC x-sectional area based on stiffness equation above

- Increase required by factor of 1.5 since sizing based on stiffness calc ignoring other flexibilities

\[ \lambda_{\text{RC,req}} = \frac{2}{3} \left( \frac{h_1 k_{\text{SRC},t} H_N \sum_i \left( C_i H_i \right) }{E_s \left( b_{\text{SRC}} + e_L \right)^2} \right) = 46.913 \text{in}^2 \]

OK as an initial design size but sizing for strength and then increasing ARC for stiffness might be better approach

Selected RC member RCSection := "W14x342"

\[ \lambda_{\text{RC}} := 101\text{in}^2 \]

\[ P_{\text{RC}} := \Omega_L N V_{pL} = 2.16 \times 10^3 \text{kip} \]

\[ M_{\text{RC}} := \Omega_L V_{pL} c_L = 1.8 \times 10^3 \text{kip-ft} \]

\[ c_b := 1.67 \]

\[ p := 0.243 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.353 \times 10^{-3} \text{ (kip-ft)}^{-1} \]

\[ D_C := \begin{cases} \frac{p |P_{\text{RC}}| + b_x |M_{\text{RC}}|}{c_b} & \text{if} \ p |P_{\text{RC}}| > 0.2 = 0.905 \\ \left[ \frac{1}{2} p |P_{\text{RC}}| + \frac{9}{8} \left( b_x |M_{\text{RC}}| \right) \right] & \text{otherwise} \end{cases} \]

Compare target stiffness factor to calculated ratio of Rehab stiffness to existing

- Design iteration converges when these are reasonably close

Final critical parameters for model

RCSection = "W14x342"

\[ b_{\text{Rehab}} = \frac{1.458}{k_{\text{oExF}}} \quad \omega_{k,t} = 2.5 \]

\[ I_L,\text{eq} := I_L \left( \frac{1}{1 + C_1} \right) = 444.826 \text{-in}^4 \]

\[ e_L = 30 \text{-in} \quad V_{pL} = 480 \text{-kip} \]

\[ I_L = 4.93 \times 10^3 \text{-in}^4 \]

A2.1-23
A2.2 6SCBF Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift

6SCBF SRC-Rehab Frame Stiffness and Drift Prediction Calculations

OBJECTIVE
The objective of this calculation is to develop an approximate approach for determining the added stiffness provided by the SRC, Links, and Reaction Column. Knowing the rehabilitated lateral stiffness of the frame allows for calculation of the rehabilitated period of vibration which, in turn, allows for prediction of the first mode lateral displacement of the nonlinear system through application of the equal displacement or equal energy rule. Given a target drift criteria for each hazard level, the critical rehabilitation parameters for achieving the necessary frame stiffness and strength can be sized.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 6SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration are used here for example calculations.

METHODOLOGY
The derived lateral stiffness of the frame includes the shear and overturning flexibility of the RC, links (including both shear and flexural deformations since they are short and likely shear controlled), and SRC considered as an equivalent moment frame. The existing BF stiffness is added to the SRC/Links/RC stiffness since the two systems act in parallel. The existing BF stiffness includes both shear and overturning deformations.

RESULTS/CONCLUSIONS
- Existing BF stiffness and period match reasonably well with the analytical models
- The added stiffness of the SRC/Links/RC is very dependent on the SRC width (bSRC), reaction column axial stiffness, and the link stiffness.
- The predicted drift is based on equal energy and equal displacements rules however all cases for the 6SCBF fall within the equal energy range.
- The predicted drifts for the Existing frame (~6%), Rehabilitated without Links (~3.75%), and Rehabilitated with links (~1.5%) seems reasonable compared with results of NLTHA.
### Input

**Earthquake Hazard**

Haz := "MCE"

**Braced Frame Columns**

BFcols := "Original"

**Stories, Floors, Modes**

\[ N := 6 \quad i := 1, 2, \ldots, N \quad m := 1, 2, \ldots, N \]

**Material Properties**

\[ E_s := 29000 \text{ksi} \quad \nu_s := 0.30 \quad F_{yc} := 55 \text{ksi} \]

\[ G_b := \frac{E_s}{2(1 + \nu_s)} = 1.115 \times 10^4 \text{ksi} \]

**Braced Frame Geometry**

\[
\begin{array}{cccccc}
18 & 18 \text{ft} & 18 \text{ft} & 2.5 \text{ft} & 25 \text{ft} \\
303 & 31 & 44 \text{ft} & 13 \text{ft} & 2.5 \text{ft} & 25 \text{ft} \\
31 & 303 & 57 \text{ft} & 13 \text{ft} & 2.5 \text{ft} & 25 \text{ft} \\
44 & 70 \text{ft} & 13 \text{ft} & 2.5 \text{ft} & 25 \text{ft} \\
303 & 83 \text{ft} & 13 \text{ft} & 0.625 \text{ft} & 28.75 \text{ft} \\
57 & 303 & 70 & \text{L}_b := \sqrt{\left(\frac{L}{2}\right)^2 + \left(h_i\right)^2} \quad \text{L} := 30 \text{ft} \\
303 & 83 & 303 & \\
\end{array}
\]

\[
\begin{align*}
\theta_{b1} := \tan \left(\frac{18 \text{ft}}{15 \text{ft}}\right) &= 50.194^-\text{deg} \\
\theta_{b2} := \tan \left(\frac{13 \text{ft}}{15 \text{ft}}\right) &= 40.914^-\text{deg}
\end{align*}
\]

\[
\theta_{bi} := \begin{cases} 
\theta_{b1} & \text{if } i = 1 \\
\theta_{b} & \text{otherwise}
\end{cases}
\]

**Mass, Weight, Gravity Loads**

\[ m_{fi} := 1.16 \text{kip-sec}^2 \text{in} \]

\[ W_f := m_{fi} \cdot g \]

\[ P_{Di} := \\
82 \text{kip} \\
82 \text{kip} \\
82 \text{kip} \\
82 \text{kip} \\
82 \text{kip} \\
77 \text{kip} \]

\[ \sum_{i=1}^{N} \left( m_{fi} \cdot g \right) = 2.687 \times 10^3 \text{kip} \]

\[ W_f = \begin{bmatrix} 447.863 \\ 447.863 \\ 447.863 \\ 447.863 \end{bmatrix} \text{kip} \]
Braced Frame Braces

\[ A_b := \begin{array}{ccc}
1107.68 \text{kip} & 877.45 \text{kip} & 263.23 \text{kip} \\
869.4 \text{kip} & 693.54 \text{kip} & 208.06 \text{kip} \\
869.4 \text{kip} & 693.54 \text{kip} & 208.06 \text{kip} \\
627.26 \text{kip} & 412.14 \text{kip} & 123.64 \text{kip} \\
507.47 \text{kip} & 270.13 \text{kip} & 81.04 \text{kip}
\end{array} \]

\[ F_{\text{BH}_i} := \begin{cases} 
\left( F_{\text{BE}_{i}} + F_{\text{BE}_{PB}} \right) \cos \theta, & \text{if } i = \text{PB} \\
\left( F_{\text{BE}_{i}} + F_{\text{BE}_{PB}} \right), & \text{otherwise}
\end{cases} \]

\[ F_{\text{BH}_i} = \begin{array}{c}
877.635 \text{-kip} \\
1.181 \times 10^3 \\
1.181 \times 10^3 \\
1.181 \times 10^3 \\
785.463 \\
587.624
\end{array} \]

Braced Frame Beams and Columns

\[ A_c := \begin{cases} 
147 & \text{if BFcols = "Improved"} \\
91.4 & \text{if BFcols = "Original"}
\end{cases} \]

\[ Z_c := \begin{cases} 
1050 & \text{if BFcols = "Improved"} \\
603 & \text{if BFcols = "Original"}
\end{cases} \]

\[ Z_{b_i} := M_{pc_i} = Z_{c_i} F_{ye} \]

\[ I_Q := 2 \begin{bmatrix} A_c \left( \frac{L}{2} \right)^2 \end{bmatrix} \]

\[ M_{pc} = \begin{bmatrix} 1.788 \times 10^3 \\
1.788 \times 10^3 \\
1.788 \times 10^3 \\
1.073 \times 10^3 \\
1.073 \times 10^3 \\
1.073 \times 10^3 \\
1.073 \times 10^3 \end{bmatrix} \text{kip ft} \]

\[ I_Q = \begin{bmatrix} 193.75 \\
193.75 \\
121.25 \\
121.25 \end{bmatrix} \text{ft}^4 \]
SRC, Links, Reaction Column

Average area and moment of inertia of SRC columns

\[ A_{SC} := 56.9 \text{in}^2 \quad I_{SC} := 2400 \text{in}^4 \]

\[ A_{d_i} := \begin{array}{ll}
62.0 \text{in}^2 & 56.8 \text{in}^2 \\
46.7 \text{in}^2 & 46.7 \text{in}^2 \\
38.8 \text{in}^2 & 38.8 \text{in}^2 \\
28.2 \text{in}^2 & 28.2 \text{in}^2 \\
23.2 \text{in}^2 & 23.2 \text{in}^2 \\
21.1 \text{in}^2 & 21.1 \text{in}^2 \\
\end{array} \]

\[ \theta_{d_i} := \tan \left( \frac{h_i}{b_{SRC}} \right) \]

\[ \theta_d = \begin{array}{ll}
56.31 \text{deg} \\
47.291 \\
47.291 \\
47.291 \\
47.291 \\
47.291 \\
\end{array} \]

Reaction Column Properties

\[ A_{RC} := 68.5 \text{in}^2 \quad I_{RC} := 3010 \text{in}^4 \]

Link Properties (see link design approach below)

Relationship between drift angle and link rotation angle
- Use to size link length to maintain 0.08 rad link rotation (assuming shear link) for target drift angle

\[ \theta_L = \left( 1 + \frac{b_{SRC}}{2 \cdot e} \right) \theta_d \]

\[ e_L := 33 \text{in} \]

\[ I_L := 3010 \text{in}^4 \]

\[ A_v := 17.12 \text{in}^2 \]

\[ V_{pL-o} := 202 \text{kip} \]
Mode shapes and periods from analysis

### Existing

<table>
<thead>
<tr>
<th>$\phi_{1Ex}$</th>
<th>$\phi_{2Ex}$</th>
<th>$\phi_{3Ex}$</th>
<th>$\phi_{4Ex}$</th>
<th>$\phi_{5Ex}$</th>
<th>$\phi_{6Ex}$</th>
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<td>0.0930</td>
<td>0.3300</td>
<td>0.4672</td>
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### Improved BF Columns

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<th>$\phi_{5Im}$</th>
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### Existing w/SRC, RC

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### Existing w/SRC, RC, Links

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<th>$\phi_{4RL}$</th>
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### Periods

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<th>$T_{Im}$</th>
<th>$T_{R}$</th>
<th>$T_{RL}$</th>
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<td>0.0541</td>
<td>0.0566</td>
<td>0.0565</td>
</tr>
</tbody>
</table>
Plastic Base Shear Capacity of Existing Frame and SRC with Links

- used later in calculation when applying equal energy approximation for inelastic system displacement calcs
- mixed mechanism formulation for determining plastic base shear capacity is considered here

**Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.**

\[
F_{bTEB_i} := \min \left( F_{bET_i}, \frac{4Z_{b_i}F_{ye}}{L_{bc_i} \sin(\theta_{b_i})} \right)
\]

\[
\text{Mech}_{EB_i} := \begin{cases} 
  \text{"Brace"} & \text{if } F_{bTEB_i} = F_{bET_i} \\
  \text{"Beam"} & \text{otherwise}
\end{cases}
\]

**Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.**

\[
F_{bTEPB_i} := \min \left( F_{bET_i}, \frac{4Z_{b_i}F_{ye}}{L_{bc_i} \sin(\theta_{b_i})} \right)
\]

\[
\text{Mech}_{EPB_i} := \begin{cases} 
  \text{"Brace"} & \text{if } F_{bTEPB_i} = F_{bET_i} \\
  \text{"Beam"} & \text{otherwise}
\end{cases}
\]

<table>
<thead>
<tr>
<th>(F_{bTEB})</th>
<th>(\text{Mech}_{EB})</th>
<th>(F_{bTEPB})</th>
<th>(\text{Mech}_{EPB})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\left(1.108 \times 10^3\right)) 869.4 869.4 627.26 507.47</td>
<td>&quot;Brace&quot; &quot;Brace&quot; &quot;Brace&quot; &quot;Brace&quot;</td>
<td>(\left(1.108 \times 10^3\right)) 869.4 869.4 627.26 507.47</td>
<td>&quot;Brace&quot; &quot;Brace&quot; &quot;Brace&quot; &quot;Brace&quot;</td>
</tr>
</tbody>
</table>

**Existing frame controlling plastic mechanism:** considers a panel mechanism and braces at EPB within the nPB story and all other stories elastic

**Brace Internal Work**
\[
W_{lb,Ex} := F_{bTEPB_{nPB}} + F_{bEPB_{nPB}} \cdot \cos(\theta_{b_{nPB}}) \cdot h_{nPB}
\]

**Column Internal Work**
\[
W_{lc,Ex} := \begin{cases} 
  2M_{pcn_{PB}} & \text{if } n_{PB} = N \\
  4M_{pcn_{PB}} & \text{otherwise}
\end{cases}
\]

**External Work**
\[
W_{E,Ex} := h_{nPB} \sum_{i=nPB}^N C_i
\]

**Internal Work = External Work**
\[
V_{p,Ex} := \frac{W_{lb,Ex} + W_{lc,Ex}}{W_{E,Ex}}
\]

**Plastic base shear of frame (existing frame)**
\[
V_{p,Ex} = 1.275 \times 10^3 \text{ kip}
\]
Rehabilitated Frame controlling plastic mechanism (without Links): considers all braces at expected tensile and expected post-buckling strengths and flexural yielding at base of columns

Brace Internal Work

\[ W_{ib,R} = \sum_{i=1}^{N} \left( F_{bET_i} + F_{bEPB_j} \right) \cos(\theta_{b_i}) \cdot h_i \]

Column Internal Work

\[ W_{ic,R} = 2M_{pC_i} \]

External Work

\[ W_{E,R} = \sum_{i=1}^{N} (H_i \cdot C_j) \]

Internal Work = External Work

\[ V_{p,Rehab} = \frac{W_{ib,R} + W_{ic,R}}{W_{E,R}} \]

Plastic base shear of frame (rehabilitated without Links)

\[ V_{p,Rehab} = 1.067 \times 10^3 \text{kip} \]

Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected post-buckling strengths, flexural yielding at base of columns, and yielding links with strength \( V_{p,L0} \) at each floor elevation

Brace Internal Work

\[ W_{ib,RL} = \sum_{i=1}^{N} \left( F_{bET_i} + F_{bEPB_j} \right) \cos(\theta_{b_i}) \cdot h_i \]

Column Internal Work

\[ W_{ic,RL} = 2M_{pC_i} \]

Link Internal Work

\[ W_{II,RL} = N \cdot V_{p,L0} \left( b_{SRC} + c_L \right) \]

External Work

\[ W_{E,RL} = \sum_{i=1}^{N} (H_i \cdot C_j) \]

Internal Work = External Work

\[ V_{p,Rehab,RL} = \frac{W_{ib,RL} + W_{ic,RL} + W_{II,RL}}{W_{E,RL}} \]

Plastic base shear of frame (rehabilitated with Links)

\[ V_{p,Rehab,RL} = 1.363 \times 10^3 \text{kip} \]
Existing Building Stiffness

Shear stiffness of each story of BF (note 0.65 factor for rigid offsets of braces)

\[ k_{ExF,i} := 2 \cdot \frac{E_i \cdot A_i}{\sqrt{0.65 \left( \frac{L}{2} \right)^2 + \left( h_i \right)^2}} \left( \cos \left( \theta_{bi} \right) \right)^2 \]

Inter-story shear force and overturning moment on BF

\[ V_{BF} := \sum_{j=1}^{N} C_j \quad M_{BF} := \sum_{j=1}^{N} (C_j \cdot h_j) \]

Overturing or “flexural” stiffness of each story of BF using moment-area method to calculate deflections under unit lateral forces

\[ k_{ExF,f} := \frac{1}{2} \frac{E_s \cdot I_o}{M_{BF} \cdot h_i \cdot \left( \frac{2}{3} \right) h_j} \]

Stiffness of existing braced frame (series spring stiffnesses of shear and flexural frame stiffnesses)

\[ k_{ExF} = \frac{1}{k_{ExF,s} + k_{ExF,f}} \]

Effective first mode height

\[ h_{mEX} = \frac{1}{\sum_{i=1}^{N} \left( m_i \cdot \phi_{Ex,i} \right)} \]

\[ f_{hEx} := \frac{H_6}{h_{mEX}} = 1.349 \]

Effective lateral stiffness of BF (derived based on calculating lateral deformation at roof due to unit base shear force)

\[ k_{oEx} := f_{hEx} \left[ \sum_{i=1}^{N} \left( k_{ExF,i} \cdot h_i \cdot H_i \right) - \sum_{i=2}^{N} \left( k_{ExF,i} \cdot h_i \cdot H_{i-1} \right) \right] = 599.722 \text{ kip/in} \]

Effective first mode weight

\[ W_{1Ex} := \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{1Ex} \right) \right]^2 = 2.156 \times 10^6 \text{ lb}^2 \text{ ft} \]

\[ T_{Pred.Ex} := 2 \pi \sqrt{\frac{W_{1Ex}}{g \cdot k_{oEx}}} \]

\[ T_{Pred.Ex} = 0.606 \text{ s} \]

\[ \sum_i W_{f_i} = 0.802 \]
Stiffness of improved braced frame

Effective first mode height

\[ h_{nlm} := \sum_{i=1}^{N} \left( \frac{H_i m_{f_i} \phi_{1lm_i}}{m_{f_i} \phi_{1lm_i}} \right) \]

\[ f_{nlm} := \frac{H_6}{h_{nlm}} = 1.37 \]

Effective lateral stiffness of BF (derived based on calculating lateral deformation at roof due to unit base shear force)

\[ k_{o1inf} := f_{nlm} \left[ \sum_{i=1}^{N} \left( \frac{k_{ExF_i} h_i - H_i}{N} \right) - \sum_{i=2}^{N} \left( \frac{k_{ExF_i} h_i H_{i-1}}{N} \right) \right] \]

\[ k_{o1inf} = 609.142 \text{ kip/in} \]

Effective first mode weight

\[ W_{1lm} := \sum_{i=1}^{N} \frac{m_{f_i} \phi_{1lm_i}^2}{\sum_{i=1}^{N} \left( m_{f_i} \phi_{1lm_i}^2 \right)^2} \]

\[ W_{1lm} = 2.228 \times 10^3 \text{ kip} \]

\[ W_{1lm} = 0.829 \]

Existing building first mode period

\[ T_{pred,lm} := 2\pi \sqrt{\frac{W_{1lm}}{g k_{o1inf}}} \]

\[ T_{pred,lm} = 0.612 \text{ s} \]

BFcols = "Original"
Existing Period Calculation using 3DOF Matrix (simply for comparison with SDOF approximation presented previously)

\[ K_{ExF} = \begin{pmatrix}
  k_{ExF_1} + k_{ExF_2} & -k_{ExF_2} & 0 & 0 & 0 \\
  -k_{ExF_2} & k_{ExF_2} + k_{ExF_3} & -k_{ExF_3} & 0 & 0 \\
  0 & -k_{ExF_3} & k_{ExF_3} + k_{ExF_4} & -k_{ExF_4} & 0 \\
  0 & 0 & -k_{ExF_4} & k_{ExF_4} + k_{ExF_5} & -k_{ExF_5} \\
  0 & 0 & 0 & -k_{ExF_5} & k_{ExF_5} + k_{ExF_6} & -k_{ExF_6} \\
  \end{pmatrix} \]

\[ m_m = \begin{pmatrix}
  1.16 & 0 & 0 & 0 & 0 \\
  0 & 1.16 & 0 & 0 & 0 \\
  0 & 0 & 1.16 & 0 & 0 \\
  0 & 0 & 0 & 1.16 & 0 \\
  0 & 0 & 0 & 0 & 1.16 \\
  \end{pmatrix} \]

\[ \omega_{Ex} = \sqrt{\text{eigenvals}(K_{ExF}^{-1}m_m)} \]

\[ \omega_{Ex} = \begin{pmatrix}
  10.033 \\
  27.485 \\
  44.547 \\
  58.687 \\
  71.579 \\
  83.841 \\
  \end{pmatrix} \text{ rad} \quad \begin{pmatrix}
  \omega_{Ex} \text{ rad} \quad 2\pi \\
  1.597 \\
  4.374 \\
  7.090 \\
  9.340 \\
  11.392 \\
  13.344 \\
  \end{pmatrix} \text{ sec} \quad \frac{1}{f_{Ex}} = \frac{1}{\text{EigVec}_{Ex}^{T} \cdot m_m \cdot \text{EigVec}_{Ex}} = \begin{pmatrix}
  6.26 \text{ s} \\
  0.229 \\
  0.141 \\
  0.088 \\
  0.075 \\
  \end{pmatrix} \]

\[ T_{Ex2} = 2\pi \sqrt{m_m \cdot \text{EigVec}_{Ex}^{T} \cdot K_{ExF} \cdot \text{EigVec}_{Ex}} = 6.26 \text{ s} \]

\[ \omega_{Ex} = \begin{pmatrix}
  0.154 & -0.358 & -0.508 & 0.475 & 0.507 & -0.327 \\
  0.265 & -0.494 & -0.382 & 3.334 \times 10^{-3} & -0.431 & 0.596 \\
  0.361 & -0.439 & 0.13 & -0.473 & -0.243 & -0.614 \\
  0.439 & -0.214 & 0.513 & -0.118 & 0.584 & 0.379 \\
  0.515 & 0.207 & 0.324 & 0.652 & -0.38 & -0.129 \\
  0.564 & 0.589 & -0.46 & -0.333 & 0.112 & 0.026 \\
  \end{pmatrix} \]

A2.2-10
Rehabilitated stiffness calculation that includes link and RC flexibility

factor for correction of flexural beam stiffness to include shear flexibility

\[ A_{eq} = \frac{N \sum b_i}{b_{SRC}^2 + (b_i)^2}^{3/2} + \frac{b_{SRC}^2}{A_{SB}} + \frac{2}{3} \frac{A_{SRC} b_{SRC}^2}{b_{SRC}^2} \]

\[ x_{NA} = \frac{A_{RC}(b_{effSRC} + \varepsilon_L)}{A_{RC} + A_{eq}} \]

\[ I_{of} = A_{eq}(x_{NA} + A_{RC}(\varepsilon_L + b_{effSRC} - x_{NA})^2 \cdot x_{NA} \cdot \varepsilon_L) \]

\[ \Delta_B = \frac{1}{1818 E_s} \left[ h_2^3 \left( \frac{166}{l_{of_6}} + \frac{1512}{l_{of_5}} + \frac{5220}{l_{of_4}} + \frac{12248}{l_{of_3}} + \frac{22994}{l_{of_2}} \right) + \frac{h_1}{l_{of_1}} \left( 2576 (b_1)^2 + 20305 h_2 h_1 + 14775 (b_2)^2 \right) \right] \]

\[ \Delta_B = 0.013 \text{ in/kip} \]

\[ k_1 = \frac{3 E_s A_{d1}}{h_1^3 \left( b_{ SRC}^2 + (h_i)^2 \right)^{3/2}} \left( \cos^2 \theta_{d1} \right) \]

\[ k_2 = \frac{12 E_s A_{RC}}{h_2^3 \left( b_{ SRC}^2 + (h_i)^2 \right)^{3/2}} \left( \cos^2 \theta_{d2} \right) \]

\[ k_3 = \frac{12 E_s A_{RC}}{h_3^3 \left( b_{ SRC}^2 + (h_i)^2 \right)^{3/2}} \left( \cos^2 \theta_{d3} \right) \]

\[ k_4 = \frac{12 E_s A_{RC}}{h_4^3 \left( b_{ SRC}^2 + (h_i)^2 \right)^{3/2}} \left( \cos^2 \theta_{d4} \right) \]

\[ k_5 = \frac{12 E_s A_{RC}}{h_5^3 \left( b_{ SRC}^2 + (h_i)^2 \right)^{3/2}} \left( \cos^2 \theta_{d5} \right) \]

\[ k_6 = \frac{12 E_s A_{RC}}{h_6^3 \left( b_{ SRC}^2 + (h_i)^2 \right)^{3/2}} \left( \cos^2 \theta_{d6} \right) \]

\[ b_{effSRC} = \frac{1}{2} b_{SRC} \]

\[ C_1 = \frac{12 E_s h_i^3}{G_s A_e c L} = 5.037 \]

<table>
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<th>Value</th>
<th>Unit</th>
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</table>

A2.2-11
\[
\Delta_S := \frac{1}{k_{s_1}} + \frac{285}{303} \frac{1}{k_{s_2}} + \frac{254}{303} \frac{1}{k_{s_3}} + \frac{210}{303} \frac{1}{k_{s_4}} + \frac{153}{303} \frac{1}{k_{s_5}} + \frac{83}{303} \frac{1}{k_{s_6}} + \frac{\sum_i (C_i H_i) H_N}{N b_{SRC}^2} + \frac{12 E_s I_L}{c_L^3} \left( \frac{1}{1 + C_1} \right)
\]

\[
\Delta_S = 2.937 \times 10^{-3} \text{ in/kip}
\]

\[
\frac{\Delta_B}{\Delta_B + \Delta_S} = 0.812 \quad \frac{\Delta_S}{\Delta_B + \Delta_S} = 0.188
\]

Effective first mode height
\[
h_{mRL} := \frac{\sum_{i=1}^{N} \left( H_i m_i \phi_{1RL_i} \right)}{\sum_{i=1}^{N} \left( m_i \phi_{1RL_i} \right)}
\]

\[f_{hRL} := \frac{H_6}{h_{mRL}} = 1.342\]

Effective first mode weight
\[
W_{1RL} := \sqrt{\sum_{i=1}^{N} \left[ W_{f_i} \phi_{1RL_i} \right]^2} \sum_{i=1}^{N} W_{f_i} \phi_{1RL_i}^2
\]

\[W_{1RL} = 2.132 \times 10^3 \text{ kip} \quad \frac{W_{1RL}}{\sum_{i} W_{f_i}} = 0.793\]

Link transverse stiffness including flexural and shear deformations
\[
k_{L,0} := \frac{12 E_s I_L}{c_L^3} \left( \frac{1}{1 + C_1} \right)
\]

\[k_{L,0} = 4.828 \times 10^{-3} \text{ kip/in}\]

Rotational stiffness of links (moment to cause unit transverse disp.)
\[
k_{L,R,0} := \frac{6 E_s I_L}{c_L^2} \left( \frac{1}{1 + C_1} \right)
\]

\[k_{L,R,0} = 7.966 \times 10^{-4} \text{ kip-in/in}\]

\[k_{oExF} = 0.875 \quad k_{SRC} = 0.125\]

\[k_{Rehab} = 685.474 \text{ kip/in}\]

Axial stiffness of RC
\[
k_{RC,0} := \frac{E_s A_{RC}}{h_1}
\]

\[k_{RC,0} = 9.197 \times 10^{-3} \text{ kip/in}\]

\[T_{Rehab} = 2 \pi \sqrt{\frac{W_{1RL}}{g t_{Rehab}}}\]

\[T_{Rehab} = 0.564 \text{ s}\]
Effective Modal weights

\[
W_{E_1} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{1E})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{1E})]^2} \quad W_{I_1} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{1I})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{1I})]^2} \quad W_{R_1} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{1R})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{1R})]^2}
\]

\[
W_{E_2} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{2E})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{2E})]^2} \quad W_{I_2} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{2I})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{2I})]^2} \quad W_{R_2} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{2R})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{2R})]^2}
\]

\[
W_{E_3} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{3E})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{3E})]^2} \quad W_{I_3} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{3I})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{3I})]^2} \quad W_{R_3} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{3R})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{3R})]^2}
\]

\[
W_{E_4} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{4E})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{4E})]^2} \quad W_{I_4} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{4I})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{4I})]^2} \quad W_{R_4} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{4R})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{4R})]^2}
\]

\[
W_{E_5} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{5E})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{5E})]^2} \quad W_{I_5} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{5I})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{5I})]^2} \quad W_{R_5} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{5R})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{5R})]^2}
\]

\[
W_{E_6} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{6E})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{6E})]^2} \quad W_{I_6} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{6I})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{6I})]^2} \quad W_{R_6} := \frac{\sum_{i=1}^{N} [W_{f_1}(\phi_{6R})]^2}{\sum_{i=1}^{N} [W_{f_1}(\phi_{6R})]^2}
\]
Modal Participation
Factors

\[
\Gamma_{E1} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{1E})]}{N} \quad \Gamma_{I1} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{1I})]}{N} \quad \Gamma_{R1} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{1R})]}{N} \quad \Gamma_{RL1} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{1RL})]}{N}
\]

\[
\Gamma_{E2} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{2E})]}{N} \quad \Gamma_{I2} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{2I})]}{N} \quad \Gamma_{R2} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{2R})]}{N} \quad \Gamma_{RL2} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{2RL})]}{N}
\]

\[
\Gamma_{E3} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{3E})]}{N} \quad \Gamma_{I3} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{3I})]}{N} \quad \Gamma_{R3} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{3R})]}{N} \quad \Gamma_{RL3} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{3RL})]}{N}
\]

\[
\Gamma_{E4} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{4E})]}{N} \quad \Gamma_{I4} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{4I})]}{N} \quad \Gamma_{R4} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{4R})]}{N} \quad \Gamma_{RL4} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{4RL})]}{N}
\]

\[
\Gamma_{E5} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{5E})]}{N} \quad \Gamma_{I5} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{5I})]}{N} \quad \Gamma_{R5} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{5R})]}{N} \quad \Gamma_{RL5} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{5RL})]}{N}
\]

\[
\Gamma_{E6} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{6E})]}{N} \quad \Gamma_{I6} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{6I})]}{N} \quad \Gamma_{R6} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{6R})]}{N} \quad \Gamma_{RL6} = \frac{\sum_{i=1}^{N} [w_{f_i}(\phi_{6RL})]}{N}
\]

\[
\Gamma_{E1} = 2.363 \quad \Gamma_{I1} = 2.402 \quad \Gamma_{R1} = 2.377 \quad \Gamma_{RL1} = 2.35
\]

\[
\Gamma_{E2} = 1.03 \quad \Gamma_{I2} = 0.919 \quad \Gamma_{R2} = 1.006 \quad \Gamma_{RL2} = 1.063
\]

\[
\Gamma_{E3} = 0.462 \quad \Gamma_{I3} = 0.461 \quad \Gamma_{R3} = 0.44 \quad \Gamma_{RL3} = 0.453
\]

\[
\Gamma_{E4} = 0.245 \quad \Gamma_{I4} = 0.196 \quad \Gamma_{R4} = 0.25 \quad \Gamma_{RL4} = 0.25
\]

\[
\Gamma_{E5} = 0.178 \quad \Gamma_{I5} = 0.208 \quad \Gamma_{R5} = 0.184 \quad \Gamma_{RL5} = 0.18
\]

\[
\Gamma_{E6} = 0.098 \quad \Gamma_{I6} = 0.13 \quad \Gamma_{R6} = 0.097 \quad \Gamma_{RL6} = 0.099
\]
\begin{align*}
\text{Spectral acceleration values} & \\
S_1 := & \begin{cases} 1.2g \text{ if Haz } = \text{ "MCE"} \\ 1.6g \text{ if Haz } = \text{ "NF"} \end{cases} & S_1 = 1.2g \\
S_s := & \begin{cases} 1.6g \text{ if Haz } = \text{ "MCE"} \\ 1.4g \text{ if Haz } = \text{ "NF"} \end{cases} & S_s = 1.6g \\
T_s := & \frac{S_1}{S_s} \text{ sec } = 0.75 \text{ s} \\
T_0 := & 0.2 \cdot T_s = 0.15 \text{ s} \\
\text{Coefficient to adjust 5\% damped spectral acceleration values to} & \\
\text{2\% damped with } T > T_0 \text{ (ASCE 7-10)} & \\
B_{d,To} := & 0.8 \\
B_{d,Ex} := & 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{Ex_i} \\
B_{d,Ex} = & \begin{bmatrix} 0.817 \\ 0.937 \\ 0.965 \\ 0.975 \\ 0.98 \\ 0.983 \end{bmatrix} \\
B_{d,Im} := & 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{Im_i} \\
B_{d,Im} = & \begin{bmatrix} 0.836 \\ 0.942 \\ 0.967 \\ 0.976 \\ 0.982 \\ 0.986 \end{bmatrix} \\
B_{d,Ri} := & 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{Ri} \\
B_{d,Ri} = & \begin{bmatrix} 0.818 \\ 0.946 \\ 0.971 \\ 0.979 \\ 0.982 \\ 0.985 \end{bmatrix} \\
B_{d,RLi} := & 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{RL_i} \\
B_{d,RLi} = & \begin{bmatrix} 0.838 \\ 0.948 \\ 0.971 \\ 0.979 \\ 0.982 \\ 0.985 \end{bmatrix}
\end{align*}
Existing Building Drift (Soft Story)

define a dummy stiffness matrix that has a soft first story to produce orthogonal mode shapes and calculate acceptable "plastic" mode shapes and other modal properties

\[
k_p = \begin{pmatrix}
  k_{p1} & k_{p2} & 0 & 0 & 0 & 0 \\
  -k_{p2} & k_{p2} + k_{p3} & -k_{p3} & 0 & 0 & 0 \\
  0 & -k_{p3} & k_{p3} + k_{p4} & -k_{p4} & 0 & 0 \\
  0 & 0 & -k_{p4} & k_{p4} + k_{p5} & -k_{p5} & 0 \\
  0 & 0 & 0 & -k_{p5} & k_{p5} + k_{p6} & -k_{p6} \\
  0 & 0 & 0 & 0 & 0 & k_{p6}
\end{pmatrix}
\]

\[
\omega_p := \sqrt{\text{eigenvals}(k_p m^{-1})}, \quad \omega_p = \begin{pmatrix}
  0.038 \\
  15.198 \\
  29.361 \\
  41.523 \\
  50.855 \\
  56.721
\end{pmatrix} \text{ rad/second}
\]

\[
f_p := \frac{\omega_p}{2\pi} = \begin{pmatrix}
  6.033 \times 10^{-3} \\
  2.419 \\
  4.673 \\
  6.609 \\
  8.094 \\
  9.027
\end{pmatrix} \quad \text{1/s}
\]

\[
f_p = \begin{pmatrix}
  6.033 \times 10^{-3} \\
  2.419 \\
  4.673 \\
  6.609 \\
  8.094 \\
  9.027
\end{pmatrix}
\]

\[
T_p := \frac{1}{f_p} = \begin{pmatrix}
  165.763 \\
  0.413 \\
  0.214 \\
  0.151 \\
  0.124 \\
  0.111
\end{pmatrix} \quad T_p = \begin{pmatrix}
  165.763 \\
  0.413 \\
  0.214 \\
  0.151 \\
  0.124 \\
  0.111
\end{pmatrix} \text{ s}
\]
\[ \phi_p := \text{eigenvector}(k_p n_m^{-1}) \]

\[ \phi_p = \begin{pmatrix} -0.408 & 0.558 & -0.5 & 0.408 & 0.289 & -0.149 \\ -0.408 & 0.408 & -3.75 \times 10^{-6} & -0.408 & -0.577 & 0.408 \\ -0.408 & 0.149 & 0.5 & -0.408 & 0.289 & -0.558 \\ -0.408 & -0.149 & 0.5 & 0.408 & 0.289 & 0.558 \\ -0.408 & -0.408 & 1.25 \times 10^{-6} & 0.408 & -0.577 & -0.408 \\ -0.408 & -0.558 & -0.5 & 0.408 & 0.289 & 0.149 \end{pmatrix} \]

\[ W_{p,m} := \sum_{i=1}^{N} \left[ W_{T_i} \left( \phi_p \langle m \rangle \right)_i \right]^2 \]

\[ W_p = \begin{pmatrix} 2.687 \times 10^3 \\ 1.94 \times 10^{-7} \\ 1.12 \times 10^{-8} \\ 1.866 \times 10^{-9} \\ 4.147 \times 10^{-10} \\ 7.18 \times 10^{-11} \end{pmatrix} \]

\[ \Gamma_{p,m} := \frac{W_{Ex,m}}{\sum_{i=1}^{N} \left[ W_{T_i} \left( \phi_p \langle m \rangle \right)_i \right]^2} \]

\[ \Gamma_p = \begin{pmatrix} -1.965 \\ 4.393 \times 10^4 \\ -3.684 \times 10^4 \\ 2.53 \times 10^4 \\ 2.823 \times 10^4 \\ -2.06 \times 10^4 \end{pmatrix} \]

\[ S_{a,i} := \frac{S_a}{B_{d,Ex_i}} \begin{cases} 0.4 + 0.6 \frac{T_{Ex_i}}{T_0} & \text{if } T_{Ex_i} < T_0 \\ S_a & \text{if } T_0 < T_{Ex_i} < T_s \\ \frac{S_1 \sec}{T_{Ex_i} B_{d,To}} & \text{otherwise} \end{cases} \]

\[ S_a = \begin{pmatrix} 1.958 \\ 1.708 \\ 1.533 \\ 1.285 \\ 1.15 \\ 1.058 \end{pmatrix} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,1} = \begin{pmatrix} \frac{W_{p,Ex}}{2 k_{o,ExF}} \left( \frac{S_{a}}{g} \right)^2 & \text{if } T_{Ex} < \frac{S_1 \sec}{S_a} \\ \frac{W_{p,Ex}}{2 k_{o,ExF}} + \frac{1}{2 V_{p,Ex}} k_{o,ExF} & \text{if } T_{Ex} < \frac{S_1 \sec}{S_a} \\ \frac{W_{p,Ex}}{g S_a} & \text{otherwise} \end{pmatrix} \]

\[ n_{PB} = 1 \]

\[ S_{d,1} = 19.164 \text{-in} \]
Higher modes

\[ S_{d_2} := \left( \frac{TP_2}{4\pi^2} \right)^2 S_{a_2} \]
\[ S_{d_3} := \left( \frac{TP_3}{4\pi^2} \right)^2 S_{a_3} \]
\[ S_{d_4} := \left( \frac{TP_4}{4\pi^2} \right)^2 S_{a_4} \]
\[ S_{d_5} := \left( \frac{TP_5}{4\pi^2} \right)^2 S_{a_5} \]
\[ S_{d_6} := \left( \frac{TP_6}{4\pi^2} \right)^2 S_{a_6} \]

\[ S_d = \{ 2.854 \text{ in}, 0.687 \text{ in}, 0.288 \text{ in}, 0.172 \text{ in}, 0.127 \text{ in} \} \]

\[ S_{d, SRSS} := \sqrt{S_{d_1}^2 + S_{d_2}^2 + S_{d_3}^2 + S_{d_4}^2 + S_{d_5}^2 + S_{d_6}^2} \]
\[ S_{d, SRSS} = 19.391 \text{ in} \]
\[ \frac{S_{d_1}}{S_{d, SRSS}} = 0.988 \]

"Plastic" mode shape for 1st soft-story existing building

\[ \phi_{p1} := \phi_p \]

\[ u_{max,e} := \Gamma_{p1} \phi_{p1} S_{d, SRSS} \]
\[ u_{max,e} = \begin{cases} 15.558 \text{ in} & \text{if } i = 1 \\ 15.558 \text{ in} & \text{if } i = 2 \\ 15.558 \text{ in} & \text{if } i = 3 \\ 15.558 \text{ in} & \text{if } i = 4 \\ 15.558 \text{ in} & \text{if } i = 5 \\ 15.558 \text{ in} & \text{if } i = 6 \end{cases} \]

\[ \theta_{existing} = \begin{cases} u_{max,e_i} & \text{if } i = 1 \\ u_{max,e_i} - u_{max,e_{i-1}} & \text{otherwise} \end{cases} \]

\[ \theta_{existing} = \begin{cases} 7.203 \% & \text{if } \max(\theta_{existing}) > 2.0\% \\ 8.311 \times 10^{-5} \% & \text{if } 2.0\% > \max(\theta_{existing}) > 1.5\% \\ 6.649 \times 10^{-5} \% & \text{if } 1.5\% > \max(\theta_{existing}) > 0.5\% \\ 4.987 \times 10^{-5} \% & \text{if } 0.5\% > \max(\theta_{existing}) \end{cases} \]

PerfCat = "Collapse" if \( \theta_{existing} \) > 2.0%

Haz = "MCE"

BFcols = "Original"
Improved Building Drift (Equal Disp and Equal Energy Calcs)

\[
S_{d_{\im_1}} := \begin{cases} 
\frac{S_a}{B_{d_{\im_1}}} (0.4 + 0.6 \frac{T_{\im_1}}{T_0}) & \text{if } T_{\im_1} < T_C \\
\frac{S_s}{B_{d_{\im_1}}} & \text{if } T_0 < T_{\im_1} < T_s \\
\frac{S_s}{T_1 \cdot \sec B_{d_{\im_1}}} & \text{otherwise}
\end{cases}
\]

\[
T_{\im} = \begin{pmatrix} 
0.616 \\
0.218 \\
0.124 \\
0.089 \\
0.068 \\
0.054 \\
1.914 \\
1.699 \\
1.484 \\
1.242 \\
1.097 \\
1.001 
\end{pmatrix}
\]

First mode displacement applying equal energy or equal displacement rule

\[
S_{d_{\im_1}} := \frac{V_{p,\,Rehab}}{2k_{olmF}} \frac{k_{olmF}}{V_{p,\,Rehab}} \left( \frac{W_{\im_1}}{g} \right) \left( \frac{S_{d_{\im_1}}}{S_s} \right)^2 \quad \text{if } T_{\im_1} \leq \frac{S_1}{S_s} \quad S_{d_{\im_1}} = 28.867 \text{ in}
\]

Higher modes

\[
S_{d_{\im_2}} := \frac{\left( T_{\im_2} \right)^2}{4\pi^2} S_{d_{\im_3}} := \frac{\left( T_{\im_3} \right)^2}{4\pi^2} S_{d_{\im_4}} := \frac{\left( T_{\im_4} \right)^2}{4\pi^2} S_{d_{\im_5}} := \frac{\left( T_{\im_5} \right)^2}{4\pi^2} S_{d_{\im_6}} := \frac{\left( T_{\im_6} \right)^2}{4\pi^2} S_{d_{\im_7}} := \frac{\left( T_{\im_7} \right)^2}{4\pi^2} S_{d_{\im_8}} := \frac{\left( T_{\im_8} \right)^2}{4\pi^2}
\]

\[
S_{d_{\im_1}} = 0.787 \text{ in} \\
S_{d_{\im_2}} = 0.224 \text{ in} \\
S_{d_{\im_3}} = 0.058 \text{ in} \\
S_{d_{\im_4}} = 0.029 \text{ in}
\]

\[
S_{d_{\im,SRSS}} := \left( \frac{S_{d_{\im_1}}^2}{S_{d_{\im_1}}^2} \right)^{1/2} + \left( \frac{S_{d_{\im_2}}^2}{S_{d_{\im_2}}^2} \right)^{1/2} + \left( \frac{S_{d_{\im_3}}^2}{S_{d_{\im_3}}^2} \right)^{1/2} + \left( \frac{S_{d_{\im_4}}^2}{S_{d_{\im_4}}^2} \right)^{1/2} + \left( \frac{S_{d_{\im_5}}^2}{S_{d_{\im_5}}^2} \right)^{1/2} + \left( \frac{S_{d_{\im_6}}^2}{S_{d_{\im_6}}^2} \right)^{1/2}
\]

\[
u_{\max_{\im_1}} := \frac{\nu_{\max_{\im_1}}}{S_{d_{\im,SRSS}}} \quad S_{d_{\im,SRSS}} = 28.879 \text{ in} \\
u_{\max_{\im_1}} = 3.366 \\
\theta_{\im_1} := \begin{cases} 
\theta_{\max_{\im_1}} & \text{if } i = 1 \\
\frac{\nu_{\max_{\im_1}} - \nu_{\max_{\im_{i-1}}}}{h_i} & \text{otherwise}
\end{cases}
\]

PerfCat := "Collapse" if max(\theta_{\im}) > 2.0% \\
"CP" if 2.0% > max(\theta_{\im}) > 1.5% \\
"LS" if 1.5% > max(\theta_{\im}) > 0.5% \\
"IO" if 0.5% > max(\theta_{\im}) \\
Haz = "MCE" \\
BFCols = "Original"

A2.2-20
Rehabilitated Building Drift (Equal Disp and Equal Energy Calcs)

First check if added stiffness from SRCRehab needed or pin-pin links can be used

\[ S_{a_1} = \begin{cases} \frac{S_4}{B_{d,R_1}} \left( 0.4 + 0.6 \frac{T_{R_1}}{T_0} \right) & \text{if } T_{R_1} < T_0 \\ \frac{S_s}{B_{d,R_1}} & \text{if } T_0 < T_{R_1} < T_s \\ S_1 \cdot \text{sec} & \text{otherwise} \end{cases} \]

\[ T_R = \begin{cases} 0.683 \frac{T_{R_1}}{T_0} & \text{if } T_{R_1} < T_0 \\ 0.203 & \text{if } T_0 < T_{R_1} < T_s \\ 0.11 & \text{if } T_s < T_{R_1} \end{cases} \]

\[ S_a = \begin{cases} 1.956 \frac{T_{R_1}}{T_0} & \text{if } T_{R_1} < T_0 \\ 1.692 & \text{if } T_0 < T_{R_1} < T_s \\ 1.387 & \text{if } T_s < T_{R_1} \end{cases} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,R_1} = \frac{V_{p,Rehab}}{2 \cdot k_0 \cdot \text{ExF}} + \frac{W_{R_1} \cdot S_{a_1}}{k_0 \cdot \text{ExF}} \]

\[ \text{if } T_{R_1} < \frac{S_1}{S_s} \]

\[ S_{d,R_1} = 29.335 \text{ in} \]

Higher modes

\[ S_{d,R_2} = \frac{\left( T_{R_2} \right)^2 S_{a_2}}{4 \pi^2} \]

\[ S_{d,R_3} = \frac{\left( T_{R_3} \right)^2 S_{a_3}}{4 \pi^2} \]

\[ S_{d,R_4} = \frac{\left( T_{R_4} \right)^2 S_{a_4}}{4 \pi^2} \]

\[ S_{d,R_5} = \frac{\left( T_{R_5} \right)^2 S_{a_5}}{4 \pi^2} \]

\[ S_{d,R_6} = \frac{\left( T_{R_6} \right)^2 S_{a_6}}{4 \pi^2} \]

\[ S_{d,R_{SRSS}} = \sqrt{S_{d,R_1}^2 + S_{d,R_2}^2 + S_{d,R_3}^2 + S_{d,R_4}^2 + S_{d,R_5}^2 + S_{d,R_6}^2} \]

\[ u_{\text{max},R} = \Gamma_{R_1} \cdot \phi_{1, R} \cdot S_{d,R_{SRSS}} \]

\[ S_{d,R_{SRSS}} = 29.343 \text{ in} \]

\[ u_{\text{max},R} = \begin{cases} u_{\text{max},R_1} & \text{if } i = 1 \\ u_{\text{max},R_i} - u_{\text{max},R_{i-1}} & \text{otherwise} \end{cases} \]

\[ \theta_{\text{Rehab}} = \begin{cases} \frac{u_{\text{max},R_1}}{h_i} & \text{if } i = 1 \\ \frac{u_{\text{max},R_i} - u_{\text{max},R_{i-1}}}{h_i} & \text{otherwise} \end{cases} \]

\[ \text{PerfCat} = \begin{cases} \text{"Collapse" if max(}\theta_{\text{Rehab}}\text{) > 2.0\%} \\ \text{"CP" if 2.0\% > max(}\theta_{\text{Rehab}}\text{) > 1.5\%} \\ \text{"LS" if 1.5\% > max(}\theta_{\text{Rehab}}\text{) > 0.5\%} \\ \text{"IO" if 0.5\% > max(}\theta_{\text{Rehab}}\text{)} \end{cases} \]

Haz = "MCE"

BFcols = "Original"
Rehabilitated Building Drift with Links (Equal Disp and Equal Energy Calcs)

\[ S_{d,RL_1} = \begin{cases} \frac{S_a}{B_{d,RL_1}} \left( 0.4 + 0.6 \frac{T_{RL_3}}{T_0} \right) & \text{if } T_{RL_3} < T_0 \\ \frac{S_a}{B_{d,RL_1}} & \text{if } T_0 < T_{RL_3} < T_s \end{cases} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,RL_1} = \frac{V_{p,Rehab,L}}{2k_{Rehab}} + \frac{k_{Rehab}}{2V_{p,Rehab,L}} \left( \frac{W_{RL_1}}{g} \right)^2 \]

if \( T_{RL_1} < \frac{S_a}{S_3} \)

Higher modes

\[ S_{d,RL_2} = \frac{T_{RL_2}^2 S_a}{4\pi^2} \]

\[ S_{d,RL_3} = \frac{T_{RL_3}^2 S_a}{4\pi^2} \]

\[ S_{d,RL_4} = \frac{T_{RL_4}^2 S_a}{4\pi^2} \]

\[ S_{d,RL_5} = \frac{T_{RL_5}^2 S_a}{4\pi^2} \]

\[ S_{d,RL_6} = \frac{T_{RL_6}^2 S_a}{4\pi^2} \]

\[ S_{d,RL,SRSS} = \sqrt{\left( S_{d,RL_1} \right)^2 + \left( S_{d,RL_2} \right)^2 + \left( S_{d,RL_3} \right)^2 + \left( S_{d,RL_4} \right)^2 + \left( S_{d,RL_5} \right)^2 + \left( S_{d,RL_6} \right)^2} \]

\[ u_{\max,RL,i} = \frac{u_{\max,RL,i} \text{ if } i \neq 1}{\text{otherwise}} \]

\[ \theta_{Rehab,L_i} = \frac{u_{\max,RL,i} \text{ if } i \neq 1}{b_i} \]

PerfCat :=

"Collapse" if \( \max(\theta_{Rehab,L}) > 2.0\% \)

"CP" if \( 2.0\% > \max(\theta_{Rehab,L}) > 1.5\% \)

"LS" if \( 1.5\% > \max(\theta_{Rehab,L}) > 0.5\% \)

"IO" if \( 0.5\% > \max(\theta_{Rehab,L}) \)

Haz = "MCE"

BFeols = "Original"
Summary of Critical Values

Existing

\[ T_{Ex} = \begin{pmatrix} 0.685 \\ 0.237 \\ 0.131 \\ 0.096 \\ 0.076 \\ 0.063 \end{pmatrix} \quad \theta_{Ex,MCE} = \begin{pmatrix} 7.203 \\ 8.311 \times 10^{-5} \\ 6.649 \times 10^{-5} \\ 6.649 \times 10^{-5} \\ 4.987 \times 10^{-5} \\ 3.324 \times 10^{-5} \end{pmatrix} \]

\[ k_{o,Ex} = 599.722 \ \text{kip/in} \]

\[ V_{p,Ex} = 1.275 \times 10^3 \ \text{kip} \]

Improved

\[ T_{Im} = \begin{pmatrix} 0.616 \\ 0.218 \\ 0.124 \\ 0.089 \\ 0.068 \\ 0.054 \end{pmatrix} \quad \theta_{Im,MCE} = \begin{pmatrix} 3.052 \\ 4.237 \\ 3.947 \\ 3.681 \\ 3.596 \\ 2.927 \end{pmatrix} \]

\[ k_{o,Im} = 617.777 \ \text{kip/in} \]

\[ V_{p,Im} = 1.067 \times 10^3 \ \text{kip} \]

Existing with SRC, Pin-Pin Links

\[ T_{R} = \begin{pmatrix} 0.683 \\ 0.203 \\ 0.11 \\ 0.081 \\ 0.067 \\ 0.057 \end{pmatrix} \quad \theta_{R,MCE} = \begin{pmatrix} 3.261 \\ 4.247 \\ 4.332 \\ 4.323 \\ 4.461 \\ 4.05 \end{pmatrix} \]

\[ k_{o,R} = 599.722 \ \text{kip/in} \]

\[ V_{p,Rehab} = 1.067 \times 10^3 \ \text{kip} \]

Existing with SRC, Fix-Fix Links

\[ T_{RL} = \begin{pmatrix} 0.607 \\ 0.196 \\ 0.108 \\ 0.08 \\ 0.066 \\ 0.057 \end{pmatrix} \quad \theta_{RL,MCE} = \begin{pmatrix} 0.96 \\ 1.406 \\ 1.483 \\ 1.535 \\ 1.584 \\ 1.43 \end{pmatrix} \]

\[ k_{Rehab} = 685.474 \ \text{kip/in} \]

\[ V_{p,Rehab,L} = 1.363 \times 10^3 \ \text{kip} \]
Design Approach for Links and Reaction Column to Achieve Target Drift

Note: Final selected link and reaction column sections used in stiffness and drift calculations above.

existing frame plastic base shear strength considering soft-story mechanism

\[ V_{p,Ex} = 1.275 \times 10^3 \text{kip} \]

frame plastic base shear strength considering all story mechanism with comp. brace at EPB

\[ V_{p,Rehab} = 1.067 \times 10^3 \text{kip} \]

Predicted story drift with pin-pin SRC

\[ \theta_{Rehab} = \begin{pmatrix} 3.261 \\ 4.247 \\ 4.332 \\ 4.323 \\ 4.461 \\ 4.05 \end{pmatrix} \%

Predicted story drift of existing sub-standard frame

\[ \theta_{existing} = \begin{pmatrix} 7.203 \\ 8.311 \times 10^{-5} \\ 6.649 \times 10^{-5} \\ 4.987 \times 10^{-5} \\ 3.324 \times 10^{-5} \\ 1.662 \times 10^{-5} \end{pmatrix} \%

\[ \frac{k_{Rehab}}{k_{0ExF}} = 1.143 \]

Target interstory drift for rehabilitated building

\[ \theta_t := 2\% \]

Maximum allowable link plastic rotation

\[ \gamma_L := 0.08 \]

Lateral stiffness increase factor -iterated design variable since both lateral strength and stiffness important for calculating drift for frame in equal energy region

\[ \omega_k := 1.125 .. 4 \]
Required frame plastic base shear strength to achieve target rotation (assume equal energy, function of rehabilitated stiffness, see derivation)

\[ V_{pl}(\omega_k) = \frac{2}{3} \omega_k \theta_1 H_N k_oExF - \omega_k k_oExF \sqrt{\frac{4 \theta_1^2 \left(H_N\right)^2}{9} \left( \frac{W_{RL1} S_1}{\delta p_{d,To}} \right)} \]

\[ V_{pl,1}(\omega_k) = \sum \frac{(C_i H_i)}{N(b_{SRC} + \epsilon)} \left(V_{pl}(\omega_k) - V_{p,Rehab}\right) \]

N = 6

<table>
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<tr>
<th>( \omega_k )</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sum (C_i H_i) )</td>
<td>1</td>
<td>1.25</td>
<td>1.5</td>
<td>1.75</td>
<td>2</td>
<td>2.25</td>
<td>2.5</td>
<td>2.75</td>
<td>3</td>
<td>3.25</td>
<td>3.5</td>
<td>3.75</td>
<td>4</td>
</tr>
</tbody>
</table>

Range of solutions for VpL with each frame stiffness factor

Note that only positive, real values are physically possible.

Ideally, the smallest value of VpL is ideal since it reduces the RC, SRC, and foundation forces that need to be capacity designed for however such a small link may not achieve the stiffness needed or the axial capacity of a small link may not be adequate to transfer the interaction force. There, a few iterations trying the smallest VpL and checking the stiffness is needed (see below).

trial stiffness factor \( \omega_{k,t} = 0.75 \)

corresponding VpL with stiffness factor to achieve target drift \( V_{pl,1}(\omega_{k,t}) = 492.233 \text{-kip} \)
**Link Selection and Checks (designed like an EBF shear link)**

maximum axial force
on link from SRC
design calculation incl.
higher mode effects
and plastic interaction forces

\[ A_{\text{Lmax}} := 569.664 \text{kip} \]

Required "squash" load of
link to limit axial demand
to 1/2 of yield strength

\[ P_{y,\text{req}} := 2 \cdot A_{\text{Lmax}} = 1.139 \times 10^3 \text{kip} \]

Selected link section

\[ \text{LinkSection} := \text{"W10x112"} \]
\[ d_{L} := 11.4 \text{in} \]

Link plastic shear strength

\[ V_{pL} := 202 \text{kip} \]
\[ M_{pL} := 7350 \text{kip-in} \]
\[ \Omega_{L} := 1.5 \]

Yield strength of section

\[ P_{y} := 1650 \text{kip} \]

LinkConn := "fixfix"

\[ I_{L} := \begin{cases} 716 \text{in}^4 & \text{if LinkConn = "fixfix"} \\ 4 \text{in}^4 & \text{if LinkConn = "fixpin"} \end{cases} \]
\[ F_{y} := 50 \text{ksi} \]
\[ A_{v} := \frac{V_{pL}}{0.6 \cdot F_{y}} = 6.733 \text{in}^2 \]

Required link length to
limit link rotation

\[ e_{L} := \max \left[ d_{L} \left( \frac{\theta_{L}}{\gamma_{L}} \right) \right] = 48 \text{in} \]

\[ \text{LinkClass} := \begin{cases} \text{"ShortLink"} & \text{if } e_{L} < \frac{1.6 \cdot M_{pL}}{V_{pL}} \\ \text{"Long Link"} & \text{if } e_{L} > \frac{2.6 \cdot M_{pL}}{V_{pL}} \\ \text{"Intermediate Link"} & \text{otherwise} \end{cases} \]

expected link plastic rotation with selected
link length

\[ \gamma_{L} := \left( 1 + \frac{b_{\text{SRC}}}{e_{L}} \right) \cdot \theta_{1} = 0.08 \]
target SRC/link/RC added stiffness

\[ k_{SRC,t} := (\omega_{kt} - 1) \cdot k_0 \cdot ExF = -149.93 \, \text{kip/in} \]

Approximate Stiffness Calculation only including flexibility of RC (use to size RC, more complete stiffness calc below)

\[ k_{SRC} = \frac{3}{2} \frac{E_s \cdot A_{RC} \cdot (b_{SRC} + e_L)^2}{h_1} \cdot \frac{H_N}{\sum_i \left( C_i \cdot H_i \right)} \]

Required RC x-sectional area based on stiffness equation above

\[ A_{RC,req} := 1.5 \cdot \left( \frac{2}{3} \frac{h_1 \cdot k_{SRC,t} \cdot H_N}{E_s \cdot (b_{SRC} + e_L)^2} \right) = -21.818 \, \text{in}^2 \]

OK as an initial design size but sizing for strength and then increasing ARC for stiffness might be better approach

Selected RC member

\[ RC_{section} := "W14\times233" \]

\[ A_{RC} := 68.5 \, \text{in}^2 \]

\[ P_{RC} := \Omega_t \cdot N \cdot V_{at} = 1.818 \times 10^3 \, \text{kip} \]

\[ M_{RC} := \Omega_L \cdot V_{pl} \cdot e_L = 1.212 \times 10^3 \, \text{kip-ft} \]

\[ C_b := 1.67 \]

\[ p := 0.397 \times 10^{-3} \, \text{kip}^{-1} \]

\[ b_x := 0.553 \times 10^{-3} \, \text{(kip-ft)}^{-1} \]

\[ DC := \begin{cases} \frac{p \cdot P_{RC} + \frac{b_x \cdot M_{RC}}{C_b}}{p \cdot P_{RC}} \quad \text{if} \quad p \cdot P_{RC} > 0.2 = 1.123 \\ \frac{1}{2} \frac{p \cdot P_{RC}}{P_{RC}} + \frac{9}{8} \left( \frac{b_x \cdot M_{RC}}{P_{RC}} \right) \quad \text{otherwise} \end{cases} \]

Compare target stiffness factor to calculated ratio of Rehab stiffness to existing

- design iteration converges when these are reasonably close

\[ \frac{k_{Rehab}}{k_0 \cdot ExF} = 1.143 \quad \omega_{kt} = 0.75 \]

Final critical parameters for model

\[ RC_{section} = "W14\times233" \]

\[ Link_{section} = "W10\times112" \]

\[ e_L = 48 \, \text{in} \quad V_{pl} = 202 \, \text{kip} \]

\[ I_{L,eq} := I_L \left( \frac{1}{1 + C_1} \right) = 118.598 \, \text{in}^4 \quad I_L = 716 \, \text{in}^4 \]
A2.3 3NCBF-WB Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift

3NCBF-WB SRC-Rehab Frame Stiffness and Drift Prediction Calculations

OBJECTIVE
The objective of this calculation is to develop an approximate approach for determining the added stiffness provided by the SRC, Links, and Reaction Column. Knowing the rehabilitated lateral stiffness of the frame allows for calculation of the rehabilitated period of vibration which, in turn, allows for prediction of the first mode lateral displacement of the nonlinear system through application of the equal displacement or equal energy rule. Given a target drift criteria for each hazard level, the critical rehabilitation parameters for achieving the necessary frame stiffness and strength can be sized.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration was modified to represent an older braced frame design based on a historic SEAOC seismic design procedure. This 3NCBF frame includes under-designed braced frame beams which are expected to yield in flexure prior to the development of the full expected tensile force in each brace. This version of the prototype frame is referred to as 3NCBF-WB (weak beam).

METHODOLOGY
The derived lateral stiffness of the frame includes the shear and overturning flexibility of the RC, links (including both shear and flexural deformations since they are short and likely shear controlled), and SRC considered as an equivalent moment frame. The existing BF stiffness is added to the SRC/Links/RC stiffness since the two systems act in parallel. The existing BF stiffness includes both shear and overturning deformations.

RESULTS/CONCLUSIONS
- Existing BF stiffness and period match reasonably well with the analytical models.
- The added stiffness of the SRC/Links/RC is very dependent on the SRC width (bSRC), reaction column axial stiffness, and the link stiffness.
- The predicted drift is based on equal energy and equal displacements rules however all cases for the 3NCBF-WB fall within the equal energy range.
- The predicted drifts for the Existing frame (~5%), Rehabilitated without Links (~3%), and Rehabilitated with links (~1%) seems reasonable compared with results of NLTHA.
Input

Earthquake Hazard  Haz := "MCE"

Stories, Floors, Modes  N := 3
  i := 1, 2..N
  m := 1, 2..N

Material Properties  \( E_s := 29000 \text{ksi} \)
  \( \nu_s := 0.30 \)
  \( G_s := \frac{E_s}{2\left(1 + \nu_s\right)} = 1.115 \times 10^4 \text{ksi} \)
  \( F_{yc} := 55 \text{ksi} \)

Braced Frame Geometry

\[
\begin{array}{c|c|c|c}
  h_i & L_{bei} & C_i & H_i := \sum_{j=1}^{i} h_j \\
  \hline
  13\text{ft} & 28.75\text{ft} & 1/6 & 26 \text{ ft} \\
  13\text{ft} & 28.75\text{ft} & 1/3 & 39 \text{ ft} \\
  28.75\text{ft} & 28.75\text{ft} & 1/2 & \\
  \hline
  L := 30\text{ft} & H = \begin{pmatrix} 13 \\ 26 \\ 39 \end{pmatrix} & \\
\end{array}
\]

\[
L_{b1} := \sqrt{\left(\frac{L}{2}\right)^2 + (h_i)^2}
\]

\[
\theta_{b1} := \tan^{-1}\left(\frac{(2h_i)}{L}\right)
\]

\[
L_{b1} = \begin{pmatrix} 19.849 \text{ ft} \\ 19.849 \\ 19.849 \end{pmatrix}
\]

\[
\theta_b = \begin{pmatrix} 40.914 \text{ deg} \\ 40.914 \end{pmatrix}
\]
Mass, Weight, Gravity Loads

\[ m_{f_{i}} = \frac{6.7 \text{kip sec}^2}{\text{in}} \]

\[ W_{f_{i}} = m_{f_{i}} \cdot g \]

\[ P_{D_{i}} = \begin{cases} 82 \text{kip} \\
82 \text{kip} \\
77 \text{kip} \end{cases} \]

\[ \sum_{i=1}^{N} (m_{f_{i}} \cdot g) = 1.94 \times 10^3 \text{kip} \]

\[ W_{f} = \begin{cases} 646.698 \text{kip} \\
646.698 \text{kip} \end{cases} \]

Braced Frame Braces

\[ A_{h_{i}} = \begin{cases} 5.77 \text{in}^2 \\
6.27 \text{in}^2 \\
5.07 \text{in}^2 \end{cases} \]

\[ F_{bET_{i}} = \begin{cases} 594.82 \text{kip} \\
395.63 \text{kip} \\
234.02 \text{kip} \end{cases} \]

\[ F_{bEC_{i}} = \begin{cases} 118.45 \text{kip} \\
118.69 \text{kip} \\
70.2 \text{kip} \end{cases} \]

\[ F_{bEPB_{i}} = \begin{cases} 526.05 \text{kip} \\
772.987 \text{kip} \\
560.336 \text{kip} \end{cases} \]

Existing BF story with braces at "post-buckling" strength

\[ F_{BH_{i}} = \begin{cases} \left( F_{bET_{i}} + F_{bEPB_{i}} \right) \cos \left( \theta_{b_{i}} \right) & \text{if } i = n_{PB} \\
\left( F_{bET_{i}} + F_{bEPB_{i}} \right) \cos \left( \theta_{b_{i}} \right) & \text{otherwise} \end{cases} \]

Braced Frame Beams and Columns

\[ Z_{b_{i}} = 28.2 \text{in}^3 \]

\[ A_{c_{i}} = 28.2 \text{in}^3 \]

\[ Z_{c} = 147 \text{in}^3 \]

\[ M_{pc} = Z_{c} \cdot F_{ye} \]

\[ I_{Q} = 2 \left[ A_{c_{i}} \left( \frac{L}{2} \right)^2 \right] \]

\[ M_{pc} = 673.75 \text{kip-ft} \]

\[ I_{Q} = \begin{cases} 88.125 \text{ ft}^4 \\
88.125 \text{ ft}^4 \end{cases} \]
SRC, Links, Reaction Column

Average area and moment of inertia of SRC columns

\[
\begin{align*}
\Lambda_{SC} &:= 34.25 \text{in}^2 \\
I_{SC} &:= 1347.5 \text{in}^4 \\
\Lambda_{d_i} &:= \\
\Lambda_{SB_i} &:= \\
\begin{array}{c|c}
42.7 \text{in}^2 & 42.7 \text{in}^2 \\
29.1 \text{in}^2 & 29.1 \text{in}^2 \\
23.2 \text{in}^2 & 23.2 \text{in}^2 \\
\end{array}
\end{align*}
\]

\[
\theta_{d_i} := \tan^{-1} \left( \frac{h_i}{b_{SRC}} \right)
\]

\[
\theta_d = \left( \frac{47.291}{47.291} \right) \text{deg}
\]

Reaction Column Properties

\[
\Lambda_{RC} = 42.7 \text{in}^2 \\
I_{RC} = 1710 \text{in}^4
\]

Link Properties (see link design approach below)

Relation between drift angle and link rotation angle

\[
\delta_L = \left( 1 + \frac{b_{SRC}}{2c_L} \right) \theta_d
\]

\[
c_L := 48 \text{in}
\]

\[
I_L := 534 \text{in}^4
\]

\[
\Lambda_v := 6.534 \text{in}^2
\]

\[
V_{pL_o} := 160 \text{kip}
\]

Mode shapes and periods from analysis

Existing

\[
\begin{align*}
\phi_{1Ex} &:= 0.223 \\
\phi_{2Ex} &:= 0.4289 \\
\phi_{3Ex} &:= 0.0452 \\
T_{Ex} &:= 0.5220 \\
\end{align*}
\]

\[
\begin{align*}
\phi_{2Ex} &:= 0.3527 \\
\phi_{3Ex} &:= -0.5359 \\
T_{Ex} &:= 0.1885 \\
\end{align*}
\]

\[
\begin{align*}
\phi_{1Ex} &:= 0.2105 \\
\phi_{2Ex} &:= 0.3652 \\
\phi_{3Ex} &:= -0.5244 \\
T_{Ex} &:= 0.1408 \\
\end{align*}
\]

Existing w/SRC, RC

\[
\begin{align*}
\phi_{1R} &:= 0.4205 \\
\phi_{2R} &:= 0.6122 \\
\phi_{3R} &:= -0.4316 \\
\end{align*}
\]

\[
\begin{align*}
\phi_{1R} &:= 0.2049 \\
\phi_{2R} &:= 0.3610 \\
\phi_{3R} &:= -0.5356 \\
T_{RL} &:= 0.1399 \\
\end{align*}
\]

Existing w/SRC, RC, Links

\[
\begin{align*}
\phi_{1RL} &:= 0.4223 \\
\phi_{2RL} &:= 0.6130 \\
\phi_{3RL} &:= -0.4267 \\
\end{align*}
\]

\[
\begin{align*}
\phi_{1RL} &:= 0.2049 \\
\phi_{2RL} &:= 0.3610 \\
\phi_{3RL} &:= -0.5356 \\
T_{RL} &:= 0.1399 \\
\end{align*}
\]
Plastic Base Shear Capacity of Existing Frame and SRC with Links

-used later in calculation when applying equal energy approximation for inelastic system displacement calcs
-mixed mechanism formulation for determining plastic base shear capacity is considered here

**Max force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.**

\[
F_{BT1} = \min \left( F_{BE1}, \frac{4 \cdot Z_{b1} \cdot F_{yc}}{L_{be} \cdot \sin(\theta_{b1})} \right) \quad F_{BT1} = 577.67 \text{kip} \\
F_{BE1} = \begin{cases} 
"Brace" & \text{if } F_{BT1} = F_{BE1} \\
"Beam" & \text{otherwise}
\end{cases} \\
\text{Mech}_{EB} = \begin{cases} 
"Brace" & \text{} \\
"Brace" & \text{otherwise}
\end{cases}
\]

**Max force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.**

\[
F_{BT1} = \min \left( F_{BE1}, \frac{4 \cdot Z_{b1} \cdot F_{yc}}{L_{be} \cdot \sin(\theta_{b1})} \right) \quad F_{BT1} = 522.52 \text{kip} \\
F_{BE1} = \begin{cases} 
"Brace" & \text{if } F_{BT1} = F_{BE1} \\
"Beam" & \text{otherwise}
\end{cases} \\
\text{Mech}_{EB} = \begin{cases} 
"Beam" & \text{} \\
"Beam" & \text{otherwise}
\end{cases}
\]

**Existing frame controlling plastic mechanism:** considers a panel mechanism and braces at EPB within the nPB story and all other stories elastic

**Brace Internal Work**

\[
W_{IB,Ex} := F_{EPB} \cdot \sin(\theta_{bPB}) \cdot \frac{L}{2} + F_{EPB} \cdot \cos(\theta_{bPB}) \cdot h_{PB}
\]

**Beam Internal Work**

\[
W_{IB,Ex} := 2 \cdot Z_{bPB} \cdot F_{yc}
\]

**Column Internal Work**

\[
W_{IC,Ex} := \begin{cases} 
2M_{pc} & \text{if } n_{PB} = N \\
4M_{pc} & \text{otherwise}
\end{cases}
\]

**External Work**

\[
W_{E,Ex} := h_{PB} \sum_{i=n_{PB}}^{N} C_i
\]

**Internal Work = External Work**

\[
V_{p,Ex} := \frac{W_{IB,Ex} + W_{IB,Ex} + W_{IB,Ex}}{W_{E,Ex}}
\]

**Plastic base shear of frame (existing frame)**

\[
V_{p,Ex} = 678.959 \text{kip}
\]
Rehabilitated Frame controlling plastic mechanism (without Links): considers all braces at expected tensile and expected post-buckling strengths and flexural yielding at base of columns

**Brace Internal Work**

\[ W_{ib.R} := \sum_{i=1}^{N} \left( F_{bEPB_i} \sin\left(\theta_{b_{ij}}\right) \frac{L}{2} + F_{bEPB_i} \cos\left(\theta_{b_{ij}}\right) h_{ij} \right) \]

**Beam Internal Work**

\[ W_{ib.B.R} := \sum_{i=1}^{N} \left( 2Z_{b_i} F_{ye}\right) \]

**Column Internal Work**

\[ W_{ic.R} := 2M_{pc} \]

**External Work**

\[ W_{E,R} := \sum_{i=1}^{N} \left( H_i C_i \right) \]

Internal Work = External Work

\[ V_{p,Rehab} := \frac{W_{ib.R} + W_{ib.B.R} + W_{ic.R}}{W_{E,R}} \]

Plastic base shear of frame (rehabilitated without Links)

\[ V_{p,Rehab} = 635.448 \text{kip} \]

Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected beam pulldown and expected post-buckling strengths, flexural yielding at base of columns, and yielding links with strength \( V_{p,Lo} \) at each floor elevation

**Brace Internal Work**

\[ W_{ib.RL} := \sum_{i=1}^{N} \left( F_{bEPB_i} \sin\left(\theta_{b_{ij}}\right) \frac{L}{2} + F_{bEPB_i} \cos\left(\theta_{b_{ij}}\right) h_{ij} \right) \]

**Beam Internal Work**

\[ W_{ib.B.RL} := \sum_{i=1}^{N} \left( 2Z_{b_i} F_{ye}\right) \]

**Column Internal Work**

\[ W_{ic.RL} := 2M_{pc} \]

**Link Internal Work**

\[ W_{ll.RL} := N V_{p,Lo} \left( h_{SRC} + q_t \right) \]

**External Work**

\[ W_{E,RL} := \sum_{i=1}^{N} \left( H_i C_i \right) \]

Internal Work = External Work

\[ V_{p,Rehab.L} := \frac{W_{ib.RL} + W_{ib.B.RL} + W_{ic.RL} + W_{ll.RL}}{W_{E,RL}} \]

Plastic base shear of frame (rehabilitated with Links)

\[ V_{p,Rehab.L} = 888.635 \text{kip} \]

\[ W_{ib.RL} = 1.1 \]
Existing Building Stiffness

shear stiffness of each story of BF (note 0.65 factor for rigid offsets of braces)

\[ k_{\text{ExF,i}} = \frac{E_s/A_{b_i}}{\sqrt{0.65 \left( \frac{L}{2} \right)^2 + (h_i)^3}} \left( \cos(\theta_{b_i}) \right)^2 \]

inter-story shear force and overturning moment on BF

\[ V_{BF,i} = \sum_{j=1}^{N} C_i \]

\[ M_{BF,i} = \sum_{j=1}^{N} \left( C_i h_j \right) \]

overturning or "flexural" stiffness of each story of BF using moment-area method to calculate deflections under unit lateral forces

\[ k_{\text{ExF,f,i}} = \frac{E_s I_{O_i}}{\left( \frac{1}{2} M_{BF,i} h_i \right)^2} \]

Stiffness of existing braced frame (series spring stiffnesses of shear and flexural frame stiffnesses)

\[ k_{\text{ExF,\Sigma}} = \frac{1}{k_{\text{ExF,s,i}}} + \frac{1}{k_{\text{ExF,f,i}}} \]

Effective first mode height

\[ h_{m\text{EX}} = \frac{\sum_{i=1}^{N} \left( H_i m_{f,i} \phi_{1\text{Ex},i} \right)}{\sum_{i=1}^{N} \left( m_{f,i} \phi_{1\text{Ex},i} \right)} \]

Effective lateral stiffness of BF (derived based on calculating lateral deformation at roof due to unit base shear force)

\[ k_{\text{oEX}} = \frac{f_{h\text{EX}}}{7} \left( \sum_{i=1}^{N} k_{\text{ExF,i}} \right) = 829.324 \text{ kip/in} \]

Effective first mode weight

\[ W_{1\text{Ex}} = \frac{1}{7} \sum_{i=1}^{N} \left[ W_{f,i} \left( \phi_{1\text{Ex},i} \right) \right]^2 \]

Existing building first mode period

\[ T_{\text{Pred.Ex}} = 2\pi \sqrt{\frac{W_{1\text{Ex}}}{g k_{\text{oEX}}}} \]

\[ T_{\text{Pred.Ex}} = 0.459 \text{ s} \]

\[ V_{\text{BF}} = \begin{bmatrix} 1 \\ 0.833 \\ 0.5 \end{bmatrix} \]

\[ M_{\text{BF}} = \begin{bmatrix} 13 \\ 10.833 \text{ ft} \\ 6.5 \end{bmatrix} \]

\[ k_{\text{ExF,s}} = \begin{bmatrix} 1.547 \times 10^3 \\ 1.68 \times 10^3 \\ 1.359 \times 10^3 \end{bmatrix} \text{ kip/in} \]
Existing Period Calculation using 3DOF Matrix (simply for comparison with SDOF approximation presented previously)

\[
K_{\text{ExF}} := \begin{pmatrix}
{k_{\text{ExF}_1} + k_{\text{ExF}_2}} & -k_{\text{ExF}_2} & 0 \\
-k_{\text{ExF}_2} & {k_{\text{ExF}_2} + k_{\text{ExF}_3}} & -k_{\text{ExF}_3} \\
0 & -k_{\text{ExF}_3} & {k_{\text{ExF}_3}}
\end{pmatrix}
\]

\[
m_{\text{m}} := \begin{pmatrix}
1.675 & 0 & 0 \\
0 & 1.675 & 0 \\
0 & 0 & 1.675
\end{pmatrix}
\]

\[
\omega_{\text{Ex}} := \sqrt{\text{eigenvals}(K_{\text{ExF}}m_{\text{m}}^{-1})}
\]

\[
f_{\text{Ex}} := \frac{\omega_{\text{Ex}}}{2\pi} = \frac{2.131}{5.759} \frac{1}{8.629} \text{ s}^{-1}
\]

\[
T_{\text{Ex}2} := \frac{1}{f_{\text{Ex}}} = \begin{pmatrix}
0.469 \\
0.174 \\
0.116
\end{pmatrix}
\]

\[
\text{EigenVec}_{\text{Ex}} := \text{eigenvectors}(K_{\text{ExF}}m_{\text{m}}^{-1})
\]

\[
\phi_{\text{m}} := \text{submatrix}(\text{EigenVec}_{\text{Ex}}, 1, 3, 1)
\]

\[
T_{\text{Ex}2} := 2\pi \sqrt{\frac{\phi_{\text{m}}^T m_{\text{m}} \phi_{\text{m}}}{\phi_{\text{m}}^T K_{\text{ExF}} \phi_{\text{m}}}} = 0.469 \text{ s}
\]

\[
\omega_{\text{Ex}} = \begin{pmatrix}
13.39 \\
36.187
\end{pmatrix} \text{ rad sec}^{-1}
\]

\[
f_{\text{Ex}} = \begin{pmatrix}
2.131 \\
5.759
\end{pmatrix} \text{ s}^{-1}
\]

\[
T_{\text{Ex}2} = \begin{pmatrix}
0.469 \\
0.174 \\
0.116
\end{pmatrix}
\]

\[
\text{EigenVec}_{\text{Ex}} = \begin{pmatrix}
-0.333 & 0.688 & -0.645 \\
-0.578 & 0.391 & 0.716 \\
-0.745 & -0.611 & -0.267
\end{pmatrix}
\]

\[
\phi_{\text{m}} = \begin{pmatrix}
-0.333 \\
-0.578 \\
-0.745
\end{pmatrix}
\]

\[
T_{\text{Ex}2} = 0.469 \text{ s}
\]
Rehabilitated stiffness calculation that includes link and RC flexibility

factor for correction of flexural beam stiffness to include shear flexibility

\[ C_1 = \frac{12 \cdot E_s \cdot I_L}{G_s \cdot A_{v} \cdot e_L^2} \]

\[ C_1 = 1.107 \]

\[ A_{eq} = \frac{\sum_{i} h_i}{\sqrt{b_{SRC}^2 + (h_i)^2}} + \frac{b_{SRC}}{A_{SB_1}} + \frac{2 \cdot (h_i)^3}{3 \cdot A_{SC} \cdot b_{SRC}^2} \]

\[ A_{eq} = \frac{739.129}{57.612 \text{ in}^2} \]

\[ b_{effSRC} = \frac{1}{2} b_{SRC} \]

\[ \lambda_{NA_i} = \frac{\lambda_{RC}(b_{effSRC} + e_L)}{\lambda_{RC} + A_{eq_i}} \]

\[ \lambda_{NA} = \frac{42.059}{51.081 \text{ in}} \]

\[ l_{of_i} = A_{eq} (\lambda_{NA_i})^2 + A_{RC} (e_L + b_{effSRC} - \lambda_{NA_i})^2 \]

\[ l_{of} = \frac{3.994 \times 10^5}{3.531 \times 10^5} \text{ in}^4 \]

\[ \Delta_B = 6.27 \frac{(h_i)^3}{E_s \cdot l_{of_1}} = 2.055 \times 10^{-3} \text{ in/kip} \]

\[ k_{s1} = \frac{3 \cdot E_s \cdot I_{RC} (h_i)^3}{(h_i)^3} + \frac{3 \cdot E_s \cdot I_{SC}}{(h_i)^3} + \frac{E_s \cdot A_{d_1}}{\sqrt{b_{SRC}^2 + (h_i)^2}} \left( \cos \left( \frac{\theta_{d_1}}{2} \right) \right)^2 \]

\[ k_{s1} = 2.753 \times 10^3 \text{ kip/in} \]

\[ k_{s2} = \frac{12 \cdot E_s \cdot I_{RC}(h_i)^3}{(h_i)^3} + \frac{12 \cdot E_s \cdot I_{SC}}{(h_i)^3} + \frac{E_s \cdot A_{d_2}}{\sqrt{b_{SRC}^2 + (h_i)^2}} \left( \cos \left( \frac{\theta_{d_2}}{2} \right) \right)^2 \]

\[ k_{s2} = 2.109 \times 10^3 \text{ kip/in} \]

\[ k_{s3} = \frac{12 \cdot E_s \cdot I_{RC}(h_i)^3}{(h_i)^3} + \frac{12 \cdot E_s \cdot I_{SC}}{(h_i)^3} + \frac{E_s \cdot A_{d_3}}{\sqrt{b_{SRC}^2 + (h_i)^2}} \left( \cos \left( \frac{\theta_{d_3}}{2} \right) \right)^2 \]

\[ k_{s3} = 1.738 \times 10^3 \text{ kip/in} \]

\[ \Delta_S = \frac{1}{k_{s1}} + \frac{5}{k_{s2}^3} + \frac{3}{k_{s3}} + \frac{\sum_{i=1}^{N} \left( C_i \cdot H_i \right)}{N \cdot b_{SRC} \cdot e_L^3} \left( \frac{1}{1 + C_1} \right) \]

\[ \Delta_S = 4.479 \times 10^{-3} \text{ in/kip} \]
Effective first mode height \( h_{mRL} \) = \[ \frac{\sum_{i=1}^{N} \left( m_{i} \cdot \phi_{1RL,i} \right)}{\sum_{i=1}^{N} \left( m_{i} \cdot \phi_{1RL,i} \right)} \]

Effective first mode weight \( w_{1RL} \) = \[ \frac{\sum_{i=1}^{N} \left[ w_{f,i} \cdot \left( \phi_{1RL,i} \right) \right]}{\sum_{i=1}^{N} \left[ w_{f,i} \cdot \left( \phi_{1RL,i} \right)^{2} \right]} \]

\( f_{hRL} = \frac{H_{3}}{h_{mRL}} \) = 1.288

\( w_{1RL} = 1.669 \times 10^{3}\text{-kip} \)

\( \frac{\Delta_{B}}{\Delta_{B} + \Delta_{S}} = 0.315 \)

\( \frac{\Delta_{S}}{\Delta_{B} + \Delta_{S}} = 0.685 \)

\( k_{SRC} = \frac{f_{hRL}}{\Delta_{B} + \Delta_{S}} \)

\( k_{SRC} = 197.119 \text{ kip/in} \)

\( k_{Rehab} = k_{oExF} + k_{SRC} \)

\( k_{Rehab} = 1.026 \times 10^{3}\text{ kip/in} \)

\( T_{Pred.Rehab} = 2\pi \sqrt{\frac{w_{1RL}}{g \cdot k_{Rehab}}} \)

\( T_{Pred.Rehab} = 0.408 \text{ s} \)
Effective Modal weights

\[
\begin{align*}
W_{E1} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{1Ei})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{1Ei})^2} \right]^2 \\
W_{R1} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{1RI})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{1RI})^2} \right]^2 \\
W_{RL1} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{1RLi})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{1RLi})^2} \right]^2 \\
W_{E2} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{2Ei})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{2Ei})^2} \right]^2 \\
W_{R2} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{2RI})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{2RI})^2} \right]^2 \\
W_{RL2} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{2RLi})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{2RLi})^2} \right]^2 \\
W_{E3} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{3Ei})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{3Ei})^2} \right]^2 \\
W_{R3} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{3RI})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{3RI})^2} \right]^2 \\
W_{RL3} &= \sum_{i=1}^{N} \left[ \frac{w_{f_i}(\phi_{3RLi})^2}{\sum_{i=1}^{N} w_{f_i}(\phi_{3RLi})^2} \right]^2
\end{align*}
\]

\[
\begin{align*}
W_{E1} &= 1.706 \times 10^3 \text{kip} \\
W_{E2} &= 201.001 \text{kip} \\
W_{E3} &= 33.123 \text{kip} \\
W_{R1} &= 1.669 \times 10^3 \text{kip} \\
W_{R2} &= 228.779 \text{kip} \\
W_{R3} &= 33.964 \text{kip} \\
W_{RL1} &= 1.669 \times 10^3 \text{kip} \\
W_{RL2} &= 236.34 \text{kip} \\
W_{RL3} &= 34.975 \text{kip}
\end{align*}
\]

\[
\begin{align*}
\sum_{i} w_{f_i} &= 0.879 \\
\sum_{i} w_{f_i} &= 0.86 \\
\sum_{i} w_{f_i} &= 0.86 \\
\sum_{i} w_{f_i} &= 0.104 \\
\sum_{i} w_{f_i} &= 0.118 \\
\sum_{i} w_{f_i} &= 0.122 \\
\sum_{i} w_{f_i} &= 0.017 \\
\sum_{i} w_{f_i} &= 0.018 \\
\sum_{i} w_{f_i} &= 0.018
\end{align*}
\]
Modal Participation Factors

\[ \Gamma_{Ex_1} := \frac{W_{Ex_1}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{1Ex})_i]} \quad \Gamma_{R_1} := \frac{W_{R_1}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{1R})_i]} \quad \Gamma_{RL_1} := \frac{W_{RL_1}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{1RL})_i]} \quad \Gamma_{Ex_1} = 2.104 \]

\[ \Gamma_{Ex_2} := \frac{W_{Ex_2}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{2Ex})_i]} \quad \Gamma_{R_2} := \frac{W_{R_2}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{2R})_i]} \quad \Gamma_{RL_2} := \frac{W_{RL_2}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{2RL})_i]} \quad \Gamma_{Ex_2} = 0.722 \]

\[ \Gamma_{Ex_3} := \frac{W_{Ex_3}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{3Ex})_i]} \quad \Gamma_{R_3} := \frac{W_{R_3}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{3R})_i]} \quad \Gamma_{RL_3} := \frac{W_{RL_3}}{\sum_{i=1}^{N} [Wf_i \cdot (\Phi_{3RL})_i]} \quad \Gamma_{Ex_3} = 0.293 \]

Spectral acceleration values

\[ S_1 := \begin{cases} 1.2g & \text{if Haz = "MCE"} \\ 0.68g & \text{if Haz = "DBE"} \end{cases} \quad S_1 = 1.2g \]

\[ S_S := \begin{cases} 1.6g & \text{if Haz = "MCE"} \\ 1.1g & \text{if Haz = "DBE"} \end{cases} \quad S_S = 1.6g \]

\[ T_s := \frac{S_1}{S_S} = 0.75s \]

\[ T_0 := 0.2 \cdot T_s = 0.15s \]

Coefficient to adjust 5% damped spectral acceleration values to 2% damped with \( T > T_0 \) (ASCE 7-10)

\[ B_{d,To} := 0.8 \]

\[ B_{d,Ex_i} := 1.0 - \frac{1.0 - B_{d,To} \cdot T_{Ex_i}}{T_s} \quad B_{d,Ex} = \begin{pmatrix} 0.88 \\ 0.955 \\ 0.972 \end{pmatrix} \]

\[ B_{d,R_i} := 1.0 - \frac{1.0 - B_{d,To} \cdot T_{R_i}}{T_s} \quad B_{d,R} = \begin{pmatrix} 0.881 \\ 0.962 \\ 0.975 \end{pmatrix} \]

\[ B_{d,RL_i} := 1.0 - \frac{1.0 - B_{d,To} \cdot T_{RL_i}}{T_s} \quad B_{d,RL} = \begin{pmatrix} 0.896 \\ 0.963 \\ 0.975 \end{pmatrix} \]
Existing Building Drift (Soft Story)

\[ \text{SiteClass := "B"} \]
\[ T_{Ex} = \begin{bmatrix} 0.45 \\ 0.17 \\ 0.11 \end{bmatrix} \]
\[ V_{p,Ex} = 678.959 \text{-kip} \]

Define a dummy stiffness matrix that has a soft first story to produce orthogonal modal shapes and calculate acceptable "plastic" mode shapes and other modal properties:

\[ k_p = \begin{bmatrix} k_{p,1} + k_{p,2} & -k_{p,2} & 0 \\ -k_{p,2} & k_{p,2} + k_{p,3} & -k_{p,3} \\ 0 & -k_{p,3} & k_{p,3} \end{bmatrix} \]
\[ k_p = \begin{bmatrix} 1 \times 10^3 & -1 \times 10^3 & 0 \\ -1 \times 10^3 & 2 \times 10^3 & -1 \times 10^3 \\ 0 & -1 \times 10^3 & 1 \times 10^3 \end{bmatrix} \text{kip} \]
\[ \omega_p := \sqrt{\text{eigenvals}(k_p)} \]
\[ \omega_p = \begin{bmatrix} 0.045 \\ 24.434 \end{bmatrix} \text{rad/sec} \]
\[ \omega_p = \begin{bmatrix} \frac{0.045}{42.321} \\ \frac{24.434}{42.321} \end{bmatrix} \]

\[ f_p \text{ := } \frac{\omega_p}{2 \pi} = \begin{bmatrix} 7.1 \times 10^{-3} \\ 3.889 \end{bmatrix} \text{rad/sec} \]
\[ f_p = \begin{bmatrix} 7.1 \times 10^{-3} \\ 3.889 \end{bmatrix} \text{Hz} \]
\[ T_p := \frac{1}{f_p} = \begin{bmatrix} 140.847 \\ 0.257 \end{bmatrix} \text{s} \]
\[ T_p = \begin{bmatrix} 140.847 \\ 0.257 \end{bmatrix} \text{s} \]

\[ \phi_p := \text{eigvecs}(k_p) \]
\[ \phi_p = \begin{bmatrix} -0.577 \\ 0.707 \\ -0.408 \end{bmatrix} \]

\[ T_p = 2 \pi \sqrt{\phi_p^T \cdot \delta_m \cdot \phi_p} = \ldots \]
\[ T_p = \begin{bmatrix} 140.847 \\ 0.257 \end{bmatrix} \text{s} \]
\[ W_p = \left( \begin{array}{c} 1.94 \times 10^3 \\ 3.233 \times 10^{-8} \\ 1.198 \times 10^{-9} \end{array} \right) \text{kip} \]
\[ \sum_{i=1}^{N} \frac{W_{f_i}}{\left( \phi_{p_i}^{(m)} \right)^2} \]
\[ \Gamma_{p_m} := \frac{\sum_{i=1}^{N} \frac{W_{f_i}}{\left( \phi_{p_i}^{(m)} \right)^2}}{\sum_{i=1}^{N} \frac{W_{f_i}}{\left( \phi_{p_i}^{(m)} \right)^2}} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d_1} := \begin{cases} \frac{V_{p,Ex}}{2k_{0,ExF}} \left( \frac{W_{p_1}}{g} \frac{S_{a_1}}{k_{0,ExF}} \right)^2 & \text{if } T_{Ex1} < T_c \left( -1.523 \right) \\ \frac{V_{p,Ex}}{2k_{0,ExF}} \left( \frac{k_{0,ExF}}{k_{0,ExF}} \right) & \text{if } T_0 < T_{Ex1} < T_s \left( 1.818 \right) \\ \frac{W_{p_1}}{g} \frac{S_{a_1}}{k_{0,ExF}} & \text{if } T_{Ex1} \text{ otherwise} \left( 1.357 \right) \end{cases} \]

Higher modes

\[ S_{d_2} := \frac{\left( T_{p_2} \right)^2 S_{a_2}}{4\pi^2} \quad S_{d_3} := \frac{\left( T_{p_3} \right)^2 S_{a_3}}{4\pi^2} \]

\[ S_{d,SRSS} := \sqrt{\left( S_{d_1} \right)^2 + \left( S_{d_2} \right)^2 + \left( S_{d_3} \right)^2} \]

\[ S_{d,SRSS} = 11.512 \text{ in} \quad \frac{S_{d_1}}{S_{d,SRSS}} = 0.995 \]
"Plastic" mode shape for 1st soft-story existing building

\[ \phi_{p1} \triangleq \phi_p^{(1)} \]

\[ u_{\text{max.e}} := \Gamma_{p1} \cdot \phi_{p1} \cdot S_{d, \text{SRSS}} \]

\[ u_{\text{max.e}} = \begin{cases} 10.122 \text{ in} & \text{otherwise} \\ 10.122 \text{ in} & \text{if } i = 1 \end{cases} \]

\[ \theta_{\text{existing}} = \begin{cases} 6.488 \% & \text{otherwise} \\ 4.326 \times 10^{-5} \% & \text{if } i = 1 \end{cases} \]

Haz = "MCE"

PerfCat :=

- "Collapse" if \( \max(\theta_{\text{existing}}) > 2.0\% \)
- "CP" if \( 2.0\% > \max(\theta_{\text{existing}}) > 1.5\% \)
- "LS" if \( 1.5\% > \max(\theta_{\text{existing}}) > 0.5\% \)
- "IO" if \( 0.5\% > \max(\theta_{\text{existing}}) \)
Rehabilitated Building Drift (Equal Disp and Equal Energy Calcs)

First check if added stiffness from SRCRehab needed or pin-pin links can be used

\[ S_{d,1} := \begin{cases} \frac{S_e}{B_{d,R_1}} \left( \frac{T_{R_1}}{T_0} \right)^{0.4 + 0.6 \frac{T_{R_1}}{T_0}} & \text{if } T_{R_1} < T_0 \quad T_R = 0.448 \quad 0.141 \text{ s} \\ \frac{S_e}{B_{d,R_1}} & \text{if } T_0 < T_{R_1} < T_s \\ S_1 \text{ sec} & \text{otherwise} \end{cases} \]

\[ S_{d,1} = \begin{cases} 1.817 & \text{if } T_{R_1} < T_0 \\ 1.601 & \text{if } T_0 < T_{R_1} < T_s \\ 1.284 & \text{otherwise} \end{cases} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,R_1} := \begin{cases} V_{p,Rehab} + \frac{k_{oExF}}{2V_{p,Rehab}} & \text{if } T_{R_1} < \frac{S_1}{S_s} \\ \sqrt{\frac{W_{R_1}}{g S_{a_1}}} & \text{if } T_{R_1} < \frac{S_1}{S_s} \\ \sqrt{\frac{W_{R_1}}{g S_{a_1}}} & \text{otherwise} \end{cases} \]

\[ S_{d,R_1} = (17.825) \text{ in} \]

Higher modes

\[ S_{d,R_2} := \frac{(T_{R_2})^2 S_{a_2}}{4\pi^2} \quad S_{d,R_3} := \frac{(T_{R_3})^2 S_{a_3}}{4\pi^2} \]

\[ S_{d,R_2} = 0.31 \text{ in} \quad S_{d,R_3} = 0.115 \text{ in} \]

\[ S_{d,R,SRSS} := \sqrt{S_{d,R_1}^2 + S_{d,R_2}^2 + S_{d,R_3}^2} \]

\[ S_{d,R,SRSS} = 17.828 \text{ in} \quad \frac{S_{d,R_1}}{S_{d,R,SRSS}} = 1 \]

\[ u_{\max,R} := \frac{T_{R_1}}{\Phi_1 R} S_{d,R,SRSS} \]

\[ u_{\max,R} = \begin{cases} 7.789 \text{ in} & \text{if } i = 1 \\ 15.559 \text{ in} & \text{otherwise} \end{cases} \]

\[ \theta_{Rehab} := \begin{cases} u_{\max,R_i} & \text{if } i = 1 \\ \frac{u_{\max,R_i} - u_{\max,R_{i-1}}}{h_i} & \text{otherwise} \end{cases} \]

\[ \theta_{Rehab} = \begin{cases} 4.993 \text{ %} & \text{if } \text{PerfCat} = "Collapse" \\ 4.981 \text{ %} & \text{otherwise} \end{cases} \]

PerfCat := "Collapse" if \( \max(\theta_{Rehab}) > 2.0\% \)

"CP" if \( 2.0\% > \max(\theta_{Rehab}) > 1.5\% \)

"LS" if \( 1.5\% > \max(\theta_{Rehab}) > 0.5\% \)

"IO" if \( 0.5\% > \max(\theta_{Rehab}) \)

Haz = "MCE"
Rehabilitated Building Drift with Links (Equal Disp and Equal Energy Calcs)

\[ S_{d,RL} := \frac{S_s}{B_{d,RL}} \left( 0.4 + 0.6 \frac{T_{RL}}{T_0} \right) \text{ if } T_{RL} < T_0 \]
\[ = \frac{S_s}{B_{d,RL}} \text{ if } T_0 < T_{RL} < T_s \]
\[ = \frac{S_s}{B_{d,RL}} \text{ otherwise} \]

First mode displacement

\[
S_{d,RL_1} := \frac{V_{p,Rehab,L}}{2k_{Rehab}} + k_{Rehab} \left( \frac{W_{RL_1}^2 \cdot S_{a_1}}{g} \right) \text{ if } T_{RL_1} < \frac{S_1}{S_s} \]
\[ = \frac{W_{RL_1} \cdot S_{a_1}}{g} \text{ otherwise} \]

Higher modes

\[ S_{d,RL_2} := \frac{T_{RL_2} \cdot S_{a_2}}{4\pi^2}, \quad S_{d,RL_3} := \frac{T_{RL_3} \cdot S_{a_3}}{4\pi^2} \]
\[ S_{d,RL_1} = 0.305 \text{ in}, \quad S_{d,RL_2} = 0.113 \text{ in} \]
\[ S_{d,RL_{SRSS}} = 5.312 \text{ in} \]
\[ u_{\text{max},R.L.} := \gamma_{RL_1} \cdot \frac{S_{d,RL_{SRSS}}}{S_{d,RL_1}} \]
\[ u_{\text{max},R.L.} = \frac{2.264}{6.775} \cdot \frac{S_{d,RL_1}}{S_{d,RL_{SRSS}}} = 0.998 \]

\[ \theta_{Rehab.L_1} := \frac{u_{\text{max},R.L.}}{h_i} \text{ if } i = 1 \]
\[ \theta_{Rehab.L_1} := \frac{u_{\text{max},R.L.} - u_{\text{max},R.L.} - 1}{h_i} \text{ otherwise} \]

PerfCat := "Collapse" if \( \max(\theta_{Rehab.L}) > 2.0\% \)
"CP" if \( 2.0\% > \max(\theta_{Rehab.L}) > 1.5\% \)
"LS" if \( 1.5\% > \max(\theta_{Rehab.L}) > 0.5\% \)
"IO" if \( 0.5\% > \max(\theta_{Rehab.L}) \)

Haz = "MCE"
Summary of Critical Values

Existing

\[
T_{Ex} = \begin{bmatrix} 0.45 \\ 0.169 \\ 0.106 \end{bmatrix} \quad k_{ExF} = 829.324 \frac{kip}{in} \quad V_{p,Ex} = 678.959 \text{kip} \quad \theta_{Ex,MCE} = \begin{bmatrix} 6.488 \\ 4.326 \times 10^{-5} \end{bmatrix} \% \\
2.163 \times 10^{-5} \%
\]

With SRC, Pin-Pin Links

\[
T_{R} = \begin{bmatrix} 0.448 \\ 0.141 \\ 0.096 \end{bmatrix} \quad k_{RF} = 829.324 \frac{kip}{in} \quad V_{p,Rehab} = 635.448 \text{kip} \quad \theta_{R,MCE} = \begin{bmatrix} 4.993 \\ 4.981 \% \end{bmatrix} \\
4.547 \%
\]

With SRC, Fix-Fix Links

\[
T_{RL} = \begin{bmatrix} 0.391 \\ 0.141 \\ 0.095 \end{bmatrix} \quad k_{Rehab} = 1.026 \times 10^{3} \frac{kip}{in} \quad V_{p,Rehab,L} = 888.635 \text{kip} \quad \theta_{RL,MCE} = \begin{bmatrix} 1.452 \\ 1.54 \% \end{bmatrix} \\
1.351 \%
\]
Design Approach for Links and Reaction Column to Achieve Target Drift

Note: Final selected link and reaction column sections used in stiffness and drift calculations above.

existing frame plastic base shear strength considering soft-story mechanism

$$V_{p,Ex} = 678.959 \text{-kip}$$

frame plastic base shear strength considering all story mechanism with comp. brace at EPB, Rehabilitated w/ pin-pin connected SRC

$$V_{p,Rehab} = 635.448 \text{-kip}$$

Predicted story drift of existing sub-standard frame

$$\theta_{\text{existing}} = \left[\begin{array}{c}
6.488 \\
4.326 \times 10^{-5} \\
2.163 \times 10^{-5}
\end{array}\right] \text{\%}$$

Predicted story drift with pin-pin SRC

$$\theta_{\text{Rehab}} = \left[\begin{array}{c}
4.993 \\
4.981 \cdot \% \\
4.547
\end{array}\right]$$

Target interstory drift for rehabilitated building

$$\theta_t := 2.0\%$$

Maximum allowable link plastic rotation

$$\gamma_L := 0.08$$

Lateral stiffness increase factor -iterated design variable since both lateral strength and stiffness important for calculating drift for frame in equal energy region

$$\omega_k := 1, 1.25, 1.5$$
Required frame plastic base shear strength to achieve target rotation (assumed equal energy function, see derivation)

$$V_{p1}(\omega_k) = \sqrt{\frac{2}{3} \omega_k \theta_t H_N k_{oExF} - \omega_k k_{oExF} \left( \frac{4 \cdot \theta_t^2}{9} \frac{H_N}{g} \frac{S_8}{B_{d,T0}} \right)^2}$$

Required plastic shear strength of each link

$$V_{pL1}(\omega_k) = \frac{1}{N \left(b_{SRC} + c_L\right)} \left(V_{p1}(\omega_k) - V_{pR,Rehab}\right)$$

For design of the link, the existing all story mechanism frame strength with braces at EPB are conservatively assumed. The brace strength is likely dependent on the magnitude of the target drift.

$$N = 3$$

$$b_{SRC} = 12 \text{ ft}$$

$$\sum_i \left(C_i H_i\right) = 30.333 \text{ ft}$$

$$\omega_k = \begin{bmatrix}
1 \\
1.25 \\
1.5 \\
1.75 \\
2 \\
2.25 \\
2.5 \\
2.75 \\
3 \\
3.25 \\
3.5 \\
3.75 \\
4 \\
4.25 \\
4.5 \\
\vdots \\
1 \\
2 \\
3 \\
4 \\
5 \\
6 \\
7 \\
8 \\
9 \\
10 \\
11 \\
12 \\
13 \\
14 \\
15 \\
16 \\
17 \\
18 \\
19 \\
20 \\
\end{bmatrix}$$

$$\begin{array}{c|c}
\text{V}_{pL1}(\omega_k) & \text{kips} \\
1 & 369.358 \\
2 & 184.489 \\
3 & 74.949 \\
4 & 1.211 \\
5 & -52.208 \\
6 & -92.843 \\
7 & -124.862 \\
8 & -150.777 \\
9 & -172.2 \\
10 & -190.218 \\
11 & -205.588 \\
12 & -218.858 \\
13 & -230.434 \\
14 & -240.622 \\
15 & -249.66 \\
16 & -257.732 \\
17 & -264.986 \\
\end{array}$$

Range of solutions for VpL with each frame stiffness factor

Note that only positive, real values are physically possible.

Ideally, the smallest value of VpL is ideal since it reduces the RC, SRC, and foundation forces that need to be capacity designed for however such a small link may not achieve the stiffness needed or the axial capacity of a small link may not be adequate to transfer the interaction force. There, a few iterations trying the smallest VpL and checking the stiffness is needed (see below).

trial stiffness factor

$$\omega_{k,t} = 1.5$$

corresponding VpL with stiffness factor to achieve target drift

$$V_{pL1}(\omega_{k,t}) = 74.949 \text{kips}$$

A2.3-20
Link Selection and Checks (designed like an EBF shear link)

maximum axial force on link from SRC
design calculation incl. higher mode effects and plastic interaction forces

\[ A_{L,\text{max}} = 587.346 \text{kip} \]

Required "squash" load of link to limit axial demand to 1/2 of yield strength

\[ P_{y,\text{req}} = 2 \cdot A_{L,\text{max}} = 1.175 \times 10^3 \text{kip} \]

Selected link section

\[ \text{LinkSection} := "W10x88" \]

\[ d_L := 10.8 \text{in} \]

Link plastic shear strength

\[ V_{pL} := 160 \text{kip} \]

\[ M_{pL} := 5650 \text{kip-in} \]

\[ \Omega_L := 1.5 \]

Yield strength of section

\[ P_y := 1300 \text{kip} \]

\[ \text{LinkConn} := "fixfix" \]

DC\_LinkShear := \frac{V_{pL} \left( \frac{\omega_{SL}}{V_{pL}} \right)}{V_{pL}} = 0.468

DC\_LinkAxial := \frac{P_{y,\text{req}}}{P_y} = 0.904

adjust link IXX since stiffness calculation below derived based on fix-fix links

\[ I_L := \begin{cases} 534 \text{in}^4 & \text{if LinkConn = "fixfix"} \\ 163 & \text{if LinkConn = "fixpin"} \end{cases} = 534 \text{in}^4 \]

\[ F_y := 50 \text{ksi} \]

\[ A_v := \frac{V_{pL}}{0.6 \cdot F_y} = 5.333 \text{in}^2 \]

Required link length to limit link rotation

\[ \varepsilon_L := \max \left[ d_L \cdot \frac{b_{SR}}{\left( \gamma_L - \theta_t \right)} \right] = 48 \text{in} \]

\[ \text{LinkClass} := \begin{cases} "\text{ShortLink}" & \text{if } \varepsilon_L \leq \frac{1.6 \cdot M_{pL}}{V_{pL}} \\ "\text{Long Link}" & \text{if } \varepsilon_L > \frac{2.6 \cdot M_{pL}}{V_{pL}} \end{cases} \]

\[ \text{Intermediate Link} \text{ otherwise} \]

\[ \gamma_L := \left( 1 + \frac{b_{SR}}{\varepsilon_L} \right) \theta_t = 0.08 \]
target SRC/link/RC added stiffness

Approximate Stiffness Calculation only including flexibility of RC (use to size RC, more complete stiffness calc below)

Required RC x-sectional area based on stiffness equation above -increase required by factor of 1.5 since sizing based on stiffness calc ignoring other flexibilities

Selected RC member

\[
k_{SRC,t} := \left(\omega_{k,t} - 1\right)k_{0ExF} = 414.662 \text{ kip in} \]

\[
k_{SRC} = \frac{3}{2} \left(\frac{E_{2}^\prime A_{RC}}{h_{1}}\right)\left(b_{SRC} + e_{L}\right)^{2}
\]

\[
A_{RC,req} = 1.5 \left(\frac{b_{1} k_{SRC,t} H_{N} \left[\sum_{1}^{N} (C_{i} H_{i})\right]}{E_{2}^\prime \left(b_{SRC} + e_{L}\right)^{2}}\right) = 10.308 \text{ in}^{2}
\]

OK as an initial design size but sizing for strength and then increasing ARC for stiffness might be better approach

RCSection := "W14x145"

\[A_{RC} = 42.7 \text{ in}^{2}\]

\[P_{RC} := \Omega_{L,N} V_{PL} = 720 \text{ kip}\]

\[M_{RC} := \Omega_{L,N} V_{PL} e_{L} = 960 \text{ kip-ft}\]

\[C_{b} = 1.67\]

\[p = 0.582 \times 10^{-3} \text{ kip}^{-1}\]

\[b_{x} = 0.912 \times 10^{-3} \text{ (kip-ft)}^{-1}\]

\[DC := \left\{\begin{array}{ll}
\left[p |P_{RC}| + \frac{b_{x} |M_{RC}|}{C_{b}}\right] & \text{if } p |P_{RC}| > 0.2 = 0.943 \\
\left[\frac{1}{2} p |P_{RC}| + \frac{9}{8} (b_{x} |M_{RC}|)\right] & \text{otherwise}
\end{array}\right.\]

Compare target stiffness factor to calculated ratio of Rehab stiffness to existing -design iteration converges when these are reasonably close

Final critical parameters for model

RCSection = "W14x145"

\[\omega_{k,t} = 1.5\]

\[\frac{k_{Rehab}}{k_{0ExF}} = 1.238\]

\[I_{L_{eq}} := I_{L} \left(\frac{1}{1 + C_{1}}\right) = 253.476 \text{ in}^{4}\]

LinkSection = "W10x88"

\[e_{L} = 48 \text{ in} \quad V_{PL} = 160 \text{ kip}\]

\[I_{L} = 534 \text{ in}^{4}\]

A2.3-22
A2.4 3NCBF-BC Drift Predictions and SRC, Link, and Reaction Column Designs to Achieve Target Drift

3NCBF-BC SRC-Rehab Frame Stiffness and Drift Prediction Calculations

OBJECTIVE
The objective of this calculation is to develop an approximate approach for determining the added stiffness provided by the SRC, Links, and Reaction Column. Knowing the rehabilitated lateral stiffness of the frame allows for calculation of the rehabilitated period of vibration which, in turn, allows for prediction of the first mode lateral displacement of the nonlinear system through application of the equal displacement or equal energy rule. Given a target drift criteria for each hazard level, the critical rehabilitation parameters for achieving the necessary frame stiffness and strength can be sized.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3NCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration was modified to represent an older braced frame design based on a historic SEAOC seismic design procedure. This 3NCBF frame includes under-designed braced frame brace-to-gusset welds which are expected to fracture prior to the development of the full expected tensile force in each brace. This version of the prototype frame is referred to as 3NCBF-BC (brittle connection).

METHODOLOGY
The derived lateral stiffness of the frame includes the shear and overturning flexibility of the RC, links (including both shear and flexural deformations since they are short and likely shear controlled), and SRC considered as an equivalent moment frame. The existing BF stiffness is added to the SRC/Links/RC stiffness since the two systems act in parallel. The existing BF stiffness includes both shear and overturning deformations.

RESULTS/CONCLUSIONS
- Existing BF stiffness and period match reasonably well with the analytical models.
- The added stiffness of the SRC/Links/RC is very dependent on the SRC width (bSRC), reaction column axial stiffness, and the link stiffness.
- The predicted drift is based on equal energy and equal displacements rules however all cases for the 3NCBF-BC fall within the equal energy range.
- The predicted drifts for the Existing frame (~10%), Rehabilitated without Links (~10%), and Rehabilitated with links (~0.75%) seems reasonable compared with results of NLTHA.
**Input**

Earthquake Hazard

| Haz := "MCE"

Stories, Floors, Modes

| N := 3 |
| i := 1, 2 .. N |
| m := 1, 2 .. N |

Material Properties

| E_s := 29000ksi |
| ν_s := 0.30 |

\[ G_s := \frac{E_s}{2(1 + ν_s)} = 1.115 \times 10^4 \text{ksi} \]

F_{yc} := 55ksi

Braced Frame Geometry

<table>
<thead>
<tr>
<th>( h_i := )</th>
<th>( L_{be_i} := )</th>
<th>( C_i := )</th>
</tr>
</thead>
<tbody>
<tr>
<td>13ft</td>
<td>28.75ft</td>
<td>( \frac{1}{6} )</td>
</tr>
<tr>
<td>13ft</td>
<td>28.75ft</td>
<td>( \frac{1}{3} )</td>
</tr>
<tr>
<td>13ft</td>
<td>28.75ft</td>
<td>( \frac{1}{2} )</td>
</tr>
</tbody>
</table>

\[ L := 30 \text{ft} \]

\[ H_i := \sum_{j=1}^{i} h_j \]

\[ H = \begin{pmatrix} 13 \\ 26 \text{ ft} \\ 39 \end{pmatrix} \]

\[ L_{b_i} := \frac{\sqrt{L_i^2 + (2h_i)^2}}{2} \]

\[ θ_{b_i} := \text{atan} \left[ \frac{(2h_i)}{L_i} \right] \]

\[ L_{b_i} = \begin{pmatrix} 19.849 \text{ ft} \\ 19.849 \\ 19.849 \end{pmatrix} \]

\[ θ_{b} = \begin{pmatrix} 40.914 \text{ deg} \end{pmatrix} \]
Mass, Weight, Gravity Loads

\[
m_f := \frac{6.7 \text{kip sec}^2}{4} \quad \text{in}
\]

\[
W_f := m_f g
\]

\[
P_{D_i} :=
\]

\[
\begin{align*}
\text{82kip} \\
\text{82kip} \\
\text{77kip}
\end{align*}
\]

Braced Frame Braces

\[
A_{b_i} := \quad F_{bET_i} := \quad F_{bEC_i} := \quad F_{bEPB_i} :=
\]

\[
\begin{align*}
8.97 \text{ in}^2 & \quad 427.9 \text{kip} \\
9.74 \text{ in}^2 & \quad 464.6 \text{kip} \\
7.88 \text{ in}^2 & \quad 375.9 \text{kip} \\
\end{align*}
\]

Existing BF story with braces at "post-buckling" strength

\[
F_{BH_i} = \begin{cases} 
(F_{bET_i} + F_{bEPB_i} \cos(\theta_{b_i})) & \text{if } i = n_{PB} \\
(F_{bET_i} + F_{bEC_i} \cos(\theta_{b_i})) & \text{otherwise}
\end{cases}
\]

\[
F_{BH_i} = \begin{bmatrix} 
621.721 \\
650.066 \\
460.91
\end{bmatrix} \text{kip}
\]

Braced Frame Beams and Columns

\[
Z_{b_i} := \quad A_{c_i} := 28.2 \text{ in}^2
\]

\[
\begin{align*}
415 \text{ in}^3 \\
467 \text{ in}^3 \\
415 \text{ in}^3
\end{align*}
\]

\[
Z_c := 147 \text{ in}^3
\]

\[
M_{pc} := Z_c F_{ye}
\]

\[
M_{pc} = 673.75 \text{kip ft}
\]

\[
I_{O_i} := 2 \left[ A_{c_i} \left( \frac{L}{2} \right)^2 \right]
\]

\[
I_O = \begin{bmatrix} 
88.125 \\
88.125 \\
88.125
\end{bmatrix} \text{ ft}^4
\]

A2.4-3
SRC, Links, Reaction Column

Average area and moment of inertia of SRC columns

\[ A_{SC} := 57.35 \text{in}^2 \quad I_{SC} := 8450 \text{in}^4 \]

\[ A_{d_1} := \begin{bmatrix} 68.5 \text{in}^2 \\ 56.8 \text{in}^2 \\ 42.7 \text{in}^2 \end{bmatrix} \quad A_{SB_1} := \begin{bmatrix} 75.6 \text{in}^2 \\ 62.0 \text{in}^2 \\ 26.5 \text{in}^2 \end{bmatrix} \]

\[ \theta_{d_1} := \text{atan} \left( \frac{h_1}{b_{SRC}} \right) \quad \theta_d = \left( \frac{47.291 \text{deg}}{47.291} \right) \]

Reaction Column Properties

\[ A_{RC} := 134 \text{in}^2 \quad I_{RC} := 7190 \text{in}^4 \]

Link Properties (see link design approach below)

Relationship between drift angle and link rotation angle

\[ \theta_L = 1 + \left( \frac{b_{SRC}}{2 \cdot e_L} \right) \cdot \theta_d \]

\[ e_L := 48 \text{in} \]

\[ I_L := 6680 \text{in}^4 \]

\[ A_v := 18.4 \text{in}^2 \]

\[ V_{pL\_L} := 553 \text{kip} \]

Mode shapes and periods from analysis

Existing

\[ \phi_{1Ex} := \begin{bmatrix} 0.223 \\ 0.4289 \\ 0.6019 \end{bmatrix} \quad \phi_{2Ex} := \begin{bmatrix} 0.5227 \\ -0.4451 \end{bmatrix} \quad \phi_{3Ex} := \begin{bmatrix} 0.5220 \\ 0.1885 \end{bmatrix} \quad T_{Ex} := \begin{bmatrix} 0.4498 \\ 0.1060 \end{bmatrix} \]

Existing w/SRC, RC

\[ \phi_{1R} := \begin{bmatrix} 0.2065 \\ 0.4175 \\ 0.6157 \end{bmatrix} \quad \phi_{2R} := \begin{bmatrix} 0.5343 \\ -0.4242 \end{bmatrix} \quad \phi_{3R} := \begin{bmatrix} 0.5170 \\ 0.1923 \end{bmatrix} \quad T_R := \begin{bmatrix} 0.4469 \\ 0.0908 \end{bmatrix} \]

Existing w/SRC, RC, Links

\[ \phi_{1RL} := \begin{bmatrix} 0.1973 \\ 0.4169 \\ 0.6191 \end{bmatrix} \quad \phi_{2RL} := \begin{bmatrix} 0.5602 \\ -0.4077 \end{bmatrix} \quad \phi_{3RL} := \begin{bmatrix} 0.4927 \\ 0.2156 \end{bmatrix} \quad T_{RL} := \begin{bmatrix} 0.3021 \\ 0.0895 \end{bmatrix} \]
Plastic Base Shear Capacity of Existing Frame and SRC with Links

- used later in calculation when applying equal energy approximation for inelastic system displacement calcs
- mixed mechanism formulation for determining plastic base shear capacity is considered here

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.

\[ F_{bTEB_i} = \min \left( F_{bET_i} + \frac{4Z_{b_i} F_{ye}}{L_{bc} \sin \theta_{b_i}} \right) \]

\[ F_{bTEB} = \begin{cases} 427.9 \text{ kip} \\ 375.9 \text{ kip} \end{cases} \]

\[ F_{bTEB} = \frac{464.6}{375.9} \text{ kip} \]

Mech_{EB} := \begin{cases} "Brace" \text{ if } F_{bTEB_i} = F_{bET_i} \\ "Beam" \text{ otherwise} \end{cases}

Mech_{EB} = \begin{cases} "Brace" \\ "Brace" \\ "Brace" \end{cases}

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.

\[ F_{bTEPB_i} := \min \left( 0, F_{bEPB_i} + \frac{4Z_{b_i} F_{ye}}{L_{bc} \sin \theta_{b_i}} \right) \]

\[ F_{bTEPB} = \begin{cases} 0 \text{ kip} \\ 0 \text{ kip} \end{cases} \]

Mech_{EPB_i} := \begin{cases} "Brace Fracture" \text{ if } F_{bTEPB_i} = 0 \\ "Beam" \text{ otherwise} \end{cases}

Mech_{EPB} = \begin{cases} "Brace Fracture" \\ "Brace Fracture" \\ "Brace Fracture" \end{cases}

Existing frame controlling plastic mechanism: considers a panel mechanism with fractured braces within the nPB story and all other stories elastic

Column Internal Work

\[ W_{ic,Ex} := \begin{cases} 2M_{pc} \text{ if } n_{PB} = N \\ 4M_{pc} \text{ otherwise} \end{cases} \]

External Work

\[ W_{E,Ex} := h \sum_{i=n_{PB}}^{N} C_i \]

Internal Work = External Work

\[ V_{p,Ex} := \frac{W_{ic,Ex}}{W_{E,Ex}} \]

Plastic base shear of frame (existing frame)

\[ V_{p,Ex} = 207.308 \text{ kip} \]
Rehabilitated Frame controlling plastic mechanism (without Links): considers all braces fractured and flexural yielding at base of columns

Column Internal Work \( W_{ic,R} := 2 \cdot M_{pc} \)

External Work \( W_{E,R} := \sum_{i=1}^{N} (H_i \cdot C_i) \)

Internal Work = External Work \( V_{p,Rehab} := \frac{W_{ic,R}}{W_{E,R}} \)

Plastic base shear of frame (rehabilitated without Links) \( V_{p,Rehab} = 44,423 \text{kip} \)

Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces fractured, flexural yielding at base of columns, and yielding Links with strength \( V_{p,Lo} \) at each floor elevation

Column Internal Work \( W_{ic,RL} := 2 \cdot M_{pc} \)

Link Internal Work \( W_{il,RL} := N \cdot V_{p,Lo} \cdot (b_{SRC} + c_L) \)

External Work \( W_{E,RL} := \sum_{i=1}^{N} (H_i \cdot C_i) \)

Internal Work = External Work \( V_{p,Rehab,RL} := \frac{W_{ic,RL} + W_{il,RL}}{W_{E,RL}} \)

Plastic base shear of frame (rehabilitated with Links) \( V_{p,Rehab,RL} = 919.5 \text{kip} \)
Existing Building Stiffness

Shear stiffness of each story of BF (note 0.65 factor for rigid offsets of braces)

\[ k_{ExF,s_i} := 2 \frac{E_p A b_i}{\sqrt{0.65 \left( \frac{L}{2} \right)^2 + (h_i)^2}} (\cos(\theta_{b_{i,j}})^2) \]

\[ k_{ExF,s} = \begin{pmatrix} 1.547 \times 10^3 \\ 1.68 \times 10^3 \\ 1.359 \times 10^3 \end{pmatrix} \text{kip in} \]

Inter-story shear force and overturning moment on BF

\[ V_{BF} := \sum_{j=1}^{N} C_j \]

\[ M_{BF} := \sum_{j=1}^{N} (C_j h_j) \]

\[ V_{BF} = \begin{pmatrix} 1 \\ 0.833 \\ 0.5 \end{pmatrix} \text{kips} \]

\[ M_{BF} = \begin{pmatrix} 13 \\ 10.833 \\ 6.5 \end{pmatrix} \text{ft kips} \]

Overturning or "flexural" stiffness of each story of BF using moment-area method to calculate deflections under unit lateral forces

\[ k_{ExF,f_i} := \frac{E_p k_{O_i}}{2 \cdot M_{BF} \cdot h_i} \cdot \left( \frac{2}{3} - \frac{h_i}{h} \right) \]

\[ k_{ExF,f} = \begin{pmatrix} 4.188 \times 10^4 \\ 5.025 \times 10^4 \\ 8.375 \times 10^4 \end{pmatrix} \text{kip in} \]

Stiffness of existing braced frame (series spring stiffnesses of shear and flexural frame stiffnesses)

\[ k_{ExF} = \frac{1}{\frac{1}{k_{ExF,s_i}} + \frac{1}{k_{ExF,f_i}}} \]

\[ k_{ExF} = \begin{pmatrix} 1.492 \times 10^3 \\ 1.626 \times 10^3 \\ 1.337 \times 10^3 \end{pmatrix} \text{kip in} \]

Effective first mode height

\[ h_{mEX} := \frac{\sum_{i=1}^{N} (H_i \cdot m_{f_i} \cdot \phi_{1Ex_i})}{\sum_{i=1}^{N} (m_{f_i} \cdot \phi_{1Ex_i})} \]

\[ f_{hEX} := \frac{H_3}{h_{mEX}} = 1.303 \]

Effective lateral stiffness of BF (derived based on calculating lateral deformation at roof due to unit base shear force)

\[ k_{0ExF} := \frac{f_{hEX}}{7} \left( \sum_{i=1}^{N} k_{ExF_i} \right) = 829.324 \text{kip in} \]

\[ W_{1Ex} := \frac{\sum_{i=1}^{N} \left( W_{f_i} \cdot (\phi_{1Ex_i})^2 \right)^2}{\sum_{i=1}^{N} \left( W_{f_i} \cdot (\phi_{1Ex_i})^2 \right)} \]

\[ W_{1Ex} = 1.706 \times 10^3 \text{kip} \]

\[ \sum_{i} W_{1Ex} = 0.879 \]

Effective first mode weight

\[ W_{1Ex} := \frac{\sum_{i=1}^{N} \left( W_{f_i} \cdot (\phi_{1Ex_i})^2 \right)}{\sum_{i=1}^{N} \left( W_{f_i} \cdot (\phi_{1Ex_i})^2 \right)} \]

Existing building first mode period

\[ T_{Pred.Ex} = 2\pi \sqrt{\frac{W_{1Ex}}{g \cdot k_{0ExF}}} \]

\[ T_{Pred.Ex} = 0.459 \text{s} \]
Existing Period Calculation using 3DOF Matrix (simply for comparison with SDOF approximation presented previously)

\[ K_{EF} := \begin{pmatrix} k_{ExF_1} + k_{ExF_2} & -k_{ExF_2} & -k_{ExF_3} \\ -k_{ExF_2} & k_{ExF_2} + k_{ExF_3} & -k_{ExF_3} \\ -k_{ExF_3} & -k_{ExF_3} & k_{ExF_3} \end{pmatrix} \begin{pmatrix} \text{kip} \\ \text{kip} \\ \text{kip} \end{pmatrix} \begin{pmatrix} \text{in} \\ \text{in} \end{pmatrix} \]

\[ m_m := \begin{pmatrix} 1.675 & 0 & 0 \\ 0 & 1.675 & 0 \\ 0 & 0 & 1.675 \end{pmatrix} \begin{pmatrix} \text{kip sec}^2 \\ \text{kip sec}^2 \end{pmatrix} \begin{pmatrix} \text{in} \\ \text{in} \end{pmatrix} \]

\[ \omega_{Ex} := \sqrt{\text{eigenvals}(K_{EF} m_m - I)} \]

\[ \omega_{Ex} = \begin{pmatrix} 13.39 \\ 36.187 \end{pmatrix} \begin{pmatrix} \text{rad} \\ \text{sec} \end{pmatrix} \]

\[ f_{Ex} = \frac{\omega_{Ex}}{2\pi} = \begin{pmatrix} 2.131 \\ 5.759 \end{pmatrix} \begin{pmatrix} \frac{1}{s} \\ \frac{1}{s} \end{pmatrix} \]

\[ T_{Ex2} := \frac{1}{f_{Ex}} = \begin{pmatrix} 0.469 \\ 0.174 \\ 0.116 \end{pmatrix} \begin{pmatrix} \text{s} \\ \text{s} \end{pmatrix} \]

\[ \text{EigVec}_{Ex} := \text{eigenvect}(K_{EF} m_m - I) \]

\[ \text{EigVec}_{Ex} = \begin{pmatrix} -0.333 & 0.688 & -0.645 \\ -0.578 & 0.391 & 0.716 \\ -0.745 & -0.611 & -0.267 \end{pmatrix} \]

\[ \phi_{m_1} := \text{submatrix}(\text{EigVec}_{Ex}^{-1}, 1, 1, 1) \]

\[ \phi_{m_1} = \begin{pmatrix} -0.333 \\ -0.578 \\ -0.745 \end{pmatrix} \]

\[ T_{Ex2} = 2\pi \sqrt{\frac{\phi_{m_1}^T m_m \phi_{m_1}}{\phi_{m_1}^T K_{ExF} \phi_{m_1}}} = 0.469 \text{s} \]

\[ T_{Ex2} = 0.469 \text{s} \]
Rehabilitated stiffness calculation that includes link and RC flexibility

Factor for correction of flexural beam stiffness to include shear flexibility

\[ C_1 := \frac{12E_sL}{G_sA_vc_L^2} \]

\[ A_{eq} := \frac{N \sum h_i}{\sqrt{b_{SRC}^2 + (h_i)^2}^3 + \frac{b_{SRC}}{A_{SB_1}} + 2(h_i)^3} \]

\[ A_{eq} = \frac{130.366}{111.649 \text{ in}^2} \]

\[ b_{effSRC} := \frac{A_{RC}}{2} \cdot b_{SRC} \]

\[ x_{NA_i} := \frac{A_{RC}(b_{effSRC} + c_L)}{A_{RC} + A_{eq_i}} \]

\[ x_{NA} = \left[ \begin{array}{c}
60.825 \\
65.459 \text{ in}
\end{array} \right] \]

\[ x_{NA} = \left[ \begin{array}{c}
5.185 \times 10^{-5}
8.77 \times 10^{-5} \text{ in}^4
7.009 \times 10^{-5}
\end{array} \right] \]

\[ l_{of_1} := A_{eq_1} \left( x_{NA_1} \right)^2 + A_{RC} \left( c_L + b_{effSRC} - x_{NA_1} \right)^2 \]

\[ l_{of_1} = \frac{627(h_1)^3}{E_sL_{of_1}} = 8.626 \times 10^{-4} \text{ in kip} \]

\[ \Delta_B := 6.27 \frac{(h_1)^3}{E_sL_{of_1}} = 8.626 \times 10^{-4} \text{ in kip} \]

\[ k_1 := \frac{3E_sL_{RC}}{(h_1)^3} + \frac{3E_sL_{ISC}}{(h_1)^3} + \frac{E_sA_{d_1}}{\sqrt{b_{SRC}^2 + (h_1)^2}^3} \left( \cos \left( \theta_{d_1} \right) \right)^2 \]

\[ k_1 = 4.663 \times 10^{-3} \text{ kip in} \]

\[ k_2 := \frac{12E_sL_{RC}}{(h_2)^3} + \frac{12E_sL_{ISC}}{(h_2)^3} + \frac{E_sA_{d_2}}{\sqrt{b_{SRC}^2 + (h_2)^2}^3} \left( \cos \left( \theta_{d_2} \right) \right)^2 \]

\[ k_2 = 5.003 \times 10^{-3} \text{ kip in} \]

\[ k_3 := \frac{12E_sL_{RC}}{(h_3)^3} + \frac{12E_sL_{ISC}}{(h_3)^3} + \frac{E_sA_{d_3}}{\sqrt{b_{SRC}^2 + (h_3)^2}^3} \left( \cos \left( \theta_{d_3} \right) \right)^2 \]

\[ k_3 = 4.117 \times 10^{-3} \text{ kip in} \]

\[ \Delta_S = \frac{1}{k_1} + \frac{3}{k_2} + \frac{3}{k_3} + \frac{N \sum (C_iH_i)}{b_{SRC}^3 \left( \frac{1}{1 + C_1} \right)} \]

\[ \Delta_S = 1.273 \times 10^{-3} \text{ in kip} \]
Effective first mode height \[ h_{mRL} := \frac{\sum_{i=1}^{N} \left( H_i m_{f_i} \phi_{1RL} \right)}{\sum_{i=1}^{N} \left( m_{f_i} \phi_{1RL} \right)} \]

Effective first mode weight \[ w_{1RL} := \frac{\left[ \sum_{i=1}^{N} \left[ W_{f_i} \left( \frac{\phi_{1RL}}{h_{m RL}} \right) \right] \right]^2}{\sum_{i=1}^{N} \left[ W_{f_i} \left( \frac{\phi_{1RL}}{h_{m RL}} \right) \right]^2} \]

\[ k_{SRC} := \frac{f_{h RL}}{\Delta_B + \Delta_S} \]

\[ k_{SRC} = 599.748 \text{ kip/in} \]

\[ k_{Rehab} = k_{oExF} + k_{SRC} \]

\[ k_{Rehab} = 1.429 \times 10^3 \text{ kip/in} \]

\[ T_{Pred.Rehab} := 2\pi \sqrt{\frac{w_{1RL}}{g k_{Rehab}}} \]

\[ T_{Pred.Rehab} = 0.344 \text{ s} \]

\[ f_{h RL} := \frac{H_3}{h_{m RL}} = 1.281 \]

\[ w_{1RL} = 1.65 \times 10^3 \text{ kip} \]

\[ \frac{\Delta_B}{\Delta_B + \Delta_S} = 0.404 \]

\[ \frac{\Delta_S}{\Delta_B + \Delta_S} = 0.596 \]

\[ \frac{k_{oExF}}{k_{Rehab}} = 0.58 \quad \frac{k_{SRC}}{k_{Rehab}} = 0.42 \]
Effective Modal weights

\[ w_{E1} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{1E})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{1E})_{i}]^2} \]

\[ w_{R1} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{1R})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{1R})_{i}]^2} \]

\[ w_{RL1} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{1RL})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{1RL})_{i}]^2} \]

\[ w_{E2} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{2E})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{2E})_{i}]^2} \]

\[ w_{R2} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{2R})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{2R})_{i}]^2} \]

\[ w_{RL2} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{2RL})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{2RL})_{i}]^2} \]

\[ w_{E3} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{3E})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{3E})_{i}]^2} \]

\[ w_{R3} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{3R})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{3R})_{i}]^2} \]

\[ w_{RL3} := \frac{\sum_{i=1}^{N} [w_{f_i} (\phi_{3RL})_{i}]^2}{\sum_{i=1}^{N} [w_{f_i} (\phi_{3RL})_{i}]^2} \]

\[ w_{E1} = 1.706 \times 10^{-3} \text{kip} \quad w_{E2} = 201.001 \text{kip} \quad w_{E3} = 33.123 \text{kip} \]

\[ w_{R1} = 1.65 \times 10^{-3} \text{kip} \quad w_{R2} = 240.991 \text{kip} \quad w_{R3} = 31.405 \text{kip} \]

\[ w_{RL1} = 1.65 \times 10^{-3} \text{kip} \quad w_{RL2} = 263.5 \text{kip} \quad w_{RL3} = 26.134 \text{kip} \]

\[ \frac{w_{E1}}{\sum w_{f_i}} = 0.879 \quad \frac{w_{R1}}{\sum w_{f_i}} = 0.851 \quad \frac{w_{RL1}}{\sum w_{f_i}} = 0.851 \]

\[ \frac{w_{E2}}{\sum w_{f_i}} = 0.104 \quad \frac{w_{R2}}{\sum w_{f_i}} = 0.124 \quad \frac{w_{RL2}}{\sum w_{f_i}} = 0.136 \]

\[ \frac{w_{E3}}{\sum w_{f_i}} = 0.017 \quad \frac{w_{R3}}{\sum w_{f_i}} = 0.016 \quad \frac{w_{RL3}}{\sum w_{f_i}} = 0.013 \]
Modal Participation Factors

\[
\Gamma_{Ex_1} := \frac{W_{Ex_1}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{1Ex_i}) \right]}
\quad \Gamma_{R_1} := \frac{W_{R_1}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{1R_i}) \right]}
\quad \Gamma_{RL_1} := \frac{W_{RL_1}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{1RL_i}) \right]}
\quad \Gamma_{Ex_1} = 2.104
\]

\[
\Gamma_{Ex_2} := \frac{W_{Ex_2}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{2Ex_i}) \right]}
\quad \Gamma_{R_2} := \frac{W_{R_2}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{2R_i}) \right]}
\quad \Gamma_{RL_2} := \frac{W_{RL_2}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{2RL_i}) \right]}
\quad \Gamma_{Ex_2} = 0.722
\]

\[
\Gamma_{Ex_3} := \frac{W_{Ex_3}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{3Ex_i}) \right]}
\quad \Gamma_{R_3} := \frac{W_{R_3}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{3R_i}) \right]}
\quad \Gamma_{RL_3} := \frac{W_{RL_3}}{\sum_{i=1}^{N} \left[ W_{f_i} \cdot (\phi_{3RL_i}) \right]}
\quad \Gamma_{Ex_3} = 0.293
\]

Spectral acceleration values

\[
S_1 := \begin{cases} 
1.2g & \text{if Haz = "MCE"} \\
0.68g & \text{if Haz = "DBE"}
\end{cases}
\quad S_1 = 1.2g
\]
\[
S_s := \begin{cases} 
1.6g & \text{if Haz = "MCE"} \\
1.1g & \text{if Haz = "DBE"}
\end{cases}
\quad S_s = 1.6g
\]

\[
T_s := \frac{S_1}{S_s} \quad T_s = 0.75 \text{s}
\]
\[
T_0 := 0.2 \cdot T_s \quad T_0 = 0.15 \text{s}
\]

Coefficient to adjust 5% damped spectral acceleration values to 2% damped with \(T > T_0\) (ASCE 7-10)

\[
B_{d,To} := 0.8
\]
\[
B_{d,Ex_i} := 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{Ex_i}
\]
\[
B_{d,Ex} = \begin{pmatrix} 
0.88 \\
0.955 \\
0.972 
\end{pmatrix}
\]
\[
B_{d,R_1} := 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{R_1}
\]
\[
B_{d,R} = \begin{pmatrix} 
0.881 \\
0.966 \\
0.976 
\end{pmatrix}
\]
\[
B_{d,RL_1} := 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_{RL_1}
\]
\[
B_{d,RL} = \begin{pmatrix} 
0.919 \\
0.966 \\
0.976 
\end{pmatrix}
\]
Existing Building Drift (Soft Story)

SiteClass := "B"

\[ T_{Ex} = \begin{bmatrix} 0.45 \\ 0.17 \\ 0.11 \end{bmatrix} \]

\[ V_{p,Ex} = 207.308 \text{ kip} \]

define a dummy stiffness matrix that has a soft first story to produce orthogonal mode shapes and calculate acceptable "plastic" mode shapes and other modal properties

\[ k_p := \begin{bmatrix} 0.01 \text{ kip in} & k_{p2} := 1000 \text{ kip in} & k_{p3} := 1000 \text{ kip in} \end{bmatrix} \]

\[ k_p := \begin{bmatrix} k_{p1} + k_{p2} & -k_{p2} & 0 \text{ kip in} \\ -k_{p2} & k_{p2} + k_{p3} & -k_{p3} \\ 0 \text{ kip in} & -k_{p3} & k_{p3} \end{bmatrix} \]

\[ k_p = \begin{bmatrix} 1 \times 10^3 & -1 \times 10^3 & 0 \\ -1 \times 10^3 & 2 \times 10^3 & -1 \times 10^3 \\ 0 & -1 \times 10^3 & 1 \times 10^3 \end{bmatrix} \text{ kip in} \]

\[ \omega_p := \sqrt{\text{eigvals}(k_p \cdot m^{-1})} \]

\[ \omega_p = \begin{bmatrix} 0.045 \text{ rad sec}^{-1} \\ 24.434 \text{ rad sec}^{-1} \\ 42.321 \text{ rad sec}^{-1} \end{bmatrix} \]

\[ f_p := \frac{\omega_p}{2\pi} = \begin{bmatrix} 7.1 \times 10^{-3} \\ 3.889 \\ 6.736 \end{bmatrix} \text{ s}^{-1} \]

\[ f_p = \begin{bmatrix} 7.1 \times 10^{-3} \\ 3.889 \\ 6.736 \end{bmatrix} \text{ s}^{-1} \]

\[ T_p := \frac{1}{f_p} = \begin{bmatrix} 140.847 \\ 0.257 \text{ s} \\ 0.148 \text{ s} \end{bmatrix} \]

\[ T_p = \begin{bmatrix} 140.847 \\ 0.257 \text{ s} \\ 0.148 \text{ s} \end{bmatrix} \]

\[ \phi_p := \text{eigenvectors}(k_p \cdot m^{-1}) \]

\[ \phi_p = \begin{bmatrix} -0.577 & 0.707 & -0.408 \\ 0.577 & 0.707 & 0.408 \\ -3.536 \times 10^{-6} & 0.816 \end{bmatrix} \]

\[ T_p := 2\pi \cdot \sqrt{\frac{\phi_p^T \cdot m \cdot \phi_p \cdot \phi_p^T \cdot m}{\phi_p^T \cdot k_p \cdot \phi_p \cdot \phi_p^T \cdot m}} = \end{bmatrix} \]

\[ T_p = \begin{bmatrix} 140.847 \\ 0.257 \text{ s} \\ 0.148 \text{ s} \end{bmatrix} \]
\( w_{p_m} = \frac{\sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_p \left( m \right) \right) \right]^2}{\sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_p \left( m \right) \right) \right]^2} \)

\( \Gamma_{p_m} = \frac{W_{E_{m}}}{\sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_p \left( m \right) \right) \right]^2} \)

\( w_{p} = \begin{pmatrix} 1.94 \times 10^3 \\ 3.233 \times 10^{-8} \\ 1.198 \times 10^{-9} \end{pmatrix} \text{kip} \)

\( \sum_{i} W_{f_i} = \begin{pmatrix} 1 \\ 1.667 \times 10^{-11} \\ 6.173 \times 10^{-13} \end{pmatrix} \)

\( S_{a_i} := \begin{cases} \frac{S_s}{B_{d,Ex_i}} \left( 0.4 + 0.6 \frac{T_{Ex_i}}{T_0} \right) & \text{if } T_{Ex_i} < T_0 \\ \frac{S_s}{B_{d,Ex_i}} & \text{if } T_0 < T_{Ex_i} < T_s \\ \frac{S_{1,sec}}{T_{Ex_i} \cdot B_{d,To}} & \text{otherwise} \end{cases} \)

\( S_a = \begin{pmatrix} -1.523 \\ -3.964 \times 10^4 \\ -3.764 \times 10^4 \end{pmatrix} \)

First mode displacement applying equal energy or equal displacement rule

\( S_{d_1} = \begin{cases} \frac{V_{p,Ex}}{2 \cdot k_{oExF}} + \frac{\left( \frac{W_{p_1}}{S_{a_1}} \right)^2}{2 \cdot V_{p,Ex}} & \text{if } T_{Ex_1} < S_{1,sec} \\ \frac{W_{p_1}}{g} \cdot S_{a_1} & \text{otherwise} \end{cases} \)

\( n_{PB} = 1 \)

\( S_{d_1} = 36.307 \text{ in} \)

Higher modes

\( S_{d_2} = \frac{T_{p_2}^2 \cdot S_{a_2}}{4 \pi^2} \quad S_{d_3} = \frac{T_{p_3}^2 \cdot S_{a_3}}{4 \pi^2} \)

\( S_{d_2} = 1.084 \text{ in} \quad S_{d_3} = 0.292 \text{ in} \)

\( S_{d,SRSS} = \sqrt{\left( S_{d_1} \right)^2 + \left( S_{d_2} \right)^2 + \left( S_{d_3} \right)^2} \quad S_{d,SRSS} = 36.325 \text{ in} \quad \frac{S_{d_1}}{S_{d,SRSS}} = 1 \)
"Plastic" mode shape for 1st soft-story existing building

\[ \phi_{p1} := \phi_p \]

\[ u_{\text{max,e}} := \Gamma_{p1} \phi_{p1} \tilde{S}_{d,SRSS} \]

\[ u_{\text{max,e}} = \begin{pmatrix} 31.938 \\ 31.939 \end{pmatrix} \text{ in} \]

\[ \theta_{\text{existing}} := \begin{cases} \frac{u_{\text{max,e}_1}}{h_1} & \text{if } i = 1 \\ \frac{u_{\text{max,e}_i} - u_{\text{max,e}_{i-1}}}{h_i} & \text{otherwise} \end{cases} \]

\[ \theta_{\text{existing}} = \begin{pmatrix} 20.473 \\ 1.365 \times 10^{-4} \end{pmatrix} \%
\begin{pmatrix} 6.824 \times 10^{-5} \end{pmatrix} \]

PerfCat := "Collapse" if \( \max(\theta_{\text{existing}}) > 2.0\% \)
"CP" if \( 2.0\% > \max(\theta_{\text{existing}}) > 1.5\% \)
"LS" if \( 1.5\% > \max(\theta_{\text{existing}}) > 0.5\% \)
"IO" if \( 0.5\% > \max(\theta_{\text{existing}}) \)

Haz = "MCE"
Rehabilitated Building Drift (Equal Disp and Equal Energy Calc)

First check if added stiffness from SRCRehab needed or pin-pin links can be used

\[ S_{a_1} := \begin{cases} \frac{S_s}{B_{d,R_1}} \left( 0.4 + 0.6 \frac{T_{R_1}}{T_0} \right) & \text{if } T_{R_1} < T_0 \quad T_R = \begin{cases} 0.447 \quad \text{s} \\ 0.129 \quad \text{s} \end{cases} \\ \frac{S_s}{B_{d,R_1}} & \text{if } T_0 < T_{R_1} < T_s \\ S_{1,\text{sec}} & \text{otherwise} \end{cases} \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,R_1} := \begin{cases} \frac{V_{p,\text{Rehab}}}{2k_{oExF}} + \frac{W_{R_1}}{g} & \text{if } T_{R_1} < \frac{S_1}{S_s} \\ \frac{W_{R_1}}{g} S_{a_1} & \text{otherwise} \end{cases} \]

\[ S_{d,R_1} = 243.969 \text{-in} \]

Higher modes

\[ \begin{align*} S_{d,R_2} &= \frac{\left( T_{R_2} \right)^2 S_{a_2}}{4\pi^2} \\ S_{d,R_3} &= \frac{\left( T_{R_3} \right)^2 S_{a_3}}{4\pi^2} \end{align*} \]

\[ S_{d,R_2} = 0.248 \text{-in} \quad S_{d,R_3} = 0.101 \text{-in} \]

\[ S_{d,R,\text{SRSS}} = \sqrt{S_{d,R_1}^2 + S_{d,R_2}^2 + S_{d,R_3}^2} \]

\[ S_{d,R,\text{SRSS}} = 243.969 \text{-in} \]

\[ u_{\text{max},R} = T_R \Phi R S_{d,R,\text{SRSS}} \]

\[ u_{\text{max},R} = \begin{cases} u_{\text{max},R_1} & \text{if } i \neq 1 \\ \left( u_{\text{max},R_1} - u_{\text{max},R_{i-1}} \right) & \text{otherwise} \end{cases} \]

\[ \theta_{\text{Rehab}_i} = \frac{u_{\text{max},R_i}}{h_i} \]

\[ \theta_{\text{Rehab}} = \begin{cases} 66.48 \quad \text{\%} \\ 67.929 \quad \text{\%} \end{cases} \]

\[ \text{PerfCat := "Collapse" if max(\theta_{\text{Rehab}}) > 2.0\%} \]

\[ \text{"CP" if 2.0\% > max(\theta_{\text{Rehab}}) > 1.5\%} \]

\[ \text{"LS" if 1.5\% > max(\theta_{\text{Rehab}}) > 0.5\%} \]

\[ \text{"IO" if 0.5\% > max(\theta_{\text{Rehab}}) \}

Haz = "MCE"

A2.4-16
Rehabilitated Building Drift with Links (Equal Disp and Equal Energy Calcs)

\[ S_{d,RL_1} := S_s \left( \frac{T_{RL_1}}{T_0} \right) \quad \text{if} \quad T_{RL_1} < T_0 \]
\[ \frac{T_{RL_1}}{T_0} \]
\[ S_s \quad \text{if} \quad T_0 < T_{RL_1} < T_s \]
\[ S_s = 1.498 \quad \text{g} \]
\[ S_1, \quad \sec \quad \text{otherwise} \]
\[ T_{RL_1}, \quad B_d, T_o \]

First mode displacement applying equal energy or equal displacement rule

\[ S_{d,RL_1} := \frac{V_{p, rehab, L} + k_{rehab} \left( \frac{W_{RL_1}}{g} \right)^2}{2 \cdot k_{rehab}} \quad \text{if} \quad T_{RL_1} < \frac{S_1}{S_s} \]
\[ S_{d,RL_1} = 3.46 \quad \text{in} \]

Higher modes

\[ S_{d,RL_2} = \frac{T_{RL_2}^2 \cdot S_{a_2}}{4 \pi^2} \]
\[ S_{d,RL_3} = \frac{T_{RL_3}^2 \cdot S_{a_3}}{4 \pi^2} \]
\[ S_{d,RL_2} = 0.233 \quad \text{in} \]
\[ S_{d,RL_3} = 0.097 \quad \text{in} \]

\[ S_{d,RL_{SRSS}} := \sqrt{S_{d,RL_1}^2 + S_{d,RL_2}^2 + S_{d,RL_3}^2} \]
\[ S_{d,RL_{SRSS}} = 3.469 \quad \text{in} \]

\[ u_{\max, R.L.} := \frac{V_{RL_1} \cdot S_{d,RL_{SRSS}}}{2 \cdot k_{rehab}} \]
\[ u_{\max, R.L.} = \left( \frac{1.416}{2.993} \right) \cdot \frac{S_{d,RL_1}}{S_{d,RL_{SRSS}}} \]
\[ u_{\max, R.L.} = 0.997 \]

\[ \theta_{Rehab, L_i} := \frac{u_{\max, R.L_i} - u_{\max, R.L_{i-1}}}{h_i} \quad \text{otherwise} \]
\[ \theta_{Rehab, L} = \left( \frac{0.908}{1.011} \right) \cdot \% \]

Haz = "MCE"

"Collapse" if \( \max(\theta_{Rehab, L}) > 2.0\% \)
"CP" if \( 2.0\% > \max(\theta_{Rehab, L}) > 1.5\% \)
"LS" if \( 1.5\% > \max(\theta_{Rehab, L}) > 0.5\% \)
"IO" if \( 0.5\% > \max(\theta_{Rehab, L}) \)
Summary of Critical Values

Existing

\[ T_{Ex} = \begin{pmatrix} 0.45 \\ 0.169 \\ 0.106 \end{pmatrix} \text{ s} \]

\[ k_{0ExF} = 829.324 \frac{\text{kip}}{\text{in}} \]

\[ V_{p,Ex} = 207.308 \text{ kip} \]

\[ \theta_{Ex,MCE} := \begin{pmatrix} 20.473 \\ 1.365 \times 10^{-4} \\ 6.824 \times 10^{-5} \end{pmatrix} \%

With SRC, Pin-Pin Links

\[ T_R = \begin{pmatrix} 0.447 \\ 0.129 \\ 0.091 \end{pmatrix} \text{ s} \]

\[ k_{0RF} = 829.324 \frac{\text{kip}}{\text{in}} \]

\[ V_{p,Rehab} = 44.423 \text{ kip} \]

\[ \theta_{R,MCE} := \begin{pmatrix} 66.48 \\ 67.929 \end{pmatrix} \%

With SRC, Fix-Fix Links

\[ T_{RL} = \begin{pmatrix} 0.302 \\ 0.126 \\ 0.09 \end{pmatrix} \text{ s} \]

\[ k_{Rehab} = 1.429 \times 10^3 \frac{\text{kip}}{\text{in}} \]

\[ V_{p,Rehab, L} = 919.5 \text{ kip} \]

\[ \theta_{RL,MCE} := \begin{pmatrix} 0.721 \\ 0.739 \end{pmatrix} \% \]
Design Approach for Links and Reaction Column to Achieve Target Drift

Note: Final selected link and reaction column sections used in stiffness and drift calculations above.

existing frame plastic base shear strength considering soft-story mechanism

\[ V_{p,Ex} = 207.308 \text{-kip} \]

frame plastic base shear strength considering all story mechanism with comp. brace at EPB, Rehabilitated w/ pin-pin connected SRC

\[ V_{p,Rehab} = 44.423 \text{-kip} \]

Predicted story drift of existing sub-standard frame

\[ \theta_{\text{existing}} = \begin{bmatrix} 20.473 \\ 1.365 \times 10^{-4} \\ 6.824 \times 10^{-5} \end{bmatrix} \ % \]

Predicted story drift with pin-pin SRC

\[ \theta_{\text{Rehab}} = \begin{bmatrix} 66.48 \\ 67.929 \ % \\ 63.808 \end{bmatrix} \]

Target interstory drift for rehabilitated building

\[ \theta_t := 2.0\% \]

Maximum allowable link plastic rotation

\[ \gamma_L := 0.08 \]

Lateral stiffness increase factor -iterated design variable since both lateral strength and stiffness important for calculating drift for frame in equal energy region

\[ \omega_k := 1.125 \ldots 5 \]
Required frame plastic base shear strength to achieve target rotation
(assume equal energy, function of rehabilitated stiffness, see derivation)

\[ V_{pL}(\omega_k) := \frac{2}{3} \omega_k \theta_{1} \theta_{N} k_{oExF} - \omega_k k_{oExF} \sqrt{\frac{4\theta_{1}^{2}(H_{N})^{2}}{9} \left( \frac{W_{RL} S_{s}}{g B_{d,T0}} \right)^{2}} \]

Required plastic shear strength of each link

\[ V_{pL,i}(\omega_k) = \frac{\sum (C_i H_j)}{N (V_{SRC} + \epsilon_L)} (V_{pL}(\omega_k) - V_{p,Rehab}) \]

For design of the link, the existing all story mechanism frame strength with braces at EPB are conservatively assumed. The brace strength is likely dependent on the magnitude of the target drift.

Range of solutions for VpL with each frame stiffness factor

Note that only positive, real values are physically possible.

Ideally, the smallest value of VpL is ideal since it reduces the RC, SRC, and foundation forces that need to be capacity designed for however such a small link may not achieve the stiffness needed or the axial capacity of a small link may not be adequate to transfer the interaction force. There, a few iterations trying the smallest VpL and checking the stiffness is needed (see below).

\[ V_{pL,i}(\omega_k) = \frac{1}{N (V_{SRC} + \epsilon_L)} (V_{pL}(\omega_k) - V_{p,Rehab}) \]

trial stiffness factor \( \omega_{k,t} := 1.25 \)

corresponding VpL with stiffness factor to achieve target drift

\[ V_{pL,i}(\omega_{k,t}) = 544.146 \text{ kip} \]
Link Selection and Checks (designed like an EBF shear link)

maximum axial force on link from SRC design calculation incl. higher mode effects and plastic interaction forces

\[ A_{\text{Lmax}} := 773.281 \text{kip} \]

Required "squash" load of link to limit axial demand to 1/2 of yield strength

\[ P_{y,\text{req}} := 2 \cdot A_{\text{Lmax}} = 1.547 \times 10^3 \text{kip} \]

Selected link section

\[ \text{LinkSection} := "W30x148" \]

\[ d_L := 30.7 \text{in} \]

Link plastic shear strength

\[ V_{pL} := 553 \text{kip} \]

\[ M_{pL} := 25000 \text{kip-in} \]

\[ \Omega_L := 1.5 \]

Yield strength of section

\[ P_y := 2180 \text{kip} \]

LinkConn := "fixfix"

adjust link IXX since stiffness calculation below derived based on fix-fix links

\[ I_L := \begin{cases} 6680 \text{in}^4 & \text{if LinkConn = "fixfix"} \\ 4 & \text{if LinkConn = "fixpin"} \end{cases} = 6.68 \times 10^3 \text{in}^4 \]

\[ F_y := 50 \text{ksi} \]

\[ A_v := \frac{V_{pL}}{0.6 \cdot F_y} = 18.433 \text{in}^2 \]

Required link length to limit link rotation

\[ \epsilon_L := \max \left[ \frac{b_{\text{SRC}} \theta_t}{d_L \left( \Omega_L - \theta_t \right)} \right] = 48 \text{in} \]

\[ \text{LinkClass} := \begin{cases} "\text{ShortLink}" & \text{if } \epsilon_L < \frac{1.6 \cdot M_{pL}}{V_{pL}} \\ "\text{Long Link}" & \text{if } \epsilon_L > \frac{2.6 \cdot M_{pL}}{V_{pL}} \\ "\text{Intermediate Link}" & \text{otherwise} \end{cases} \]

\[ \gamma_L := \left( 1 + \frac{b_{\text{SRC}}}{\epsilon_L} \right) \theta_t = 0.08 \]
A target SRC/link/RC added stiffness

\[ k_{SRC,t} := (\omega_{k,t} - 1) \cdot k_{oExF} = 207,331 \text{kip in} \]

Approximate Stiffness Calculation only including flexibility of RC (use to size RC, more complete stiffness calc below)

\[ k_{SRC} = \frac{3}{2} \cdot \frac{E_s A_{RC}}{h_l} \left( \frac{h_{SRC} + e_L}{H_N} \right)^2 \sum_i \left( C_i H_i \right)^4 \]

Required RC x-sectional area based on stiffness equation above -increase required by factor of 1.5 since sizing based on stiffness calc ignoring other flexibilities

\[ A_{RC, req} := \frac{2}{3} \cdot \frac{h_l k_{SRC,t} H_N \left( \sum_i \left( C_i H_i \right)^4 \right)}{E_s \left( h_{SRC} + e_L \right)^2} = 5.154 \text{in}^2 \]

OK as an initial design size but sizing for strength and then increasing ARC for stiffness might be better approach

Selected RC member

\[ RC_{Section} := "W14\times455" \]

\[ A_{RC} := 134 \text{in}^2 \]

\[ P_{RC} := \Omega L N V_{PL} = 2.489 \times 10^3 \text{kip} \]

\[ M_{RC} := \Omega L V_{PL} e_L = 3.318 \times 10^3 \text{kip ft} \]

\[ C_b = 1.67 \]

\[ p := 0.182 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.253 \times 10^{-3} \text{(kip ft)}^{-1} \]

\[ DC := \begin{cases} \left( p \left| P_{RC} \right| + \frac{b_x \left| M_{RC} \right|}{C_b} \right) & \text{if } p \left| P_{RC} \right| > 0.2 = 0.956 \\ \left[ \frac{1}{2} p \left| P_{RC} \right| + \frac{9}{8} (b_x \left| M_{RC} \right|) \right] & \text{otherwise} \end{cases} \]

Compare target stiffness factor to calculated ratio of Rehab stiffness to existing -design iteration converges when these are reasonably close

\[ \frac{k_{Rehab}}{k_{oExF}} = 1.723 \quad \omega_{k,t} = 1.25 \]

Final critical parameters for model

\[ RC_{Section} = "W14\times455" \]

\[ Link_{Section} = "W30\times148" \]

\[ e_L = 48\text{ in} \quad V_{PL} = 553\text{ kip} \]

\[ I_{L, eq} := I_l \left( \frac{1}{1 + C_1} \right) = 1.129 \times 10^3 \text{ in}^4 \]

\[ I_L = 6.68 \times 10^3 \text{ in}^4 \]

A2.4-22
A2.5 3SCBF SRC Design Inter-Story Shear Force

OBJECTIVE
The objective of this calculation is to calculate the forces for design of the SRC. The design forces include the plastic mechanism interaction forces considering some degree of non-uniform LFRS degradation and initial differences in nominal frame strength. Additionally, higher mode force effects are added to the plastic mechanism SRC force demands based on elastic higher mode dynamics.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration are used here. Assuming a SRC is added to the frame, the soft-story mechanism will not form and plastic deformations will be limited to the braces and/or beam.

METHODOLOGY
The calculation first evaluates the controlling frame mechanism considering the compression brace at the expected buckling strength (EB) and then at the expected post-buckling (EPB) strength. The expected tension brace force is calculated considering the compression brace at EB or EPB which may result in the tension brace reaching its expected tension strength or the force to develop the beam “pull-down” mechanism.

The plastic base shear capacity of the frame can then be calculated using plastic analysis concepts assuming an inverted triangular load distribution. The differential shear demand can then be calculated by assuming degradation of a single story’s compression brace to EB while all others are at EB. Assuming all at EB or EPB results in little differential shear. The demands on BF columns are calculated simply to determine if the existing columns are adequate. Considering a single story at EPB and all other at EB results in a significant bending demand on the BF columns (considering no SRC). Once the SRC is added, the differential shear is assumed to be distributed among the SRC, braced frame columns, gravity column, and reaction column based on their relative interstory shear stiffness.

The final part of the calculation considers the portion of the differential shear taken to the SRC based on the relative stiffness of the braced frame columns, gravity column, reaction column, and SRC. Once the differential shear to develop the plastic mechanism is known (including the effects of non-uniform degradation and differences in nominal strength), the higher mode forces are added to the mechanism forces. The higher mode forces are combined with a SRSS modal combination rule and added directly to the plastic mechanism differential shear forces. The direct addition of the plastic mechanism forces and higher mode forces is likely appropriate since the structure responds within the plastic deformation “regime” during a significant amount of its cyclic hysteretic response.

RESULTS/CONCLUSIONS
Input

Earthquake Hazard
Haz := "MCE"

Link Connectivity
Links := "FixFix"

Gravity Column
GravCol := "None"

Contribution ("None," "Weak," or "Strong")

Stories, Floors, Modes
N := 3 i := 1, 2..N j := 1, 2..N
m := 1, 2..N n := 1, 2..N

Geometry
L := 30ft bSRC := 12ft

\[ C_i := h_i := \]
\[ H_i := \sum_{j=1}^{i} h_j \]
\[ \theta_{bi} := \tan \left( \frac{(2 \cdot h_i)}{L} \right) \]
\[ \theta_b := \tan \left( \frac{(2 \cdot 40.914)}{40.914} \right) \]
\[ e_{bi} := \]
\[ L_{bc_i} := \]

BF Sections and Material Properties

\[ Z_{bi} := \]
\[ Zc := 147\text{in}^3 \]
\[ F_{ye} := 55\text{ksi} \]

\[ I_c := 833\text{in}^4 \]
\[ E_s := 29000\text{ksi} \]

\[ A_{ci} := 28.2\text{in}^2 \]
\[ M_{pc} := Z_c \cdot F_{ye} \]
\[ M_{pc} = 673.75\text{-kip-ft} \]

Gravity Column

\[ A_{GC} := \begin{cases} 
143.5\text{in}^2 & \text{if GravCol = "Strong"} \\
143.5\text{in}^2 & \text{if GravCol = "Weak"} \\
0\text{in}^2 & \text{if GravCol = "None"}
\end{cases} \]

\[ I_{GC} := \begin{cases} 
4782.5\text{in}^4 & \text{if GravCol = "Strong"} \\
1033.2\text{in}^4 & \text{if GravCol = "Weak"} \\
0\text{in}^4 & \text{if GravCol = "None"}
\end{cases} \]
\[ A_{d_i} := \begin{array}{c} 56.8in^2 \\ 46.7in^2 \\ 35.3in^2 \end{array} \quad \begin{array}{c} 68.5in^2 \\ 56.8in^2 \\ 26.5in^2 \end{array} \]

\[ \theta_{d_i} := \tan \left( \frac{h_i}{b_{SRC}} \right) \]

\[ \theta_d = \begin{pmatrix} 47.291 \\ 47.291 \end{pmatrix} \quad \text{deg} \]

Column boundary condition (for stiffness calculation below)

\[ B_{RC} := \begin{cases} 1 & \text{if Links = "FixFix"} \\ 4 & \text{otherwise} \end{cases} \]

Average of SRC column areas and moments of inertia

\[ \begin{align*} A_{SC} & := 40.75in^2 \\ I_{SC} & := 5265in^4 \end{align*} \]

Reaction Column Properties

\[ \begin{align*} A_{RC} & := 101in^2 \\ I_{RC} & := 4900in^4 \end{align*} \]

Link Properties: size considered to achieve target drift (see design and prediction calculations)

\[ V_{pL} := \begin{cases} 480\text{kip} & \text{if Links = "FixFix"} \\ 0\text{kip} & \text{otherwise} \end{cases} \]

\[ V_{pL} = 480\text{-kip} \]

\[ e_{L} := 30\text{in} \]

Mass and Gravity Loads

\[ m_{f_i} := \frac{6.7 \text{kip sec}^2}{4} \]

\[ \sum_{i=1}^{N} \left( m_{f_i} \cdot g \right) = 1.94 \times 10^3\text{-kip} \]

\[ W_{f_i} := m_{f_i} \cdot g \]

\[ W_{f} = \begin{pmatrix} 646.698 \\ 646.698 \end{pmatrix} \text{-kip} \]

\[ P_{D_i} := \begin{array}{c} 82\text{kip} \\ 82\text{kip} \\ 77\text{kip} \end{array} \]
Expected buckling capacity of compression brace

\[ F_{bEC_i} := \]

<table>
<thead>
<tr>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>693.54 kip</td>
</tr>
<tr>
<td>693.54 kip</td>
</tr>
<tr>
<td>372.4 kip</td>
</tr>
</tbody>
</table>

Expected post-buckling capacity of compression brace

\[ F_{bEPB_i} := \]

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</tr>
</thead>
<tbody>
<tr>
<td>208.06 kip</td>
</tr>
<tr>
<td>208.06 kip</td>
</tr>
<tr>
<td>111.72 kip</td>
</tr>
</tbody>
</table>

Expected tension capacity of tension brace

\[ F_{bET_i} := \]

<table>
<thead>
<tr>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>869.4 kip</td>
</tr>
<tr>
<td>869.4 kip</td>
</tr>
<tr>
<td>488.2 kip</td>
</tr>
</tbody>
</table>

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.

\[
F_{bTEB_i} := \min \left( F_{bET_i}, F_{bEC_i} + \frac{4 \cdot Z_{bi} \cdot F_{ye}}{L_{be} \cdot \sin(\theta_{bi})} \right) \quad F_{bTEB} = \begin{cases} 869.4 \text{ kip} & \\
869.4 \text{ kip} \end{cases}
\]

Mech\(_{EB_i}\) =

| "Brace" if \( F_{bTEB_i} = F_{bET_i} \) | "Brace“ |
| "Beam” otherwise                 | “Brace“ |

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.

\[
F_{bTEPB_i} := \min \left( F_{bET_i}, F_{bEPB_i} + \frac{4 \cdot Z_{bi} \cdot F_{ye}}{L_{be} \cdot \sin(\theta_{bi})} \right) \quad F_{bTEPB} = \begin{cases} 869.4 \text{ kip} & \\
869.4 \text{ kip} \end{cases}
\]

Mech\(_{EPB_i}\) =

| "Brace” if \( F_{bTEPB_i} = F_{bET_i} \) | "Brace“ |
| "Beam” otherwise                    | “Brace“ |
Relative shear stiffness calculations for determining distribution of brace interstory shear demands to SRC, RC, GC, and BF columns.

**SRC**

\[
k_{s1} = \frac{3 \cdot E_s \cdot l_{SC}}{h_1^3} + \frac{E_s \cdot A_d}{\sqrt{b_{SRC}^2 + h_1^2}} \left( \cos^2 \left( \frac{\theta}{2} \right) \right)^{\frac{1}{2}} \quad k_{s1} = 3.69 \times 10^3 \text{ kip in}
\]

\[
k_{s2} = \frac{12 \cdot E_s \cdot l_{SC}}{h_2^3} + \frac{E_s \cdot A_d}{\sqrt{b_{SRC}^2 + h_2^2}} \left( \cos^2 \left( \frac{\theta}{2} \right) \right)^{\frac{1}{2}} \quad k_{s2} = 3.417 \times 10^3 \text{ kip in}
\]

\[
k_{s3} = \frac{12 \cdot E_s \cdot l_{SC}}{B_{RC} h_3^3} + \frac{E_s \cdot A_d}{\sqrt{b_{SRC}^2 + h_3^2}} \left( \cos^2 \left( \frac{\theta}{2} \right) \right)^{\frac{1}{2}} \quad k_{s3} = 2.701 \times 10^3 \text{ kip in}
\]

**Reaction Column**

\[
k_{r1} = \frac{3 \cdot E_s \cdot l_{RC}}{h_1^3} = 112.29 \text{ kip in} \quad k_{r2} = \frac{12 \cdot E_s \cdot l_{RC}}{h_2^3} = 449.16 \text{ kip in} \quad k_{r3} = \frac{12 \cdot E_s \cdot l_{RC}}{B_{RC} h_3^3} = 449.16 \text{ kip in}
\]

**Gravity Column**

\[
k_{g1} = \frac{3 \cdot E_s \cdot l_{GC}}{h_1^3} = 0 \text{ kip in} \quad k_{g2} = \frac{12 \cdot E_s \cdot l_{GC}}{h_2^3} = 0 \text{ kip in} \quad k_{g3} = \frac{3 \cdot E_s \cdot l_{GC}}{h_3^3} = 0 \text{ kip in}
\]

**Braced Frame Columns**

\[
k_{b1} = \frac{12 \cdot E_s \cdot l_{BC}}{h_1^3} = 152.715 \text{ kip in} \quad k_{b2} = \frac{12 \cdot E_s \cdot l_{BC}}{h_2^3} = 152.715 \text{ kip in} \quad k_{b3} = \frac{3 \cdot E_s \cdot l_{BC}}{h_3^3} = 38.179 \text{ kip in}
\]

**Relative Contributions**

\[
k_{i} = k_{s1} + k_{s2} + k_{s3} + k_{b1} + k_{g1} + k_{g2} + k_{g3} + k_{b2} + k_{b3}
\]

\[
k_i = \begin{bmatrix} 3.955 \times 10^3 \\ 4.019 \times 10^3 \\ 3.188 \times 10^3 \end{bmatrix} \text{ kip in}
\]

\[
P_s = \begin{bmatrix} 0.933 \\ 0.85 \\ 0.847 \end{bmatrix} \quad P_r = \begin{bmatrix} 0.028 \\ 0.112 \\ 0.141 \end{bmatrix} \quad P_g = \begin{bmatrix} 0 \end{bmatrix} \quad P_b = \begin{bmatrix} 0.039 \\ 0.038 \\ 0.012 \end{bmatrix}
\]

A2.5-5
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected buckling strengths, flexural yielding at base of columns, and yielding links with strength V_pL at each floor elevation

Brace Internal Work

\[ W_{ib,EB} := \sum_{i=1}^{N} \left[ F_{b,ET_i} + F_{b,EC_i} \cos(\theta_{b_i}) \right] h_i \]

Column Internal Work

\[ W_{ic,EB} := 2M_{pc} \]

Link Internal Work

\[ W_{il,EB} := N \cdot V_pL \left( h_{SRC} + \epsilon_{L} \right) \]

External Work

\[ W_{E,EB} := \sum_{i=1}^{N} (H_i \cdot C_i) \]

Internal Work = External Work

\[ V_p := \frac{W_{ib,EB} + W_{ic,EB} + W_{il,EB}}{W_{E,EB}} \]

Plastic base shear of frame with all comp. braces at Expected Buckling capacity

\[ V_p = 2.024 \times 10^3 \text{-kip} \]

Differential Shear Demand

\[ \Delta V_i := \sum_{j=1}^{N} \left( C_j \cdot V_p \right) - \left( F_{b,EC_i} + F_{b,TEB_{ij}} \cos(\theta_{b_i}) \right) \]

\[ \Delta V_i = \begin{array}{c} 842.766 \text{-kip} \\ 505.456 \\ 361.585 \end{array} \]

\[ F_{BH_i} := \left( F_{b,EC_i} + F_{b,TEB_{ij}} \cos(\theta_{b_i}) \right) \]

\[ F_BH = \begin{pmatrix} 1.181 \times 10^3 \\ 1.181 \times 10^3 \\ 650.346 \end{pmatrix} \text{kip} \]

\[ F_{BV_i} := \left( -F_{b,EC_i} + F_{b,TEB_{ij}} \sin(\theta_{b_i}) \right) \]

\[ F_BV = \begin{pmatrix} 115.176 \\ 115.176 \\ 75.841 \end{pmatrix} \text{kip} \]
\[
A_n := \sum_{i=n}^{N} p_i + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \begin{cases} \sum_{i=n+1}^{N} \left( F_{BE_i} \cdot \sin(\theta_i) \right) & \text{if } n < N \\ 0 \text{kip} & \text{otherwise} \end{cases}
\]

\[
M_{T_n} := \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i \cdot V_i \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( p_i \cdot L_i + F_{BV_i} \cdot \frac{L_i}{2} - A_n \cdot L \right)
\]

\[
M_{B_n} = M_{T_n} + \frac{\Delta V_n}{2} \cdot h_n
\]

\[
M_{T} = \begin{pmatrix}
5.636 \times 10^3 \\
2.35 \times 10^3 \\
0
\end{pmatrix} \text{kip-ft}
\]

\[
M_{B} = \begin{pmatrix}
1.111 \times 10^4 \\
5.636 \times 10^3 \\
2.35 \times 10^3
\end{pmatrix} \text{kip-ft}
\]

\[
M_{\text{max}} := M_{\text{max}} \leftarrow \begin{cases} M_{T_n} & \text{if } n = 1 \\ M_{T_n} \text{ if } (n > 1) \land \left( |M_{T_n}| > |M_{B_n}| \right) \\ M_{B_n} & \text{otherwise} \end{cases}
\]

\[
M_{\text{max}} = \begin{cases} 5.636 \times 10^3 \text{-kip-ft} \\
1.111 \times 10^4 \text{-kip-ft} \\
5.636 \times 10^3 \text{-kip-ft}
\end{cases}
\]

\[
A_{\text{max}} := A_{\text{max}} \leftarrow \begin{cases} A_{n} & \text{if } |A_{n}| > A_{\text{max}} \\ A_{\text{max}} & \text{otherwise} \end{cases}
\]

\[
A_{\text{max}} = 1.092 \times 10^3 \text{-kip}
\]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected buckling strengths, flexural yielding at base of columns, and yielding links with strength VpL at each floor elevation

Brace Internal Work
\[ W_{iB,EB} = \sum_{i=1}^{N} \left[ \left( F_{EB,i} + F_{EBP,i} \right) \cos(\theta_{b,i}) h_i \right] \]

Column Internal Work
\[ W_{iC,EB} = 2M_{pc} \]

Link Internal Work
\[ W_{iLEP} = N \cdot V_{pl} \left( b_{SRP} + c_L \right) \]

External Work
\[ W_{E,EB} = \sum_{i=1}^{N} \left( H_i - C_i \right) \]

Internal Work = External Work
\[ V_p = \frac{W_{iB,EB} + W_{iC,EB} + W_{iLEP}}{W_{E,EB}} \]

Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity
\[ V_p = 1.625 \times 10^3 \text{-kip} \]

Single Column Shear Demand
\[ \Delta V_i = \sum_{j=1}^{N} \left( C_j \cdot V_p - \left( F_{EBP,i} + F_{EB,TEP,i} \cos(\theta_{b,i}) \right) \right) \]
\[ \Delta V_i = \begin{array}{c}
810.751 \\
539.922 \\
359.135 \\
\end{array} \text{-kip} \]

\[ F_{BH,i} = \left( F_{EBP,i} + F_{EB,TEP,i} \cos(\theta_{b,i}) \right) \]
\[ F_{BH} = \begin{array}{c}
814.225 \\
453.353 \\
\end{array} \text{-kip} \]

\[ F_{BV,i} = \left( -F_{EBP,i} + F_{EB,TEP,i} \sin(\theta_{b,i}) \right) \]
\[ F_{BV} = \begin{array}{c}
433.132 \\
246.568 \\
\end{array} \text{-kip} \]
\[ A_n = \sum_{i=n}^{N} P_{Di} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \sum_{i=n+1}^{N} \left( F_{bEPB_i} \sin(\theta_{b_i}) \right) \text{ if } n < N \]

\[ A_n = \begin{cases} 1.007 \times 10^3 \text{ kip} \\ 572.019 \text{ kip} \\ 200.284 \text{ kip} \end{cases} \]

Moment on BF column with no SRC (all DV to columns) - use to check if existing columns are adequate.

\[ M_{T_n} := \frac{1}{2} \left[ \sum_{i=n+1}^{N} \left[ C_i \cdot V_p \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{Di} \cdot L + F_{BV_n} \cdot \frac{L}{2} - A_n \cdot L \right) \right] \text{ if } n < N \]

\[ M_{B_n} := M_{T_n} + \frac{\Delta V_n}{2} h_n \]

\[ M_T = \begin{cases} 5.844 \times 10^3 \text{ kip-ft} \\ 2.334 \times 10^3 \text{ kip-ft} \\ 0 \end{cases} \]

\[ M_B = \begin{cases} 1.111 \times 10^4 \text{ kip-ft} \\ 5.844 \times 10^3 \text{ kip-ft} \\ 2.334 \times 10^3 \text{ kip-ft} \end{cases} \]

\[ M_{\max} := \left\{ \begin{array}{ll} M_{T_n} \text{ if } n = 1 \\ M_{T_n} \text{ if } (n > 1) \land (M_{T_n} > M_{B_n}) \\ M_{B_n} \text{ otherwise} \end{array} \right. \]

\[ M_{\max} \leftarrow M_{\max1} \text{ if } M_{\max1} > M_{\max} \]

\[ M_{\max} \text{ otherwise} \]

\[ A_{\max} := \left\{ \begin{array}{ll} A_n \text{ if } A_n > A_{\max} \\ A_{\max} \text{ otherwise} \end{array} \right. \]

\[ A_{\max} = 1.007 \times 10^3 \text{ kip} \]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all tension braces at expected tensile strength, one compression brace at expected post-buckling strength, all other compression braces at expected buckling strength, flexural yielding at base of columns, and yielding links with strength VpL at each floor elevation.

Brace Internal Work
\[ W_{\text{lb,nPB}} := \sum_{i=1}^{N} \left( \begin{array}{l} \left( F_{bET_{i}} + F_{bEPB_{i}} \right) \cos \left( \theta_{b_{i}} \right) h_{i} \text{ if } i \neq \text{nPB} \\ \left( F_{bET_{i}} + F_{bEC_{i}} \right) \cos \left( \theta_{b_{i}} \right) h_{i} \text{ otherwise} \end{array} \right) \]

Column Internal Work
\[ W_{\text{lc,nPB}} := 2M_{pc} \]

Link Internal Work
\[ W_{\text{ll,nPB}} := N V_{pL} (b_{SRC} + c_{L}) \]

External Work
\[ W_{\text{E,nPB}} := \sum_{i=1}^{N} (H_{i} C_{i}) \]

Internal Work = External Work
\[ V_{p} := \frac{W_{\text{lb,nPB}} + W_{\text{lc,nPB}} + W_{\text{ll,nPB}}}{W_{\text{E,nPB}}} \]

Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity
\[ V_{p} = 1.867 \times 10^{3} \text{ kip} \]

\[ F_{BH_{i}} := \left( \begin{array}{l} \left( F_{bEPB_{i}} + F_{bTEPB_{i}} \right) \cos \left( \theta_{b_{i}} \right) \text{ if } i \neq \text{nPB} \\ \left( F_{bEC_{i}} + F_{bTEB_{i}} \right) \cos \left( \theta_{b_{i}} \right) \text{ otherwise} \end{array} \right) \]

\[ F_{BV_{i}} := \left( \begin{array}{l} \left( -F_{bEPB_{i}} + F_{bTEPB_{i}} \right) \sin \left( \theta_{b_{i}} \right) \text{ if } i \neq \text{nPB} \\ \left( -F_{bEC_{i}} + F_{bTEB_{i}} \right) \sin \left( \theta_{b_{i}} \right) \text{ otherwise} \end{array} \right) \]

Single Column Shear Demand
\[ \Delta V_{i} := \sum_{j=1}^{N} \left( C_{j} V_{p} \right) - F_{BH_{i}} \]

\[ \Delta V_{i} = \begin{array}{c} 1.052 \times 10^{3} \text{ kip} \\ 374.43 \\ 282.97 \end{array} \]
\[
A_n = \sum_{i=n}^{N} P_i + \sum_{i=n}^{N} \frac{F_{BV,i}}{2} + \sum_{i=n+1}^{N} \left( F_{BP,i} \sin(\theta_i) \right) \text{ if } n < N \\
(0 \text{ kip) otherwise}
\]

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\[
M_T^n = \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i V_p \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D,i} L/2 + F_{BV,i} V_{n} L - A_n L \right) \text{ if } n < N \\
\sum_{i=n}^{N} \left( P_{D,i} L/2 + F_{BV,i} V_{n} L - A_n L \right) \text{ otherwise}
\]

\[
M_B^n = M_T^n + \frac{\Delta V_n n}{2} \\
M_T = \begin{pmatrix} 1.16 \times 10^4 \\ 4.4 \times 10^3 \end{pmatrix} \text{kip-ft} \\
M_B = \begin{pmatrix} 1.844 \times 10^4 \\ 6.834 \times 10^3 \end{pmatrix} \text{kip-ft}
\]

\[
M_{max} = \left\{ \begin{array}{ll}
M_{max} & \text{if } n = 1 \\
M_T^n & \text{if } (n > 1) \land M_T^n > M_B^n \\
M_B^n & \text{otherwise}
\end{array} \right.
\]

\[
A_{max} = \left\{ \begin{array}{ll}
A_n & \text{if } A_n > A_{max} \\
A_{max} & \text{otherwise}
\end{array} \right.
\]

\[
\text{A}\_{max} = 762.508 \text{kip}
\]

\[
\text{M}\_\text{max} = 1.16 \times 10^4 \text{kip-ft}
\]

\[
\text{A}\_{max} = 762.508 \text{kip}
\]
Add higher mode forces to differential shear forces for design of the SRC

\[ \phi_1 := \begin{pmatrix}
0.1633 \\
0.3932 \\
0.644 \\
0.1872 \\
0.4031 \\
0.6313 \\
\end{pmatrix} \text{ if } \text{Links} = \"FixFix\"
\]

\[ \phi_2 := \begin{pmatrix}
0.5128 \\
0.4253 \\
-0.3896 \\
0.4830 \\
0.4321 \\
-0.4192 \\
\end{pmatrix} \text{ if } \text{Links} = \"PinPin\"
\]

\[ \phi_3 := \begin{pmatrix}
0.5528 \\
-0.5099 \\
0.1712 \\
0.5718 \\
-0.4963 \\
0.1473 \\
\end{pmatrix} \text{ if } \text{Links} = \"PinPin\"
\]

\[ T := \begin{pmatrix}
0.2952 \\
0.1148 \text{ sec} \\
0.0748 \\
0.3822 \\
0.1202 \text{ sec} \\
0.0762 \\
\end{pmatrix} \text{ if } \text{Links} = \"PinPin\" \]
Design Spectral Acceleration

\[ S_{D1} := \begin{cases} 1.2g & \text{if Haz = "MCE"} \\ 1.6g & \text{if Haz = "NF"} \end{cases} \]

\[ S_{DS} := \begin{cases} 1.6g & \text{if Haz = "MCE"} \\ 1.4g & \text{if Haz = "NF"} \end{cases} \]

\[ T_s := \frac{S_{D1}}{S_{DS}} \]

\[ T_0 := 0.2 \times T_s \]

Coefficient to adjust 5% damped spectral acceleration values to 2% damped with \( T > T_0 \) (ASCE 7-10)

\[ B_{d,T0} := 0.8 \]

\[ B_{d,Ex} := \begin{cases} 1.0 & \text{if } T_m < T_0 \\ 1.0 - \frac{0.9}{T_s/T_i} & \text{if } T_0 < T_m < T_i \end{cases} \]

\[ B_{d,Ex} = \left( \begin{array}{c} 0.921 \\ 0.98 \end{array} \right) \]

Effective Modal weights

\[ W_m := \left( \sum_{i=1}^{N} \left[ W_{f_i} \left[ \phi_{m,i} \right] \right]^2 \right)^{1/2} \]

\[ W := \left( \sum_{i=1}^{N} W_{f_i} \left[ \phi_{m,i} \right]^2 \right)^{1/2} \]

\[ W = \left( \begin{array}{c} 1.564 \times 10^3 \\ 49.83 \end{array} \right) \]

Modal Participation Factors

\[ r_m := \frac{W_m}{\sum_{i=1}^{N} \left[ W_{f_i} \left[ \phi_{m,i} \right] \right]} \]

\[ r := \frac{2.014}{0.36} \]

Response Coefficient of each mode

\[ C_{s,m} := \begin{cases} \frac{S_{DS}}{g B_{d,Ex_m}} \left( 0.4 + 0.6 \frac{T_m}{T_0} \right) & \text{if } T_m < T_0 \\ \frac{S_{DS}}{g B_{d,Ex_m}} & \text{if } T_0 < T_m < T_i \\ \frac{T_m}{S_{D1}} & \text{otherwise} \end{cases} \]

\[ C_s = \left( \begin{array}{c} 1.737 \\ 1.141 \end{array} \right) \]

Modal base shear

\[ V_m := C_{s,m} \cdot W_m \]

\[ V = \left( \begin{array}{c} 2.716 \times 10^3 \\ 56.881 \end{array} \right) \text{kip} \]
Elastic System Forces

Story shear forces of each mode

\[ F_{i,m} := W_{f_i} \left( \phi_{m} \right) \frac{f_{m}^{r}}{W_{m}} \]

- 1st mode not relevant

\[ V_{SRC,m_{j,m}} := \begin{cases} \Delta V_{j} & \text{if } m = 1 \\ \sum_{i=j}^{3} F_{i,m} & \text{otherwise} \end{cases} \]

\[ V_{SRC,m} = \begin{pmatrix} 369.424 \\ 889.514 \\ 1457 \times 10^3 \end{pmatrix} \text{kip} \]

\[ V_{SRC,m} = \begin{pmatrix} 433.077 \\ 359.181 \\ -329.031 \end{pmatrix} \text{kip} \]

\[ V_{SRC,m} = \begin{pmatrix} 146.865 \\ -135.467 \\ 45.483 \end{pmatrix} \text{kip} \]

Links = "FixFix"

Haz = "MCE"

GravCol = "None"

\[ V_{SRC,j} := \sum_{i=j}^{3} \left( \frac{V_{j,m}}{f_{m}} \right) \]

\[ V_{SRC} := \begin{pmatrix} 1.052 \times 10^3 \\ 374.43 \\ 282.97 \end{pmatrix} \text{kip} \]

\[ V_{SRC} := \begin{pmatrix} 463.227 \\ 30.15 \\ -89.984 \end{pmatrix} \text{kip} \]

\[ V_{SRC} := \begin{pmatrix} 56.881 \\ 282.97 \\ 45.483 \end{pmatrix} \text{kip} \]

\[ nPB = 1 \]

\[ nPB = 2 \]

\[ nPB = 3 \]

Pin-Pin Links, MCE Hazard
\[ V_{PM,nPB1} := \begin{pmatrix} 722.895 \\ 141.688 \\ 253.597 \end{pmatrix} \text{kip} \]
\[ V_{PM,nPB2} := \begin{pmatrix} 380.604 \\ 453.623 \\ 253.597 \end{pmatrix} \text{kip} \]
\[ V_{PM,nPB3} := \begin{pmatrix} 193.274 \\ 448.531 \\ 472.886 \end{pmatrix} \text{kip} \]

Pin-Pin Links, NF Hazard
\[ V_{PN,nPB1} := \begin{pmatrix} 611.176 \\ 50.922 \\ 161.322 \end{pmatrix} \text{kip} \]
\[ V_{PN,nPB2} := \begin{pmatrix} 268.886 \\ 362.856 \\ 161.322 \end{pmatrix} \text{kip} \]
\[ V_{PN,nPB3} := \begin{pmatrix} 102.508 \\ 336.813 \\ 380.611 \end{pmatrix} \text{kip} \]

Fix-Fix Links, MCE Hazard
\[ V_{FM,nPB1} := \begin{pmatrix} 1.409 \times 10^3 \\ 644.754 \\ 521.107 \end{pmatrix} \text{kip} \]
\[ V_{FM,nPB2} := \begin{pmatrix} 1.066 \times 10^3 \\ 956.688 \\ 521.107 \end{pmatrix} \text{kip} \]
\[ V_{FM,nPB3} := \begin{pmatrix} 1.134 \times 10^3 \\ 696.34 \\ 718.828 \end{pmatrix} \text{kip} \]

Fix-Fix Links, NF Hazard
\[ V_{FN,nPB1} := \begin{pmatrix} 1.285 \times 10^3 \\ 550.416 \\ 438.728 \end{pmatrix} \text{kip} \]
\[ V_{FN,nPB2} := \begin{pmatrix} 942.626 \\ 862.35 \\ 438.728 \end{pmatrix} \text{kip} \]
\[ V_{FN,nPB3} := \begin{pmatrix} 1.011 \times 10^3 \\ 602.002 \\ 636.449 \end{pmatrix} \text{kip} \]
peak axial force on links

Pin-Pin Links, MCE
Hazard

\[ \Lambda_{PM.nPB1} := \begin{cases} \sum_{k=j}^{N} \Lambda_{PM1_k} & \text{if } j = N \\ \Lambda_{PM1} & \text{otherwise} \end{cases} \]

\[ \Lambda_{PM.nPB1} = \begin{pmatrix} 581.207 \\ -111.909 \cdot \text{kip} \\ 253.597 \end{pmatrix} \]

\[ \Lambda_{PM.nPB2} := \begin{cases} \sum_{k=j}^{N} \Lambda_{PM1_k} & \text{if } j = N \\ \Lambda_{PM1} & \text{otherwise} \end{cases} \]

\[ \Lambda_{PM.nPB2} = \begin{pmatrix} -73.019 \\ 200.026 \cdot \text{kip} \\ 253.597 \end{pmatrix} \]

\[ \Lambda_{PM.nPB3} := \begin{cases} \sum_{k=j}^{N} \Lambda_{PM1_k} & \text{if } j = N \\ \Lambda_{PM1} & \text{otherwise} \end{cases} \]

\[ \Lambda_{PM.nPB3} = \begin{pmatrix} 255.257 \\ -279.612 \cdot \text{kip} \\ 472.886 \end{pmatrix} \]

\[ \Lambda_{PM.max} := \max(\Lambda_{PM.nPB1}, \Lambda_{PM.nPB2}, \Lambda_{PM.nPB3}) \]

\[ \Lambda_{PM.min} := \min(\Lambda_{PM.nPB1}, \Lambda_{PM.nPB2}, \Lambda_{PM.nPB3}) \]

\[ \Lambda_{PM.max} = 581.207 \cdot \text{kip} \]

\[ \Lambda_{PM.min} = -279.612 \cdot \text{kip} \]

\[ \Lambda_{PM} := \max(|\Lambda_{PM}max|, |\Lambda_{PM.min}|) = 581.207 \cdot \text{kip} \]
Pin-Pin Links, NF Hazard

\[ A_{PN,nPB1} := \begin{cases} \text{for } j \in N, N - 1 \ldots 1 \\ \forall j \neq N \\ \forall j = N \\ \text{otherwise} \end{cases} \]

\[ A_{PN1,j} \leftarrow V_{PN,nPB1,j} \]

\[ A_{PN1,j} \leftarrow V_{PN,nPB1,j} - \sum_{k=j}^{N} A_{PN1,k} \]

\[ A_{PN,nPB1} = \begin{cases} 560.254 \text{ kip} \\ -110.4 \text{ kip} \\ 161.322 \text{ kip} \end{cases} \]

\[ A_{PN,nPB2} := \begin{cases} \text{for } j \in N, N - 1 \ldots 1 \\ \forall j \neq N \\ \forall j = N \\ \text{otherwise} \end{cases} \]

\[ A_{PN1,j} \leftarrow V_{PN,nPB2,j} \]

\[ A_{PN1,j} \leftarrow V_{PN,nPB2,j} - \sum_{k=j}^{N} A_{PN1,k} \]

\[ A_{PN,nPB2} = \begin{cases} 201.534 \text{ kip} \\ -93.97 \text{ kip} \\ 161.322 \text{ kip} \end{cases} \]

\[ A_{PN,nPB3} := \begin{cases} \text{for } j \in N, N - 1 \ldots 1 \\ \forall j \neq N \\ \forall j = N \\ \text{otherwise} \end{cases} \]

\[ A_{PN1,j} \leftarrow V_{PN,nPB3,j} \]

\[ A_{PN1,j} \leftarrow V_{PN,nPB3,j} - \sum_{k=j}^{N} A_{PN1,k} \]

\[ A_{PN,nPB3} = \begin{cases} 380.611 \text{ kip} \\ -278.103 \text{ kip} \\ 234.305 \text{ kip} \end{cases} \]

\[ A_{PN,max} := \max\left( A_{PN,nPB1}, A_{PN,nPB2}, A_{PN,nPB3} \right) \]

\[ A_{PN,min} := \min\left( A_{PN,nPB1}, A_{PN,nPB2}, A_{PN,nPB3} \right) \]

\[ A_{PN,max} = 560.254 \text{ kip} \]

\[ A_{PN,min} = -278.103 \text{ kip} \]

\[ A_{PN} := \max\left( \left| A_{PN,max} \right|, \left| A_{PN,min} \right| \right) = 560.254 \text{ kip} \]
Fix-Fix Links, MCE Hazard

\[ \lambda_{FM,nPB1} \overset{\text{for } j \in \{N, N-1, \ldots, 1\}}{=} \begin{cases} \lambda_{FM1} & \text{if } j = N \\ \lambda_{FM1} & \text{if } j = N \\ \lambda_{FM1} & \text{otherwise} \\ \lambda_{FM1} & \text{otherwise} \end{cases} \]

\[ \lambda_{FM,nPB1} = \begin{cases} 764.246 \text{-kip} \\ 123.647 \text{-kip} \\ 521.107 \text{-kip} \end{cases} \]

\[ \lambda_{FM,nPB2} \overset{\text{for } j \in \{N, N-1, \ldots, 1\}}{=} \begin{cases} \lambda_{FM1} & \text{if } j = N \\ \lambda_{FM1} & \text{if } j = N \\ \lambda_{FM1} & \text{otherwise} \\ \lambda_{FM1} & \text{otherwise} \end{cases} \]

\[ \lambda_{FM,nPB2} = \begin{cases} 109.312 \text{-kip} \\ 435.581 \text{-kip} \\ 521.107 \text{-kip} \end{cases} \]

\[ \lambda_{FM,nPB3} \overset{\text{for } j \in \{N, N-1, \ldots, 1\}}{=} \begin{cases} \lambda_{FM1} & \text{if } j = N \\ \lambda_{FM1} & \text{if } j = N \\ \lambda_{FM1} & \text{otherwise} \\ \lambda_{FM1} & \text{otherwise} \end{cases} \]

\[ \lambda_{FM,nPB3} = \begin{cases} 437.66 \text{-kip} \\ -22.488 \text{-kip} \\ 718.828 \text{-kip} \end{cases} \]

\[ \lambda_{FM} := \max(\lambda_{FM,nPB1} \cdot \lambda_{FM,nPB2} \cdot \lambda_{FM,nPB3}) \]

\[ \lambda_{FM} := \min(\lambda_{FM,nPB1} \cdot \lambda_{FM,nPB2} \cdot \lambda_{FM,nPB3}) \]

\[ \lambda_{FM} = 764.246 \text{-kip} \]

\[ \lambda_{FM} = -22.488 \text{-kip} \]

\[ \lambda_{FM} := \max(\lambda_{FM} \cdot \lambda_{FM}) = 764.246 \text{-kip} \]
Fix-Fix Links, NF Hazard

\[ A_{\text{FN},nPB1} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ A_{\text{FN1}} \leftarrow \begin{cases} V_{\text{FN,nPB1}}_j & \text{if } j = N \\ V_{\text{FN,nPB1}}_j - \sum_{k=j}^{N} A_{\text{FN1}} & \text{otherwise} \end{cases} \]
\[ A_{\text{FN,nPB1}} = \begin{pmatrix} \frac{734.584}{111.688} \\ \frac{438.728}{438.728} \end{pmatrix} \]

\[ A_{\text{FN,nPB2}} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ A_{\text{FN1}} \leftarrow \begin{cases} V_{\text{FN,nPB2}}_j & \text{if } j = N \\ V_{\text{FN,nPB2}}_j - \sum_{k=j}^{N} A_{\text{FN1}} & \text{otherwise} \end{cases} \]
\[ A_{\text{FN,nPB2}} = \begin{pmatrix} \frac{80.276}{423.622} \\ \frac{438.728}{438.728} \end{pmatrix} \]

\[ A_{\text{FN,nPB3}} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ A_{\text{FN1}} \leftarrow \begin{cases} V_{\text{FN,nPB3}}_j & \text{if } j = N \\ V_{\text{FN,nPB3}}_j - \sum_{k=j}^{N} A_{\text{FN1}} & \text{otherwise} \end{cases} \]
\[ A_{\text{FN,nPB3}} = \begin{pmatrix} \frac{-34.447}{636.449} \end{pmatrix} \]

\[ A_{\text{FN, max}} := \max(A_{\text{FN,nPB1}}, A_{\text{FN,nPB2}}, A_{\text{FN,nPB3}}) \]
\[ A_{\text{FN, min}} := \min(A_{\text{FN,nPB1}}, A_{\text{FN,nPB2}}, A_{\text{FN,nPB3}}) \]
\[ A_{\text{FN, max}} = 734.584 \text{ kip} \]
\[ A_{\text{FN, min}} = -34.447 \text{ kip} \]
\[ A_{\text{FN}} := \max(|A_{\text{FN, max}}|, |A_{\text{FN, min}}|) = 734.584 \text{ kip} \]
Summary

Earthquake Hazard
Haz = “MCE”

Link Connectivity
Links = “FixFix”

Gravity Frame Contribution
GravCol = "None"

Plastic Shear Strength of Links
$V_{pL} = 480 \text{-kip}$

Controlling Plastic Mechanism at EB
Mech$_{EB} =$

Controlling Plastic Mechanism at EPB
Mech$_{EPB} =$

Story at EPB
$n_{PB} = 1$

Plastic Base Shear
$V_p = 1.867 \times 10^3 \text{-kip}$

Decimal percentage of differential interstory shear taken by SRC (s), reaction column (r), braced frame columns (b), and gravity column (g) on each story based on relative interstory stiffness of each component

Note: >80% of differential shear is taken by SRC

Values for Design

Pin-Pin Links, MCE Hazard

SRC Shear $V_{PM, Max} = \begin{pmatrix} 722.895 \\ 453.623 \end{pmatrix} \text{-kip}$

Link Axial Force $A_{PM} = 581.207 \text{kip}$

Pin-Pin Links, NF Hazard

SRC Shear $V_{PN, Max} = \begin{pmatrix} 611.176 \\ 362.856 \end{pmatrix} \text{-kip}$

Link Axial Force $A_{PN} = 560.254 \text{kip}$

Fix-Fix Links, MCE Hazard

SRC Shear $V_{FM, Max} = \begin{pmatrix} 1.409 \times 10^3 \\ 956.688 \end{pmatrix} \text{-kip}$

Link Axial Force $A_{FM} = 764.246 \text{kip}$

Fix-Fix Links, NF Hazard

SRC Shear $V_{FN, Max} = \begin{pmatrix} 1.285 \times 10^3 \\ 862.35 \end{pmatrix} \text{-kip}$

Link Axial Force $A_{FN} = 734.584 \text{kip}$
A2.6 6SCBF SRC Design Inter-Story Shear Force

6SCBF SRC Strength Design Calculations

OBJECTIVE

The objective of this calculation is to calculate the forces for design of the SRC. The design forces include the plastic mechanism interaction forces considering some degree of non-uniform LFRS degradation and initial differences in nominal frame strength. Additionally, higher mode force effects are added to the plastic mechanism SRC force demands based on elastic higher mode dynamics.

REFERENCES


BACKGROUND AND ASSUMPTIONS

The 6SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration are used here. Assuming a SRC is added to the frame, the soft-story mechanism will not form and plastic deformations will be limited to the braces and/or beam.

METHODOLOGY

The calculation first evaluates the controlling frame mechanism considering the compression brace at the expected buckling strength (EB) and then at the expected post-buckling (EPB) strength. The expected tension brace force is calculated considering the compression brace at EB or EPB which may result in the tension brace reaching its expected tension strength or the force to develop the beam “pull-down” mechanism.

The plastic base shear capacity of the frame can then be calculated using plastic analysis concepts assuming an inverted triangular load distribution. The differential shear demand can then be calculated by assuming degradation of a single story’s compression brace to EPB while all others are at EB. Assuming all at EB or EPB results in little differential shear. The demands on BF columns are calculated simply to determine if the existing columns are adequate. Considering a single story at EPB and all other at EB results in a significant bending demand on the BF columns (considering no SRC). Once the SRC is added, the differential shear is assumed to be distributed among the SRC, braced frame columns, gravity column, and reaction column based on their relative interstory shear stiffness.

The final part of the calculation considers the portion of the differential shear taken to the SRC based on the relative stiffness of the braced frame columns, gravity column, reaction column, and SRC. Once the differential shear to develop the plastic mechanism is known (including the effects of non-uniform degradation and differences in nominal strength), the higher mode forces are added to the mechanism forces. The higher mode forces are combined with a SRSS modal combination rule and added directly to the plastic mechanism differential shear forces. The direct addition of the plastic mechanism forces and higher mode forces is likely appropriate since the structure responds within the plastic deformation “regime” during a significant amount of its cyclic hysteretic response.

RESULTS/CONCLUSIONS
Input

Earthquake Hazard: Haz := "MCE"
Link Connectivity: Links := "FixFix"
Gravity Column Contribution: GravCol := "None"

Stories, Floors, Modes:
N := 6  i := 1, 2..N  j := 1, 2..N
m := 1, 2..N  n := 1, 2..N

Geometry:
L := 30ft  b_{SRC} := 12ft
C_i :=  h_i :=  H_i :=

\begin{array}{|c|c|}
\hline
i & h_i \\
\hline
18 & 18ft \\
303 & 13ft \\
31 & 44ft \\
303 & 13ft \\
44 & 70ft \\
303 & 13ft \\
57 & 83ft \\
303 & 13ft \\
70 & 13ft \\
303 & 13ft \\
83 & 13ft \\
303 & 13ft \\
\hline
\end{array}

\begin{array}{|c|c|}
\hline
i & C_i \\
\hline
18 & 18ft \\
303 & 13ft \\
31 & 44ft \\
303 & 13ft \\
44 & 70ft \\
303 & 13ft \\
57 & 83ft \\
303 & 13ft \\
70 & 13ft \\
303 & 13ft \\
83 & 13ft \\
303 & 13ft \\
\hline
\end{array}

\begin{align*}
\theta_{b1} & := \tan\left(\frac{18ft}{15ft}\right) = 50.194\text{-deg} \\
\theta_b & := \tan\left(\frac{13ft}{15ft}\right) = 40.914\text{-deg} \\
\theta_{b1} & := \begin{cases} 
\theta_{b1} & \text{if } i = 1 \\
\theta_b & \text{otherwise}
\end{cases}
\end{align*}

\begin{array}{|c|c|}
\hline
i & \theta_{b1} \\
\hline
18 & 50.194 \\
303 & 40.914 \\
31 & 40.914 \\
303 & 40.914 \\
44 & 40.914 \\
303 & 40.914 \\
57 & 40.914 \\
303 & 40.914 \\
70 & 40.914 \\
303 & 40.914 \\
83 & 40.914 \\
303 & 40.914 \\
\hline
\end{array}

\begin{array}{|c|c|}
\hline
i & e_{bi} \\
\hline
2.5ft & 25ft \\
2.5ft & 25ft \\
2.5ft & 25ft \\
2.5ft & 25ft \\
2.5ft & 25ft \\
0.625ft & 28.75ft \\
\hline
\end{array}

\begin{array}{|c|c|}
\hline
i & L_{bc_i} \\
\hline
2.5ft & 25ft \\
2.5ft & 25ft \\
2.5ft & 25ft \\
2.5ft & 25ft \\
2.5ft & 25ft \\
0.625ft & 28.75ft \\
\hline
\end{array}
BF Sections and Material Properties

\[ Z_{bh_1} = Z_{c_1} = I_{c_1} = F_{ye} = 55 \text{ksi} \]

\[ E_s = 29000 \text{ksi} \]

\[ M_{pc} = Z_c F_{ys} \]

\[ M_{pc} = 1.788 \times 10^3 \quad \text{kips-ft} \]

\[ 1.788 \times 10^3 \]

\[ 1.073 \times 10^3 \]

Gravity Column

\[ A_{GC} := \begin{cases} 
155.82 & \text{in}^2 \text{ if GravCol } = \text{"Strong"} \\
77.45 \\
77.45 \\
77.45 \\
155.82 \\
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155.82 \\
77.45 \\
77.45 \\
77.45 \\
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SRC, Links, Reaction Column

\[ A_{d_i} = \begin{array}{c}
62.0 \text{in}^2 \\
46.7 \text{in}^2 \\
38.8 \text{in}^2 \\
28.2 \text{in}^2 \\
23.2 \text{in}^2 \\
21.1 \text{in}^2
\end{array} \quad \begin{array}{c}
56.8 \text{in}^2 \\
46.7 \text{in}^2 \\
38.8 \text{in}^2 \\
28.2 \text{in}^2 \\
23.2 \text{in}^2 \\
21.1 \text{in}^2
\end{array} \]

\[ \theta_{d_i} := \tan\left(\frac{h_i}{b_{SRC}}\right) \]

Column boundary condition (for stiffness calculation below)

\[ B_{RC} := \begin{cases} 
1 & \text{if Links = "FixFix"} \\
4 & \text{otherwise}
\end{cases} \]

Average of SRC column areas and moments of inertia

\[ A_{SC} := 56.9 \text{in}^2 \quad I_{SC} := 2400 \text{in}^4 \]

Reaction Column Properties

\[ A_{RC} := 68.5 \text{in}^2 \quad I_{RC} := 3010 \text{in}^4 \]

Link Properties: size considered to achieve target drift (see design and prediction calculations)

\[ V_{PL} := \begin{cases} 
202 \text{kip} & \text{if Links = "FixFix"} \\
0 \text{kip} & \text{otherwise}
\end{cases} \quad V_{PL} = 202 \text{kip} \]

\[ e_{L} := 33 \text{in} \]

Mass and Gravity Loads

\[ m_i := 1.16 \frac{\text{kip} \cdot \text{sec}^2}{\text{in}} \]

\[ W_{f_i} := m_i \cdot g \]

\[ p_{D_i} = \begin{cases} 
82 \text{kip} \\
82 \text{kip} \\
82 \text{kip} \\
82 \text{kip} \\
82 \text{kip} \\
77 \text{kip}
\end{cases} \]

\[ \sum_{i=1}^{N} (m_i \cdot g) = 2.687 \times 10^3 \text{kip} \]

\[ W_f = \begin{cases} 
447.863 \text{kip} \\
447.863 \text{kip} \\
447.863 \text{kip} \\
447.863 \text{kip} \\
447.863 \text{kip} \\
447.863 \text{kip}
\end{cases} \]

\[ \theta_d = \begin{cases} 
56.31 \text{deg} \\
47.291 \text{deg} \\
47.291 \text{deg} \\
47.291 \text{deg} \\
47.291 \text{deg}
\end{cases} \]
<table>
<thead>
<tr>
<th>Expected buckling capacity of compression brace</th>
<th>Expected post-buckling capacity of compression brace</th>
<th>Expected tension capacity of tension brace</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{bEC_i} :=$</td>
<td>$F_{bEPB_i} :=$</td>
<td>$F_{bET_i} :=$</td>
</tr>
<tr>
<td>877.45kip</td>
<td>263.23kip</td>
<td>1107.68kip</td>
</tr>
<tr>
<td>693.54kip</td>
<td>208.06kip</td>
<td>869.4kip</td>
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<tr>
<td>693.54kip</td>
<td>208.06kip</td>
<td>869.4kip</td>
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<tr>
<td>693.54kip</td>
<td>208.06kip</td>
<td>869.4kip</td>
</tr>
<tr>
<td>412.14kip</td>
<td>123.64kip</td>
<td>627.26kip</td>
</tr>
<tr>
<td>270.13kip</td>
<td>81.04kip</td>
<td>507.47kip</td>
</tr>
</tbody>
</table>

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.

$$F_{bTEB_i} := \min \left( F_{bET_i}, F_{bEC_i} + \frac{4Z_{b_i}F_{yc}}{I_{bc_i}\sin(\theta_{b_i})} \right)$$

Mech$_{EB_i}$ :=

- "Brace" if $F_{bTEB_i} = F_{bET_i}$
- "Beam" otherwise

$$F_{bTEB} = \begin{cases} 
1.108 \times 10^3 \text{kip} \\
869.4 \\
869.4 \\
869.4 \\
627.26 \\
507.47 
\end{cases}$$

Mech$_EB$ = "Brace" 

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.

$$F_{bTEPB_i} := \min \left( F_{bET_i}, F_{bEPB_i} + \frac{4Z_{b_i}F_{yc}}{I_{bc_i}\sin(\theta_{b_i})} \right)$$

Mech$_{EPB_i}$ :=

- "Brace" if $F_{bTEPB_i} = F_{bET_i}$
- "Beam" otherwise

$$F_{bTEPB} = \begin{cases} 
1.108 \times 10^3 \text{kip} \\
869.4 \\
869.4 \\
869.4 \\
627.26 \\
507.47 
\end{cases}$$

Mech$_{EPB}$ = "Brace"
Relative shear stiffness calculations for determining distribution of brace interstory shear demands to SRC, RC, GC, and BF columns.

**SRC**

\[
k_1 = \frac{3E_s l_{SC}}{(h_1)^3} \frac{E_s A_d}{\sqrt{b_{SRC}^2 + (h_1)^2}} (\cos \theta_{d_1})^2 \quad k_1 = 2.152 \times 10^3 \text{ kip in}
\]

\[
k_2 = \frac{12E_s l_{SC}}{(h_2)^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + (h_2)^2}} (\cos \theta_{d_2})^2 \quad k_2 = 3.155 \times 10^3 \text{ kip in}
\]

\[
k_3 = \frac{12E_s l_{SC}}{(h_3)^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + (h_3)^2}} (\cos \theta_{d_3})^2 \quad k_3 = 2.658 \times 10^3 \text{ kip in}
\]

\[
k_4 = \frac{12E_s l_{SC}}{(h_4)^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + (h_4)^2}} (\cos \theta_{d_4})^2 \quad k_4 = 1.992 \times 10^3 \text{ kip in}
\]

\[
k_5 = \frac{12E_s l_{SC}}{(h_5)^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + (h_5)^2}} (\cos \theta_{d_5})^2 \quad k_5 = 1.678 \times 10^3 \text{ kip in}
\]

\[
k_6 = \frac{12E_s l_{SC}}{(h_6)^3} + \frac{E_s A_d}{(b_{RC}) (h_6)^2} (\cos \theta_{d_6})^2 \quad k_6 = 1.546 \times 10^3 \text{ kip in}
\]

**Reaction Column**

\[
k_1 = \frac{3E_s l_{RC}}{(h_1)^3} = 25.913 \text{ kip in} \quad k_2 = \frac{12E_s l_{RC}}{(h_2)^3} = 275.913 \text{ kip in} \quad k_3 = \frac{12E_s l_{RC}}{(h_3)^3} = 275.913 \text{ kip in}
\]

\[
k_4 = \frac{12E_s l_{RC}}{(h_4)^3} = 275.913 \text{ kip in} \quad k_5 = \frac{12E_s l_{RC}}{(h_5)^3} = 275.913 \text{ kip in} \quad k_6 = \frac{12E_s l_{RC}}{(h_6)^3} = 275.913 \text{ kip in}
\]

**Gravity Column**

\[
k_1 = \frac{3E_s l_{GC_1}}{(h_1)^3} = 0 \text{ kip in} \quad k_2 = \frac{12E_s l_{GC_2}}{(h_2)^3} = 0 \text{ kip in} \quad k_3 = \frac{12E_s l_{GC_3}}{(h_3)^3} = 0 \text{ kip in}
\]

\[
k_4 = \frac{12E_s l_{GC_4}}{(h_4)^3} = 0 \text{ kip in} \quad k_5 = \frac{12E_s l_{GC_5}}{(h_5)^3} = 0 \text{ kip in} \quad k_6 = \frac{3E_s l_{GC_6}}{(h_6)^3} = 0 \text{ kip in}
\]
Braced Frame Columns

\[
\begin{align*}
kb_1 &= \frac{12F_s - 2lc_1}{(h_1)^3} = 183.709 \text{kip in} \\
k_b_2 &= \frac{12F_s - 2lc_2}{(h_2)^3} = 487.66 \text{kip in} \\
k_b_3 &= \frac{12F_s - 2lc_3}{(h_3)^3} = 487.66 \text{kip in} \\
k_b_4 &= \frac{12F_s - 2lc_4}{(h_4)^3} = 280.496 \text{kip in} \\
k_b_5 &= \frac{12F_s - 2lc_5}{(h_5)^3} = 280.496 \text{kip in} \\
k_b_6 &= \frac{3F_s - 2lc_6}{(h_6)^3} = 70.124 \text{kip in}
\end{align*}
\]

Relative Contributions

\[
\begin{align*}
k_i &= k_{i_1} + k_{i_2} + k_{i_3} + k_{i_4} \\
k_t &= \frac{k_{t_1}}{k_{t_1}} + \frac{k_{t_2}}{k_{t_1}} + \frac{k_{t_3}}{k_{t_1}} + \frac{k_{t_4}}{k_{t_1}} \\
PB &= \begin{pmatrix}
2.362 \times 10^3 \\
3.918 \times 10^3 \\
3.422 \times 10^3 \\
2.549 \times 10^3 \\
2.234 \times 10^3 \\
1.892 \times 10^3
\end{pmatrix}
\end{align*}
\]

\[
\begin{align*}
P_s &= \begin{pmatrix}
0.911 \\
0.805 \\
0.777 \\
0.782 \\
0.751 \\
0.817
\end{pmatrix} \\
P_t &= \begin{pmatrix}
0.011 \\
0.07 \\
0.081 \\
0.108 \\
0.123 \\
0.146
\end{pmatrix} \\
P_g &= \begin{pmatrix}
0 \\
0 \\
0 \\
0 \\
0 \\
0
\end{pmatrix} \\
P_b &= \begin{pmatrix}
0.078 \\
0.124 \\
0.143 \\
0.11 \\
0.126 \\
0.037
\end{pmatrix}
\end{align*}
\]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected buckling strengths, flexural yielding at base of columns, and yielding links with strength $V_{pl}$ at each floor elevation.

**Brace Internal Work**

$$W_{lb,EB} := \sum_{i=1}^{N} \left[ (F_{lbET_i} + F_{lbEC_i}) \cdot \cos(\theta_{bl,i}) \right]$$

**Column Internal Work**

$$W_{lc,EB} := 2 M_{pC_i}$$

**Link Internal Work**

$$W_{ll,EB} := N \cdot V_{pl} \cdot (b_{SRC} + e_{L})$$

**External Work**

$$W_{E,EB} := \sum_{i=1}^{N} (H_i \cdot C_i)$$

**Internal Work = External Work**

$$V_p := \frac{W_{lb,EB} + W_{lc,EB} + W_{ll,EB}}{W_{E,EB}}$$

Plastic base shear of frame with all comp. braces at Expected Buckling capacity

$$V_p = 1.796 \times 10^3 \text{kip}$$

**Differential Shear Demand**

$$\Delta V_i := \sum_{j=1}^{N} \left( C_j \cdot V_p \right) - \left( F_{lbEC_i} + F_{lbETE_i} \right) \cdot \cos(\theta_{bl,i}) \cdot \Delta V_i = \begin{bmatrix} 525.349 \\ 508.396 \\ 324.627 \\ 63.793 \\ 121.528 \\ -95.596 \end{bmatrix} \text{kip}$$

$$F_{BH_i} = \left( F_{lbEC_i} + F_{lbETE_i} \right) \cdot \cos(\theta_{bl,i})$$

$$F_{BV_i} = \left( F_{lbEC_i} + F_{lbETE_i} \right) \cdot \sin(\theta_{bl,i})$$

$$F_{BH} = \begin{bmatrix} 1.271 \times 10^3 \\ 1.181 \times 10^3 \\ 1.181 \times 10^3 \\ 1.181 \times 10^3 \\ 785.463 \\ 587.624 \end{bmatrix} \text{kip}$$

$$F_{BV} = \begin{bmatrix} 176.686 \\ 115.176 \\ 115.176 \\ 140.889 \\ 155.441 \end{bmatrix} \text{kip}$$
\[ A_n := \sum_{i=n}^{N} p_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \begin{cases} \sum_{i=n+1}^{N} \left( F_{bE_i} \cdot \sin(\theta_{b_i}) \right) & \text{if } n < N \\ 0 \text{kip} & \text{otherwise} \end{cases} \]

\[ A_n = \begin{array}{c} 2.706 \times 10^3 \\ 2.081 \times 10^3 \\ 1.467 \times 10^3 \\ 893.592 \\ 484.081 \\ 154.721 \end{array} \]

Moment on BF column with no SRC (all DV to columns)
- use to check if existing columns are adequate

\[ M_{T_n} := \frac{1}{2} \sum_{i=n+1}^{N} \left( C_i \cdot \left( \sum_{j=n+1}^{i} h_j \right) \right) + \frac{3}{2} \sum_{i=n}^{N} \left( p_{D_i} \cdot L \right) + F_{BV} \cdot \frac{L}{4} - A_n \cdot L \]

\[ M_{T_n} = \begin{cases} \frac{5.998 \times 10^3}{2} & \text{kip-ft} \\ \frac{2.693 \times 10^3}{583.211} & \text{kip-ft} \\ \frac{1.68556}{-621.374} & \text{kip-ft} \\ 0 & \end{cases} \]

\[ M_{B_n} := M_{T_n} + \frac{\Delta V_n}{h_n} \]

\[ M_{B_n} = \begin{cases} \frac{5.998 \times 10^3}{2} & \text{kip-ft} \\ \frac{2.693 \times 10^3}{583.211} & \text{kip-ft} \\ \frac{1.68556}{-621.374} & \text{kip-ft} \\ 0 & \end{cases} \]

\[ M_{max} := M_{max} \leftarrow 0 \text{kip-ft} \]

for \( n = 0, 1, 2, \ldots N \)

\[ M_{max} \leftarrow \begin{cases} M_{T_n} & \text{if } n = 1 \\ M_{max} \leftarrow \left| M_{T_n} \right| & \text{if } (n > 1) \wedge \left| M_{T_n} \right| > \left| M_{B_n} \right| \\ M_{B_n} & \text{otherwise} \end{cases} \]

\[ M_{max} = \begin{cases} 5.998 \times 10^3 \text{kip-ft} & \end{cases} \]

\[ \Lambda_{max} := \Lambda_{max} \leftarrow 0 \text{kip} \]

for \( n = 0, 1, 2, \ldots N \)

\[ \Lambda_{max} \leftarrow \begin{cases} \left| A_n \right| & \text{if } \left| A_n \right| > \Lambda_{max} \\ \Lambda_{max} & \text{otherwise} \end{cases} \]

\[ \Lambda_{max} = 2.706 \times 10^3 \text{kip} \]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected buckling strengths, flexural yielding at base of columns, and yielding links with strength $V_{PL}$ at each floor elevation.

**Brace Internal Work**

\[ W_{B,i,EPB} = \sum_{i=1}^{N} \left[ F_{bET,i} + F_{bEPB,i} \cos(\theta_{b_i}) \right] \]

**Column Internal Work**

\[ W_{C,i,EPB} = 2 M_{pc,i} \]

**Link Internal Work**

\[ W_{L,i,EPB} = N V_{pL,i} (b_{SRC} + c_{L}) \]

**External Work**

\[ W_{E,EPB} = \sum_{i=1}^{N} \left( H_{i} - C_{i} \right) \]

**Internal Work = External Work**

\[ V_p = \frac{W_{B,i,EPB} + W_{C,i,EPB} + W_{L,i,EPB}}{W_{E,EPB}} \]

Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity

\[ V_p = 1.363 \times 10^3 \text{kip} \]

**Single Column Shear Demand**

\[ \Delta V_i = \sum_{j=1}^{N} \left( C_j V_p - \left( F_{bEPB,i} + F_{bEPB,i} \cos(\theta_{b_j}) \right) \right) \]

\[ \Delta V_i = \begin{cases} 525.349 & \text{kip} \\ 508.396 & \text{kip} \\ 324.627 & \text{kip} \\ 63.793 & \text{kip} \\ 121.528 & \text{kip} \\ -95.596 & \text{kip} \end{cases} \]

\[ F_{BH,i} = \left( F_{bEPB,i} + F_{bEPB,i} \cos(\theta_{b_j}) \right) \]

\[ F_{BH} = \begin{cases} 877.635 & \text{kif} \\ 814.225 & \text{kif} \\ 814.225 & \text{kif} \\ 567.447 & \text{kif} \end{cases} \]

\[ F_{BV,i} = \left( -F_{bEPB,i} + F_{bEPB,i} \sin(\theta_{b_j}) \right) \]

\[ F_{BV} = \begin{cases} 648.724 & \text{kif} \\ 433.132 & \text{kif} \\ 433.132 & \text{kif} \\ 329.836 & \text{kif} \\ 279.282 & \text{kif} \end{cases} \]

A2.6-10
\[ A_n = \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \sum_{i=n+1}^{N} \left( F_{bEP_i} \sin(\theta_{i+1}) \right) \] if \( n < N \)
\[ A_n = 0 \text{kip} \] otherwise

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\[ M_{T_n} := \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i V_i \left( \sum_{j=n}^{i-1} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} - A_{nL} \right) \] if \( n < N \)
\[ M_{T_n} := a_n \] otherwise

\[ M_{B_n} = M_{T_n} + \frac{\Delta V_n}{2} \left( \frac{3.31 \times 10^3}{323.595} \right) \]
\[ M_T = \begin{pmatrix} 1.073 \times 10^4 \\ 3.31 \times 10^3 \end{pmatrix} \] kip-ft
\[ M_B = \begin{pmatrix} 6.354 \times 10^3 \\ -463.087 \end{pmatrix} \] kip-ft

\[ M_{\text{max}} := \begin{cases} M_{\text{max}} & \text{if } n \text{ even} \\ M_{T_n} & \text{if } n = 1 \\ M_{B_n} & \text{if } (n > 1) \wedge M_{T_n} > M_{B_n} \\ M_{\text{max}} & \text{otherwise} \end{cases} \]

\[ M_{\text{max}} \]

\[ A_{\text{max}} := \begin{cases} A_{\text{max}} & \text{if } n \text{ even} \\ A_n & \text{if } A_n > A_{\text{max}} \\ A_{\text{max}} & \text{otherwise} \end{cases} \]

\[ A_{\text{max}} = 2.308 \times 10^3 \text{kip} \]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all tension braces at expected tensile strength, one compression brace at expected post-buckling strength, all other compression braces at expected buckling strength, flexural yielding at base of columns, and yielding links with strength $V_{PL}$ at each floor elevation.

**Brace Internal Work**

$$W_{Ib,nPB} := \sum_{i=1}^{N} \left[ \begin{array}{ll} \left( F_{bET_1} + F_{bEPB_{i,j}} \cos(\theta_{b_{i,j}}) \right) & \text{if } i = n_{PB} \\ \left( F_{bEC_{i,j}} \cos(\theta_{b_{i,j}}) \right) & \text{otherwise} \end{array} \right] h_{i}$$

**Column Internal Work**

$$W_{Ic,nPB} := 2 M_{pc_i}$$

**Link Internal Work**

$$W_{IL,nPB} := N V_{PL}(b_{SRC} + e_{L})$$

**External Work**

$$W_{E,nPB} := \sum_{i=1}^{N} (H_i - C_i)$$

**Internal Work = External Work**

$$V_p := \frac{W_{Ib,nPB} + W_{Ic,nPB} + W_{IL,nPB}}{W_{E,nPB}}$$

**Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity**

$$V_p = 1.679 \times 10^3 \text{ kip}$$

$$F_{BH_i} := \begin{cases} \left( F_{bEPB_1} + F_{bEPB_{i,j}} \cos(\theta_{b_{i,j}}) \right) & \text{if } i = n_{PB} \\ \left( F_{bEC_{1,j}} + F_{bEC_{i,j}} \cos(\theta_{b_{i,j}}) \right) & \text{otherwise} \end{cases}$$

$$F_{BV_i} := \begin{cases} \left( -F_{bEPB_1} + F_{bEPB_{i,j}} \sin(\theta_{b_{i,j}}) \right) & \text{if } i = n_{PB} \\ \left( -F_{bEC_{1,j}} + F_{bEC_{i,j}} \sin(\theta_{b_{i,j}}) \right) & \text{otherwise} \end{cases}$$

**Single Column Shear Demand**

$$\Delta V_i := \sum_{j=1}^{N} (C_{i,j} \cdot V_p) - F_{BH_i} \cdot \Delta V_i = \begin{cases} 801.109 \text{ kip} & \text{if } i = n_{PB} \\ 397.92 & 1.181 \times 10^3 \text{ kip} \\ 226.167 & 785.463 \\ -17.611 & 587.624 \\ 62.219 & \text{kip} \\ -127.77 & \text{kip} \end{cases}$$

$$F_{BV} = \begin{cases} 648.724 & 115.176 \\ 115.176 & 115.176 \\ 140.889 & 155.441 \end{cases}$$
\[ A_n := \sum_{i=n}^{N} P_{D_{i}} + \sum_{i=n}^{N} \frac{F_{BV_{i}}}{2} + \sum_{i=n+1}^{N} \left( F_{bE_{PB_{i}}} \sin(\theta_{v_{i}}) \right) \text{ if } n < N \]
\[ (0 \text{ kip}) \text{ otherwise} \]

\[ M_{T_{n}} := \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_{i} V_{p} \left( \sum_{j=n+1}^{i} h_{j} \right) \right] + \sum_{i=n}^{N} \left( P_{D_{i}} L_{i} \right) + F_{BV_{n}} \frac{L}{2} - A_{n} L \text{ if } n < N \]

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\[ M_{B_{n}} := M_{T_{n}} + \frac{\Delta V_{n}}{2} h_{n} \]

\[ M_{T} = \begin{pmatrix}
2.252 \times 10^4 \\
1.516 \times 10^4 \\
8.921 \times 10^3 \\
4.266 \times 10^3 \\
1.027 \times 10^3 \\
0
\end{pmatrix} \text{ kip-ft} \]

\[ M_{B} = \begin{pmatrix}
2.973 \times 10^4 \\
1.775 \times 10^4 \\
1.039 \times 10^4 \\
4.151 \times 10^3 \\
1.432 \times 10^3 \\
-830.504
\end{pmatrix} \text{ kip-ft} \]

\[ A_{n} = \begin{pmatrix}
1.675 \times 10^3 \\
1.133 \times 10^3 \\
856.657 \\
580.804 \\
360.241 \\
154.721
\end{pmatrix} \text{ kip} \]

\[ M_{\text{max}} = 2.252 \times 10^4 \text{ kip-ft} \]

\[ A_{\text{max}} = 1.675 \times 10^3 \text{ kip} \]
Add higher mode forces to differential shear forces for design of the SRC

\[
\begin{align*}
\Phi_1 &:= \\
\Phi_2 &:= \begin{cases} 
0.0893 & \text{if Links = "FixFix"} \\
0.1838 & \\
0.2835 & \\
0.3867 & \\
0.4932 & \\
0.5893 & \\
0.1010 & \text{if Links = "PinPin"} \\
0.1960 & \\
0.2929 & \\
0.3896 & \\
0.4894 & \end{cases} \\
\Phi_3 &:= \begin{cases} 
0.4487 & \text{if Links = "FixFix"} \\
0.3848 & \\
-0.0771 & \\
-0.4794 & \\
-0.3109 & \\
0.4241 & \\
0.4430 & \\
0.3875 & \\
-0.0709 & \text{if Links = "PinPin"} \\
-0.4825 & \\
-0.3178 & \\
0.4201 & \end{cases} \\
\Phi_4 &:= \begin{cases} 
0.4940 & \text{if Links = "FixFix"} \\
0.0285 & \\
-0.4730 & \\
-0.1060 & \\
0.5905 & \\
-0.2740 & \end{cases} \\
\Phi_5 &:= \begin{cases} 
0.4504 & \text{if Links = "FixFix"} \\
0.0343 & \\
-0.4717 & \\
-0.1148 & \\
0.5894 & \\
-0.2722 & \end{cases} \\
\Phi_6 &:= \begin{cases} 
0.3411 & \text{if Links = "FixFix"} \\
-0.5697 & \\
0.5400 & \\
-0.3310 & \text{if Links = "FixFix"} \\
0.1296 & \\
-0.0251 & \end{cases} \\
\end{align*}
\]

T := 
\[
\begin{align*}
0.6073 & \\
0.1955 & \\
0.1081 & \\
0.0800 & \\
0.0663 & \\
0.0565 & \\
0.6827 & \\
0.2035 & \\
0.1103 & \text{sec if Links = "PinPin"} \\
0.0806 & \\
0.0667 & \\
0.0566 & \\
\end{align*}
\]

A2.6-14
Design Spectral Acceleration

\[
S_{D1} := \begin{cases} 
1.2g & \text{if Haz = "MCE"} \\
1.6g & \text{if Haz = "NF"} \\
1.4g & \text{if Haz = "NF"} 
\end{cases} \quad S_{D1} = 1.2g \\
S_{DS} := \begin{cases} 
1.6g & \text{if Haz = "MCE"} \\
1.4g & \text{if Haz = "NF"} 
\end{cases} \quad S_{DS} = 1.6g
\]

\[
T_s := \frac{S_{D1}}{S_{DS}} \quad T_s = 0.75s \\
T_0 := 0.2T_s \quad T_0 = 0.15s
\]

coefficient to adjust 5% damped spectral acceleration values to 2% damped with \( T > T_0 \) (ASCE 7-10)

\[
B_{d,To} := 0.8 \\
B_{d,Ex} := 1.0 - \frac{1.0 - B_{d,To}}{T_s/T_i}
\]

Effective Modal weights

\[
W_m = \sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{m_i} \right) \right]^2
\]

\[
W = \begin{bmatrix} 2.132 \times 10^3 \\
435.913 \\
79.135 \\
24.041 \\
12.472 \\
3.756 
\end{bmatrix} \quad \text{kip} \\
\sum_{i} W_{f_i} = \begin{bmatrix} 0.793 \\
0.162 \\
0.029 \\
8.946 \times 10^{-3} \\
4.641 \times 10^{-3} \\
1.398 \times 10^{-3} \end{bmatrix}
\]

Modal Participation Factors

\[
\gamma_m := \frac{W_m}{\sum_{i=1}^{N} \left[ W_{f_i} \left( \phi_{m_i} \right) \right]^2}
\]

\[
\gamma = \begin{bmatrix} 2.35 \\
1.063 \\
1.043 \\
0.453 \\
0.25 \\
0.18 \end{bmatrix} \quad \begin{bmatrix} \frac{T_m}{T_0} \end{bmatrix} \quad \begin{bmatrix} 0.099 \end{bmatrix}
\]

Response Coefficient of each mode

\[
C_{s,m} := \begin{cases} 
\frac{S_{DS}}{gB_{d,Ex_m}} \left( 0.4 + 0.6 \frac{T_m}{T_0} \right) & \text{if } T_m < T_0 \\
\frac{S_{DS}}{gB_{d,Ex_m}} & \text{if } T_0 < T_m < T_s \\
\frac{S_{D1}}{T_m \text{sec} B_{d,Ex_m}} & \text{otherwise}
\end{cases}
\]

\[
C_s = \begin{bmatrix} 1.909 \\
1.688 \\
1.371 \\
1.177 \\
1.083 \\
1.017 \end{bmatrix}
\]

Modal base shear

\[
V_m := C_{s,m} W_m \\
V = \begin{bmatrix} 4.07 \times 10^3 \\
735.821 \\
108.523 \\
28.299 \\
13.513 \\
3.819 \end{bmatrix} \quad \text{kip}
\]
Elastic System Forces

Story shear forces of each mode
- 1st mode not relevant

\[ F_{i,m} = \frac{W_{i,j}}{W_{m}} \left( \phi_{m,i} \right)^{\frac{\Gamma_{m}}{W_{m}}} V_{m} \]

\[ F = \begin{bmatrix}
179.407 & 269.587 & 124.794 & 59.098 & 41.036 & 15.345 \\
776.896 & 203.797 & -133.332 & -13.952 & 47.539 & -14.891 \\
990.858 & -82.981 & -86.468 & 77.722 & -33.267 & 5.83 \\
1.184 \times 10^3 & -398.114 & 117.952 & -36.064 & 10.213 & -1.129 \\
\end{bmatrix} \text{kip} \]

\[ V_{SRC,m,j,m} = \begin{cases} 
\Delta V_j & \text{if } m = 1 \\
\sum_{i=j}^{6} F_{i,m} & \text{otherwise}
\end{cases} \]

\[ V_{SRC,m} = \begin{bmatrix}
226.167 & 82.981 & -123.292 & -34.551 & 3.465 & 14.104 \\
62.219 & -481.095 & 31.483 & 41.658 & -23.054 & 4.701 \\
\end{bmatrix} \text{kip} \]

\[ V_{SRC,j} := P_{sj} \left[ \Delta V_j + \sum_{m=2}^{6} \left( F_{j,m} \right)^2 \right] \]

Links = "FixFix"
Haz = "MCE"
GravCol = "None"
\[ n_{PB} = 1 \]

\[ V_{SRC} = \begin{bmatrix}
1.009 \times 10^3 \\
642.397 \\
461.314 \\
180.85 \\
156.953 \\
236.259 \\
\end{bmatrix} \text{kip} \]
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<td>636.814</td>
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<tr>
<td>306.624</td>
<td>252.777</td>
<td>517.728</td>
<td>455.586</td>
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<tr>
<td>56.695</td>
<td>14.579</td>
<td>227.782</td>
<td>181.579</td>
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<tr>
<td>79.059</td>
<td>51.125</td>
<td>189.803</td>
<td>161.611</td>
</tr>
<tr>
<td>331.8</td>
<td>255.575</td>
<td>372.408</td>
<td>296.873</td>
</tr>
</tbody>
</table>
peak axial force on links

Pin-Pin Links, MCE Hazard

\[ A_{PM,nPB1} := \text{for } j \in N, N - 1 \ldots \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB1,j} \text{ if } j = N \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB1,j} - \sum_{k=j}^{N} A_{PM1,k} \text{ otherwise} \]
\[ A_{PM1} \]

\[ A_{PM,nPB2} := \text{for } j \in N, N - 1 \ldots \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB2,j} \text{ if } j = N \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB2,j} - \sum_{k=j}^{N} A_{PM1,k} \text{ otherwise} \]
\[ A_{PM1} \]

\[ A_{PM,nPB3} := \text{for } j \in N, N - 1 \ldots \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB3,j} \text{ if } j = N \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB3,j} - \sum_{k=j}^{N} A_{PM1,k} \text{ otherwise} \]
\[ A_{PM1} \]

\[ A_{PM,nPB4} := \text{for } j \in N, N - 1 \ldots \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB4,j} \text{ if } j = N \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB4,j} - \sum_{k=j}^{N} A_{PM1,k} \text{ otherwise} \]
\[ A_{PM1} \]

\[ A_{PM,nPB5} := \text{for } j \in N, N - 1 \ldots \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB5,j} \text{ if } j = N \]
\[ A_{PM1,j} \leftarrow V_{PM,nPB5,j} - \sum_{k=j}^{N} A_{PM1,k} \text{ otherwise} \]
\[ A_{PM1} \]
\[ A_{PM,nPB6} := \text{for } j \in N, N - 1 \ldots 1 \]
\[ A_{PM,j} \leftarrow V_{PM,nPB6,j} \quad \text{if } j = N \]
\[ A_{PM,j} \leftarrow V_{PM,nPB6,j} - \sum_{k=j}^{N} A_{PM,k} \quad \text{otherwise} \]

\[
A_{PM,nPB1} = \begin{pmatrix}
322.484 \\
149.76 \\
240.447 \\
-36.446 \\
-134.215 \\
180.424
\end{pmatrix} \text{kip} \quad A_{PM,nPB2} = \begin{pmatrix}
-325.297 \\
449.203 \\
244.641 \\
-30.219 \\
-129.222 \\
189.958
\end{pmatrix} \text{kip} \quad A_{PM,nPB3} = \begin{pmatrix}
-29.917 \\
153.823 \\
-42.136 \\
256.558 \\
-129.222 \\
189.958
\end{pmatrix} \text{kip}
\]

\[
A_{PM,nPB4} = \begin{pmatrix}
-29.917 \\
153.823 \\
-42.136 \\
256.558 \\
-129.222 \\
189.958
\end{pmatrix} \text{kip} \quad A_{PM,nPB5} = \begin{pmatrix}
-24.975 \\
157.23 \\
248.155 \\
-188.724 \\
38.689 \\
197.95
\end{pmatrix} \text{kip} \quad A_{PM,nPB6} = \begin{pmatrix}
-22.481 \\
158.949 \\
249.929 \\
-22.364 \\
-252.741 \\
331.8
\end{pmatrix} \text{kip}
\]

\[ A_{PM,max} := \max( A_{PM,nPB1}, A_{PM,nPB2}, A_{PM,nPB3}, A_{PM,nPB4}, A_{PM,nPB5}, A_{PM,nPB6} ) \]
\[ A_{PM,min} := \min( A_{PM,nPB1}, A_{PM,nPB2}, A_{PM,nPB3}, A_{PM,nPB4}, A_{PM,nPB5}, A_{PM,nPB6} ) \]
\[ A_{PM,max} = 529.648 \text{ kip} \]
\[ A_{PM,min} = -325.297 \text{ kip} \]
\[ A_{PM} := \max( |A_{PM,max}|, |A_{PM,min}| ) = 529.648 \text{ kip} \]
Pin-Pin Links, NF Hazard

\[ \text{APN}.nP_{B1} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ \begin{align*}
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B1}} \quad \text{if } j = N \\
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B1}} - \sum_{k=j}^{N} \text{APN}_{1k} \quad \text{otherwise}
\end{align*} \]
\[ \text{APN}_{1} \]

\[ \text{APN}.nP_{B2} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ \begin{align*}
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B2}} \quad \text{if } j = N \\
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B2}} - \sum_{k=j}^{N} \text{APN}_{1k} \quad \text{otherwise}
\end{align*} \]
\[ \text{APN}_{1} \]

\[ \text{APN}.nP_{B3} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ \begin{align*}
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B3}} \quad \text{if } j = N \\
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B3}} - \sum_{k=j}^{N} \text{APN}_{1k} \quad \text{otherwise}
\end{align*} \]
\[ \text{APN}_{1} \]

\[ \text{APN}.nP_{B4} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ \begin{align*}
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B4}} \quad \text{if } j = N \\
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B4}} - \sum_{k=j}^{N} \text{APN}_{1k} \quad \text{otherwise}
\end{align*} \]
\[ \text{APN}_{1} \]

\[ \text{APN}.nP_{B5} := \text{for } j \in \{N, N-1, \ldots, 1\} \]
\[ \begin{align*}
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B5}} \quad \text{if } j = N \\
\text{APN}_{1j} &\leftarrow V_{\text{PN}.nP_{B5}} - \sum_{k=j}^{N} \text{APN}_{1k} \quad \text{otherwise}
\end{align*} \]
\[ \text{APN}_{1} \]
\[ \text{APN}_{n\text{PB}6} := \begin{cases} \text{APN}_{n1} & \text{if } j = N \\ \text{APN}_{n1} & \sum_{k=j}^{N} \text{APN}_{n1k} \text{ otherwise} \end{cases} \]

\[
\begin{array}{c|c|c}
\text{APN}_{n\text{PB}1} = & \text{kip} & \text{APN}_{n\text{PB}2} = \begin{pmatrix} 327.847 \\ 141.002 \\ 228.715 \\ -50.627 \\ -85.924 \\ 104.199 \end{pmatrix} \begin{pmatrix} -319.935 \\ 440.445 \\ 232.909 \\ -44.4 \\ -80.931 \\ 113.733 \end{pmatrix} \begin{pmatrix} -24.555 \\ -139.942 \\ 517.916 \\ -44.4 \\ -80.931 \\ 113.733 \end{pmatrix} \\
\text{APN}_{n\text{PB}4} = & \text{kip} & \text{APN}_{n\text{PB}5} = \begin{pmatrix} -24.555 \\ 145.065 \\ -53.868 \\ 242.377 \\ -80.931 \\ 113.733 \end{pmatrix} \begin{pmatrix} -19.613 \\ 148.472 \\ 236.424 \\ -202.906 \\ 86.981 \\ 121.724 \end{pmatrix} \begin{pmatrix} -17.119 \\ 150.191 \\ 238.198 \\ -36.546 \\ -204.45 \\ 255.575 \end{pmatrix} \\
\end{array}
\]

\[ \text{APN}_{\text{max}} := \max \left( \text{APN}_{n\text{PB}1}, \text{APN}_{n\text{PB}2}, \text{APN}_{n\text{PB}3}, \text{APN}_{n\text{PB}4}, \text{APN}_{n\text{PB}5}, \text{APN}_{n\text{PB}6} \right) \]

\[ \text{APN}_{\text{min}} := \min \left( \text{APN}_{n\text{PB}1}, \text{APN}_{n\text{PB}2}, \text{APN}_{n\text{PB}3}, \text{APN}_{n\text{PB}4}, \text{APN}_{n\text{PB}5}, \text{APN}_{n\text{PB}6} \right) \]

\[ \text{APN}_{\text{max}} = 517.916 \text{ kip} \]

\[ \text{APN}_{\text{min}} = -319.935 \text{ kip} \]

\[ \text{APN} := \max \left( |\text{APN}_{\text{max}}|, |\text{APN}_{\text{min}}| \right) = 517.916 \text{ kip} \]
Fix-Fix Links, MCE Hazard

$A_{FM,nPB1} := \text{for } j \in N, N - 1..1$

$A_{FM1_j} \leftarrow V_{FM,nPB1_j}$ if $j = N$

$A_{FM1_j} \leftarrow V_{FM,nPB1_j} - \sum_{k = j}^{N} A_{FM1_k}$ otherwise

$A_{FM1}$

$A_{FM,nPB2} := \text{for } j \in N, N - 1..1$

$A_{FM1_j} \leftarrow V_{FM,nPB2_j}$ if $j = N$

$A_{FM1_j} \leftarrow V_{FM,nPB2_j} - \sum_{k = j}^{N} A_{FM1_k}$ otherwise

$A_{FM1}$

$A_{FM,nPB3} := \text{for } j \in N, N - 1..1$

$A_{FM1_j} \leftarrow V_{FM,nPB3_j}$ if $j = N$

$A_{FM1_j} \leftarrow V_{FM,nPB3_j} - \sum_{k = j}^{N} A_{FM1_k}$ otherwise

$A_{FM1}$

$A_{FM,nPB4} := \text{for } j \in N, N - 1..1$

$A_{FM1_j} \leftarrow V_{FM,nPB4_j}$ if $j = N$

$A_{FM1_j} \leftarrow V_{FM,nPB4_j} - \sum_{k = j}^{N} A_{FM1_k}$ otherwise

$A_{FM1}$

$A_{FM,nPB5} := \text{for } j \in N, N - 1..1$

$A_{FM1_j} \leftarrow V_{FM,nPB5_j}$ if $j = N$

$A_{FM1_j} \leftarrow V_{FM,nPB5_j} - \sum_{k = j}^{N} A_{FM1_k}$ otherwise

$A_{FM1}$
\[ \text{AFM} : \max \left( |\text{AFM}_{\text{max}}|, |\text{AFM}_{\text{min}}| \right) = 569.664 \text{ kip} \]
Fix-Fix Links, NF Hazard

\[ A_{FN,nPB1} := \text{for } j \in N, N - 1..1 \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB1,j} \text{ if } j = N \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB1,j} - \sum_{k=j}^{N} A_{FN1,k} \text{ otherwise} \]
\[ A_{FN1} \]

\[ A_{FN,nPB2} := \text{for } j \in N, N - 1..1 \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB2,j} \text{ if } j = N \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB2,j} - \sum_{k=j}^{N} A_{FN1,k} \text{ otherwise} \]
\[ A_{FN1} \]

\[ A_{FN,nPB3} := \text{for } j \in N, N - 1..1 \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB3,j} \text{ if } j = N \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB3,j} - \sum_{k=j}^{N} A_{FN1,k} \text{ otherwise} \]
\[ A_{FN1} \]

\[ A_{FN,nPB4} := \text{for } j \in N, N - 1..1 \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB4,j} \text{ if } j = N \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB4,j} - \sum_{k=j}^{N} A_{FN1,k} \text{ otherwise} \]
\[ A_{FN1} \]

\[ A_{FN,nPB5} := \text{for } j \in N, N - 1..1 \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB5,j} \text{ if } j = N \]
\[ A_{FN1,j} \leftarrow V_{FN,nPB5,j} - \sum_{k=j}^{N} A_{FN1,k} \text{ otherwise} \]
\[ A_{FN1} \]
\[
\lambda_{F_{N,nPB6}} := \text{for } j \in N, N - 1..1
\]
\[
\begin{align*}
\lambda_{F_{N1,j}} & \leftarrow V_{F_{N,nPB6,j}} \quad \text{if } j = N \\
\lambda_{F_{N1,j}} & \leftarrow V_{F_{N,nPB6,j}} - \sum_{k=j}^{N} \lambda_{F_{N1,k}} \quad \text{otherwise}
\end{align*}
\]

\[
\lambda_{F_{N,nPB1}} = \begin{bmatrix}
373.534 \\
172.039 \\
264.524 \\
5.886 \\
-31.962 \\
160.724
\end{bmatrix}
\]

\[
\lambda_{F_{N,nPB2}} = \begin{bmatrix}
-274.247 \\
471.482 \\
268.718 \\
12.113 \\
-26.009 \\
169.298
\end{bmatrix}
\]

\[
\lambda_{F_{N,nPB3}} = \begin{bmatrix}
21.132 \\
-108.904 \\
553.725 \\
12.113 \\
-26.009 \\
169.298
\end{bmatrix}
\]

\[
\lambda_{F_{N,nPB4}} = \begin{bmatrix}
21.132 \\
176.103 \\
-18.059 \\
298.89 \\
-26.009 \\
169.298
\end{bmatrix}
\]

\[
\lambda_{F_{N,nPB5}} = \begin{bmatrix}
26.075 \\
179.509 \\
272.233 \\
-146.392 \\
142.705 \\
176.486
\end{bmatrix}
\]

\[
\lambda_{F_{N,nPB6}} = \begin{bmatrix}
28.569 \\
181.228 \\
274.007 \\
19.968 \\
-135.262 \\
296.873
\end{bmatrix}
\]

\[
\lambda_{F_{N,\text{max}}} := \max(\lambda_{F_{N,nPB1}}, \lambda_{F_{N,nPB2}}, \lambda_{F_{N,nPB3}}, \lambda_{F_{N,nPB4}}, \lambda_{F_{N,nPB5}}, \lambda_{F_{N,nPB6}})
\]

\[
\lambda_{F_{N,\text{min}}} := \min(\lambda_{F_{N,nPB1}}, \lambda_{F_{N,nPB2}}, \lambda_{F_{N,nPB3}}, \lambda_{F_{N,nPB4}}, \lambda_{F_{N,nPB5}}, \lambda_{F_{N,nPB6}})
\]

\[
\lambda_{F_{N,\text{max}}} = 553.725 \text{ kip}
\]

\[
\lambda_{F_{N,\text{min}}} = -274.247 \text{ kip}
\]

\[
\lambda_{F_{N}} := \max(\mid \lambda_{F_{N,\text{max}}}, \mid \lambda_{F_{N,\text{min}}}) = 553.725 \text{ kip}
\]
<table>
<thead>
<tr>
<th>Summary</th>
<th>Values for Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake Hazard</td>
<td>Pin-Pin Links, MCE Hazard</td>
</tr>
<tr>
<td>Haz = “MCE”</td>
<td>(722.454)</td>
</tr>
<tr>
<td>Link Connectivity</td>
<td>724.361</td>
</tr>
<tr>
<td>Links = “FixFix”</td>
<td>560.165</td>
</tr>
<tr>
<td>Gravity Frame</td>
<td>SRC Shear $V_{FM, Max} = 317.294 \text{kip}$</td>
</tr>
<tr>
<td>Contribution</td>
<td>236.639</td>
</tr>
<tr>
<td>Plastic Shear Strength of Links</td>
<td>(331.8)</td>
</tr>
<tr>
<td>$V_{pL} = 202$-kip</td>
<td></td>
</tr>
<tr>
<td>Controlling Plastic Mechanism at EB</td>
<td>Link Axial Force $A_{PM} = 529.648$-kip</td>
</tr>
<tr>
<td>$\text{Mech}_{EB} =$</td>
<td>Pin-Pin Links, NF Hazard</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>(665.212)</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>661.756</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>SRC Shear $V_{PN, Max} = 275.179 \text{kip}$</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>208.705</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>(255.575)</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td></td>
</tr>
<tr>
<td>Controlling Plastic Mechanism at EPB</td>
<td>Link Axial Force $A_{PN} = 517.916$-kip</td>
</tr>
<tr>
<td>$\text{Mech}_{EB} =$</td>
<td>Fix-Fix Links, MCE Hazard</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>(1.009 $\times 10^3$)</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>966.789</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>771.269</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>SRC Shear $V_{FM, Max} = 488.381 \text{kip}$</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>347.383</td>
</tr>
<tr>
<td>&quot;Brace&quot;</td>
<td>(372.408)</td>
</tr>
<tr>
<td>Story at EPB</td>
<td>Link Axial Force $A_{FM} = 569.664$-kip</td>
</tr>
<tr>
<td>$n_{PB} = 1$</td>
<td>Fix-Fix Links, NF Hazard</td>
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<tr>
<td>Plastic Base Shear</td>
<td>(944.745)</td>
</tr>
<tr>
<td>$V_{p} = 1.679 \times 10^3$-kip</td>
<td>895.602</td>
</tr>
<tr>
<td>Decimal percentage of differential interstory shear taken by SRC</td>
<td>SRC Shear $V_{FN, Max} = 442.179 \text{kip}$</td>
</tr>
<tr>
<td>$P_s =$</td>
<td>709.127</td>
</tr>
<tr>
<td>$0.911$</td>
<td>319.191</td>
</tr>
<tr>
<td>$0.805$</td>
<td>(296.873)</td>
</tr>
<tr>
<td>$0.011$</td>
<td></td>
</tr>
<tr>
<td>$0.817$</td>
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<td>$0.077$</td>
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<tr>
<td>$0.751$</td>
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</tr>
<tr>
<td>$0.081$</td>
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</tr>
<tr>
<td>$0.108$</td>
<td></td>
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<tr>
<td>$0.123$</td>
<td></td>
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<tr>
<td>$0$</td>
<td></td>
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<tr>
<td>$0$</td>
<td></td>
</tr>
<tr>
<td>Note: &gt;75% of differential shear is taken by SRC</td>
<td>Link Axial Force $A_{FN} = 553.725$-kip</td>
</tr>
</tbody>
</table>
A2.7 3NCBF-WB SRC Design Inter-Story Shear Force

OBJECTIVE
The objective of this calculation is to calculate the forces for design of the SRC. The design forces include the plastic mechanism interaction forces considering some degree of non-uniform LFRS degradation and initial differences in nominal frame strength. Additionally, higher mode force effects are added to the plastic mechanism SRC force demands based on elastic higher mode dynamics.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration was modified to represent an older braced frame design based on a historic SEAOC seismic design procedure. This 3NCBF frame includes under-designed braced frame beams which are expected to yield in flexure prior to the development of the full expected tensile force in each brace. This version of the prototype frame is referred to as 3NCBF-WB (weak beam). Assuming a SRC is added to the frame, the soft-story mechanism will not form and plastic deformations will be limited to the braces and/or beam.

METHODOLOGY
The calculation first evaluates the controlling frame mechanism considering the compression brace at the expected buckling strength (EB) and then at the expected post-buckling (EPB) strength. The expected tension brace force is calculated considering the compression brace at EB or EPB which may result in the tension brace reaching its expected tension strength or the force to develop the beam “pull-down” mechanism.

The plastic base shear capacity of the frame can then be calculated using plastic analysis concepts assuming an inverted triangular load distribution. The differential shear demand can then be calculated by assuming degradation of a single story’s compression brace to EPB while all others are at EB. Assuming all at EB or EPB results in little differential shear. The demands on BF columns are calculated simply to determine if the existing columns are adequate. Considering a single story at EPB and all other at EB results in a significant bending demand on the BF columns (considering no SRC). Once the SRC is added, the differential shear is assumed to be distributed among the SRC, braced frame columns, gravity column, and reaction column based on their relative interstory shear stiffness.

The final part of the calculation considers the portion of the differential shear taken to the SRC based on the relative stiffness of the braced frame columns, gravity column, reaction column, and SRC. Once the differential shear to develop the plastic mechanism is known (including the effects of non-uniform degradation and differences in nominal strength), the higher mode forces are added to the mechanism forces. The higher mode forces are combined with a SRSS modal combination rule and added directly to the plastic mechanism differential shear forces. The direct addition of the plastic mechanism forces and higher mode forces is likely appropriate since the structure responds within the plastic deformation “regime” during a significant amount of its cyclic hysteretic response.
Input

Earthquake Hazard
Haz := "MCE"

Link Connectivity
Links := "FixFix"

Gravity Column Contribution ("None," "Weak," or "Strong")
GravCol := "None"

Stories, Floors, Modes
N := 3 i := 1, 2 .. N j := 1, 2 .. N
m := 1, 2 .. N n := 1, 2 .. N

Geometry
L := 30ft b_{SRC} := 12ft

\[ C_i := h_i := \frac{1}{6} \]
\[ 13ft \]
\[ \frac{1}{3} \]
\[ \frac{1}{2} \]

\[ H_i := \sum_{j=1}^{i} h_j \]
\[ \theta_{b_i} := \text{atan} \left( \frac{(2-h_i)}{L} \right) \]
\[ \theta_{b} := \left( \begin{array}{c}
40.914 \\
40.914 \cdot \text{deg}
\end{array} \right) \]
\[ c_{bi} := \frac{1}{2} \text{be}_i := \text{be}_i := \left( \begin{array}{c}
.625ft \\
\cdot 28.75ft
\end{array} \right) \]

BF Sections and Material Properties
Z_{b_i} := Z_c := 147 in^3
F_{ye} := 55ksi

\[ I_c := 833in^4 \]
\[ E_s := 29000 ksi \]

\[ A_{c_i} := 28.2in^2 \]
\[ M_{pc} := Z_c \cdot F_{ye} \]
M_{pc} = 673.75 kip-ft

Gravity Column
A_{GC} :=
\begin{align*}
143.5in^2 & \text{ if GravCol = "Strong"} \\
143.5in^2 & \text{ if GravCol = "Weak"} \\
0in^2 & \text{ if GravCol = "None"}
\end{align*}

I_{GC} :=
\begin{align*}
4782.5in^4 & \text{ if GravCol = "Strong"} \\
1033.2in^4 & \text{ if GravCol = "Weak"} \\
0in^4 & \text{ if GravCol = "None"}
\end{align*}
SRC, Links, Reaction Column

\[ A_{d_1} := \begin{cases} 42.7 \text{ in}^2 \\ 29.1 \text{ in}^2 \\ 23.2 \text{ in}^2 \end{cases} \]

\[ A_{SB_1} := \begin{cases} 42.7 \text{ in}^2 \\ 29.1 \text{ in}^2 \\ 23.2 \text{ in}^2 \end{cases} \]

\[ \theta_{d_1} := \text{atan} \left( \frac{h_i}{b_{\text{SRC}}} \right) \]

\[ B_{RC} := \begin{cases} 1 \text{ if Links = "FixFix"} \\ 4 \text{ otherwise} \end{cases} \]

\[ \theta_d = \begin{cases} 47.291 \text{ deg} \end{cases} \]

Column boundary condition (for stiffness calculation below)

Average of SRC column areas and moments of inertia

\[ A_{SC} := 34.25 \text{ in}^2 \]
\[ I_{SC} := 1347 \text{ in}^4 \]

Reaction Column Properties

\[ A_{RC} := 42.7 \text{ in}^2 \]
\[ I_{RC} := 1710 \text{ in}^4 \]

Link Properties: size considered to achieve target drift (see design and prediction calculations)

\[ V_{PL} := \begin{cases} 160 \text{kip} \text{ if Links = "FixFix"} \\ 0 \text{kip} \text{ otherwise} \end{cases} \]

\[ c_L := 48 \text{in} \]

Mass and Gravity Loads

\[ m_{f_i} := \frac{6.7 \text{kip} \cdot \text{sec}^2}{4} \]

\[ m_{f_i} \cdot g \]

\[ W_{f_i} := \text{P}_{D_1} := \begin{cases} 82 \text{kip} \\ 82 \text{kip} \\ 77 \text{kip} \end{cases} \]

\[ 82 \text{kip} \]

\[ W_f = \begin{cases} 646.698 \text{ kip} \\ 646.698 \text{ kip} \end{cases} \]

\[ \mathbf{N} = 1.94 \times 10^3 \text{kip} \]
Expected buckling capacity of compression brace

\[
F_{bEC_i} :=
\begin{align*}
394.82 \text{kip} \\
395.63 \text{kip} \\
234.02 \text{kip}
\end{align*}
\]

Expected post-buckling capacity of compression brace

\[
F_{bEPB_i} :=
\begin{align*}
118.45 \text{kip} \\
118.69 \text{kip} \\
70.2 \text{kip}
\end{align*}
\]

Expected tension capacity of tension brace

\[
F_{bET_i} :=
\begin{align*}
577.67 \text{kip} \\
627.26 \text{kip} \\
507.47 \text{kip}
\end{align*}
\]

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.

\[
F_{bTEB_i} := \min \left( F_{bET_i} \cdot F_{bEC_i} + \frac{4 \cdot Z_{b_i} \cdot F_{ye}}{L_{be_i} \cdot \sin(\theta_{b_{i,j}})} \right)
\]

\[
F_{bTEB} = \begin{cases} 
577.67 \\ 
627.26 \\ 
507.47
\end{cases} \cdot \text{kip}
\]

Mech_{EB} :=
\begin{cases} 
"Brace" & \text{if } F_{bTEB_i} = F_{bET_i} \\
"Beam" & \text{otherwise}
\end{cases}

Mech_{EB} = \begin{cases} 
"Brace" \\ 
"Brace"
\end{cases}

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.

\[
F_{bTEPB_i} := \min \left( F_{bET_i} \cdot F_{bEPB_i} + \frac{4 \cdot Z_{b_i} \cdot F_{ye}}{L_{be_i} \cdot \sin(\theta_{b_{i,j}})} \right)
\]

\[
F_{bTEPB} = \begin{cases} 
522.52 \\ 
573.39 \\ 
474.27
\end{cases} \cdot \text{kip}
\]

Mech_{EPB} :=
\begin{cases} 
"Brace" & \text{if } F_{bTEPB_i} = F_{bET_i} \\
"Beam" & \text{otherwise}
\end{cases}

Mech_{EPB} = \begin{cases} 
"Beam" \\ 
"Beam"
\end{cases}
Relative shear stiffness calculations for determining distribution of brace interstory shear demands to SRC, RC, GC, and BF columns.

**SRC**

\[ \begin{align*}
  k_1 & = \frac{3E_s I_{SC}}{(h_1)^3} + \frac{E_s A_{d1}}{\sqrt{b_{SRC} \cdot (h_1)^2}} \cos^2 \theta_d \\
  k_2 & = \frac{12E_s I_{SC}}{(h_2)^3} + \frac{E_s A_{d2}}{\sqrt{b_{SRC} \cdot (h_2)^2}} \cos^2 \theta_d \\
  k_3 & = \frac{12E_s I_{SC}}{B_{RC}(h_3)^3} + \frac{E_s A_{d3}}{\sqrt{b_{SRC} \cdot (h_3)^2}} \cos^2 \theta_d \\
  k_1 & = 2.714 \times 10^3 \text{ kip/in} \\
  k_2 & = 1.952 \times 10^3 \text{ kip/in} \\
  k_3 & = 1.581 \times 10^3 \text{ kip/in}
\end{align*} \]

**Reaction Column**

\[ \begin{align*}
  k_{r1} & = \frac{3E_s I_{RC}}{(h_1)^3} = 39.187 \text{ kip/in} \\
  k_{r2} & = \frac{12E_s I_{RC}}{(h_2)^3} = 156.748 \text{ kip/in} \\
  k_{r3} & = \frac{12E_s I_{RC}}{B_{RC}(h_3)^3} = 156.748 \text{ kip/in}
\end{align*} \]

**Gravity Column**

\[ \begin{align*}
  k_{g1} & = \frac{3E_s I_{GC}}{(h_1)^3} = 0 \text{ kip/in} \\
  k_{g2} & = \frac{12E_s I_{GC}}{(h_2)^3} = 0 \text{ kip/in} \\
  k_{g3} & = \frac{3E_s I_{GC}}{(h_3)^3} = 0 \text{ kip/in}
\end{align*} \]

**Braced Frame Columns**

\[ \begin{align*}
  k_{b1} & = \frac{12E_s I_{2Lc}}{(h_1)^3} = 152.715 \text{ kip/in} \\
  k_{b2} & = \frac{12E_s I_{2Lc}}{(h_2)^3} = 152.715 \text{ kip/in} \\
  k_{b3} & = \frac{3E_s I_{2Lc}}{(h_3)^3} = 38.179 \text{ kip/in}
\end{align*} \]

**Relative Contributions**

\[ \begin{align*}
  k_i & = k_{r1} + k_{g1} + k_{b1} + k_{b2} + k_{b3} \\
  k_i & = \begin{pmatrix} 2.906 \times 10^3 \\ 2.262 \times 10^3 \\ 1.776 \times 10^3 \end{pmatrix} \text{ kip/in}
\end{align*} \]

\[ \begin{align*}
  P_i & = \begin{pmatrix} 0.934 \\ 0.013 \\ 0.89 \end{pmatrix} \\
  P_r & = \begin{pmatrix} 0.013 \\ 0.0869 \\ 0 \end{pmatrix} \\
  P_g & = \begin{pmatrix} 0 \end{pmatrix} \\
  P_b & = \begin{pmatrix} 0.053 \\ 0.068 \\ 0.021 \end{pmatrix}
\end{align*} \]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected tensile and expected buckling strengths, flexural yielding at base of columns, and yielding links with strength VpL at each floor elevation.

Brace Internal Work
\[ W_{ib,EB} := \sum_{i=1}^{N} \left( F_{ibET_i} + F_{ibEC_i} \cos(\theta_{b_{i_j}} \cdot h_i) \right) \]

Column Internal Work
\[ W_{ic,EB} := 2M_{pc} \]

Link Internal Work
\[ W_{IL,EB} := V_{pL} \left( h_{SRC} + c_L \right) \]

External Work
\[ W_{E,EB} := \sum_{i=1}^{N} \left( H_{i} \cdot C_{i} \right) \]

Internal Work = External Work
\[ V_p := \frac{W_{ib,EB} + W_{ic,EB} + W_{IL,EB}}{W_{E,EB}} \]

Plastic base shear of frame with all comp. braces at Expected Buckling capacity
\[ V_p = 1.184 \times 10^3 \text{kip} \]

Plastic base shear of frame with all comp. braces at Expected Buckling capacity
\[ V_p = 1.184 \times 10^3 \text{kip} \]

Differential Shear Demand
\[ \Delta V_i := \sum_{j=i}^{N} \left( C_{j} \cdot V_{p} \right) - \left( F_{ibEC_i} + F_{ibTEB_i} \right) \cos(\theta_{b_{i_j}}) \]

\[ \Delta V_i = \begin{array}{c} 449.091 \text{-kip} \\ 213.672 \text{-kip} \\ 31.66 \text{-kip} \end{array} \]

\[ F_{BH_i} := \left( F_{ibEC_i} + F_{ibTEB_i} \right) \cos(\theta_{b_{i_j}}) \]
\[ F_{BH} = \begin{pmatrix} 734.9 \text{-kip} \\ 772.987 \text{-kip} \\ 560.336 \text{-kip} \end{pmatrix} \]

\[ F_{BV_i} := \left( -F_{ibEC_i} + F_{ibTEB_i} \right) \sin(\theta_{b_{i_j}}) \]
\[ F_{BV} = \begin{pmatrix} 119.754 \text{-kip} \\ 151.702 \text{-kip} \\ 179.091 \text{-kip} \end{pmatrix} \]
\[ A_n = \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \sum_{i=n+1}^{N} \left( F_{BEi} \sin(\theta_i) \right) \text{ if } n < N \]
\[ A_n = (0 \text{kip}) \text{ otherwise} \]

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\[ M_{T_n} = \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i \cdot V_{p_i} \left( \sum_{j=n+1}^{i} b_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} \cdot L_i \right) + F_{BV_n} \cdot \frac{L}{2} - A_n \cdot L \text{ if } n < N \]
\[ M_{T_n} = (0 \text{kip-ft}) \text{ otherwise} \]

\[ M_{B_n} = M_{T_n} + \frac{\Delta V_n}{2} \cdot h_n \]

\[ M_T = \begin{pmatrix}
1.595 \times 10^3 \\
205.788 \\
0
\end{pmatrix} \text{ kip-ft} \]

\[ M_B = \begin{pmatrix}
4.514 \times 10^3 \\
1.595 \times 10^3 \\
205.788
\end{pmatrix} \text{ kip-ft} \]

\[ M_{max} = \begin{cases} 
M_{max} & \text{for } n \in 1, 2, \ldots, N \\
M_{max} & \text{if } n = 1 \\
M_{T_{n-1}} & \text{if } (n > 1) \land M_{T_{n-1}} > M_B \\
M_{B_{n-1}} & \text{if } M_{max} > M_{max1} \\
M_B & \text{otherwise} \\
M_{max} & \text{otherwise}
\end{cases} \]

\[ M_{max} = 1.595 \times 10^3 \text{ kip-ft} \]

\[ A_{max} = \begin{cases} 
A_{max} & \text{for } n \in 1, 2, \ldots, N \\
A_{max} & \text{if } A_n > A_{max} \\
A_n & \text{if } A_n < A_{max} \\
A_{max} & \text{otherwise}
\end{cases} \]

\[ A_{max} = 878.65 \text{ kip} \]

A2.7-7
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected beam pulldown and expected post-buckling strengths, flexural yielding at base of columns, and yielding links with strength $V_{P,L}$ at each floor elevation.

Brace Internal Work

$$W_{b,EBP} := \sum_{i=1}^{N} \left( F_{bEBP_i} \sin(\theta_{b_{i,j}}) \frac{L_{j}}{2} + F_{bEBP_i} \cos(\theta_{b_{i,j}}) h_{j} \right)$$

Beam Internal Work

$$W_{IB,EBP} := \sum_{i=1}^{N} \left( 2Z_{b_{i,j}} \cdot F_{y_{c_{j}}} \right)$$

Column Internal Work

$$W_{c,EBP} := 2 \cdot M_{pc}$$

Link Internal Work

$$W_{L,EBP} := N \cdot V_{P,L} \left( b_{SRC} + e_{L} \right)$$

External Work

$$W_{E,EBP} := \sum_{i=1}^{N} \left( H_{i} \cdot C_{i} \right)$$

Internal Work = External Work

$$V_{p} := \frac{W_{b,EBP} + W_{IB,EBP} + W_{c,EBP} + W_{L,EBP}}{W_{E,EBP}}$$

Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity

$$V_{p} = 888.635 \text{ kip}$$

Single Column Shear Demand

$$\Delta V_{i} := \sum_{j=1}^{N} \left( C_{j} \cdot V_{p} \right) - \left( F_{bEBP_i} + F_{bTEPB_{i,j}} \right) \cdot \cos(\theta_{b_{i,j}}) \Delta V_{i} =$$

$$\begin{align*}
\Delta V_{1} &= 404.261 \text{ kip} \\
\Delta V_{2} &= 217.532 \\
\Delta V_{3} &= 32.868
\end{align*}$$

$$F_{BH_{1}} := \left( F_{bEBP_{1}} + F_{bTEPB_{1,j}} \right) \cdot \cos(\theta_{b_{1,j}})$$

$$F_{BH} = \begin{align*}
\text{F}_{BH_{1}} &= 484.374 \text{ kip} \\
\text{F}_{BH_{2}} &= 522.998 \text{ kip} \\
\text{F}_{BH_{3}} &= 411.45
\end{align*}$$

$$F_{BV_{1}} := -\left( F_{bEBP_{1}} + F_{bTEPB_{1,j}} \right) \cdot \sin(\theta_{b_{1,j}})$$

$$F_{BV} = \begin{align*}
\text{F}_{BV_{1}} &= 264.638 \text{ kip} \\
\text{F}_{BV_{2}} &= 297.797 \text{ kip} \\
\text{F}_{BV_{3}} &= 264.638
\end{align*}$$
\[ A_n := \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \left\{ \begin{array}{ll}
(\text{kip}) & \text{if } n < N \\
0 & \text{otherwise}
\end{array} \right. \]

\[ A_n = \begin{cases}
778.246 & \text{kip} \\
486.194 & \\
209.319 &
\end{cases}
\]

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\[ M_T := \frac{1}{2} \sum_{i=n}^{N} \left[ C_i V_{p_i} \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} L + F_{BV_i} \frac{L}{n_2} - A_n L \right) \]

\[ M_T = \begin{cases}
1.628 \times 10^3 & \text{kip-ft} \\
213.639 & \\
0 & \\
4.255 \times 10^3 & \text{kip-ft}
\end{cases}
\]

\[ M_B := M_T \frac{\Delta V_n}{2} - h_n \]

\[ M_B = \begin{cases}
1.628 \times 10^3 & \text{kip-ft} \\
213.639 &
\end{cases}
\]

\[ M_{\text{max}} := \begin{cases}
M_{\text{max}} & \text{if } n = 1 \\
|M_T| & \text{if } n > 1 \land (n+1) \land |M_T| > |M_B| \\
|M_B| & \text{otherwise}
\end{cases}
\]

\[ M_{\text{max}} = 1.628 \times 10^3 \text{kip-ft} \]

\[ A_{\text{max}} := \begin{cases}
A_{\text{max}} & \text{if } n = 1 \\
|A_n| & \text{if } |A_n| > A_{\text{max}} \\
A_{\text{max}} & \text{otherwise}
\end{cases}
\]

\[ A_{\text{max}} = 778.246 \text{ kip} \]
Rehabilitated Frame controlling plastic mechanism (with Links): considers one stories' braces at expected beam pulldown and expected post-buckling strengths, other story braces at expected strengths, flexural yielding at base of columns, and yielding links with strength $V_{P,L}$ at each floor elevation.

**Brace Internal Work**

$$W_{ib,nPB} = \sum_{i=1}^{N} \left[ \left( F_{bEPB_{nPB}} \sin(\theta_{b_{nPB}}) \frac{L}{2} + F_{bEPB_{nPB}} \cos(\theta_{b_{nPB}}) h_{nPB} \right) \text{if } i \neq n_{PB} \right]$$

**Beam Internal Work**

$$W_{IB,nPB} = 2Z_{b_{nPB}} F_{ye}$$

**Column Internal Work**

$$W_{IC,nPB} = 2 M_{pc}$$

**Link Internal Work**

$$W_{II,nPB} = N V_{pL} \left( b_{SRC} + c_{L} \right)$$

**External Work**

$$W_{E,nPB} = \sum_{i=1}^{N} \left( H_{i} - C_{i} \right)$$

**Internal Work = External Work**

$$V_{p} = \frac{W_{ib,nPB} + W_{IB,nPB} + W_{IC,nPB} + W_{II,nPB}}{W_{E,nPB}}$$

**Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity**

$$V_{p} = 1,071 \times 10^{3} \text{-kip}$$

$$F_{BH_i} = \left[ \left( F_{bEPB_i} + F_{bTEPB_i} \cos(\theta_{b_{ij}}) \right) \text{if } i \neq n_{PB} \right]$$

$$F_{BH_i} = \left[ \left( F_{bEPB_i} + F_{bTEPB_i} \cos(\theta_{b_{ij}}) \right) \text{otherwise} \right]$$

$$F_{BV_i} = \left[ \left( -F_{bEPB_i} + F_{bTEPB_i} \sin(\theta_{b_{ij}}) \right) \text{if } i \neq n_{PB} \right]$$

$$F_{BV_i} = \left[ \left( -F_{bEPB_i} + F_{bTEPB_i} \sin(\theta_{b_{ij}}) \right) \text{otherwise} \right]$$

**Single Column Shear Demand**

$$\Delta V_i = \sum_{j=i}^{N} \left( C_{j} \cdot V_{p} \right) - F_{BH_i}$$

$$\Delta V_i = 586.796 \text{-kip}$$

$$\Delta V_i = 119.655 \text{-kip}$$

$$\Delta V_i = -24.751 \text{-kip}$$
\[ A_n := \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \sum_{i=n+1}^{N} \left( \frac{F_{bEPB_i}}{2} \sin(\theta_{b_i}) \right) \text{ if } n < N \quad A_n = \begin{cases} \text{662.425 kip} & \text{if } n = N \\ \text{370.372 kip} & \text{otherwise} \\ \text{166.545 kip} & \end{cases} \]

Moment on BF column with no SRC (all DV to columns)
- use to check if existing columns are adequate

\[ M_{T_n} := \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i V_p \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} L_i + F_{BV_i} \frac{L}{2} - A_n L \right) \text{ if } n < N \]

\[ M_{B_n} := M_{T_n} + \frac{\Delta V_n}{2} h_n \]

\[ M_T = \begin{cases} 4.947 \times 10^3 \text{-kip-ft} & M_B = \begin{cases} 8.761 \times 10^3 \text{-kip-ft} & M_{max} = 4.947 \times 10^3 \text{-kip-ft} \\ 1.448 \times 10^3 \text{-kip-ft} & \\ 0 \end{cases} \\ -2.226 \times 10^3 \text{-kip-ft} \\ -160.881 \end{cases} \]

\[ M_{max} := \begin{cases} M_{max} \leftarrow 0 \text{-kip-ft} & \text{for } n \in 1, 2, .. N \\ M_{max1} \leftarrow M_{T_n} \text{ if } n = 1 \\ M_{max1} \leftarrow M_{T_n} \text{ if } (n > 1) \wedge M_{T_n} > M_{B_n} \text{ otherwise} \\ M_{max} \leftarrow M_{max1} \text{ if } M_{max1} > M_{max} \\ M_{max} \text{ otherwse} \end{cases} \]

\[ A_{max} := \begin{cases} A_{max} \leftarrow 0 \text{-kip} & \text{for } n \in 1, 2, .. N \\ A_{max} \leftarrow A_n \text{ if } A_n > A_{max} \\ A_{max} \text{ otherwise} \end{cases} \]

\[ A_{max} = 662.425 \text{-kip} \]
Add higher mode forces to differential shear forces for design of the SRC

\[ \phi_1 := \begin{bmatrix} 0.2049 \\ 0.4223 \text{ if Links = "FixFix"} \\ 0.6130 \\ 0.2105 \\ 0.4205 \text{ if Links = "PinPin"} \\ 0.6122 \end{bmatrix} \]

\[ \phi_2 := \begin{bmatrix} 0.5323 \\ 0.3610 \text{ if Links = "FixFix"} \\ -0.4267 \\ 0.5255 \\ 0.3652 \text{ if Links = "PinPin"} \\ -0.4316 \end{bmatrix} \]

\[ \phi_3 := \begin{bmatrix} 0.5197 \\ -0.5356 \text{ if Links = "FixFix"} \\ 0.1953 \\ 0.5244 \\ -0.5342 \text{ if Links = "PinPin"} \\ 0.1866 \end{bmatrix} \]

\[ T := \begin{bmatrix} 0.3906 \\ 0.1399 \text{ sec if Links = "FixFix"} \\ 0.0949 \\ 0.4476 \\ 0.1408 \text{ sec if Links = "PinPin"} \\ 0.0955 \end{bmatrix} \]
Design Spectral Acceleration

\[ S_{D1} := \begin{cases} 1.2g & \text{if Haz = "MCE"} \\ 0.68g & \text{if Haz = "DBE"} \end{cases} \]

\[ S_{DS} := \begin{cases} 1.6g & \text{if Haz = "MCE"} \\ 1.1g & \text{if Haz = "DBE"} \end{cases} \]

\[ T_s := \frac{S_{D1}}{S_{DS}} \]

\[ T_0 := 0.2T_s \]

coefficient to adjust 5\% damped spectral acceleration values to 2\% damped with \( T > T_0 \) (ASCE 7-10)

\[ B_{d,\text{To}} := 0.8 \]

\[ B_{d,\text{Ex}} := 1.0 - \frac{1.0 - B_{d,\text{To}}}{T_s} \]

\[ B_{d,\text{Ex}} = \begin{pmatrix} 0.896 \\ 0.963 \\ 0.975 \end{pmatrix} \]

Effective Modal weights

\[ W_m := \sum_{i=1}^{N} \left[ W_{f_i} (\phi_{m_i}) \right] \]

\[ W = \begin{pmatrix} 1.669 \times 10^3 \\ 236.34 \\ 34.975 \end{pmatrix} \]

\[ \sum_{i=1}^{N} W_{f_i} = \begin{pmatrix} 0.86 \\ 0.122 \\ 0.018 \end{pmatrix} \]

Modal Participation Factors

\[ \Gamma_m := \frac{W_m}{\sum_{i=1}^{N} W_{f_i} (\phi_{m_i})} \]

\[ \Gamma = \begin{pmatrix} 2.081 \\ 0.783 \\ 0.301 \end{pmatrix} \]

Response Coefficient of each mode

\[ C_{s,\text{m}} := \frac{S_{DS}}{gB_{d,\text{Ex}_m}} \begin{cases} 0.4 + 0.6 \frac{T_m}{T_0} & \text{if } T_m < T_0 \\ S_{DS} & \text{if } T_0 < T_m < T_s \\ S_{D1} & \text{otherwise} \end{cases} \]

\[ C_{s,\text{m}} = \begin{pmatrix} 1.786 \\ 1.595 \\ 1.28 \end{pmatrix} \]

Modal base shear

\[ V_m := C_{s,\text{m}} W_m \]

\[ V = \begin{pmatrix} 2.98 \times 10^3 \\ 376.928 \\ 44.759 \end{pmatrix} \]
Elastic System Forces

Story shear forces of each mode
- 1st mode not relevant

\[ F_{i,m} := W_{P,m} \cdot \left( \phi \left( \Phi \right) m \right) \cdot \frac{\Gamma_{m,i}}{W_{m,i}} \cdot V_{m} \]

\[ F = \begin{pmatrix} 492.394 & 430.002 & 129.662 \\ 1.015 \times 10^3 & 291.623 & -133.629 \\ 1.473 \times 10^3 & -344.696 & 48.726 \end{pmatrix} \text{kip} \]

\[ V_{SRC,m,j, m} := \begin{cases} \Delta V_{j} & \text{if } m = 1 \\ \sum_{i=j}^{3} F_{i,m} & \text{otherwise} \end{cases} \]

\[ V_{SRC,m} = \begin{pmatrix} 586.796 & 376.928 & 44.759 \\ 119.655 & -53.074 & -84.903 \cdot \text{kip} \\ -2.751 & -344.696 & 48.726 \end{pmatrix} \]

Links = "FixFix"
Haz = "MCE"
GravCol = "None"

\[ V_{SRC,j} := P_{j} \left[ \Delta V_{j} + \sum_{m=2}^{3} \left( \frac{F_{j,m}}{W_{m,j}} \right) \right] \]

\[ V_{SRC} = \begin{pmatrix} 967.518 \\ 380.172 \cdot \text{kip} \\ 287.888 \end{pmatrix} \]

\[ n \text{PB} = 1 \quad n \text{PB} = 2 \quad n \text{PB} = 3 \]

| Pin-Pin Links, MCE Hazard | V_{PM,nPB1} := \begin{pmatrix} 721.412 \\ 197.211 \text{ kip} \\ 186.417 \end{pmatrix} | V_{PM,nPB2} := \begin{pmatrix} 487.006 \\ 412.668 \text{ kip} \\ 186.202 \end{pmatrix} | V_{PM,nPB3} := \begin{pmatrix} 528.112 \\ 228.544 \text{ kip} \\ 348.653 \end{pmatrix} |
| Pin-Pin Links, DBE Hazard | V_{PD,nPB1} := \begin{pmatrix} 608.154 \\ 121.944 \text{ kip} \\ 94.116 \end{pmatrix} | V_{PD,nPB2} := \begin{pmatrix} 373.747 \\ 337.401 \text{ kip} \\ 93.901 \end{pmatrix} | V_{PD,nPB3} := \begin{pmatrix} 414.854 \\ 153.277 \text{ kip} \\ 256.352 \end{pmatrix} |
| Fix-Fix Links, MCE Hazard | V_{FM,nPB1} := \begin{pmatrix} 967.518 \\ 380.172 \text{ kip} \\ 287.888 \end{pmatrix} | V_{FM,nPB2} := \begin{pmatrix} 733.111 \\ 595.63 \text{ kip} \\ 287.686 \end{pmatrix} | V_{FM,nPB3} := \begin{pmatrix} 774.218 \\ 411.505 \text{ kip} \\ 439.826 \end{pmatrix} |
| Fix-Fix Links, DBE Hazard | V_{FD,nPB1} := \begin{pmatrix} 852.539 \\ 305.312 \text{ kip} \\ 202.112 \end{pmatrix} | V_{FD,nPB2} := \begin{pmatrix} 618.132 \\ 520.77 \text{ kip} \\ 201.91 \end{pmatrix} | V_{FD,nPB3} := \begin{pmatrix} 659.239 \\ 336.645 \text{ kip} \\ 354.05 \end{pmatrix} |
peak axial force on links

Pin-Pin Links, MCE
Hazard

\[ \begin{align*}
& \Lambda_{PM,nPB1} := \text{for } j \in N, N - 1 \ldots 1 \\
& \quad \Lambda_{PM1_j} \leftarrow V_{PM,nPB1_j} \quad \text{if } j = N \\
& \quad \Lambda_{PM1_j} \leftarrow V_{PM,nPB1_j} - \sum_{k=j}^{N} \Lambda_{PM1_k} \quad \text{otherwise} \\
& \Lambda_{PM1} \\
& \Lambda_{PM2} := \text{for } j \in N, N - 1 \ldots 1 \\
& \quad \Lambda_{PM1_j} \leftarrow V_{PM,nPB2_j} \quad \text{if } j = N \\
& \quad \Lambda_{PM1_j} \leftarrow V_{PM,nPB2_j} - \sum_{k=j}^{N} \Lambda_{PM1_k} \quad \text{otherwise} \\
& \Lambda_{PM1} \\
& \Lambda_{PM3} := \text{for } j \in N, N - 1 \ldots 1 \\
& \quad \Lambda_{PM1_j} \leftarrow V_{PM,nPB3_j} \quad \text{if } j = N \\
& \quad \Lambda_{PM1_j} \leftarrow V_{PM,nPB3_j} - \sum_{k=j}^{N} \Lambda_{PM1_k} \quad \text{otherwise} \\
& \Lambda_{PM1} \\
& \Lambda_{PM,\text{max}} := \max(\Lambda_{PM,nPB1}, \Lambda_{PM,nPB2}, \Lambda_{PM,nPB3}) \\
& \Lambda_{PM,\text{min}} := \min(\Lambda_{PM,nPB1}, \Lambda_{PM,nPB2}, \Lambda_{PM,nPB3}) \\
& \Lambda_{PM,\text{max}} = 524.201 \text{-kip} \\
& \Lambda_{PM,\text{min}} = -120.109 \text{-kip} \\
& \Lambda_{PM} := \max(\mid \Lambda_{PM,\text{max}} \mid, \mid \Lambda_{PM,\text{min}} \mid) = 524.201 \text{-kip}
\end{align*} \]
Pin-Pin Links, DBE
Hazard

\[ A_{PD,nPB1} := \begin{cases} 
A_{PD1,j} & \text{if } j = N \\
A_{PD1,j} - \sum_{k=j}^{N} A_{PD1,k} & \text{otherwise}
\end{cases} \]

\[ A_{PD,nPB1} = \begin{cases} 
27.828 & \text{kip} \\
94.116 & \text{kip}
\end{cases} \]

\[ A_{PD,nPB2} := \begin{cases} 
A_{PD1,j} & \text{if } j = N \\
A_{PD1,j} - \sum_{k=j}^{N} A_{PD1,k} & \text{otherwise}
\end{cases} \]

\[ A_{PD,nPB2} = \begin{cases} 
243.5 & \text{kip} \\
93.901 & \text{kip}
\end{cases} \]

\[ A_{PD,nPB3} := \begin{cases} 
A_{PD1,j} & \text{if } j = N \\
A_{PD1,j} - \sum_{k=j}^{N} A_{PD1,k} & \text{otherwise}
\end{cases} \]

\[ A_{PD,nPB3} = \begin{cases} 
-103.075 & \text{kip} \\
256.352 & \text{kip}
\end{cases} \]

\[ A_{PD,\max} := \max(A_{PD,nPB1}, A_{PD,nPB2}, A_{PD,nPB3}) \]
\[ A_{PD,\min} := \min(A_{PD,nPB1}, A_{PD,nPB2}, A_{PD,nPB3}) \]

\[ A_{PD,\max} = 486.21 \text{ kip} \]
\[ A_{PD,\min} = -103.075 \text{ kip} \]

\[ A_{PD} := \max(|A_{PD,\max}|, |A_{PD,\min}|) = 486.21 \text{ kip} \]
\[ \Lambda_{FM,nPB1} := \begin{cases} \Lambda_{FM,nPB1_j} & \text{if } j = N \\ \Lambda_{FM,nPB1_j} & \Lambda_{FM,nPB1} \sum_{k=j}^{N} \Lambda_{FM1_k} & \text{otherwise} \end{cases} \]

\[ \Lambda_{FM,nPB1} = \begin{pmatrix} 587.346 \\ 92.284 \cdot \text{kip} \\ 287.888 \end{pmatrix} \]

\[ \Lambda_{FM,nPB2} := \begin{cases} \Lambda_{FM,nPB2_j} & \text{if } j = N \\ \Lambda_{FM,nPB2_j} & \Lambda_{FM,nPB2} \sum_{k=j}^{N} \Lambda_{FM1_k} & \text{otherwise} \end{cases} \]

\[ \Lambda_{FM,nPB2} = \begin{pmatrix} 137.481 \\ 307.944 \cdot \text{kip} \\ 287.686 \end{pmatrix} \]

\[ \Lambda_{FM,nPB3} := \begin{cases} \Lambda_{FM,nPB3_j} & \text{if } j = N \\ \Lambda_{FM,nPB3_j} & \Lambda_{FM,nPB3} \sum_{k=j}^{N} \Lambda_{FM1_k} & \text{otherwise} \end{cases} \]

\[ \Lambda_{FM,nPB3} = \begin{pmatrix} 362.713 \\ -28.321 \cdot \text{kip} \\ 439.826 \end{pmatrix} \]

\[ \Lambda_{FM,max} := \max(\Lambda_{FM,nPB1}, \Lambda_{FM,nPB2}, \Lambda_{FM,nPB3}) \]

\[ \Lambda_{FM,\text{min}} := \min(\Lambda_{FM,nPB1}, \Lambda_{FM,nPB2}, \Lambda_{FM,nPB3}) \]

\[ \Lambda_{FM,max} = 587.346 \cdot \text{kip} \]

\[ \Lambda_{FM,\text{min}} = -28.321 \cdot \text{kip} \]

\[ \Lambda_{FM} = \max(|\Lambda_{FM,max}|, |\Lambda_{FM,\text{min}|) = 587.346 \cdot \text{kip} \]
Fix-Fix Links, DBE

Hazard

\[ A_{\text{FD.nPB1}} := \begin{cases} V_{\text{FD.nPB1}} & \text{if } j = N \\ V_{\text{FD.nPB1}} - \sum_{k=j}^{N} A_{\text{FD1}} & \text{otherwise} \end{cases} \]

\[ A_{\text{FD1}} \]

\[ A_{\text{FD.nPB2}} := \begin{cases} V_{\text{FD.nPB2}} & \text{if } j = N \\ V_{\text{FD.nPB2}} - \sum_{k=j}^{N} A_{\text{FD1}} & \text{otherwise} \end{cases} \]

\[ A_{\text{FD1}} \]

\[ A_{\text{FD.nPB3}} := \begin{cases} V_{\text{FD.nPB3}} & \text{if } j = N \\ V_{\text{FD.nPB3}} - \sum_{k=j}^{N} A_{\text{FD1}} & \text{otherwise} \end{cases} \]

\[ A_{\text{FD1}} \]

\[ A_{\text{FD.max}} := \max \left( A_{\text{FD.nPB1}}, A_{\text{FD.nPB2}}, A_{\text{FD.nPB3}} \right) \]

\[ A_{\text{FD.min}} := \min \left( A_{\text{FD.nPB1}}, A_{\text{FD.nPB2}}, A_{\text{FD.nPB3}} \right) \]

\[ A_{\text{FD.max}} = 547.227 \text{ kip} \]

\[ A_{\text{FD.min}} = -17.405 \text{ kip} \]

\[ A_{\text{FD}} := \max \left( |A_{\text{FD.max}}|, |A_{\text{FD.min}}| \right) = 547.227 \text{ kip} \]
### Summary

<table>
<thead>
<tr>
<th>Trace Hazard</th>
<th>Haz = &quot;MCE&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Link Connectivity</td>
<td>Links = &quot;FixFix&quot;</td>
</tr>
<tr>
<td>Gravity Frame Contribution</td>
<td>GravCol = &quot;None&quot;</td>
</tr>
<tr>
<td>Plastic Shear Strength of Links</td>
<td>$V_{pL} = 160$ kip</td>
</tr>
<tr>
<td>Controlling Plastic Mechanism at EB</td>
<td>Mech$_{EB} =$ (&quot;Brace&quot;, &quot;Brace&quot;)</td>
</tr>
<tr>
<td>Controlling Plastic Mechanism at EPB</td>
<td>Mech$_{EPB} =$ (&quot;Beam&quot;, &quot;Beam&quot;)</td>
</tr>
<tr>
<td>Story with Fractured Braces</td>
<td>$n_{PB} = 1$</td>
</tr>
<tr>
<td>Plastic Base Shear</td>
<td>$V_p = 1.071 \times 10^3$ kip</td>
</tr>
</tbody>
</table>

### Values for Design

<table>
<thead>
<tr>
<th>Type of Links, Hazard</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pin-Pin Links, MCE Hazard</td>
<td>$V_{PM,Max} = \begin{cases} 721.412 \text{ kip} \ 348.653 \text{ kip} \end{cases}$</td>
</tr>
<tr>
<td>Link Axial Force $A_{PM} = 524.201$ kip</td>
<td></td>
</tr>
<tr>
<td>Pin-Pin Links, DBE Hazard</td>
<td>$V_{PD,Max} = \begin{cases} 608.154 \text{ kip} \ 256.352 \text{ kip} \end{cases}$</td>
</tr>
<tr>
<td>Link Axial Force $A_{PD} = 486.21$ kip</td>
<td></td>
</tr>
<tr>
<td>Fix-Fix Links, MCE Hazard</td>
<td>$V_{FM,Max} = \begin{cases} 967.518 \text{ kip} \ 439.826 \text{ kip} \end{cases}$</td>
</tr>
<tr>
<td>Link Axial Force $A_{FM} = 587.346$ kip</td>
<td></td>
</tr>
<tr>
<td>Fix-Fix Links, DBE Hazard</td>
<td>$V_{FD,Max} = \begin{cases} 852.539 \text{ kip} \ 354.05 \text{ kip} \end{cases}$</td>
</tr>
<tr>
<td>Link Axial Force $A_{FD} = 547.227$ kip</td>
<td></td>
</tr>
</tbody>
</table>

### Notes
- $P_s = (0.934, 0.863, 0.89)$
- $P_r = (0.013, 0.069, 0.088)$
- $P_b = (0.053, 0.068, 0.021)$
- $P_g = (0, 0, 0)$

Note: >85% of differential shear is taken by SRC
A2.8 3NCBF-BC SRC Design Inter-Story Shear Force

3NCBF-BC SRC Strength Design Calculations

OBJECTIVE
The objective of this calculation is to calculate the forces for design of the SRC. The design forces include the plastic mechanism interaction forces considering brittle fracture of LFRS connections and initial differences in nominal frame strength. Additionally, higher mode force effects are added to the plastic mechanism SRC force demands based on elastic higher mode dynamics.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration was modified to represent an older braced frame design based on a historic SEAOC seismic design procedure. This 3NCBF frame includes under-designed braced frame brace-to-gusset welds which are expected to fracture prior to the development of the full expected tensile force in each brace. This version of the prototype frame is referred to as 3NCBF-BC (brittle connection). Assuming a SRC is added to the frame, the soft-story mechanism will not form and plastic deformations will be limited to the braces and/or beam.

METHODOLOGY
The calculation first evaluates the controlling frame mechanism considering the compression brace at the expected buckling strength (EB) but not at the expected post-buckling (EPB) strength since degradation will not occur due to brittle fracture of brace connections in tension. The expected tension brace force is taken as the connection strength, or zero when considering connection fracture in tension.

The plastic base shear capacity of the frame can then be calculated using plastic analysis concepts assuming an inverted triangular load distribution. The differential shear demand can then be calculated by assuming fracture of a single story’s tension brace while all others are at their expected capacities. Assuming all at fracture or EB/weld strength force results in little differential shear. The demands on BF columns are calculated simply to determine if the existing columns are adequate. Considering a single story with fractured braces and all other at EB/weld strength results in a significant bending demand on the BF columns (considering no SRC). Once the SRC is added, the differential shear is assumed to be distributed among the SRC, braced frame columns, gravity column, and reaction column based on their relative interstory shear stiffness.

The final part of the calculation considers the portion of the differential shear taken to the SRC based on the relative stiffness of the braced frame columns, gravity column, reaction column, and SRC. Once the differential shear to develop the plastic mechanism is known (including the effects of non-uniform degradation and differences in nominal strength), the higher mode forces are added to the mechanism forces. The higher mode forces are combined with a SRSS modal combination rule and added directly to the plastic mechanism differential shear forces. The direct addition of the plastic mechanism forces and higher mode forces is likely appropriate since the structure responds within the plastic deformation “regime” during a significant amount of its cyclic hysteretic response.
Input

Earthquake Hazard
Haz := "MCE"

Link Connectivity
Links := "FixFix"

Gravity Column Contribution ("None," "Weak," or "Strong")
GravCol := "None"

Stories, Floors, Modes
N := 3 i := 1,2..N j := 1,2..N
m := 1,2..N n := 1,2..N

Geometry
L := 30ft b_{SRC} := 12ft

\[ C_i := h_i := H_i := \sum_{j=1}^{i} h_j \]
\[ \theta_{b_i} := \text{atan} \left( \frac{(2 \cdot h_i^3)}{L} \right) \]
\[ \theta_b := \left( \begin{array}{c}
40.914 \\
40.914 \cdot \text{deg}
\end{array} \right) \]

\[ e_{b_i} := L_{be_i} := \]
\[
\begin{array}{c}
625ft \\
28.75ft \\
625ft \\
28.75ft \\
625ft \\
28.75ft
\end{array}
\]

BF Sections and Material Properties
\[ Z_{b_i} := Z_c := 147in^3 \]
\[ F_{ye} := 55ksi \]

\[ I_c := 833in^4 \]
\[ E_s := 29000ksi \]

\[ A_{c_i} := 28.2in^2 \]
\[ M_{pc} := Z_c \cdot F_{ye} \]
\[ M_{pc} = 673.75\text{-kip}\cdot\text{ft} \]

Gravity Column
\[ A_{GC} := \begin{cases} 
143.5in^2 & \text{if GravCol = "Strong"} \\
143.5in^2 & \text{if GravCol = "Weak"} \\
0in^2 & \text{if GravCol = "None"}
\end{cases} \]

\[ I_{GC} := \begin{cases} 
4782.5in^4 & \text{if GravCol = "Strong"} \\
1033.2in^4 & \text{if GravCol = "Weak"} \\
0in^4 & \text{if GravCol = "None"}
\end{cases} \]
SRC, Links, Reaction Column

\[
\begin{align*}
A_{d_1} & := \ A_{SB_1} := \\
& = \begin{pmatrix} 68.5 \text{ in}^2 \\
56.8 \text{ in}^2 \\
42.7 \text{ in}^2 \\
\end{pmatrix} \\
& = \begin{pmatrix} 75.6 \text{ in}^2 \\
62 \text{ in}^2 \\
26.5 \text{ in}^2 \\
\end{pmatrix}
\end{align*}
\]

\[
\theta_{d_1} := \text{atan} \left( \frac{h_1}{b_{SRM}} \right) \quad \theta_d = 47.291 \text{-deg}
\]

Column boundary condition (for stiffness calculation below)

\[
B_{RC} := \begin{cases} 
1 & \text{if Links = "FixFix"} \\
4 & \text{otherwise}
\end{cases}
\]

Average of SRC column areas and moments of inertia

\[
A_{SC} := 57.35 \text{ in}^2 \\
I_{SC} := 8450 \text{ in}^4
\]

Reaction Column Properties

\[
A_{RC} := 134 \text{ in}^2 \\
I_{RC} := 7190 \text{ in}^4
\]

Link Properties: size considered to achieve target drift (see design and prediction calculations)

\[
V_{pL} := \begin{cases} 
553 \text{kip} & \text{if Links = "FixFix"} \\
0 \text{kip} & \text{otherwise}
\end{cases} \\
V_{pL} = 553 \text{-kip}
\]

\[
e_L := 48 \text{in}
\]

Mass and Gravity Loads

\[
m_{f_1} := \frac{6.7 \text{ kip} \cdot \text{sec}^2}{\text{in}} \\
\sum_{i=1}^{N} (m_{f_i} \cdot g) = 1.94 \times 10^3 \text{-kip}
\]

\[
W_{f_1} := m_{f_1} \cdot g
\]

\[
P_{D_1} := \begin{pmatrix} 82 \text{kip} \\
82 \text{kip} \\
77 \text{kip} \\
\end{pmatrix}
\]

\[
W_f = \begin{pmatrix} 646.698 \\
646.698 \\
646.698 \\
\end{pmatrix}
\]

A2.8-3
Expected buckling capacity of compression brace  

\[ F_{bEC_i} := \begin{cases} 
394.82 \text{kip} \\
395.63 \text{kip} \\
234.02 \text{kip} 
\end{cases} \]

Expected post-buckling equals expected buckling (no degradation)  

\[ F_{bEPB_i} := \begin{cases} 
394.82 \text{kip} \\
395.63 \text{kip} \\
234.02 \text{kip} 
\end{cases} \]

Expected tension capacity of tension brace weld  

\[ F_{bET_i} := \begin{cases} 
427.9 \text{kip} \\
464.6 \text{kip} \\
375.9 \text{kip} 
\end{cases} \]

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Buckling and beam plastic mech.  

\[ F_{bTEB_i} := \min \left( F_{bET_i}, F_{bEC_i}, \frac{4Z_b i F_{ye}}{L_{be_i} \sin(\theta_{b_i})} \right) \quad F_{bTEB} = \begin{cases} 
427.9 \text{kip} \\
464.6 \text{kip} \\
375.9 \text{kip} 
\end{cases} \]

Mech\[\text{EB}_i := \begin{cases} 
"\text{Brace}" \text{ if } F_{bTEB_i} = F_{bET_i} \\
"\text{Beam}" \text{ otherwise} 
\end{cases} \quad \text{MechEB} = \begin{cases} 
"\text{Brace}" \\
"\text{Beam}" 
\end{cases} \]

Maximum force that can develop in tension brace based on vert. equil. with comp. brace at Exp. Post-Buckling and beam plastic mech.  

\[ F_{bTEPB_i} := \min \left( 0, F_{bEPB_i} + \frac{4Z_b i F_{ye}}{L_{be_i} \sin(\theta_{b_i})} \right) \quad F_{bTEPB} = \begin{cases} 
0 \text{ kip} 
\end{cases} \]

Mech\[\text{EPB}_i := \begin{cases} 
"\text{Brace Fracture}" \text{ if } F_{bTEPB_i} = 0 \\
"\text{Beam}" \text{ otherwise} 
\end{cases} \quad \text{MechEPB} = \begin{cases} 
"\text{Brace Fracture}" \\
"\text{Beam Fracture}" 
\end{cases} \]
Relative shear stiffness calculations for determining distribution of brace interstory shear demands to SRC, RC, GC, and BF columns.

**SRC**

\[
k_s = \frac{3E_s I_{SC}}{h_1^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + h_1^2}} \left( \cos^2 \theta_1 \right) \quad k_{s1} = 4.498 \times 10^3 \text{ kip in} \\
k_{s2} = \frac{12E_s I_{SC}}{h_2^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + h_2^2}} \left( \cos^2 \theta_2 \right) \quad k_{s2} = 4.344 \times 10^3 \text{ kip in} \\
k_{s3} = \frac{12E_s I_{SC}}{b_{RC} h_3^3} + \frac{E_s A_d}{\sqrt{b_{SRC}^2 + h_3^2}} \left( \cos^2 \theta_3 \right) \quad k_{s3} = 3.458 \times 10^3 \text{ kip in}
\]

**Reaction Column**

\[
k_{r1} = \frac{3E_s I_{RC}}{h_1^3} = 164.769 \text{ kip in} \quad k_{r2} = \frac{12E_s I_{RC}}{h_2^3} = 659.074 \text{ kip in} \quad k_{r3} = \frac{12E_s I_{RC}}{b_{RC} h_3^3} = 659.074 \text{ kip in}
\]

**Gravity Column**

\[
k_{g1} = \frac{3E_s I_{GC}}{h_1^3} = 0 \text{ kip in} \quad k_{g2} = \frac{12E_s I_{GC}}{h_2^3} = 0 \text{ kip in} \quad k_{g3} = \frac{3E_s I_{GC}}{h_3^3} = 0 \text{ kip in}
\]

**Braced Frame Columns**

\[
k_{b1} = \frac{12E_s 2I_{bc}}{h_1^3} = 152.715 \text{ kip in} \quad k_{b2} = \frac{12E_s 2I_{bc}}{h_2^3} = 152.715 \text{ kip in} \quad k_{b3} = \frac{3E_s 2I_{bc}}{h_3^3} = 38.179 \text{ kip in}
\]

**Relative Contributions**

\[
k = k_s + k_r + k_g + k_b
\]

\[
P_s = \begin{pmatrix}
0.934 & 0.034 & 0 \\
0.843 & 0.128 & 0 \\
0.832 & 0.159 & 0
\end{pmatrix}
\]

\[
P_b = \begin{pmatrix}
0.032 \\
0.03 \\
9.188 \times 10^{-3}
\end{pmatrix}
\]

A2.8-5
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces at expected weld and expected buckling strengths, flexural yielding at base of columns, and yielding links with strength \( V_{pL} \) at each floor elevation.

Brace Internal Work
\[
W_{bEB} := \sum_{i = 1}^{N} \left[ \left( F_{bET} + F_{bEC} \right) \cos(\theta_{bi}) \right] h_i
\]

Column Internal Work
\[
W_{cEB} := 2M_{pc}
\]

Link Internal Work
\[
W_{lEB} := N V_{pL} \left( b_{SRC} + c_L \right)
\]

External Work
\[
W_{EEB} := \sum_{i = 1}^{N} H_i C_i
\]

Internal Work = External Work
\[
V_p := \frac{W_{bEB} + W_{cEB} + W_{lEB}}{W_{EEB}}
\]

Plastic base shear of frame with all comp. braces at Expected Buckling capacity
\[
V_p = 1.662 \times 10^3 \text{kip}
\]

Differential Shear Demand
\[
\Delta V_i := \sum_{j = i}^{N} \left( C_j V_p \right) - \left( F_{bEC} + F_{bTEB} \right) \cos(\theta_{bi})
\]

\[
\Delta V_i = \begin{cases} 
1.04 \times 10^3 \text{kip} \\
735.004 \\
370.132
\end{cases}
\]

\[
F_{BH} := \left( F_{bEC} + F_{bTEB} \right) \cos(\theta_{bi})
\]

\[
F_{BH} = \begin{cases} 
621.721 \text{kip} \\
650.066 \\
460.91
\end{cases}
\]

\[
F_{BV} := -\left( F_{bEC} + F_{bTEB} \right) \sin(\theta_{bi})
\]

\[
F_{BV} = \begin{cases} 
21.665 \text{kip} \\
45.171 \\
92.922
\end{cases}
\]
\[ A_n = \sum_{i=n}^{N} P_{D_1} + \sum_{i=n}^{N} F_{BV_i} - \sum_{i=n+1}^{N} \left( F_{BE_{i-1}} \cdot \sin(\theta_{i-1}) \right) \text{ if } n < N, \quad A_n = 0 \text{ (kip) otherwise} \]

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\[ M_{T_n} = \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_{i} \cdot V_{p} \left( \sum_{j=n+1}^{i} h_{j} \right) \right] + \sum_{i=n}^{N} \left( P_{D_1} \cdot L_n^2 + F_{BV} \cdot \frac{L_n}{2} - A_n \cdot L_n \text{ if } n < N \right) \]

\[ M_{B_n} = M_{T_n} + \frac{\Delta V_{n}}{2} \cdot h_n \]

\[ M_T = \begin{pmatrix} 7.183 \times 10^3 \\ 2.406 \times 10^3 \end{pmatrix} \text{ kip ft} \]

\[ M_B = \begin{pmatrix} 1.395 \times 10^4 \\ 7.183 \times 10^3 \end{pmatrix} \text{ kip ft} \]

\[ M_{\max} = 7.183 \times 10^3 \text{ kip ft} \]

\[ A_{\max} = 733.256 \text{ kip} \]
Rehabilitated Frame controlling plastic mechanism (with Links): considers all braces fractured, flexural yielding at base of columns, and yielding links with strength $V_{pL}$ at each floor elevation

Column Internal Work $W_{IC\_EPB} := 2M_{pc}$

Link Internal Work $W_{IL\_EPB} := N \cdot V_{pL} \left( b_{SRC} + c_{L} \right)$

External Work
$W_{E\_EPB} := \sum_{i=1}^{N} \left( H_{i} \cdot C_{i} \right)$

Internal Work = External Work
$V_{p} := \frac{W_{IC\_EPB} + W_{IL\_EPB}}{W_{E\_EPB}}$

Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity
$V_{p} = 919.5$-kip

Single Column Shear Demand
$\Delta V_{i} := \sum_{j=1}^{N} \left( C_{j} \cdot V_{p} \right) - \left( F_{b\_EPB_{i}} + F_{b\_TEPB_{i}} \right) \cos \left( \theta_{b_{i}} \right)$

$\Delta V_{i} =$
$\begin{array}{c}
621.139 \\
467.277 \\
282.904 \\
\end{array}$

$F_{BH_{i}} := \left( F_{b\_EPB_{i}} + F_{b\_TEPB_{i}} \right) \cos \left( \theta_{b_{i}} \right)$

$F_{BH} =$
$\begin{array}{c}
298.361 \\
298.973 \\
176.846 \\
\end{array}$

$F_{BV_{i}} := -\left( F_{b\_EPB_{i}} + F_{b\_TEPB_{i}} \right) \sin \left( \theta_{b_{i}} \right)$

$F_{BV} =$
$\begin{array}{c}
-258.58 \\
-259.11 \\
-153.267 \\
\end{array}$
\[ A_n := \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{PV_i}}{2} + \sum_{i=n+1}^{N} \left( F_{bEP}_{i} \sin(\theta_{b_i}) \right) \text{ if } n < N \]

\[ A_n = \begin{cases} 317.899 \text{ kip} & \text{if } n < N \\ 106.078 & \text{if } n = N \\ 0.367 & \text{if } n > N \end{cases} \]

Moment on BF column with no SRC (all DV to columns)
- use to check if existing columns are adequate

\[ M_T := \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i V_i \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} L_i + F_{BV} \frac{L}{2} - A_n \right) \text{ if } n < N \]

\[ M_T = \sum_{i=n}^{N} \left( P_{D_i} L_i + F_{BV} \frac{L}{2} - A_n \right) \text{ if } n = N \]

\[ M_B := M_T + \frac{\Delta V}{2} h_n \]

\[ M_T = \begin{cases} 4.876 \times 10^3 \text{ kip-ft} & \text{for } n \in 1,2, \ldots, N \\ 1.839 \times 10^3 \text{ kip-ft} & \text{for } n > 1 \land M_T > M_B \\ 7.848 \times 10^{-14} \text{ kip-ft} & \text{otherwise} \end{cases} \]

\[ M_B = \begin{cases} 8.914 \times 10^3 \text{ kip-ft} & \text{for } n > 1 \land M_T > M_B \\ 1.839 \times 10^3 \text{ kip-ft} & \text{otherwise} \end{cases} \]

\[ M_{\max} = \begin{cases} M_{\max} & \text{if } n \in 1,2, \ldots, N \\ M_{\max} & \text{if } n > 1 \land M_{\max} > M_{\max} \\ M_{\max} & \text{otherwise} \end{cases} \]

\[ A_{\max} = \begin{cases} A_{\max} & \text{if } A_n > A_{\max} \\ A_{\max} & \text{otherwise} \end{cases} \]

\[ A_{\max} = 317.899 \text{ kip} \]

A2.8-9
Rehabilitated Frame controlling plastic mechanism (with Links): considers one story with fractured braces and all other stories at expected buckling and expected weld strengths, flexural yielding at base of columns, and yielding links with strength VpL at each floor elevation.

Brace Internal Work
\[ W_{lb,nPB} := \sum_{i=1}^{N} \begin{cases} 0 & \text{if } i = nPB \\ \left( F_{bET_i} + F_{bEC_i} \right) \cos(\theta_{bij}) h_i & \text{otherwise} \end{cases} \]

Column Internal Work
\[ W_{lc,nPB} := 2M_{pc} \]

Link Internal Work
\[ W_{ll,nPB} := N V_{pL} (b_{SRC} + c_L) \]

External Work
\[ W_{E,nPB} := \sum_{i=1}^{N} (H_i C_i) \]

Internal Work = External Work
\[ V_p := \frac{W_{lb,nPB} + W_{lc,nPB} + W_{ll,nPB}}{W_{E,nPB}} \]

Plastic base shear of frame with all comp. braces at Expected Post-Buckling capacity
\[ V_p = 1.396 \times 10^{-3} \text{kip} \]

\[ F_{BH_i} := \begin{cases} \left( F_{bEPB_i} + F_{bTEPB_i} \cos(\theta_{bij}) \right) & \text{if } i = nPB \\ \left( F_{bEC_i} + F_{bTEC_i} \cos(\theta_{bij}) \right) & \text{otherwise} \end{cases} \]

\[ F_{BV_i} := \begin{cases} \left( -F_{bEPB_i} + F_{bTEPB_i} \sin(\theta_{bij}) \right) & \text{if } i = nPB \\ \left( -F_{bEC_i} + F_{bTEC_i} \sin(\theta_{bij}) \right) & \text{otherwise} \end{cases} \]

Single Column Shear Demand
\[ \Delta V_i := \sum_{j=i}^{N} \left( C_j V_p \right) - F_{BH_i} \]

\[ \Delta V_i = \begin{cases} 1.097 \times 10^3 \text{kip} \\ 512.961 \text{kip} \\ 236.906 \text{kip} \end{cases} \]
\( A_n := \sum_{i=n}^{N} P_{D_i} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \sum_{i=n+1}^{N} \left( F_{bE PB_i} \sin(\theta_{b_i}) \right) \text{ if } n < N \) 

\( A_n = \begin{cases} 593.133 \text{ kip} \\ 381.313 \text{ kip} \\ 123.461 \text{ kip} \end{cases} \)

Moment on BF column with no SRC (all DV to columns)
-use to check if existing columns are adequate

\( M_{T_n} := \frac{1}{2} \left[ \sum_{i=n+1}^{N} C_{l, i} V_{p} \left( \sum_{j=i+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} L_i \right) + F_{BV_n} \frac{L}{2} - A_n L \text{ if } n < N \)

\( M_{B_n} := M_{T_n} + \frac{\Delta V_n}{2} h_n \)

\( M_T = \begin{pmatrix} 4.874 \times 10^3 \\ 1.54 \times 10^3 \text{ kip ft} \\ 0 \end{pmatrix} \)

\( M_B = \begin{pmatrix} 1.201 \times 10^4 \\ 4.874 \times 10^3 \text{ kip ft} \\ 1.54 \times 10^3 \end{pmatrix} \)

\( M_{\text{max}} := \begin{cases} M_{\text{max}} \leftarrow 0 \text{ kip ft} \\ \text{for } n \in 1, 2, \ldots, N \end{cases} \)

\( M_{\text{max1}} \leftarrow \begin{cases} |M_{T_n}| \text{ if } n = 1 \\ |M_{T_n}| \text{ if } (n > 1) \land |M_{T_n}| > |M_{B_n}| \\ |M_{B_n}| \text{ otherwise} \end{cases} \)

\( M_{\text{max}} \leftarrow \begin{cases} M_{\text{max1}} \text{ if } M_{\text{max1}} > M_{\text{max}} \\ M_{\text{max}} \text{ otherwise} \end{cases} \)

\( M_{\text{max}} = 4.874 \times 10^3 \text{ kip ft} \)

\( A_{\text{max}} := \begin{cases} A_{\text{max}} \leftarrow 0 \text{ kip} \\ \text{for } n \in 1, 2, \ldots, N \end{cases} \)

\( A_{\text{max}} \leftarrow \begin{cases} |A_n| \text{ if } |A_n| > A_{\text{max}} \\ A_{\text{max}} \text{ otherwise} \end{cases} \)

\( A_{\text{max}} = 593.133 \text{ kip} \)
Add higher mode forces to differential shear forces for design of the SRC

\[
\phi_1 := \begin{cases} 
0.1973 & \text{if Links = "FixFix"} \\
0.4169 & \\
0.6191 & \\
0.2065 & \\
0.4175 & \text{if Links = "PinPin"} \\
0.6157 & 
\end{cases}
\]

\[
\phi_2 := \begin{cases}
0.5602 & \\
0.3402 & \text{if Links = "FixFix"} \\
-0.4077 & \\
0.5343 & \\
0.3611 & \text{if Links = "PinPin"} \\
-0.4242 & 
\end{cases}
\]

\[
\phi_3 := \begin{cases}
0.4927 & \\
-0.5532 & \text{if Links = "FixFix"} \\
0.2156 & \\
0.5170 & \\
-0.5393 & \text{if Links = "PinPin"} \\
0.1923 & 
\end{cases}
\]

\[
T := \begin{cases}
0.3021 & \\
0.1262 & \text{sec if Links = "FixFix"} \\
0.0895 & \\
0.4469 & \\
0.1292 & \text{sec if Links = "PinPin"} \\
0.0908 & 
\end{cases}
\]
Design Spectral Acceleration

\[ S_{D1} := \begin{cases} 
1.2g & \text{if Haz = "MCE"} \\
0.68g & \text{if Haz = "DBE"} 
\end{cases} \]

\[ S_{DS} := \begin{cases} 
1.6g & \text{if Haz = "MCE"} \\
1.1g & \text{if Haz = "DBE"} 
\end{cases} \]

\[ T_s := \frac{S_{D1}}{S_{DS}} \]

\[ T_0 := 0.2T_s \]

Coefficient to adjust 5% damped spectral acceleration values to 2% damped with T > T0 (ASCE 7-10)

\[ B_{d,To} = 0.8 \]

\[ B_{d,Ex} := 1.0 - \frac{1.0 - B_{d,To}}{T_s} \cdot T_i \]

Effective Modal Weights

\[ W_m := \left( \sum_{i=1}^{N} \left[ W_{r_i} \left( \phi_m \right)_i \right]^2 \right)^{\frac{1}{2}} \]

\[ W = \begin{pmatrix} 1.65 \times 10^3 \\ 263.5 \end{pmatrix} \text{kip} \]

\[ \sum_{i} W_{r_i} = \begin{pmatrix} 0.851 \\ 0.136 \end{pmatrix} \]

Modal Participation Factors

\[ \Gamma_m := \frac{W_m}{\sum_{i=1}^{N} \left[ W_{r_i} \left( \phi_m \right)_i \right]} \]

\[ \Gamma = \begin{pmatrix} 2.069 \\ 0.827 \end{pmatrix} \]

Response Coefficient of each mode

\[ C_{s,m} := \frac{S_{DS} \cdot \left( 0.4 + 0.6 \frac{T_m}{T_0} \right) \text{ if } T_m < T_0}{g \cdot B_{d,Ex}} \]

\[ C_s := \begin{cases} 
1.74 & \text{if } T_0 < T_m < T_s \\
1.498 & \text{otherwise} 
\end{cases} \]

Modal base shear

\[ V_m := C_{s,m} \cdot W_m \]

\[ V = \begin{pmatrix} 2.872 \times 10^3 \\ 394.749 \end{pmatrix} \text{kip} \]

\[ \begin{pmatrix} 32.471 \end{pmatrix} \]
Elastic System Forces

Story shear forces of each mode
- 1st mode not relevant

\[ F_{i,m} := W_{ij} \left( \frac{\Gamma_m}{W_m} \right) \delta_{m}^{m} V_{m} \]

\[ F = \begin{pmatrix} 459.447 & 448.83 & 103.149 \\ 970.822 & 272.567 & -115.815 \\ 1.442 \times 10^3 & -326.647 & 45.137 \end{pmatrix} \text{ kip} \]

\[ V_{SRC,j,m} := \begin{cases} \Delta V_{j} & \text{if } m = 1 \\ \sum_{i=j}^{3} F_{i,m} & \text{otherwise} \end{cases} \]

\[ V_{SRC} := P_{j} \left[ \Delta V_{j} + \sum_{m=2}^{3} \left( F_{j,m} \right)^2 \right] \]

\[ V_{SRC} = \begin{pmatrix} 1.455 \times 10^3 \\ 681.719 \end{pmatrix} \text{ kip} \]

Links = "FixFix"
Haz = "MCE"
GravCol = "None"

\[ n_{PB} = 1 \]

\[ n_{PB} = 1 \quad n_{PB} = 2 \quad n_{PB} = 3 \]

Pin-Pin Links, MCE Hazard
\[ V_{PM,nPB1} := 611.077 \text{ kip} \]
\[ V_{PM,nPB2} := (297.687 \text{ kip}) \\
\[ 123.258 \text{ kip} \]
\[ V_{PM,nPB3} = (363.751 \text{ kip}) \\
\[ 117.584 \text{ kip} \]

Pin-Pin Links, DBE Hazard
\[ V_{PD,nPB1} := 512.646 \text{ kip} \]
\[ V_{PD,nPB2} := (199.256 \text{ kip}) \\
\[ 14.396 \text{ kip} \]
\[ V_{PD,nPB3} = (300.781 \text{ kip}) \\
\[ 41.701 \text{ kip} \]

Fix-Fix Links, MCE Hazard
\[ V_{FM,nPB1} := 1.455 \times 10^3 \text{ kip} \]
\[ V_{FM,nPB2} := (1.142 \times 10^3 \text{ kip}) \\
\[ 471.572 \text{ kip} \]
\[ V_{FM,nPB3} = (969.003 \text{ kip}) \\
\[ 466.518 \text{ kip} \]

Fix-Fix Links, DBE Hazard
\[ V_{FD,nPB1} := 1.354 \times 10^3 \text{ kip} \]
\[ V_{FD,nPB2} := (1.041 \times 10^3 \text{ kip}) \\
\[ 407.202 \text{ kip} \]
\[ V_{FD,nPB3} = (910.312 \text{ kip}) \\
\[ 402.147 \text{ kip} \]

\[ V_{FD,nPB3} = (1.116 \times 10^3 \text{ kip}) \\
\[ 671.417 \text{ kip} \]

\[ 672.277 \text{ kip} \]
peak axial force on links

**Pin-Pin Links, MCE**

**Hazard**

\[
\text{APM}_{nPB1} := \begin{cases} 
\text{APM}_{1j} & \text{if } j = N \\
\text{APM}_{1j} & \sum_{k=j}^{N} \text{APM}_{1k} & \text{otherwise} 
\end{cases}
\]

\[
\text{APM}_{nPB1} = \begin{pmatrix} 534.61 \text{ kip} \\
-46.791 \text{ kip} \\
123.258 \text{ kip} 
\end{pmatrix}
\]

\[
\text{APM}_{nPB2} := \begin{cases} 
\text{APM}_{1j} & \text{if } j = N \\
\text{APM}_{1j} & \sum_{k=j}^{N} \text{APM}_{1k} & \text{otherwise} 
\end{cases}
\]

\[
\text{APM}_{nPB2} = \begin{pmatrix} -66.064 \text{ kip} \\
246.167 \text{ kip} \\
117.584 \text{ kip} 
\end{pmatrix}
\]

\[
\text{APM}_{nPB3} := \begin{cases} 
\text{APM}_{1j} & \text{if } j = N \\
\text{APM}_{1j} & \sum_{k=j}^{N} \text{APM}_{1k} & \text{otherwise} 
\end{cases}
\]

\[
\text{APM}_{nPB3} = \begin{pmatrix} 248.554 \text{ kip} \\
-295.937 \text{ kip} \\
420.793 \text{ kip} 
\end{pmatrix}
\]

\[
\text{APM}_{\text{max}} := \max(\text{APM}_{nPB1}, \text{APM}_{nPB2}, \text{APM}_{nPB3})
\]

\[
\text{APM}_{\text{min}} := \min(\text{APM}_{nPB1}, \text{APM}_{nPB2}, \text{APM}_{nPB3})
\]

\[
\text{APM}_{\text{max}} = 534.61 \text{ kip}
\]

\[
\text{APM}_{\text{min}} = -295.937 \text{ kip}
\]

\[
\text{APM} := \max(\lvert \text{APM}_{\text{max}} \rvert, \lvert \text{APM}_{\text{min}} \rvert) = 534.61 \text{ kip}
\]
Pin-Pin Links, DBE
Hazard

\[
\begin{align*}
\text{APD, nPB1} & := \begin{cases} 
\text{APD1} & \text{for } j \in N, N - 1, \ldots, 1 \\
\text{APD1} & \leftarrow \text{V}_{\text{PD, nPB1}}j, \text{ if } j = N \\
\text{APD1} & \leftarrow \text{V}_{\text{PD, nPB1}}j - \sum_{k=j}^{N} \text{APD1}k, \text{ otherwise} \\
\text{APD1} & \end{cases} \\
\text{APD, nPB1} & = \begin{pmatrix} 499.15 \\
-33.878 \\
47.374 \\
\end{pmatrix} \cdot \text{kip}
\end{align*}
\]

\[
\begin{align*}
\text{APD, nPB2} & := \begin{cases} 
\text{APD1} & \text{for } j \in N, N - 1, \ldots, 1 \\
\text{APD1} & \leftarrow \text{V}_{\text{PD, nPB2}}j, \text{ if } j = N \\
\text{APD1} & \leftarrow \text{V}_{\text{PD, nPB2}}j - \sum_{k=j}^{N} \text{APD1}k, \text{ otherwise} \\
\text{APD1} & \end{cases} \\
\text{APD, nPB2} & = \begin{pmatrix} -101.525 \\
259.08 \\
41.701 \\
\end{pmatrix} \cdot \text{kip}
\end{align*}
\]

\[
\begin{align*}
\text{APD, nPB3} & := \begin{cases} 
\text{APD1} & \text{for } j \in N, N - 1, \ldots, 1 \\
\text{APD1} & \leftarrow \text{V}_{\text{PD, nPB3}}j, \text{ if } j = N \\
\text{APD1} & \leftarrow \text{V}_{\text{PD, nPB3}}j - \sum_{k=j}^{N} \text{APD1}k, \text{ otherwise} \\
\text{APD1} & \end{cases} \\
\text{APD, nPB3} & = \begin{pmatrix} 213.093 \\
-283.023 \\
344.909 \\
\end{pmatrix} \cdot \text{kip}
\end{align*}
\]

\[
\begin{align*}
\text{APD, max} & := \max(\text{APD, nPB1}, \text{APD, nPB2}, \text{APD, nPB3}) \\
\text{APD, min} & := \min(\text{APD, nPB1}, \text{APD, nPB2}, \text{APD, nPB3}) \\
\text{APD, max} & = 499.15 \cdot \text{kip} \\
\text{APD, min} & = -283.023 \cdot \text{kip} \\
\text{APD} & := \max(\text{APD, max}, \text{APD, min}) = 499.15 \cdot \text{kip}
\end{align*}
\]
\[A_{FM,nPB1} := \begin{cases} A_{FM1,j} & \text{if } j = N \\ A_{FM1,j} & \text{if } j = N \\ A_{FM1,j} & \sum_{k=j}^{N} A_{FM1,k} \text{ otherwise} \end{cases} \]

\[A_{FM,nPB1} = \begin{cases} \text{773.281 kip} \\ \text{210.147 kip} \end{cases} \]

\[A_{FM,nPB2} := \begin{cases} A_{FM1,j} & \text{if } j = N \\ A_{FM1,j} & \sum_{k=j}^{N} A_{FM1,k} \text{ otherwise} \end{cases} \]

\[A_{FM,nPB2} = \begin{cases} \text{172.997 kip} \\ \text{502.485 kip} \end{cases} \]

\[A_{FM,nPB3} := \begin{cases} A_{FM1,j} & \text{if } j = N \\ A_{FM1,j} & \sum_{k=j}^{N} A_{FM1,k} \text{ otherwise} \end{cases} \]

\[A_{FM,nPB3} = \begin{cases} \text{486.892 kip} \\ \text{736.647 kip} \end{cases} \]

\[A_{FM} := \max(\{A_{FM,nPB1}, A_{FM,nPB2}, A_{FM,nPB3}\}) \]

\[A_{FM} := \max(\{A_{FM,nPB1}, A_{FM,nPB2}, A_{FM,nPB3}\}) = \text{773.281 kip} \]

\[A_{FM} := \max(\{A_{FM,nPB1}, A_{FM,nPB2}, A_{FM,nPB3}\}) = \text{773.281 kip} \]
Fix-Fix Links, DBE
Hazard

\[ A_{FD,nPB1} = \begin{cases} 
V_{FD,nPB1} & \text{if } j = N \\
A_{FD1_j} & \text{otherwise}
\end{cases} \]

\[ A_{FD,nPB1} = \begin{align*}
A_{FD1} & = 730.973 \text{ kip} \\
& \quad \quad \text{407.202 kip}
\end{align*} \]

\[ A_{FD,nPB2} = \begin{cases} 
V_{FD,nPB2} & \text{if } j = N \\
A_{FD1_j} & \text{otherwise}
\end{cases} \]

\[ A_{FD,nPB2} = \begin{align*}
A_{FD1} & = 130.688 \text{ kip} \\
& \quad \quad 508.165 \text{ kip}
\end{align*} \]

\[ A_{FD,nPB3} = \begin{cases} 
V_{FD,nPB3} & \text{if } j = N \\
A_{FD1_j} & \text{otherwise}
\end{cases} \]

\[ A_{FD,nPB3} = \begin{align*}
A_{FD1} & = 444.583 \text{ kip} \\
& \quad \quad 672.277 \text{ kip}
\end{align*} \]

\[ A_{FD,max} := \max(A_{FD,nPB1}, A_{FD,nPB2}, A_{FD,nPB3}) \]

\[ A_{FD,min} := \min(A_{FD,nPB1}, A_{FD,nPB2}, A_{FD,nPB3}) \]

\[ A_{FD,max} = 730.973 \text{ kip} \]

\[ A_{FD,min} = -0.86 \text{ kip} \]

\[ A_{FD} := \max(|A_{FD,max}|, |A_{FD,min}|) = 730.973 \text{ kip} \]

EPB (nPB)

A2.8-19
Summary

Earthquake Hazard
Haz = "MCE"

Link Connectivity
Links = "FixFix"

Gravity Frame Contribution
GravCol = "None"

Plastic Shear Strength of Links
\[ V_{PL} = 553\text{-kip} \]

Controlling Plastic Mechanism at EB
\[
\text{Mech}_{EB} = \begin{cases} 
"Brace" \\
"Brace" \\
"Brace"
\end{cases}
\]

Controlling Plastic Mechanism at EPB
\[
\text{Mech}_{EB} = \begin{cases} 
"Brace Fracture" \\
"Brace Fracture" \\
"Brace Fracture"
\end{cases}
\]

Story with Fractured Braces
\[ n_{PB} = 1 \]

Plastic Base Shear
\[ V_p = 1.396 \times 10^3\text{-kip} \]

Decimal percentage of differential interstory shear taken by SRC (s), reaction column (r), braced frame columns (b), and gravity column (g) on each story based on relative interstory stiffness of each component
\[
P_s = \begin{bmatrix} 0.934 \\ 0.843 \\ 0.832 \end{bmatrix} 
\]
\[
P_r = \begin{bmatrix} 0.034 \\ 0.128 \\ 0.159 \end{bmatrix} 
\]
\[
P_b = \begin{bmatrix} 0.032 \\ 0.03 \\ 9.188 \times 10^{-3} \end{bmatrix} 
\]
\[
P_g = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} 
\]

Values for Design

Pin-Pin Links, MCE Hazard

\[ V_{PM,\text{Max}} = \begin{bmatrix} 611.077 \\ 420.793 \end{bmatrix} \]

Link Axial Force
\[ A_{PM} = 534.61\text{-kip} \]

Pin-Pin Links, DBE Hazard

\[ V_{PD,\text{Max}} = \begin{bmatrix} 512.646 \\ 344.909 \end{bmatrix} \]

Link Axial Force
\[ A_{PD} = 499.15\text{-kip} \]

Fix-Fix Links, MCE Hazard

\[ V_{FM,\text{Max}} = \begin{bmatrix} 1.455 \times 10^3 \\ 736.647 \end{bmatrix} \]

Link Axial Force
\[ A_{FM} = 773.281\text{-kip} \]

Fix-Fix Links, DBE Hazard

\[ V_{FD,\text{Max}} = \begin{bmatrix} 1.354 \times 10^3 \\ 672.277 \end{bmatrix} \]

Link Axial Force
\[ A_{FD} = 730.973\text{-kip} \]
A2.9 3SCBF Improved Braced Frame Column Design Procedure

OBJECTIVE
The objective of this calculation is to calculate appropriate loads for design of CBF columns considering non-uniform brace degradation and initial differences in nominal brace strength.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 3SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration are used here however the beams in the chevron bracing configuration are assumed adequate to resist the vertical imbalanced brace force. Nonlinear dynamic analyses suggest the columns designed per current AISC Seismic Provisions (AISC 341) do not provide adequate resistance to resist the “differential” shear force that develops over one story as the braces degrade non-uniformly and there is a large difference in the nominal brace strengths compared to the inter-story shear at that story.

METHODOLOGY
The calculation approach is to calculate the plastic base shear capacity of the frame using plastic analysis concepts assuming an inverted triangular load distribution. The plastic base shear capacity is calculated considering each set of story braces at their “expected strengths” (per AISC 341) and then a non-uniform degradation of braces at each story by considering the compression brace at each story to have degraded to its “expected post-buckling strength”.

The story shear demand on the columns can then be calculated considering horizontal equilibrium at the entire frame plastic mechanism with a horizontal section cut through the entire frame at each story knowing the applied lateral shear and horizontal component of the braces’ force.

The column moment at the top of each story is determined through moment equilibrium a similar section cut at the top of each story column assuming the moment in each column is the same (appears similar from pushover analysis in SAP2000). The column moment at the bottom of each story is determined from a FBD of each story column after calculating the moment at the top of the column and column shear force thus the bottom moment is equal to top plus the shear time story height.

The axial demand in the column at each story is based on vertical equilibrium of the column on the compression side of the frame.

The column design is based on axial compression and bending interaction per AISC 360.

RESULTS/CONCLUSIONS
It was found that both column segments’ design forces are controlled by the case in which the 1st story braces are at expected post-buckling strength.

Column = W14x211
Input

\[ N := 3 \]
\[ i := 1, 2, \ldots, N \]
\[ C_i := \frac{i}{6} \quad C_i = \begin{bmatrix} 0.167 \\ 0.333 \\ 0.5 \end{bmatrix} \]
\[ \sum_i C_i = 1 \]

\[ \theta_b := \text{atan} \left( \frac{13\text{ft}}{15\text{ft}} \right) \quad \theta_b = \frac{40.914}{40.914} \text{ deg} \]

\[ L := 30\text{ft} \]
\[ H_i := \begin{bmatrix} 13\text{ft} \\ 26\text{ft} \\ 39\text{ft} \end{bmatrix} \quad h_i := \begin{bmatrix} 13\text{ft} \\ 13\text{ft} \\ 13\text{ft} \end{bmatrix} \]

\[ P_{D_i} := \begin{bmatrix} 82\text{kip} \\ 82\text{kip} \\ 77\text{kip} \end{bmatrix} \quad F_{bET_i} := \begin{bmatrix} 869.4\text{kip} \\ 869.4\text{kip} \\ 488.2\text{kip} \end{bmatrix} \quad F_{bEC_i} := \begin{bmatrix} 693.54\text{kip} \\ 693.54\text{kip} \\ 372.4\text{kip} \end{bmatrix} \quad F_{bEPB_i} := \begin{bmatrix} 208.06\text{kip} \\ 208.06\text{kip} \\ 111.72\text{kip} \end{bmatrix} \]
story with braces at 
"post-buckling" strength

\[ n_{PB} := 1 \]

\[ F_{BH_i} := \begin{cases} (F_{bET_i} + F_{bEPB_i}) \cos(\theta_{b_i}) & \text{if } i = n_{PB} \\ (F_{bET_i} + F_{bEC_i}) \cos(\theta_{b_i}) & \text{otherwise} \end{cases} \]

\[
\begin{array}{c}
F_{BH_i} = \\
814.225 ~ \text{kip} \\
1.181 \cdot 10^3 \text{kip} \\
650.346 \text{kip}
\end{array}
\]

\[ F_{BV_i} := \begin{cases} (F_{bET_i} - F_{bEPB_i}) \sin(\theta_{b_i}) & \text{if } i = n_{PB} \\ (F_{bET_i} - F_{bEC_i}) \sin(\theta_{b_i}) & \text{otherwise} \end{cases} \]

\[
\begin{array}{c}
F_{BV_i} = \\
433.132 ~ \text{kip} \\
115.176 \text{kip} \\
75.841 \text{kip}
\end{array}
\]

Lower column plastic bending strength
\[ M_{pc} := 1460 \text{kip-ft} \]
Frame Plastic Base Shear Strength

\[
\sum_{i=1}^{N} (C_i H_i) = 30.333 \text{ ft}
\]

Plastic mech. internal work (includes all brace strength plus flexural hinges at base of columns)

\[
\sum_{i=1}^{N} \left( F_{BH_i} h_{ij} + 2 M_{pc} \right) = 3.731 \times 10^4 \text{ kip-ft}
\]

\[
V_p \approx \frac{\sum_{i=1}^{N} \left( F_{BH_i} h_{ij} + 2 M_{pc} \right)}{\sum_{i=1}^{N} (C_i H_i)} = 1.23 \times 10^3 \text{ kip}
\]

Column Design Forces

\( n := 1, 2, ..., N \)

column axial force considering a FBD of only
the column on the compression side of the BF (includes the gravity loads and vertical component of the bracing members at the appropriate expected strength)

\[
A_n = \sum_{i=n}^{N} P_{Di} + \sum_{i=n}^{N} \frac{F_{BV_i}}{2} + \left\{ \begin{array}{ll}
F_{DEPB_i} \sin(\theta_{bij}) & \text{if } i = npB \\
F_{DEBC_i} \sin(\theta_{bij}) & \text{otherwise}
\end{array} \right.

\text{if } n < N

(0 \text{ kip}) \text{ otherwise}

\[
A_n = \begin{array}{c}
1.251 \times 10^3 \\
498.405 \\
114.92
\end{array} \text{ kip}
\]
Column shear demand considering a FBD of the entire frame above the story of interest (cutting through the columns and bracing members) assuming equal shear demand to both columns -assumption seems quite reasonable from SAP pushover analysis

\[ V_n = \sum_{i=n}^{N} \left( \frac{C_i \cdot V_p}{2} \right) - \frac{F_{BH}}{2} \]

\[ V_n = \begin{align*}
207.948 \\
-77.998 \\
-17.643 
\end{align*} \text{kip} \]

Column moment at top based on FBD of entire BF above story of interest with cut line immediately below floor beam

\[ M_{T_n} := \frac{1}{2} \left( \sum_{i=n+1}^{N} \left[ C_i \cdot V_p \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( p_{D_i} \cdot L_i \right) + F_{BV} \cdot \frac{L}{2} - A_n \cdot L \text{ if } n < N \right) \]

\[ M_{T_n} = \begin{pmatrix}
-1.243 \times 10^3 \\
-229.355 \\
0
\end{pmatrix} \text{kip-ft} \]

Column moment at bottom simply considering FBD of column knowing moment at top and integrating shear over height

\[ M_{B_n} := M_{T_n} + V_n \cdot h_n \]

\[ M_{B_n} = \begin{pmatrix}
1.46 \times 10^3 \\
-1.243 \times 10^3 \\
-229.355
\end{pmatrix} \text{kip-ft} \]
consider the maximum of either the top or bottom column moment for design

\[ M_{\text{design},n} := \begin{cases} M_T^n & \text{if } n = 1 \\ M_T^n & \text{if } (n > 1) \land \left| M_T^n \right| > \left| M_B^n \right| \\ M_B^n & \text{otherwise} \end{cases} \]

\[ M_{\text{design}} = \begin{pmatrix} -1.243 \times 10^3 \\ -1.243 \times 10^3 \\ -229.355 \end{pmatrix} \text{ kip-ft} \]

\[ A = \begin{pmatrix} 1.251 \times 10^3 \\ 498.405 \\ 114.92 \end{pmatrix} \text{ kip} \]

\[ n_{PB} = 1 \]
Need to update these arrays for each change in nPB or changes in frame.

\[ M_{c,S1} := \begin{pmatrix} -1.243 \times 10^3 \\ -229.355 \end{pmatrix} \text{kip-ft} \quad M_{c,S2} := \begin{pmatrix} 1.141 \times 10^3 \\ -229.355 \end{pmatrix} \text{kip-ft} \quad M_{c,S3} := \begin{pmatrix} 668.106 \\ 1.288 \times 10^3 \end{pmatrix} \text{kip-ft} \]

\[ A_{c,S1} := \begin{pmatrix} 1.251 \times 10^3 \\ 498.405 \\ 114.92 \end{pmatrix} \text{kip} \quad A_{c,S2} := \begin{pmatrix} 933.235 \\ 657.382 \\ 114.92 \end{pmatrix} \text{kip} \quad A_{c,S3} := \begin{pmatrix} 1.007 \times 10^3 \\ 413.041 \\ 200.284 \end{pmatrix} \text{kip} \]

\[ M_{max} := \max\left(\left| M_{c,S1} \right|, \left| M_{c,S2} \right|, \left| M_{c,S3} \right| \right) \]

\[ M_{max} = 1.243 \times 10^3 \text{kip-ft} \]

\[ A_{max} := \max\left(\left| A_{c,S1} \right|, \left| A_{c,S2} \right|, \left| A_{c,S3} \right| \right) \]

\[ A_{max} = 1.251 \times 10^3 \text{kip} \]
Column Design

Use AISC Table 6-1

KL = 13 ft
Lb = 13 ft
C_b := 1.67

W14x2??

\[ p := 0.436 \times 10^{-3} \text{ kip}^{-1} \]
\[ b_x := 0.668 \times 10^{-3} \text{ (kip-ft)}^{-1} \]

\[
\text{DC} := \left\{ \begin{array}{ll}
( p \cdot |A_{\text{max}}| + \frac{b_x}{C_b} |M_{\text{max}}| ) & \text{if } p \cdot |A_1| > 0.2 \\
\frac{1}{2} p \cdot |A_{\text{max}}| + \frac{9}{8} (b_x |M_{\text{max}}|) & \text{otherwise}
\end{array} \right.
\]

W14x211

\[ p := 0.399 \times 10^{-3} \text{ kip}^{-1} \]
\[ b_x := 0.608 \times 10^{-3} \text{ (kip-ft)}^{-1} \]

\[
\text{DC} := \left\{ \begin{array}{ll}
( p \cdot |A_{\text{max}}| + \frac{b_x}{C_b} |M_{\text{max}}| ) & \text{if } p \cdot |A_1| > 0.2 \\
\frac{1}{2} p \cdot |A_{\text{max}}| + \frac{9}{8} (b_x |M_{\text{max}}|) & \text{otherwise}
\end{array} \right.
\]

W14x233

\[ p := 0.361 \times 10^{-3} \text{ kip}^{-1} \]
\[ b_x := 0.544 \times 10^{-3} \text{ (kip-ft)}^{-1} \]

\[
\text{DC} := \left\{ \begin{array}{ll}
( p \cdot |A_{\text{max}}| + \frac{b_x}{C_b} |M_{\text{max}}| ) & \text{if } p \cdot |A_1| > 0.2 \\
\frac{1}{2} p \cdot |A_{\text{max}}| + \frac{9}{8} (b_x |M_{\text{max}}|) & \text{otherwise}
\end{array} \right.
\]
A2.10 6SCBF Improved Braced Frame Column Design Procedure

6SCBF Improved Column Design Calculations

OBJECTIVE
The objective of this calculation is to calculate appropriate loads for design of CBF columns considering non-uniform brace degradation and initial differences in nominal brace strength.

REFERENCES

BACKGROUND AND ASSUMPTIONS
The 6SCBF building considered by Sabelli (2001) with inverted V (chevron) bracing configuration are used here however the beams in the chevron bracing configuration are assumed adequate to resist the vertical imbalanced brace force. Nonlinear dynamic analyses suggest the columns designed per current AISC Seismic Provisions (AISC 341) do not provide adequate resistance to resist the “differential” shear force that develops over one story as the braces degrade non-uniformly and there is a large difference in the nominal brace strengths compared to the inter-story shear at that story.

METHODOLOGY
The calculation approach is to calculate the plastic base shear capacity of the frame using plastic analysis concepts assuming an inverted triangular load distribution. The plastic base shear capacity is calculated considering each set of story braces at their “expected strengths” (per AISC 341) and then a non-uniform degradation of braces at each story by considering the compression brace at each story to have degraded to its “expected post-buckling strength”.

The story shear demand on the columns can then be calculated considering horizontal equilibrium at the entire frame plastic mechanism with a horizontal section cut through the entire frame at each story knowing the applied lateral shear and horizontal component of the braces’ force.

The column moment at the top of each story is determined through moment equilibrium a similar section cut at the top of each story column assuming the moment in each column is the same (appears similar from pushover analysis in SAP2000). The column moment at the bottom of each story is determined from a FBD of each story column after calculating the moment at the top of the column and column shear force thus the bottom moment is equal to top plus the shear time story height.

The axial demand in the column at each story is based on vertical equilibrium of the column on the compression side of the frame.

For the 6SCBF frame, there is a column splice in the 4th story and thus the critical demands in the lower and upper column segment are determined. The column design is based on axial compression and bending interaction per AISC 360.

RESULTS/CONCLUSIONS
It was found that both column segments’ design forces are controlled by the case in which the 1st story braces are at expected post-buckling strength.
Lower column = W14x500
Upper column = W14x311
N := 6

\[ i := 1, 2, \ldots, N \]

\[ C_i := \]

\[
\begin{array}{c}
18 \\
303 \\
31 \\
303 \\
44 \\
303 \\
57 \\
303 \\
70 \\
303 \\
83 \\
303
\end{array}
\]

\[ \sum_{i} C_i = 1 \]

\[ \theta_{b1} := \arctan \left( \frac{18}{15} \right) = 50.194 \text{ deg} \]

\[ \theta_{b} := \arctan \left( \frac{13}{15} \right) = 40.914 \text{ deg} \]

\[ \theta_{b_i} := \begin{cases} 
\theta_{b1} & \text{if } i = 1 \\
\theta_{b} & \text{otherwise}
\end{cases} \]

\[ L := 30 \text{ ft} \]

\[ H_i := h_i := \]

\[
\begin{array}{c|c|c}
18 \text{ ft} & 18 \text{ ft} & \\
31 \text{ ft} & 13 \text{ ft} & \\
44 \text{ ft} & 13 \text{ ft} & \\
57 \text{ ft} & 13 \text{ ft} & \\
70 \text{ ft} & 13 \text{ ft} & \\
83 \text{ ft} & 13 \text{ ft} & \\
\end{array}
\]

\[ \theta_b = \begin{pmatrix} 50.194 \\
40.914 \\
40.914 \\
40.914 \\
\end{pmatrix} \text{ deg} \]

\[ P_{D_i} := \begin{array}{c|c|c|c}
82 \text{ kip} & 1107.68 \text{ kip} & 877.45 \text{ kip} & 263.23 \text{ kip} \\
82 \text{ kip} & 869.4 \text{ kip} & 693.54 \text{ kip} & 208.06 \text{ kip} \\
82 \text{ kip} & 869.4 \text{ kip} & 693.54 \text{ kip} & 208.06 \text{ kip} \\
82 \text{ kip} & 869.4 \text{ kip} & 693.54 \text{ kip} & 208.06 \text{ kip} \\
82 \text{ kip} & 627.26 \text{ kip} & 412.14 \text{ kip} & 123.64 \text{ kip} \\
77 \text{ kip} & 507.47 \text{ kip} & 270.13 \text{ kip} & 81.04 \text{ kip}
\end{array} \]

\[ F_{bET_i} := F_{bEC_i} := F_{bEPB_i} := \]
story with braces at "post-buckling" strength

\[ n_{PB} := 1 \]

\[
F_{BH_i} := \begin{cases} 
(F_{bET_i} + F_{bEPB_i}) \cdot \cos(\theta_{bi}) & \text{if } i = n_{PB} \\
\left[N_{bET_i} + F_{bEC_i}\right] \cdot \cos(\theta_{bi}) & \text{otherwise}
\end{cases}
\]

\[
F_{BH_i} = \begin{array}{c}
877.635 \\
1.181 \times 10^3 \\
1.181 \times 10^3 \\
1.181 \times 10^3 \\
785.463 \\
587.624
\end{array} \quad \text{kip}
\]

\[
F_{BV_i} := \begin{cases} 
(F_{bET_i} - F_{bEPB_i}) \cdot \sin(\theta_{bi}) & \text{if } i = n_{PB} \\
\left[N_{bET_i} - F_{bEC_i}\right] \cdot \sin(\theta_{bi}) & \text{otherwise}
\end{cases}
\]

\[
F_{BV_i} = \begin{array}{c}
648.724 \\
115.176 \\
115.176 \\
115.176 \\
140.889 \\
155.441
\end{array} \quad \text{kip}
\]

Lower column plastic bending strength

\[ M_{pc} := 3940 \text{ kip-ft} \]

(14.73 % of moment capacity)

(needs to be iterated during design, see lower column selected below)
Frame Plastic Base Shear Stength

\[ \sum_{i=1}^{6} (C_{i} \cdot H_{i}) = 60.261 \text{ ft} \]

Plastic mech. internal work
(includes all brace strength plus flexural hinges at base of columns)

\[ \sum_{i=1}^{6} \left( F_{BH_{i}} \cdot h_{i} \right) + 2 \cdot M_{pc} = 8.759 \times 10^4 \text{ kip-ft} \]

\[ V_{p} := \frac{\sum_{i=1}^{6} \left( F_{BH_{i}} \cdot h_{i} \right) + 2 \cdot M_{pc}}{\sum_{i=1}^{6} (C_{i} \cdot H_{i})} = 1.454 \times 10^3 \text{ kip} \]

Column Design Forces

\[ n := 1, 2, \ldots, N \]

\[ A_{n} = N \sum_{i=n}^{N} P_{Di} + \frac{F_{BV}}{2} + \sum_{i=n+1}^{N} \begin{cases} 
F_{BPB} \sin\left( \theta_{i} \right) & \text{if } i = n_{PB} \\
F_{BCP} \sin\left( \theta_{i} \right) & \text{otherwise} \\
0 \text{kip} & \text{otherwise} 
\end{cases} \]

if \( n < N \)

\[ A_{n} = \begin{array}{c}
2.942 \times 10^3 \text{ kip} \\
2.081 \times 10^3 \\
1.487 \times 10^3 \\
893.592 \\
484.081 \\
154.721 
\end{array} \]
\[ V_n = \sum_{i=n}^{N} \left( \frac{C_i V_p}{2} \right) - \frac{F_{BH_n}}{2} \]

\[ V_n = \begin{cases} 287.944 \text{ kip} \\ 93.039 \\ 18.684 \\ -86.852 \\ -25.753 \\ -94.732 \end{cases} \]

\[ M_{T_n} := \frac{1}{2} \sum_{i=n+1}^{N} \left[ C_i V_p \left( \sum_{j=n+1}^{i} h_j \right) \right] + \sum_{i=n}^{N} \left( P_{D_i} L \right) + F_{BV} \frac{L}{2} - A_n L \text{ if } n < N \]

\[ \sum_{i=n}^{N} \left( P_{D_i} L \right) + F_{BV} \frac{L}{2} - A_n L \text{ otherwise} \]

\[ M_T = \begin{pmatrix} -1.243 \times 10^3 \\ -2.452 \times 10^3 \\ -2.695 \times 10^3 \\ -1.566 \times 10^3 \\ -1.232 \times 10^3 \\ 0 \end{pmatrix} \text{ kip-ft} \]

\[ M_{B_n} := M_{T_n} + V_n h_n \]

\[ M_B = \begin{pmatrix} 3.94 \times 10^3 \\ -1.243 \times 10^3 \\ -2.452 \times 10^3 \\ -2.695 \times 10^3 \\ -1.566 \times 10^3 \\ -1.232 \times 10^3 \end{pmatrix} \text{ kip-ft} \]
consider the maximum of either the top or bottom column moment for design

\[ M_{c,\text{design}} = \begin{cases} M_T & \text{if } n = 1 \\ M_T & \text{if } (n > 1) \land \left| M_T \right| > \left| M_B \right| \\ M_B & \text{otherwise} \end{cases} \]

\[ n_{PB} = 1 \]

\[ M_{c,\text{design}} = \begin{pmatrix} -1.243 \times 10^3 \\ -2.452 \times 10^3 \\ -2.695 \times 10^3 \\ -2.695 \times 10^3 \\ -1.566 \times 10^3 \\ -1.232 \times 10^3 \end{pmatrix} \text{ kip-ft} \]

\[ A = \begin{pmatrix} 2.942 \times 10^3 \\ 2.081 \times 10^3 \\ 1.487 \times 10^3 \\ 893.592 \\ 484.081 \\ 154.721 \end{pmatrix} \text{ kip} \]
Need to update these arrays for each change in nPB or changes in frame

\[
\begin{align*}
M_{c,S1} & := \begin{pmatrix}
-1.243 \times 10^3 \\
-2.452 \times 10^3 \\
-2.695 \times 10^3 \\
-2.695 \times 10^3 \\
-1.566 \times 10^3 \\
-1.232 \times 10^3
\end{pmatrix} \text{kip-ft} \\
M_{c,S2} & := \begin{pmatrix}
1.951 \times 10^3 \\
1.951 \times 10^3 \\
1.951 \times 10^3 \\
-2.329 \times 10^3 \\
-2.329 \times 10^3 \\
-2.329 \times 10^3
\end{pmatrix} \text{kip-ft} \\
M_{c,S3} & := \begin{pmatrix}
1.951 \times 10^3 \\
1.951 \times 10^3 \\
1.951 \times 10^3 \\
-2.329 \times 10^3 \\
-2.329 \times 10^3 \\
-2.329 \times 10^3
\end{pmatrix} \text{kip-ft} \\
M_{c,S4} & := \begin{pmatrix}
1.951 \times 10^3 \\
1.951 \times 10^3 \\
1.516 \times 10^3 \\
-1.372 \times 10^3 \\
-1.372 \times 10^3 \\
-1.163 \times 10^3
\end{pmatrix} \text{kip-ft} \\
M_{c,S5} & := \begin{pmatrix}
1.516 \times 10^3 \\
1.662 \times 10^3 \\
1.662 \times 10^3 \\
-604.514 \\
-604.514 \\
-1.106 \times 10^3
\end{pmatrix} \text{kip-ft} \\
M_{c,S6} & := \begin{pmatrix}
1.516 \times 10^3 \\
1.662 \times 10^3 \\
1.662 \times 10^3 \\
-604.514 \\
-604.514 \\
-1.106 \times 10^3
\end{pmatrix} \text{kip-ft}
\end{align*}
\]
\[ M_{\text{max}L} := M_{\text{max}} \leftarrow 0 \text{kip ft} \]

\[ M_{\text{max}} \]

\[ M_{\text{max}} \]

\[ M_{\text{max}L} = 2.695 \times 10^3 \text{kip ft} \]

\[ M_{\text{max}U} := M_{\text{max}} \leftarrow 0 \text{kip ft} \]

\[ M_{\text{max}} \]

\[ M_{\text{max}U} \]

\[ M_{\text{max}U} = 2.695 \times 10^3 \text{kip ft} \]

\[ A_{\text{max}L} := A_{\text{max}} \leftarrow 0 \text{kip ft} \]

\[ A_{\text{max}} \]

\[ A_{\text{max}} \]

\[ A_{\text{max}L} = 2.942 \times 10^3 \text{kip} \]

\[ A_{\text{max}U} := A_{\text{max}} \leftarrow 0 \text{kip ft} \]

\[ A_{\text{max}} \]

\[ A_{\text{max}U} \]

\[ A_{\text{max}U} = 1.053 \times 10^3 \text{kip} \]
Lower Column Segment Design

Use AISC Table 6-1

KL = 18 ft
Lb = 18 ft \quad C_b := 1.67

W14x283

\[ p := 0.325 \times 10^{-3} \text{kip}^{-1} \]
\[ b_x := 0.443 \times 10^{-3} \text{kip-ft}^{-1} \]

\[
DC := \begin{cases} 
 p \left| A_{\text{maxL}} \right| + \frac{b_x \left| M_{\text{maxL}} \right|}{C_b} & \text{if } p \left| A_{\text{maxL}} \right| > 0.2 = 1.671 \\
\frac{1}{2} p \left| A_{\text{maxL}} \right| + \frac{9}{8} (b_x \left| M_{\text{maxL}} \right|) & \text{otherwise}
\end{cases}
\]

W14x311

\[ p := 0.295 \times 10^{-3} \text{kip}^{-1} \]
\[ b_x := 0.398 \times 10^{-3} \text{kip-ft}^{-1} \]

\[
DC := \begin{cases} 
 p \left| A_{\text{maxL}} \right| + \frac{b_x \left| M_{\text{maxL}} \right|}{C_b} & \text{if } p \left| A_{\text{maxL}} \right| > 0.2 = 1.51 \\
\frac{1}{2} p \left| A_{\text{maxL}} \right| + \frac{9}{8} (b_x \left| M_{\text{maxL}} \right|) & \text{otherwise}
\end{cases}
\]

W14x342

\[ p := 0.266 \times 10^{-3} \text{kip}^{-1} \]
\[ b_x := 0.356 \times 10^{-3} \text{kip-ft}^{-1} \]

\[
DC := \begin{cases} 
 p \left| A_{\text{maxL}} \right| + \frac{b_x \left| M_{\text{maxL}} \right|}{C_b} & \text{if } p \left| A_{\text{maxL}} \right| > 0.2 = 1.357 \\
\frac{1}{2} p \left| A_{\text{maxL}} \right| + \frac{9}{8} (b_x \left| M_{\text{maxL}} \right|) & \text{otherwise}
\end{cases}
\]
\[ p := 0.246 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.325 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[ \text{DC} := \begin{cases} 
& \left( p \cdot |A_{\text{maxL}}| + \frac{b_x \cdot |M_{\text{maxL}}|}{C_b} \right) \quad \text{if} \quad p \cdot |A_{\text{maxL}}| \geq 0.2 \quad = 1.248 \\
& \left[ \frac{1}{2} \cdot p \cdot |A_{\text{maxL}}| + \frac{9}{8} \left( b_x \cdot |M_{\text{maxL}}| \right) \right] \quad \text{otherwise} 
\end{cases} \]

\[ p := 0.228 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.298 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[ \text{DC} := \begin{cases} 
& \left( p \cdot |A_{\text{maxL}}| + \frac{b_x \cdot |M_{\text{maxL}}|}{C_b} \right) \quad \text{if} \quad p \cdot |A_{\text{maxL}}| \geq 0.2 \quad = 1.152 \\
& \left[ \frac{1}{2} \cdot p \cdot |A_{\text{maxL}}| + \frac{9}{8} \left( b_x \cdot |M_{\text{maxL}}| \right) \right] \quad \text{otherwise} 
\end{cases} \]

\[ p := 0.213 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.275 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[ \text{DC} := \begin{cases} 
& \left( p \cdot |A_{\text{maxL}}| + \frac{b_x \cdot |M_{\text{maxL}}|}{C_b} \right) \quad \text{if} \quad p \cdot |A_{\text{maxL}}| \geq 0.2 \quad = 1.07 \\
& \left[ \frac{1}{2} \cdot p \cdot |A_{\text{maxL}}| + \frac{9}{8} \left( b_x \cdot |M_{\text{maxL}}| \right) \right] \quad \text{otherwise} 
\end{cases} \]

\[ p := 0.18 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.227 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[ \text{DC} := \begin{cases} 
& \left( p \cdot |A_{\text{maxL}}| + \frac{b_x \cdot |M_{\text{maxL}}|}{C_b} \right) \quad \text{if} \quad p \cdot |A_{\text{maxL}}| \geq 0.2 \quad = 0.896 \\
& \left[ \frac{1}{2} \cdot p \cdot |A_{\text{maxL}}| + \frac{9}{8} \left( b_x \cdot |M_{\text{maxL}}| \right) \right] \quad \text{otherwise} 
\end{cases} \]
Upper Column Segment Design

Use AISC Table 6-1

KL = 13 ft

Lb = 13 ft \quad C_b := 1.67

W14x233

\[ p := 0.361 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.544 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[
DC := \begin{cases} 
\left( p \left| A_{\text{max}U} \right| + \frac{b_x M_{\text{max}U}}{C_b} \right) & \text{if } p \left| A_{\text{max}U} \right| > 0.2 = 1.258 \\
\left[ \frac{1}{2} p \left| A_{\text{max}U} \right| + \frac{9}{8} (b_x M_{\text{max}U}) \right] & \text{otherwise}
\end{cases}
\]

W14x257

\[ p := 0.326 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.487 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[
DC := \begin{cases} 
\left( p \left| A_{\text{max}U} \right| + \frac{b_x M_{\text{max}U}}{C_b} \right) & \text{if } p \left| A_{\text{max}U} \right| > 0.2 = 1.129 \\
\left[ \frac{1}{2} p \left| A_{\text{max}U} \right| + \frac{9}{8} (b_x M_{\text{max}U}) \right] & \text{otherwise}
\end{cases}
\]

W14x283

\[ p := 0.296 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.437 \times 10^{-3} \text{(kip-ft)}^{-1} \]

\[
DC := \begin{cases} 
\left( p \left| A_{\text{max}U} \right| + \frac{b_x M_{\text{max}U}}{C_b} \right) & \text{if } p \left| A_{\text{max}U} \right| > 0.2 = 1.017 \\
\left[ \frac{1}{2} p \left| A_{\text{max}U} \right| + \frac{9}{8} (b_x M_{\text{max}U}) \right] & \text{otherwise}
\end{cases}
\]
\[ p := 0.269 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.393 \times 10^{-3} \text{ (kip ft)}^{-1} \]

\[
\text{DC} := \begin{cases} 
\left( p \left| A_{\max U} \right| + \frac{b_x \left| M_{\max U} \right|}{C_B} \right) & \text{if } p \left| A_{\max U} \right| > 0.2 = 0.917 \\
\left[ \frac{1}{2} p \left| A_{\max U} \right| + \frac{9}{8} \left( b_x \left| M_{\max U} \right| \right) \right] & \text{otherwise}
\end{cases}
\]

\[ p := 0.243 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.353 \times 10^{-3} \text{ (kip ft)}^{-1} \]

\[
\text{DC} := \begin{cases} 
\left( p \left| A_{\max U} \right| + \frac{b_x \left| M_{\max U} \right|}{C_B} \right) & \text{if } p \left| A_{\max U} \right| > 0.2 = 0.826 \\
\left[ \frac{1}{2} p \left| A_{\max U} \right| + \frac{9}{8} \left( b_x \left| M_{\max U} \right| \right) \right] & \text{otherwise}
\end{cases}
\]

\[ p := 0.225 \times 10^{-3} \text{kip}^{-1} \]

\[ b_x := 0.322 \times 10^{-3} \text{ (kip ft)}^{-1} \]

\[
\text{DC} := \begin{cases} 
\left( p \left| A_{\max U} \right| + \frac{b_x \left| M_{\max U} \right|}{C_B} \right) & \text{if } p \left| A_{\max U} \right| > 0.2 = 0.757 \\
\left[ \frac{1}{2} p \left| A_{\max U} \right| + \frac{9}{8} \left( b_x \left| M_{\max U} \right| \right) \right] & \text{otherwise}
\end{cases}
\]
APPENDIX 3: EQUIVALENT CROSS SECTION CALCULATIONS

A3.1 Equivalent Wide Flange Section

\[ A := 5.87 \text{in}^2 \quad d := 8\text{in} \quad \text{TOL} = 1 \times 10^{-3} \]
\[ Z_x := 11.4\text{in}^3 \quad b_f := 4\text{in} \quad \text{CTOL} = 1 \times 10^{-3} \]
\[ I_x := 23.8\text{in}^4 \quad t_f := .25\text{in} \quad \text{TOL} := 0.00 \]
\[ A_w := 1.46\text{in}^2 \quad t_w := .25\text{in} \quad \text{CTOL} := 0.00 \]

Given
\[ A = 2t_f \cdot b_f + t_w(d - 2t_f) \]
\[ A_w = d \cdot t_w \]
\[ I_x = 2\left(\frac{1}{12}b_f \cdot t_f^3\right) + \frac{1}{12}t_w(d - 2t_f)^3 + 2t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 \]
\[ Z_x = 2\left[t_w \left(\frac{d}{2} - t_f\right) \cdot \frac{d}{2} - \frac{t_f}{2}\right] + 2 \left[ b_f \cdot t_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \right] \]
\[ \begin{bmatrix} d \\ b_f \\ t_f \\ t_w \end{bmatrix} := \text{Find}(d, b_f, t_f, t_w) = \begin{bmatrix} 4.85 \\ 4.866 \\ 0.483 \\ 0.301 \end{bmatrix} \text{in} \]

\[ A := 2t_f \cdot b_f + t_w(d - 2t_f) = 5.87\text{in}^2 \]
\[ A_w := d \cdot t_w = 1.46\text{in}^2 \]
\[ I_x := 2\left(\frac{1}{12}b_f \cdot t_f^3\right) + \frac{1}{12}t_w(d - 2t_f)^3 + 2t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 = 23.973\text{in}^4 \]
\[ Z_x := 2\left[t_w \left(\frac{d}{2} - t_f\right) \cdot \frac{d}{2} - \frac{t_f}{2}\right] + 2 \left[ b_f \cdot t_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \right] = 11.4\text{in}^3 \]
A3.2 Equivalent Square HSS Section

\[
\begin{align*}
  r_y & := 0.6 \text{in} & d & := 3 \text{in} & \text{TOL} = 1 \times 10^{-3} \\
  A & := 0.96 \text{in}^2 & t & := 0.125 \text{in} & \text{CTOL} = 1 \times 10^{-3} \\
  I_y & := 0.355 \text{in}^4 & b_f & := 1.5 \text{in} & \text{TOL} := 0.0 \text{l} \\
  & & t_w & := 0.177 \text{in} & \text{CTOL} := 0.0 \\
\end{align*}
\]

Given

\[
A = 4t(d - t)
\]

\[
I_y = \frac{2t d^3}{12} + 2 \left[ \frac{(d - 2t) t^3}{12} + (d - 2t) t \left( \frac{d - t}{2} \right)^2 \right]
\]

\[
\left( \frac{d}{t} \right) := \text{Find}(d, t) = \left( \frac{1.64279}{0.16209} \right) \text{in}
\]

A := 4t(d - t) = 0.96 \text{in}^2

\[
I_y := \frac{2t d^3}{12} + 2 \left[ \frac{(d - 2t) t^3}{12} + (d - 2t) t \left( \frac{d - t}{2} \right)^2 \right] = 0.355 \text{in}^4
\]

\[
r_y := \sqrt{\frac{I_y}{A}} = 0.608 \text{in}
\]

A3.3 Verification of Desired Equivalent Scaled Section Behavior

A simple model consisting of a single cantilever beam was constructed in OpenSees and two simulations were performed to verify the equivalent cross section scaling procedure. A 100 inch long nonlinear fiber beam element with an equivalent scaled W30x116 cross section (d = 9.076 in, b_f = 11.197 in, t_f = 0.074 in, t_w = 0.177 in) was subjected to an axial and transverse load (in separate simulations) applied at the tip of the beam. The “Steel02” material type was used with a specified modulus of elasticity of 29000 ksi, yield stress of 50 ksi, and strain hardening of 0.1%. This material model gradually transitions between linear and nonlinear behavior near the specified yield stress. Figure A-Figure A-1 shows the results of these simulations. When subjected to the...
axial load, the model responded as expected, with an elastic modulus of 29000 ksi and a yield stress of 50 ksi. When the transverse load was applied, the beam also behaved as expected, with a transition between linear and nonlinear behavior at the section’s plastic moment, $M_p$. This confirms that the equivalent cross sections behave as expected.

**Figure A-1:** Equivalent Scaled Cross Section Verification Simulation Results for Axial (a) and Transverse Loading (b).
Figure A-2: 3SCBF Elevation View
Figure A-3: 3SCBF Connection Details 1
Figure A-4: 3SCBF Connection Details 2
Figure A-5: 3SCBF Connection Details 3
Figure A-6: 3NCFB Elevation View
Figure A-7: 3NCBF Connection Details 1
Figure A-8: 3NCBF Connection Details 2
Figure A-9: 3NCBF Connection Details 3
Figure A-10: 6SCF Elevation View
Figure A-11: 6SCBF Connection Details 1
Figure A-12: 6SCBF Connection Details 2
Figure A-13: SRC and Links Elevation View
Figure A-14: SRC Story 1 Details

A325 Bolts. All other bolted connections on this sheet use A490 Bolts.

2L 4x4x3/4

5/16" 5/16" 5/16" 5/16"

1 1 1 1

3/4" 3/4" 3/4" 3/4"

3/4" 3/4" 3/4" 3/4"

2L 4x4x3/4, Leg Cut Gusset, t = 7/8"

1/2" 1/2"

Column to Base Plate

4 Bolts in Line (8 Total for SRC to Rocker Bearing Connection)
Figure A-17: HSS Link Details
## Appendix 5: Specimen Material Coupon Test Results

### 3SCBF Material Coupons

<table>
<thead>
<tr>
<th>Coupon Name</th>
<th>3SCBF - ColF1E1</th>
<th>3SCBF - ColF1E2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coupon Location</strong></td>
<td>East Column Flange Story 1 (Coupon #1)</td>
<td>East Column Flange Story 1 (Coupon #2)</td>
</tr>
<tr>
<td><strong>Section Width (in)</strong></td>
<td>0.4825</td>
<td>0.3600</td>
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<tr>
<td><strong>Section Depth (in)</strong></td>
<td>0.3790</td>
<td>0.4825</td>
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<tr>
<td><strong>Reduced Section Length (in)</strong></td>
<td>3.5500</td>
<td>3.6000</td>
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<tr>
<td><strong>Cross Sectional Area (in²)</strong></td>
<td>0.1829</td>
<td>0.1737</td>
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<tr>
<td><strong>Yield Stress (ksi)</strong></td>
<td>51.50</td>
<td>47.50</td>
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<tr>
<td><strong>Modulus of Elasticity (ksi)</strong></td>
<td>29000.00</td>
<td>29000.00</td>
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<tr>
<td><strong>Ultimate Stress (ksi)</strong></td>
<td>63.49</td>
<td>60.57</td>
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</table>

![Stress-Strain Diagram](image-url)
<table>
<thead>
<tr>
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<th>3SCBF - BeF11</th>
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<tbody>
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<td><strong>Coupon Location</strong></td>
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<tr>
<td><strong>Section Width</strong></td>
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<td><strong>Section Depth</strong></td>
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<td><strong>Length (in)</strong></td>
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<td><strong>Cross Sectional</strong></td>
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<tr>
<td><strong>Area (in²)</strong></td>
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<tr>
<td><strong>Yield Stress</strong></td>
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<tr>
<td><strong>Modulus of</strong></td>
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<tr>
<td><strong>Elasticity (ksi)</strong></td>
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<tr>
<td><strong>Ultimate Stress</strong></td>
<td>64.66</td>
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<tr>
<td><strong>(ksi)</strong></td>
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<td><strong>Section Depth</strong></td>
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<td><strong>Reduced Section</strong></td>
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<td><strong>Length (in)</strong></td>
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<tr>
<td><strong>Cross Sectional</strong></td>
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<tr>
<td><strong>Area (in²)</strong></td>
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<tr>
<td><strong>Yield Stress</strong></td>
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<tr>
<td><strong>Modulus of</strong></td>
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<td><strong>Elasticity (ksi)</strong></td>
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<td><strong>Ultimate Stress</strong></td>
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<td><strong>(ksi)</strong></td>
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<td>Coupon Name</td>
<td>3SCBF - Br1E1</td>
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<tr>
<td>-------------------</td>
<td>-------------------------------</td>
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<tr>
<td>Coupon Location</td>
<td>East Brace Story 1 (Coupon #1)</td>
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<td>Cross Sectional Area (in²)</td>
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<td>Ultimate Stress (ksi)</td>
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<td>Cross Sectional Area (in²)</td>
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<tr>
<td>Yield Stress (ksi)</td>
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<td>Ultimate Stress (ksi)</td>
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### 3NCBF Material Coupons

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<td>Reduced Section Length (in)</td>
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<td>Cross Sectional Area (in^2)</td>
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<tr>
<td>Ultimate Stress (ksi)</td>
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### Graphs

- **Graph 1:** Stress vs. Strain
- **Graph 2:** Stress vs. Strain
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![Graph](chart.png)
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<td>(Coupon #2)</td>
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<td>Ultimate Stress (ksi)</td>
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### 6SCBF Material Coupons

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<td><strong>Modulus of Elasticity</strong></td>
<td>(ksi) 29000.00</td>
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<td><strong>Ultimate Stress</strong></td>
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<table>
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APPENDIX 6: INSTRUMENTATION POST PROCESSING CALCULATIONS

A6.1 Calculation of Internal Forces Using Strain Gauge Readings

\[
\begin{align*}
\sigma &= E \varepsilon \\
F &= \sigma A \\
M &= \frac{\sigma_b y_x}{y} \\
\varepsilon_a &= \frac{\varepsilon_1 + \varepsilon_2}{2} \\
\varepsilon_b &= \varepsilon_{123} - \varepsilon_a \\
S_x &= \frac{2l_{xx}}{d}
\end{align*}
\]

\[M := E S_x \varepsilon_b \quad \text{Axial} := \varepsilon_a A E \quad M_1 := E S_x (\varepsilon_2 - \varepsilon_a) \quad M_2 := E S_x (\varepsilon_3 - \varepsilon_a) \quad V := \frac{M_2 - M_1}{L}\]

A6.2 Calculation of Brace Deformations Using Krypton Coordinate Measurements

\[
\begin{align*}
\Delta A &= \left(\frac{L_2 - L_1}{L_1}\right) L_T \\
\Delta O &= \frac{y_2 - y_1}{x_2 - x_1} \left( x_2 - \frac{L_T}{2} \right) + y_2
\end{align*}
\]
A6.3 Calculation of Link Rotation Using Krypton Coordinate Measurements

\[
x_{av} := \frac{x_1 + x_2}{2} \quad z_{av} := \frac{z_1 + z_2}{2} \quad \theta_r := \sin \left( \frac{z_3 - z_{av}}{L_H - x_3 - x_{av}} \right)
\]

\[
\theta_v := \tan \left( \frac{z_3 - z_{av}}{L_H - x_3 - x_{av}} \right) \quad \theta_T := \theta_r + \theta_v
\]
References


