A STUDY OF STEEL MOMENT CONNECTIONS FOR STRUCTURES UNDER BLAST AND PROGRESSIVE COLLAPSE LOADING RATES

A Thesis in
Civil Engineering
by
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Connections are one of the most significant contributors to decide the ductility and robustness of a steel framed structure. Particularly, moment connections have been used in lateral loading resistant frame design due to their ability to reduce the relative rotation between beams and columns. However, they are vulnerable to static and dynamic loads, such as progressive collapse rate, earthquake, and blast-rate loads. Therefore, it is essential to understand and determine the steel connection behaviors in order to offer the necessary structural capabilities in design for resisting blast and progressive collapse. In present study the welded-unreinforced flange-bolted-web connections (WUF-B), one of the commonly used moment connections, were characterized with respect to the quasi-static and blast-rate pressure loads. The characterization process was carried out by finite element analyses of a full three-dimensional connection assembly. Moment-rotation curves, moment-tip displacement relationships, rotation dynamic increase factors, and moment-impulse diagrams were utilized as static and dynamic connection properties. The complicated connection configurations were simplified using infinitesimal point element with the mechanical properties. Other possible configuration of WUF-B connections were designed and analyzed in both quasi-static and short duration loading environment. The characterized resistant functions were compiled in the database.
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Chapter 1

INTRODUCTION

1.1 Problem Description and Research Significance

Abnormal loading generated from blast or impact may cause local structural damage in a building. Such local failure may propagate from element to element, and eventually to the whole structural system. The local failure and subsequent progressive collapse may cause significant casualties, and loss of property. Therefore, structures have to be designed to limit the effects of local collapse and to prevent or minimize progressive collapse. Structural ductility, continuity, and redundancy are important parameters in mitigating such consequences. This could be accomplished in steel frame structures if their connections are properly designed and constructed. Structural connections are important contributors to ductility and robustness of the structural systems. However, connections are vulnerable to static and dynamic loads, as observed in the World Trade Center collapse, and the Northridge earthquake. The post-blast behavior of such structures can be assessed by advanced numerical simulations that require accurate and numerically-efficient connection models. Such models do not exist currently, due to the lack of a full understanding of their behavior. Accordingly, an accurate understanding of steel connection behavior is essential to provide the necessary ductility and resistance in design for mitigating blast effect and progressive collapse. The present study is aimed at characterizing connection behavior based upon the finite element analyses of a full three-dimensional connection assembly and simplifying the configurations which are feasible in fast-running algorithms for structural assessment.
1.2 Research Objectives and Scope

The objectives of this study are to:

1. Derive a better understanding of how three-dimensional welded-unreinforced flange-bolted-web (WUF-B) steel moment connections behave under quasi-static and high-strain-rate loads by means of characterizing their resistance, ultimate strength, and ductility in the form of detailed moment-rotation relationships.

2. Apply the established quasi-static and high-strain-rate resistance functions for the detailed connection models to formulate connection elements to be used in simplified frame analyses.

3. Evaluate the existing connection design criteria by quasi-static and blast analyses results using numerical models.

4. Construct resistance function databases of other connection configurations for full frame analyses in a parallel investigation and express the functions in the mathematical forms.

The objectives will be accomplished within the following scope:

1. A literature review of types and performance of steel connections under a variety of loading conditions was carried out with particular emphasis on the WUF-B connections which are selected for this study.

2. Validation of representative numerical models for the WUF-B connection was conducted by comparison with known test data. Structural dimensions and material properties were based on tests by other researchers.
3. Moment-rotation relationships were determined from the validated connection model under quasi-static pressure to characterize the connection performances under progressive collapse-rate loads.

4. Information on connection behaviors under blast-rate loads was derived by applying high-strain-rate pressures.

5. Investigated properties of the connections were assigned to a finite element frame analysis with simpler beams, columns, and connector elements in lieu of full three dimensional (3D) FE models with 3D elements for these structural members. The results from the simple frame analyses were compared to those obtained from the analyses with detailed 3D FE models in order to demonstrate the approach feasibility.

6. Blast damage criteria for steel connections specified in TM5-1300 were evaluated comparing with the simulation results.

7. Strain levels of individual components were investigated for damage assessments.

8. Different cases of WUB-F connections were designed by the Load and Resistance Factor Design (LRFD) procedures of AISC.

9. Following quasi-static and high-strain-rate push-over numerical analyses were conducted on the designed connections and the results were compiled in the database. The database is expected to define the characteristics of the connections under different load conditions.

10. Discussions of the findings lead to conclusions and recommendations for future research and possible implementation into the design of steel moment connection.
Chapter 2

LITERATURE REVIEW

2.1 Introduction

As a result of a continuous effort, a great number of published papers, reports, and design guidelines have been presented in the field of steel connections. Many studies attempted to determine the effect of quasi-static or seismic loads on steel moment connections by applying continuous or cyclic loads. Studies for determining material properties of the comprised members under static and dynamic states were conducted. This chapter will review the related research and their relevance to this study.

2.2 General Definition of Steel Connections

Steel construction is categorized by the Load and Resistance Factor Design (LRFD) approach into Fully-Restrained (FR), Partially-Restrained (PR), and Unrestrained (Simple) frames (American Institute of Steel Construction, 2003). Each frame condition is defined in terms of the amount of rotational restraint at the connections. These classifications of connections are best shown in Figure 2-1 (Salmon and Johnson, 1996). When $M_{Fa}$ is the moment in the fully fixed beam end point $a$ ($\theta_a = 0$), typical FR connections would carry an end moment $M_a$, about 90% or more of $M_{Fa}$. On the other hand, simple connections may resist only 20% or less of $M_{Fa}$. PR connections are the connections capable of intermediate moment capacities between FR and simple connections.
Figure 2-1 Moment-Rotation Curves of AISC Connection Types (Salmon and Johnson, 1996)
Welded-unreinforced flange-bolted-web connections (WUF-B), one of the FR connections, are commonly used in welded steel moment-resisting frames (Figure 2-2). These connections utilize complete joint penetration (CJP) groove welds to join beam or girder flanges to column flanges. A single plate shear tab is welded to the column flange and pre-drilled beam webs are bolted to the shear tab with supplemental weld around the plate edges. Continuity plates (stiffeners) at the top and bottom of the panel zone may be present based on the capacity of flange bending and web yielding/crippling in the column panel zone. Doubler plate can reinforce the shear capacity of the panel zone.

![Diagram of Welded-Unreinforced Flange-Bolted-Web Connection](image)

1. top and bottom flange weld
2. bolted shear tab, weld to column flange with fillet weld both sides, or with complete joint penetration weld (CJP)
3. continuity plates
4. web doubler plate
5. weld access hole
6. panel zone

Figure 2-2 Welded-Unreinforced Flange-Bolted-Web Connection (Federal Emergency Management Agency, 2000)
2.3 Derivation of Moment-Rotation Curves

Moment-rotation ($M-\theta$) curves can be used to characterize steel connections and provide the properties of connector elements of a building frame model. Typically, $M-\theta$ curves were created based on experimental test results. Using extensive test results, mathematical models have been proposed (Nethercot, 1985; Kishi and Chen, 1986). Empirical models for several PR connections were also introduced based on the experimental results with constant derivation techniques, such as curve fitting and regression analysis (Kishi, 1994; Attiogbe, 1991).

Mathematical expression of connection property was performed for computational frame analysis. Especially, for the prediction of overall frame performance, it has to be account for accurate representations of the moment-rotation relationships. One of the mostly used equations for the $M-\theta$ curves is Richard-Abbot equation (also called Power model), as shown in Equation 2-1 (Richard and Abbott, 1975).

$$M = \frac{K_i \theta}{\left(1 + \left(\frac{\theta}{\theta_0}\right)^n\right)^{\frac{1}{n}}}$$  \hspace{1cm} (2-1)

where $K_i$ is initial stiffness, $\theta_0$ is plastic rotation, and $n$ is a shape factor. The shape factor may be obtained by two point analytical expression on the curve. The equation for $n$ is

$$A^n - \frac{1}{2^n} (B^n - 1) = 0$$  \hspace{1cm} (2-2)

in which $A = \frac{K_i}{(K_a - K_p)}$; $B = \frac{K_i}{(K_b - K_p)}$; $K_a = \frac{M_a}{\theta_a}$; and $K_b = \frac{M_b}{\theta_b}$. The shape factor is then determined by iteration.
Analytical models can be categorized as more advanced model for predicting connection behaviors. Several studies applied elastic analysis and limit design to simplified models of various connection types. On the basis of main deformation sources and collapse mechanisms observed in experimental settings, initial connection stiffness and ultimate moment were predicted by means of elastic and plastic analyses (Kishi and Chen, 1987; Yee and Melchers, 1986). Curve fitting of test data is also required for the calibration of curve shape factors only when the complete $M$–$\theta$ curve is desired.

Mechanical models are represented by component approaches proposed in the Eurocode 3 (European Prestandard, 1995). These models, namely spring models, are established with a set of components (spring elements) that contain inelastic constitutive properties quantified individually. These spring components are arranged in series or in parallel and the assemblage generate $M$–$\theta$ curve. The arrangement of springs, contributing deformations, and the capacity of the weakest spring determine the initial rotational stiffness and flexural resistance capacity respectively (European Prestandard, 1995; Faella et al., 2000; Tamboli, 1999; Ivany and Baniotopoulos, 2000; Tschemmerennegg, 1988; Simoes da Silva et al., 2002; Rassati et al., 2004).

A detailed, nonlinear finite element analysis (FEA) model could be a suitable approach to investigate joint performance comparing to the aforementioned models. Thanks to the powerful computation abilities of modern computers, researchers were able to gain very reliable predictions of connection behaviors using FEA models. Many studies have implemented this model for the purpose of determining or evaluating their research objectives (Joh and Chen, 1999; Chi et al., 1997; Mao et al., 2001; Yang et al., 2000; Bursi and Jaspart, 1997; Srivanich et al. 1999; Matos and Dodds Jr., 2002;
 Nevertheless, it has to be recognized that requirements, such as accurate geometrical and material nonlinearities of individual parts, bolt pre-loading and its effect, contact variability and friction, weld toughness, should be considered to achieve accurate analyses since the performance of a joint is a result of very complicated interactions among its consisting parts (Faella et al., 2000; Tamboli, 1999).

2.4 Steel Connections under Quasi-Static and Cyclic Loads

2.4.1 Experimental Studies

One of the earliest studies in the field of steel connection was conducted by Wilson et al (1917). These tests were to determine the rigidity of riveted joints which connect the members of steel framed structures. Loads were statically applied by the cross-head of the testing machine through rollers to the end of the beams. The study measured applied moment, rotations of the rectangular panel zone, rivet slip effect and expressed the rigidity of the connections in terms of their relationships.

Tests of steel beam-column subassemblages were performed by Krawinkler et al (1971). In all connections, girder flanges and shear tabs were welded to column flanges and girder webs were bolted to the shear tabs (Welded-flange-bolted-web connection). Hinges were assumed at the outer ends of the girders and the inflection points in the columns were taken at midstory height. The push-over test results showed that panel zones can develop significant shear distortions with the ability to dissipate energy. Based on the test results, they suggested a trilinear model of the shear force versus deformation behavior of the panel zone.
The damage suffered by WUF-B connections during the 1994 Northridge earthquake lead a number of research teams to carry out various studies to determine the causes of damage and to provide possible solutions. A connection testing program (Engelhardt and Sabol, 1994) was initiated by AISC that involved preliminary studies of the failures and development of interim guidelines for improving connection details to better resist induced seismic loads. Cyclic tests were conducted on cantilever beam-column specimens connected using WUF-B type, one of 9 different types of connections (Figure 2-3, Table 2-1). Slowly applied cyclic loads subjected the connections to cyclic bending moments and shear forces. Results showed that WUF-B type specimens performed poorly. They experienced brittle fractures in the groove welds at the beam flange to column face early in their loading history as shown in Figure 2-4.
Table 2-1 Property Details of Tested Connection (Engelhardt and Sabol, 1994)

<table>
<thead>
<tr>
<th>Connection Details</th>
</tr>
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<tbody>
<tr>
<td><strong>Connection Description</strong></td>
</tr>
<tr>
<td><strong>Girder Top Flange Reinforcement</strong></td>
</tr>
<tr>
<td>Standard Welded Flange - Bolted Web</td>
</tr>
</tbody>
</table>

**Selected Data For Flat Position Field Groove Welds**

<table>
<thead>
<tr>
<th>Electrode Diameter</th>
<th>Electrode Designation</th>
<th>Top Flange Backup Bar</th>
<th>Bottom Flange Backup Bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.12&quot;</td>
<td>E70T-4</td>
<td>Removed</td>
<td>Removed</td>
</tr>
</tbody>
</table>

**Girder and Column Properties**

<table>
<thead>
<tr>
<th>Member</th>
<th>Size</th>
<th>Nominal Values</th>
<th>Measured Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>W36X150</td>
<td>F_y</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F_u</td>
<td>50</td>
</tr>
<tr>
<td>Column</td>
<td>W14X455</td>
<td>Flange F_y</td>
<td>59.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flange F_u</td>
<td>77.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web F_y</td>
<td>46.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web F_u</td>
<td>67.9</td>
</tr>
</tbody>
</table>

Figure 2-4 Load-Displacement Relationships in Cyclic Test (Engelhardt and Sabol, 1994)
Whittaker et al. also performed cyclic tests on welded-flange-bolted-web (WUF-B) pre-Northridge connections to characterize the behaviors and evaluate the efficacy of a repair scheme (Whittaker et al., 2001). Three specimens were tested to failure and one of them was repaired by re-welding and re-tested. Premature brittle fractures of the groove welds were observed in all specimens. The mean beam plastic rotation capacity was 0.005 rad., one-sixth of the target value 0.03 that a joint venture of Structural Engineers Association of California (SAC), Applied Technology Council, and California Universities for Research in Earthquake Engineering, established by means of nonlinear analysis of code-compliant steel moment-resisting frames. The repaired connection, meanwhile, did not perform in a satisfactory manner. The maximum plastic rotation was even less than the smallest value of the pre-Northridge specimens. They concluded that repair of damaged welded joints with toughness-rated weld metal may not re-instate the rotation capacity.

As the principal component of the SAC Steel Project Task 7.02, a series of parametric tests on pre- and post-Northridge WUF-B were conducted by Stojadinović et al (2000). The objective of these tests were to determine possible causes of connection damage during the Northridge earthquake and to establish if a newly detailed connection could provide satisfactory behavior in future earthquakes. Pre-Northridge fully restrained connections had practically no plastic rotation capacity, i.e., their failure was quite brittle, with a crack starting at the flange groove welds. A newly detailed SAC post-Northridge connection could develop better plastic rotational behaviors, but yet, not sufficient in regions of high seismicity.
The same newly detailed post-Northridge connections were tested by another research group. Ricles et al's study (2001) involved the inelastic cyclic testing of 11 full-scale connection specimens which included modified weld access hole geometry, stronger panel zone, higher toughness weld metal. It was demonstrated that the improved connections could reliably achieve an inelastic rotation of 0.03 radians or more prior to failure. The test results indicated that a strong panel zone enhanced inelastic connection performance.

Based on a failure analysis of WUF-B connections tested at the University of Michigan and Lehigh University, Barsom and Pellegrino (2002) presented the effects of geometries of weld access holes and the roughness of its flame cut surface. They described the crack initiation and propagation at intersections of the weld access hole with the beam flange and from the flame-cut surface of the weld access hole. The paper suggested changes of the weld access hole and the surface roughness for the flame-cut surface in order to improve the performance of unreinforced post-Northridge welded steel moment-resisting connections subjected to seismic loads.

2.4.2 Numerical Studies

A number of numerical studies were carried out that investigate the behavior of steel connections. Both two- and three-dimensional finite element models were employed to understand these connection behaviors numerically. A majority of these studies, however, considered static loading only.

Joh et al. (1999) examined brittle fracture strength of WUF-B connections using linear elastic fracture mechanics and post-Northridge experiments. They calculated
effective fracture toughness at the column-weld interface using an edge crack assumption on connection tests. Then, they developed a FE model with an edge crack and calculated the maximum energy release rate at the edge crack front under iterated loads. The assumed load was determined to be the brittle fracture strength of the connection when both results matched. Using this method and the assumed crack distribution, they calculated the probability of brittle fracture. Also, they showed this method was applicable to other types of connections such as a reduced beam section (RBS) connection.

Chi et al. (1997) developed detailed 2D and 3D finite element models to determine fracture toughness requirements in WUF-B connections. Toughness demands were quantified in terms of the elastic stress intensity factor and inelastic crack tip opening displacement. This study confirmed observations from the pre-Northridge connection tests that low toughness E70T-4 weld metal was likely to fracture without significant yielding. Resultant data could describe how fracture demands were affected by the weld matching ratio, joint panel shear strength, weld reinforcing, and residual stresses.

Mao et al. (2001) performed 3D finite-element study of the inelastic behavior of unreinforced, flange-welded moment connections. The models included subassemblies with notch tough weld metal. The study addressed the effect of (1) geometry and size of the weld access hole; (2) control of inelastic panel zone deformation; and (3) benefit of a welded beam web. In conclusions, they recommended a proper shape of weld access hole, beam web-to-column flange groove weld in conjunction with shear tab fillet weld, and a stronger panel zone.
Yang et al. (2000) investigated load-displacement relationships, moment-rotation curves, and stress distributions of double angle connections using the finite element code ABAQUS. The double angle connections, welded to the beam web and bolted to the column flange, were subjected to axial tensile loads, shear loads, and a combination of these loads. Three-dimensional modeling of the angle separated from the column and modeling of the contact forces between the bolt heads and the angles were attempted. Experimental test data were compared with the simulation results to support the findings. They introduced a mechanical model and a simpler two-dimensional FE model, which can produce the complex three-dimensional FE simulation results, such as separation of outstanding legs from the column, interaction of bolts and angles, bolt slip, bolt prestressing, prying actions, inelastic behaviors, etc.

Bursi and Jaspart (1997) presented results of finite element analysis devoted to extended end-plate connections by means of ABAQUS. This study provided an overview of current developments for estimating moment-rotation characteristics and attempted to establish a methodology for finite element analysis. Calibration development of a three-dimensional FE model using test data of an elementary tee stub connection was performed initially. Next, an assemblage of beam elements was proposed to reproduce the bolt behavior of isolated extended end-plate steel connections. The comparison between computed and measured values in each phase highlighted the effectiveness and accuracy of the proposed FE models.

Srivanich et al. (1999) carried out a case study of cover-plate strengthened frames with two, five, and ten stories. The study began with the comparison of a test results from a cover-plate connection assembly with results using a detailed nonlinear FEA model by
means of ABAQUS. With a strong correlation to test results, further models were generated and analyzed in order to examine the local behavior of cover-plate profiles and compare their performance to the simplified structural model that would be used in the static and time history analysis of those of the frames. With the reliable simple structural model of the cover-plate connection assemblies capturing both elastic and inelastic behaviors, such a model was incorporated into frames to perform nonlinear static and dynamic analyses.

2.5 Steel Connections under High-Speed Load

2.5.1 High-Speed Load

High-speed loads are defined as transient and time dependent abnormal loads. Generally, high-speed loads are generated in impact and blast events, as shown in Figure 2-5. Two important parameters defining these transient loads are peak pressure (force) and impulse. The impulse is the area beneath the pressure (force) time curves from the load initiation time to the end of the positive phase,

\[ I = \int_{0}^{t_d} P(t)dt \text{ or } \int_{0}^{t_d} F(t)dt \]  \hspace{1cm} (2-3)

where \( P(t) \) or \( F(t) \) is the load-time curve and \( t_d \) is the duration of the positive load pulse.
An impact load results from the collision between a moving object and a target structure. When the impactor strikes the target, transient stresses are generated at the interface between the two bodies, and they propagate away from the impact site at speeds inherent to each material. These waves reflect multiple times as other interface are reached (Zineddin and Krauthammer, 2002). The shear stress of the transient stress pulse may cause a conical fracture surface emanating from the point of impact, and the compressive stress pulse reflects from the rear face of the target as a tensile pulse which may cause scabbing (Hibbitt, Karlsson & Sorensen Inc., 2005).

Blast load is generated by the release of large amounts of energy from chemical or nuclear explosions. High pressure and temperature created by the explosion energy move from the detonation and develop a blast wave. The general shape of an air blast shock wave pressure-time history is illustrated in Figure 2-6. The maximum (peak) incident overpressure, $P_{so}$, appears at the shock front, and the pressure behind the front decays...
exponentially. As the wave propagates away from the explosion center, the peak incident overpressure and the pressure behind the front decrease steadily. When the decreased overpressure behind the front becomes smaller than atmosphere pressure $P_o$, the pressure domain is defined as at the negative phase.

When the shock front with the peak overpressure $P_{so}$ strikes a target at time $t_a$, the pressure of the shock front increases instantaneously as peak reflected pressure, $P_r$ due to the formation of a reflected wave. Since $t_r$ is very short, an instantaneous rise to the peak pressure can be assumed. The negative phase is not important for the blast-resistant design of heavy structures, and is usually ignored. Therefore, the exponentially decaying positive phase pressure can be idealized as a positive right triangle (Ng and Krauthammer, 2004).
2.5.2 Material Behavior

Generally speaking, steel as a material shows yield stress increases with strain-rate and this influence diminishes for higher-grade steel and higher strength weldment (Matos and Dodds Jr., 2002). Shake table tests of tension-only concentrically braced steel frames supported the fact that the strength of steel is increased under higher strain rate (Filatrault and Tremblay, 1997). However, all such material properties and notch effect were different. Higher strain rates correspond to lower cyclic resistance of steel components in many tests. This can be explained by the different rate of increase between yield strength ($f_y$) and tensile strength ($f_u$). Since $f_y$ increases faster than $f_u$, i.e. $f_y/f_u$ is reduced, very high localized strains, causing low ductility and early cracking, are developed. In other words, the fracture toughness deteriorates with an increase of strain rate (Dubina and Stratan, 2002; Plumier, 2000; Arimoch, 1998). Although test results under seismic loading conditions showed only a slight rate effect on connection behavior (AIJ, 1997; Federal Emergency Management Agency, 2000a; Federal Emergency Management Agency, 2000b; Federal Emergency Management Agency, 2000c), high-speed load such as impact and blast, have much larger effects on similar connections.

Some available test results on the steel tensile behavior at rapid loading rates were compiled by Soroushian, as shown in Figure 2-7 (Soroushian and Choi, 1987). The research indicated that there were increases in the yield and ultimate stress, though the increase of ultimate strength was relatively small when compared to the increase in yield stress. In addition, steels with lower yield strength were more strain rate sensitive than those with higher yield strength. The modulus of elasticity and elongation at rupture, however, remained unaffected by the increase strain rate.
The department of U.S. Army recommended in TM5-1300 (Department of U.S. Army, 1990) that the yield and ultimate strength be increased by dynamic increase factors (DIF). DIF is a stress increase ratio caused by strain rate. Related to the DIF is the recommended dynamic design stress given by

\[ f_{dy} = c \cdot a \cdot f_y \]  

(2-4)
where \( f_{dy} \) is a dynamic yield stress, \( c \) is a dynamic increase factor on the yield stress, \( a \) is an average strength increase factor (1.1 for steel), and \( f_y \) is a static yield stress based on uniaxial tensile stress. The effect of the increased strain rate can be visualized in Figure 2-8 and Table 2-2.

Figure 2-8 Typical Stress Strain Curves for Steel (Department of U.S. Army, 1990)

Table 2-2 Dynamic Increase Factors (Department of U.S. Army, 1990)

<table>
<thead>
<tr>
<th>Material (minimum yield stress)</th>
<th>Yield stress</th>
<th>Ultimate stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>bending</td>
<td>tension and compression</td>
</tr>
<tr>
<td>low pressure (( \dot{\varepsilon} =0.10 ))</td>
<td>1.29</td>
<td>1.19</td>
</tr>
<tr>
<td>high pressure (( \dot{\varepsilon} =0.30 ))</td>
<td>1.36</td>
<td>1.24</td>
</tr>
<tr>
<td>low pressure (( \dot{\varepsilon} =0.20 ))</td>
<td>1.19</td>
<td>1.12</td>
</tr>
<tr>
<td>high pressure (( \dot{\varepsilon} =0.50 ))</td>
<td>1.24</td>
<td>1.15</td>
</tr>
<tr>
<td>high and low pressure</td>
<td>1.10</td>
<td>1.05</td>
</tr>
</tbody>
</table>
2.5.3 Charpy V-Notch Test

The Charpy V-Notch (CVN) test is conducted for the purpose of characterizing the ductile-brittle transition in steels under static and high-strain-rate load. As described in detail in ASTM A370, CVN test utilizes a swinging pendulum and a standard Type-A rectangular beam-shaped specimen with V-notch of specified geometries to assess the resistance of a material to brittle fracture (Figure 2-9). The absorbed energy is measured from either a calibrated analog scale, an encoder and digital readout, and/or an instrumented striker.

![Charpy V-Notch Impact Test Machine and Standard Type-A Specimen](Chi et al., 1997)

2.5.4 Charpy V-Notch Test Results of the Materials in a Connection

The absorbed energy versus temperature curve is used for the results of CVN tests. Figure 2-10 schematically shows the general difference in fracture for rate-dependent material such as structural steels and weld metals. The quasi-static curve refers to the fracture toughness obtained in a standard ASTM $K_{IC}$ fracture test under slow load. The
impact curve is obtained from $K_{id}$ or CVN test. The difference in the location of these curves is the temperature shift, which is a function of the yield strength of the steels (Chi et al., 1997). Namely, it can be said that the material may be more prone to brittle fracture in the temperature range where a high energy shift occurs. Shown in Figure 2-11, CVN values in case of E70T-4 weld metal and A572 Gr. 50, used for cyclic test and simulation in this study, were generally lower at 70 degrees F than at higher temperature.

![Figure 2-10 Schematic Notch Toughness Transition Curves (Chi et al., 1997)](image-url)
(a) E70T-4 Weld Metal (Barsom et al., 1999)

(b) A572 Gr. 50 Base Metal (Chi et al., 1997)

Figure 2-11 Charpy V-Notch Test Data
2.5.5 Steel Connections under High-Speed Load

 Minimal information is available related to blast-resistant steel connection design. TM5-1300 (Department of U.S. Army, 1990), the fundamental reference in the field of blast-resistant design, emphasizes special care in connection design to provide integrity to the maximum response. The design procedures estimated beam and column responses under the blast loads in terms of the maximum deflection at the middle length of the member span, $X_m$, and the corresponding rotation at the member end $\theta$ as shown in Figure 2-12.

Figure 2-12. Maximum Deflection and End Rotation in Structural Member

Dynamic response was determined using design charts including the natural period of vibration ($T_N$), resistance ($R$), and deflection ($X$) under specific blast load ($P$) and time duration ($T$). A ductility ratio, $\mu$, associated with the ratios $T/T_N$ and $P/R_U$, where $T_N$ is the natural period of vibration and $R_U$ is the ultimate resistance of the structural member, can be obtained from the design charts. Using the obtained
information, the maximum deflection \((X_m)\) and rotation \((\theta_m)\) are then determined. Generally, the blast design takes into account the structural response on the first peak cycle since the maximum deflections are attained in that cycle. Damping has not significant effect on the first peak cycle and can be neglected. Based on the end rotations, TM 5-1300 defined damage levels as, light damage in less than 2 degrees; moderate damage between 2-5 degrees; severe damage between 5-12 degrees; very bad situation if greater than 12 degrees (Department of the Army, 1990).

Little test data exists to quantify the performance of steel connections directly loaded by blast. Furthermore, numerical studies investigating blast loaded steel connections are also quite limited. Considering the complexity inherent to any dynamic analysis and the complex interaction that can occur between various parts of a steel connection, finite element analysis presents a viable approach for investigating such problems. Krauthammer et al. (2004) attempted to show the possibility of more severe deformations than allowed by the TM5-1300 design criteria using finite element analysis results of pre- and post- Northridge steel moment connections subjected to blast (Krauthammer and Oh, 1997; Krauthammer, 1999; Krauthammer et al., 2004). Large local deformation from these studies suggested that the current procedures and guidelines outlined in TM 5-1300 may need to be revised.

2.5.6 Pressure – Impulse Diagram

In assessing structural performance, final states are often the main interest for a designer, instead of whole response path. A response spectrum is typically utilized to design a dynamic system using relationship between maximum response and either
frequency or the ratio of the load duration to natural period. It is known that the response is related to the ratio between its natural period and blast load duration (Smith and Hetherington, 1994). Three regimes of structural behavior can be defined by relationships between load durations and natural period of structural element. The first situation is where a load duration is much shorter than a natural period of the component, $t_d << T$. The load is applied transiently and disappears before the structure reaches its maximum responses. Such loading is known to be impulsive loading. On the other hand, it is referred as quasi-static loading when the load duration is much larger than the natural period, $t_d >> T$. The structure shows its maximum responses before the load undergoes any significant decay. Dynamic regime exists between previous two regimes, i.e., the positive load duration and the natural period are similar. The three regimes are summarized in Figure 2-13 and Table 2-3.

![Figure 2-13 Load Regime (Department of U.S. Army, 1990)]
Table 2-3 Load Regime Summary (Smith and Hetherington, 1994)

<table>
<thead>
<tr>
<th>Loading Regime</th>
<th>Approximate Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impulsive</td>
<td>* $\omega t_d &lt; 0.4$</td>
</tr>
<tr>
<td>Quasi-Static</td>
<td>$\omega t_d &gt; 40$</td>
</tr>
<tr>
<td>Dynamic</td>
<td>$0.4 &lt; \omega t_d &lt; 40$</td>
</tr>
</tbody>
</table>

* $\omega$: natural circular frequency ($\frac{2\pi}{T}$)

To demonstrate the response spectrum of structures, an example of an undamped, linear single-degree-of-freedom (SDOF) system subjected to a transient constant load is used, as shown in Figure 2-14.

A closed form solution of the response spectrum was given by the following expressions (Tedesco et al., 1999),
\[
\frac{x_{\text{max}}}{P/k} = 2 \sin \left( \frac{\omega t_d}{2} \right) \quad 0 \leq \omega t_d \leq \pi
\]

or
\[
\frac{x_{\text{max}}}{P/k} = 2 \sin \left( \frac{\pi t_d}{T} \right) \quad 0 < \frac{t_d}{T} < \frac{1}{2} \quad \text{(free vibration phase)} \quad (2-5a)
\]

\[
\frac{x_{\text{max}}}{P/k} = 2 \quad \omega t_d > \pi
\]

or
\[
\frac{x_{\text{max}}}{P/k} = 2 \quad \frac{t_d}{T} > \frac{1}{2} \quad \text{(forced vibration phase)} \quad (2-5b)
\]

The responses of structures loaded quasi-statically and impulsively can also be evaluated by the energy balance method (Smith and Hetherington, 1994). Three energy terms should be derived first, as follows;

Work done, \( WD = P x_{\text{max}} \) \quad (2-6a)

Strain energy, \( U = \frac{1}{2} k x_{\text{max}}^2 \), where \( k \) is stiffness. \quad (2-6b)

Kinetic energy, \( KE = \frac{1}{2} m \ddot{x}_0^2 = \frac{I^2}{2m} \), \quad (2-6c)

where \( \ddot{x}_0 = \frac{I}{m} \) since impulse causes an initially stationary structure to acquire a velocity \( \dot{x}_0 \). In the case of a quasi-static loading, it is assumed that the work done on the structure is all converted to strain energy. Therefore, the following equation can be obtained;

\[
WD = U, \quad \frac{x_{\text{max}}}{P/k} = \frac{x_{\text{max}}}{x_{\text{static}}} = 2 \quad (2-7)
\]

where the value 2 represents the upper bound of a quasi-static response (Figure 2-15).
Conservation of energy is used to analyze the response by impulsive loading. It is to acknowledge that when an impulse is delivered to a structure it produces an instantaneous velocity change. Therefore, by equating the kinetic energy to the strain energy, the following is obtained,

\[
\frac{KE}{2m} = \frac{I^2}{2m} \frac{1}{k} x_{\text{max}}^2, \quad x_{\text{max}} = \frac{I}{\sqrt{km(P/k)}} = \frac{Pt_d}{\sqrt{m/(P/k)}} = \omega t_d
\]  

which is the equation of the impulsive asymptote. With the use of the both solutions, the response spectrum is plotted in Figure 2-15.
A Pressure-Impulse diagram (P-I diagram) is an alternative way to show the limit of response for a target structure (Smith and Hetherington, 1994). From the previous energy balance analyses, the quasi-static and impulsive equations can be respectively converted, as follows;

\[
\frac{2P}{kx_{\text{max}}} = 1 \quad (2-9a)
\]

\[
\frac{x_{\text{max}}}{\omega t_d P/\sqrt{k}} = 1, \quad \frac{2P}{kx_{\text{max}}} = \frac{P t_d}{k} \sqrt{\frac{k}{m}} = \frac{I}{x_{\text{max}} \sqrt{km}} \quad (2-9b)
\]

Using Equations 2-9a and 2-9b as axes, the responses spectrum curve can be replotted as shown in Figure 2-16.

Figure 2-16 Pressure-Impulse Diagram (Soh and Krauthammer, 2004)
The main difference of the two presentations is that the response spectrum shows the influence of scaled time $t_d / T$, whereas the P-I diagram emphasizes the combination of peak load and impulse that will cause failure (Smith and Hetherington, 1994). That is, if combinations of pressure and impulse fall to the left and below the curve, it will not induce failure while those to the right and above the graph will indicate damage.

P-I diagrams can be generated by collecting data points from numerical analyses. Each dynamic analysis of a structural model under a specific pressure-impulse combination (impulsive, dynamic, or quasi-static) produces maximum response which may or may not represent specific damage states. A data point will be created when the P-I combination define the transitions from damage to no-damage zones (Figure 2-17). Since the dynamic analyses are possible to be conducted with the aid of computers, this approach is useful to develop P-I diagrams for various types of structures, damage states, and complex load pulses.

![Figure 2-17 Numerical Approach to P-I Diagram](Soh and Krauthammer, 2004)
2.6 Simplified Numerical Model for Frame Analysis

In the preceding sections highly detailed connection geometries had been employed in both quasi-static and high-strain-rate simulations to obtain more accurate results compared with the experimental output. However, this will not be efficient for the analysis of complete structural frame that contains hundreds of connections. Hence, it is necessary to simplify the original detailed assemblies utilizing beam and connector elements that include the corresponding properties.

In ABAQUS (Hibbitt, Karlsson & Sorensen Inc., 2005) a beam element is a one-dimensional line element that has stiffness associated with deformation about the beam axis (Figure 2-18). These deformations consist of axial tension, compression, bending, and torsion. Beam elements offer additional flexibility associated with transverse shear deformation between the beam's axis and its cross-section directions. The main advantage of using beam elements is that they are geometrically simple and have few degrees of freedom. This simplicity is achieved by assuming that the member deformation can be estimated entirely from variables that are functions of position along the beam axis only.

Figure 2-18 Selected Beam Element Types
A connector element can define a connection between two nodes that has relative constrained and/or available components such as displacements and rotations (Hibbitt, Karlsson & Sorensen Inc., 2005). Constrained components of relative motion are displacements and rotations that are fixed by the connector element and available components are not constrained kinematically, hence, remain available for defining material-like behavior. Two kinds of connector elements were selected to describe the steel moment connection as illustrated in Figure 2-19. The orientation directions at node \(a\) (the first node on the connector element) and \(b\) (the second node on the connector element) are indicated as unit base vectors \(\mathbf{e}_i^a\) and \(\mathbf{e}_i^b\), respectively, where \(i = \{1, 2, 3\}\). According to the constraint level in Figure 2-19, the rotation about three directions is the possible relative motion in the connection. The available component contains specific properties with respect to the local motion, such as elasticity, plasticity, failure, and so forth.

(a) Translational Component  
(b) Rotational Component

Figure 2-19 Selected Connector Elements (Hibbitt, Karlsson & Sorensen Inc., 2005)
For instance, a beam-column connection can be modeled using a connector element and the moment-rotation relationship of the connection can represent structural behaviors of the connector element, as shown in Figure 2-20.

(a) Beam-Column Connection  (b) Connector Element

(c) Connector Element Behaviors in the Moment-Rotation Relationship

Figure 2-20 Applications of Connector Element
The linear elastic stiffness, $E_n$, of the connector element can be defined as shown in Equation 2-10,

$$M_i = E_n \theta_i$$  \hspace{1cm} (2-10)$$

where $M_i$ is the moment in the $i$th component of relative motion and $\theta_i$ is the connector rotation in the $i$th direction. The plasticity formulation for connectors is similar to the plasticity formulation in metal plasticity. The stress $\sigma$ corresponds to the moment $M$, the strain $\varepsilon$ corresponds to the constitutive rotation $\theta$, the plastic strain $\varepsilon_{pl}$ corresponds to the plastic relative rotation $\theta_{pl}$, and the equivalent plastic strain $\bar{\varepsilon}_{pl}$, corresponds to the equivalent plastic relative rotation $\bar{\theta}_{pl}$. The yield function $\phi$ is defined by Equation 2-11.

$$\phi(M, \bar{\theta}^\text{pl}) = P(M) - M_y \leq 0$$  \hspace{1cm} (2-11)$$

where $M$ is the summation of moments in the available components of relative motion that ultimately contribute to the yield function; the connector potential (yield function) $P(M)$, defines a magnitude of connector tractions similar to defining an equivalent state of stress in Mises plasticity; and $M_y$ is the yield moment. The connector relative rotations $\theta$ remain elastic as long as $\phi < 0$; and when plastic flow occurs, $\phi = 0$. As shown in Figure 2–21, elastic motion occurs prior to plasticity onset, and unloading from a plastic state occurs on a straight line parallel to the initial loading.
2.7 Summary

Steel connections are divided into three categories based on the degree of restraint, such as fully-rigid, partially-rigid, and simple connections. The target connection details in the current study are fully restrained connections. A number of experimental and numerical studies were carried out that investigate the static and cyclic behavior of steel connections. Very little information, however, is available on steel connections subjected to high-speed loading, such as impact and blast. This study is aimed at the determination of connection properties under both quasi-static and high-speed loads. $M-\theta$ curves and P-I diagrams will be derived in this study to accomplish the static and dynamic characterizations of the connections, respectively. The last section of this chapter introduced a numerical approach that converts highly detailed beam-column connections to a simple frame model utilizing beam and connector elements that include the corresponding properties. The proposed research approach based upon the presented background will be addressed in the next chapter.
Chapter 3

RESEARCH APPROACH

3.1 Introduction

The objective of this study is the characterization of WUF-B type steel moment connections. In order to accomplish this finite element analyses, the commercial modeling package ABAQUS (Hibbitt, Karlsson & Sorensen Inc., 2005), will be utilized. This chapter will describe macro approaches and detailed computational procedures of connection characterization.

3.2 Step 1 - Connection Design

Connection design followed all requirements of the AISC Manual of Steel Construction for Load and Resistance Factor Design. A moment connection is generally set up as an exterior girder-to-column connection for the purpose of resisting lateral loads. It consists of a shear connection between a girder web and a column flange or web and complete-joint-penetration groove welds between the top and bottom flanges of the girder and the face supporting column flange. Horizontal stiffeners and doubler plates may be required when compression or tension forces in the girder flange are greater than local column flange bending, web yielding, web crippling, and compression buckling. Although there are literally infinite numbers of possible connection configurations that can be categorized in WUF-B type, several reasonable beam-column combinations will be chosen and adequate connections designed. Loads will be determined by the probable
shear force and flexural resistant capacity of supported girders. More details are to be explained in Chapter 5.

3.3 Step 2 - Model Validation

Three-dimensional finite element models of the chosen connection are developed in ABAQUS. Before starting the simulations, numerical model validations will be preceded by comparing their predictions against experimental data obtained from AISC Northridge Test Program (Engelhardt and Sabol, 1994). Test setup and connection details are depicted in Figure 2-3 and detailed properties are presented in Table 2-1. Numerical models of these connections will be prepared using ABAQUS/Standard (Hibbitt, Karlsson & Sorensen Inc., 2005), which can be used for general static analysis. Load and displacement relationships from the simulations will be compared with results from the experiment (Figure 2-4). Once the established simulation procedures are producing valid output, characterizing work with respect to quasi-static and high-stain-rate dynamic behaviors of the connection will begin.

3.4 Step 3 - Characterizations

Connections are categorized into various types by their amount of restraint, such as fully restrained, partially restrained, and simple connection. Moment-rotation curves are generally assumed to be the best characterization of connection behavior. These moment-rotation curves will be derived using ABAQUS/Standard. Moments can easily be obtained by multiplying pressure forces on the girder top flange by lever arm lengths. Horizontal and vertical rotations (θ) are calculated from the displacements at the point
closed to the column flange surface and on the neutral axis of the beam, as shown in Figure 3-1 and Equation 3-1.

(a) Vertical Rotation

(b) Horizontal Rotation

Figure 3-1 Connection Nominal Rotations (Continued)
where \( L \) is the distance from column flange surface to a point located on the neutral axis of the beam and close to the column flange. \( u \) and \( v \) are displacements in the x and y for vertical (or z for horizontal) directions respectively. The moment and rotation curves will define properties of connector elements in the macro simulations of simplified frame structure. It should be noted that the definition of rotations in present study is different from that of TM5-1300. As introduced on literature review (Figure 2-12), TM5-1300 defines the end rotation by simply dividing maximum deflection by the beam length at the midspan. However, this definition would overestimate the rotation values due to bending effect of the beam and undermine credibility of properties of simplified connector elements. These phenomena and differences will be clarified on the simulation results using simplified connection models in Chapter 4.
In order to determine high-strain-rate behaviors of the chosen connection details, three-dimensional finite element models will be developed using ABAQUS/Explicit (Hibbitt, Karlsson & Sorensen Inc., 2005). ABAQUS/Explicit is efficient for the analysis of large models with relatively short dynamic response times and for the analysis of extremely discontinuous events or processes; on the other hand, ABAQUS/Standard is more efficient for solving smooth nonlinear problems, which was used in Step 2 and 3. ABAQUS/Explicit uses an explicit integration method while ABAQUS/Standard uses an implicit integration method. Each is one of direct time integration methods to calculate structural dynamic response history using step-by-step integration in time, without first changing the form of the dynamic equations. When the response is evaluated at instants separated by time increments \( \Delta t \), the equation of motion at the \( n \)th time step is

\[
\{F(t)\}_n = [K]\{d\}_n + [C]\{\dot{d}\}_n + [M]\{\ddot{d}\}_n
\]

(3-2)

where \( \{F\} = \sum_{\text{element}=1}^{N} f^{(\text{element})} \) is force matrix; \( [K]= \sum_{\text{element}=1}^{N} k^{(\text{element})} \) is global stiffness matrix; \( [C] \) is damping matrix; \( [M]= \sum_{\text{element}=1}^{N} m^{(\text{element})} \) is mass matrix; \( \{d\} \), \( \{\dot{d}\} \), \( \{\ddot{d}\} \) are displacement (degree of freedom), velocity, acceleration matrix respectively. Methods of direct integration calculate conditions at time step \( n+1 \) from the equation of motion, a difference expression, and known conditions at preceding time steps. In the explicit method, the displacement solution \( \{d\}_{t+\Delta t} \) is determined from equilibrium equations established at time \( t \). That is, the solution at time \( t + \Delta t \) is approximated from the equilibrium conditions formulated at time \( t \). On the other hand, integration methods
which yield a solution at time \( t + \Delta t \) based on equilibrium established at time \( t + \Delta t \) are called the implicit integration method. Also, explicit integration methods do not require a factorization of the effective stiffness methods, whereas the implicit integration methods do. Therefore, explicit methods require relatively less computational effort to solve than the implicit method. In contrast, implicit methods permit a relatively large \( \Delta t \).

Theoretically, high-speed dynamic event might be solved with ABAQUS/Standard using the implicit integration methods but would have difficulty converging because of contact or material complexities in short duration, resulting in a large number of iterations. Such analyses are expensive in ABAQUS/Standard because each iteration requires a large set of equations to be solved. ABAQUS/Explicit is using explicit integration methods, and it determines the solution without iterating by explicitly advancing the kinematic state from the previous increment. Even though the given analysis may require a large number of time increments using the explicit method, the analysis can be more efficient in ABAQUS/Explicit if the same analysis in ABAQUS/Standard requires many iterations (Hibbitt, Karlsson & Sorensen Inc., 2005).

Instead of static pressure load, an idealized blast wave expressed by pressure time-history with short duration (Figure 2-6) will be applied to same geometric model used in ABAQUS/Standard. Moment-rotations, girder deflections, and failure phenomena will be investigated from these simulations. For the blast studies, yield and ultimate strengths will be increased to account for strain rate effects using dynamic increase factors as recommended discussed in Section 2.5.2. One of the important factors that cannot be overlooked is the brittle failure model. The failure model definition includes a shear failure model, which causes this simulation program to remove elements from the
mesh as they fail. The shear failure model is based on the value of the equivalent plastic strain (PEEQ) at element integration point. In order to find the PEEQ of weld and base metal materials, simulations of Charpy V-notch (CVN) toughness tests will be conducted. As shown in the Figure 2-11, absorbed energy was measured to determine the brittleness of specimen materials. Comparing the absorbed energy in the CVN tests with energy dissipated by plastic deformation in the simulation, one will be able to assess PEEQ values.

3.5 Step 4 - Database Construction

Applying these processes into several joints assembly placed on the different resisting capability levels, finally, database of WUF-B steel moment connections with respect to static and dynamic behaviors will be constructed. Quasi-static moment rotation curves will be determined from quasi-static push-over analyses.

Next, rate dependant moment rotation curves, illustrated in Figure 3-2, and impulse failure will be studied by blast-rate analyses. Since there can be various blast load time-histories with peak force and duration variables (Figure 3-2 (a)), different shapes of moment-rotation curves representing different strain-rate effect can be obtained (Figure 3-2 (b)). Finally, these relationships and failure results will be reported incorporated into P-I diagrams, as explained in Section 2.5.
(a) Input: Various Blast Load Time-Histories

(b) Output: Rate Dependent Moment-Rotation Curves

Figure 3-2 Moment-Rotation Curve Database of Blast-Rate Analyses
3.6 Step 5 - Simplified Frame Analysis

The suitability of the aforementioned connection properties and connector models will be assessed by applying the same loading and structural conditions to a connector element in the simplified frame analysis. As stated in the introduction of this proposal, numerical analyses of a whole structural system will be complicated and computationally expensive if refined connection models are used in the three-dimensional multi-story building model. For instance, as shown in Figure 3-3, there are approximately 500 connections in the of 10-story building with 4 by 4 bays. Provided the blast-rate simulation for a single connection made by detailed model might contains 93332 elements and could require about 24 hours, the simulation for the whole building structure will be prohibitively long and expensive. Therefore, it is essential to develop a more simplified building frame model to significantly reduce the required human and computational resources but still insure acceptable accuracy.

Figure 3-3 Three-dimensional Frame Model for Building of 10 story and 4 by 4 bays
A simpler frame consists of beam elements instead of beam and column, and connector element for connection components. All geometric and mechanical properties of the complex finite element model in the preceding steps will become input data for the simplified frame model. It should be noted that the moment-rotation curve extracted from the detailed connection model will become a mechanical property for the connector element of the simplified model. Since moment-rotation curves can vary depending on loading circumstances, as shown Figure 3-2, the connector element property will become rotation-rate dependent.

A rotational displacement obtained in the simplified frame analysis will be compared to a geometrically computed rotation (Figure 3-1) in the analysis using the detailed model. The comparisons will be performed in both quasi-static and blast-rate analyses. It is expected that the simplified frame model will be an acceptable alternative which generate accurate predictions while effectively reducing simulation time.

The quasi-static and blast-rate behaviors of the connections in the vertical and horizontal directions will be determined independently. These two responses would be spoken for the situations when the blast pressures are applied normal to the connected girders. However, since the blast wave can be propagated in biaxial directions, the simplified connector elements should contain the combined properties in both directions. Therefore, the biaxial properties will be investigated by applying inclined loads to the detailed connection models and plotting the combination of forces or motions as a specific shape, such as an ellipse. Then, connector functions for the coupled behaviors in the simplified model will be developed. These analyses will be conducted in both progressive rate and blast rate loading cases.
3.7 Summary

This chapter describes research steps for this study. First, possible connection configurations in steel moment frames are designed using the LRFD methodology. The connections will be characterized using finite element method. Before analyzing the connection candidates, a finite element model will be validated by comparison with Engelhardt’s test data (Quasi-static analysis), and Charpy V-notch impact test models of weldment and base materials will be simulated for development of weld material properties (Blast-rate analysis). With the validated FEM model, computational simulations will be conducted in the quasi-static and high-strain-rate domains which represent progressive collapse and blast rates respectively. Moment-rotation curves will be extracted from the quasi-static analyses for progressive collapse simulations. And also, pressure-impulse diagrams will be determined by blast-rate analyses as a property of the connection under blast loading. These simulations will be implemented in all connection candidates and the results will be used to construct the connection property database. Finally, the database is expected to be applied to quasi-static and/or dynamic frame analyses. Figure 3-4 shows a schematic flow of this study.
Finite Element Models
- validated by comparison with Engelhardt’s test data
  (Quasi-static analysis)
  Charpy V-notch impact test models of weldment and base materials will be simulated for development of weld material properties
  (Blast-rate analysis)

Characterizing Simulations
- based upon the validated FEM model,
  - Moment-Rotation Relationships
    - Initial Stiffness
    - Plastic Moment and Rotation
    - Failure Moment and Rotation
    - Dynamic Increase Factors for Rate-Dependent Properties
  - Pressure-Impulse Diagrams

Database Construction
- will be constructed with respect to the possible connection assembly.

Simplified Frame Analysis
- The connection database can be applied to the static and/or dynamic frame analyses.

LRFD Design
- possible connection configurations are built in terms of LRFD design code.

Figure 3-4 Flow Chart for Research Approach
Chapter 4

RESULTS AND DISCUSSIONS

4.1 Introduction

Based upon the research approach required to meet the objective of this study, a preliminary study was conducted using a previously tested WUF-B connection model (Engelhardt and Sabol, 1994), as discussed on Figure 2-3 and Table 2-1 in Chapter 2. The results showed that this approach could clearly provide connection characteristics and that the connection characteristics defined by full meshed models could effectively be applied to simplified frame model.

4.2 Model Validation

A finite element model of the aforementioned structure was established, as shown in Figure 4-1. An elasto-plastic material property with isotropic hardening was selected to simulate the material behavior of all the components of the finite element model. The material models are shown in Table 4-1 and Figure 4-2. An elastic modulus of 29000 ksi and Poisson’s ratio of 0.3 was used.

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>Beam</th>
<th>Column</th>
<th>Shear Tabs</th>
<th>Bolt/Nut</th>
<th>Weld</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_y$ (ksi)</td>
<td>50</td>
<td>62.5</td>
<td>50</td>
<td>92</td>
<td>70</td>
</tr>
<tr>
<td>$f_u$ (ksi)</td>
<td>64.8</td>
<td>78.9</td>
<td>64.8</td>
<td>120</td>
<td>80</td>
</tr>
</tbody>
</table>
Figure 4-1 Numerical Model using ABAQUS/Standard

Figure 4-2 Elasto-Plastic with Isotropic Strain Hardening Model for Numerical Study
The element types mainly used for the finite element models in this study were eight-noded continuum (brick) elements with reduced integration (C3D8R). Six-noded wedge elements (C3D6) were also used to model the curved parts that included the weld access holes, bolt holes and welds. Figure 4-3 shows the shapes of the brick and wedge elements.

(a) Eight-noded Solid Element (C3D8R)    (b) Six-noded Solid Element (C3D6)

Figure 4-3 Element Types

The total number of elements was 93,066 for the different components used for the model, and the numbers of elements of individual members are defined in Table 4-2.

Table 4-2 Number of Elements for Various Components

<table>
<thead>
<tr>
<th>Component</th>
<th>Number of Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>21280</td>
</tr>
<tr>
<td>Column</td>
<td>39964</td>
</tr>
<tr>
<td>Bolts</td>
<td>6784</td>
</tr>
<tr>
<td>Nuts</td>
<td>1024</td>
</tr>
<tr>
<td>Shear Tabs</td>
<td>16680</td>
</tr>
<tr>
<td>Welds</td>
<td>7158</td>
</tr>
<tr>
<td>End Plate</td>
<td>176</td>
</tr>
</tbody>
</table>
Load and displacement relationships obtained from the simulation were compared with those from experiment in Table 4-3 and Figure 4-4.

Table 4-3 Result Comparison between Test and Simulation

<table>
<thead>
<tr>
<th>Cyclic Loads (kips)</th>
<th>Displacement (in.)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cyclic Test</td>
<td>Simulation</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>75</td>
<td>0.375</td>
<td>0.382</td>
<td></td>
</tr>
<tr>
<td>-75</td>
<td>-0.375</td>
<td>-0.390</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>0.75</td>
<td>0.737</td>
<td></td>
</tr>
<tr>
<td>-150</td>
<td>-0.75</td>
<td>-0.778</td>
<td></td>
</tr>
<tr>
<td>175</td>
<td>1.0</td>
<td>0.975</td>
<td></td>
</tr>
<tr>
<td>-175</td>
<td>-1.0</td>
<td>-0.985</td>
<td></td>
</tr>
<tr>
<td>195</td>
<td>1.5</td>
<td>1.51</td>
<td></td>
</tr>
<tr>
<td>-198</td>
<td>-1.4</td>
<td>-1.55</td>
<td></td>
</tr>
</tbody>
</table>

(a) Load-Displacement Relationships in Test (Engelhardt and Sabol, 1994)

Figure 4-4 Test and Simulation Results (Continued)
(b) Load-Displacement Relationships in Simulation and Comparison with Test

Figure 4-4 Test and Simulation Results
As shown in result comparisons, the simulation using the proposed model provided displacements with respect to the applied cyclic loads within 5 to 10 percents of experimental displacements. Considering finite element analysis provides only an estimation of real structure response, it was concluded that the numerical model gave a good approximation of structural response and acceptable to characterize other connections in design.

4.3 Characterizations

4.3.1 Quasi-Static Properties

Moment-rotation curves were extracted from the validated model, as illustrated in Figure 4-5. Push-over analyses where pressures were statically applied to the girder top or side surface were conducted using ABAQUS/Standard. Moments could be obtained by multiplying the applied pressure on the girder top flange by lever arm lengths. As shown in Figure 3-1, connection rotations, $\theta_{\text{vertical}}$ and $\theta_{\text{horizontal}}$, were calculated from the displacements at the point close to the column flange surface and on the neutral axis of the beam. The connection property can be defined similarly to a uniaxial stress-strain material property. Table 4-4 summarized the property factors in the vertical and horizontal directions. The strength and stiffness of the connection were higher in the vertical way than in the horizontal way. However, it exhibited high ductile behavior in the horizontal direction.
(a) Vertical Direction (90 degrees)

(b) Horizontal Direction (0 degree)

Figure 4-5 Moment-Rotation Curves by FEA and Mathematical Equation
Table 4-4 Quasi-Static Properties of the Selected Connection

<table>
<thead>
<tr>
<th>Properties</th>
<th>Vertical</th>
<th>Horizontal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffness (Elasticity) [k·in/rad]</td>
<td>2.71E+07</td>
<td>1.64E+06</td>
</tr>
<tr>
<td>Yield Moment [k·in]</td>
<td>2.36E+04</td>
<td>3.52E+03</td>
</tr>
<tr>
<td>Yield Rotation [rad] (deg)</td>
<td>1.12E-03 (0.064)</td>
<td>2.46E-03 (0.14)</td>
</tr>
<tr>
<td>Ultimate Moment [k·in]</td>
<td>2.77E+04</td>
<td>4.11E+03</td>
</tr>
<tr>
<td>Ultimate Rotation [rad] (deg)</td>
<td>0.79E-02 (0.45)</td>
<td>1.60E-02 (0.92)</td>
</tr>
</tbody>
</table>

The moment-rotation curves obtained by finite element analyses were represented as mathematical formulations using Equation 2-1 and 2-2. Using the parameters in Table 4-5, the curves were calculated and given in Figure 4-5. The equations introduced in Section 2.3 provided more than 98 percent accuracy in representing the vertical and horizontal moment-rotation relationships of finite element analyses. Therefore, the Power model was evaluated as an accurate mathematical representation for connection property through these comparisons.

Table 4-5 Parameters of Mathematical Representation

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Vertical Direction</th>
<th>Horizontal Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_i$</td>
<td>2.71E+10</td>
<td>1.64E+09</td>
</tr>
<tr>
<td>$K_p$</td>
<td>6.75E+07</td>
<td>1.54E+06</td>
</tr>
<tr>
<td>$K_t$</td>
<td>2.70E+10</td>
<td>1.64E+09</td>
</tr>
<tr>
<td>$M_u$</td>
<td>2.72E+07</td>
<td>4.11E+06</td>
</tr>
<tr>
<td>$\theta_0$</td>
<td>0.0010</td>
<td>0.0025</td>
</tr>
<tr>
<td>$M_a$</td>
<td>2.36E+07</td>
<td>3.52E+06</td>
</tr>
<tr>
<td>$\theta_a$</td>
<td>0.0011</td>
<td>0.0025</td>
</tr>
<tr>
<td>$K_a$</td>
<td>2.10E+10</td>
<td>1.43E+09</td>
</tr>
<tr>
<td>$M_b$</td>
<td>2.54E+07</td>
<td>3.84E+06</td>
</tr>
<tr>
<td>$\theta_b$</td>
<td>0.0022</td>
<td>0.0051</td>
</tr>
<tr>
<td>$K_b$</td>
<td>1.14E+10</td>
<td>7.52E+08</td>
</tr>
<tr>
<td>$A$</td>
<td>1.2917</td>
<td>1.1463</td>
</tr>
<tr>
<td>$B$</td>
<td>2.3805</td>
<td>2.1801</td>
</tr>
<tr>
<td>$n$</td>
<td>1.962</td>
<td>2.908</td>
</tr>
</tbody>
</table>
The connection properties of various angles between vertical and horizontal directions were investigated. Different magnitudes of pressures were applied on the top and side surfaces considering inclined directions of 30 through 60 degrees when the horizontal and vertical directions were assumed 0 and 90 degrees respectively. The connection properties under the inclined quasi-static pressures were expressed in terms of vertical and horizontal moment-rotation curves (Figure 4-6). According to the results, the horizontal properties dominated the behaviors as the directions were closed to 0 degree. However, the properties were more affected by vertical properties as the pressures tilted to the gravity direction.

(a) 30 degrees

Figure 4-6 Moment-Rotation Curves on the Inclined Directions (Continued)
(b) 45 degrees

(c) 60 degrees

Figure 4-6 Moment-Rotation Curves on the Inclined Directions
4.3.2 High-Strain-Rate Properties

To achieve one of the objectives in this study, investigation of dynamic behaviors of the WUF-B moment connection under blast load, finite element analyses were conducted using ABAQUS/Explicit. As stated in Chapter 2, little test data with respect to blast performance exist. Therefore, material properties of the finite element models were adjusted based upon dynamic material test data of the configuring members. In order to establish a steel connection model applicable to the blast loading environment, it is necessary to understand material behaviors subjected to dynamic forces. As presented in Section 2.5, the yield and ultimate strength are increased by dynamic increase factors (DIF), and the fracture toughness of the steel material deteriorates with increase of strain rate. The increased yield and ultimate strengths for the various components of the connections used in the numerical model were summarized in Table 4-6.

Table 4-6 Connection Component Yield and Ultimate Strengths

<table>
<thead>
<tr>
<th>Connection Component</th>
<th>( f_y ) (ksi) (w/o DIF)</th>
<th>DIF</th>
<th>( f_y ) (ksi) (w/ DIF)</th>
<th>( f_u ) (ksi) (w/o DIF)</th>
<th>DIF</th>
<th>( f_u ) (ksi) (w/ DIF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>50</td>
<td>1.29</td>
<td>64.5</td>
<td>64.8</td>
<td>1.1</td>
<td>71.28</td>
</tr>
<tr>
<td>Column</td>
<td>62.5</td>
<td>1.1</td>
<td>68.75</td>
<td>78.9</td>
<td>1.05</td>
<td>82.85</td>
</tr>
<tr>
<td>Shear Tab</td>
<td>50</td>
<td>1.29</td>
<td>64.5</td>
<td>64.8</td>
<td>1.1</td>
<td>71.28</td>
</tr>
<tr>
<td>Bolt/Nut</td>
<td>92</td>
<td>1.1</td>
<td>101.2</td>
<td>120.0</td>
<td>1.05</td>
<td>126.0</td>
</tr>
<tr>
<td>Weld</td>
<td>70</td>
<td>1.1</td>
<td>77</td>
<td>80.0</td>
<td>1.05</td>
<td>84</td>
</tr>
</tbody>
</table>

A parametric study was conducted comparing energy dissipated by plastic deformation, one of the factors in the energy balance equation, with the absorbed energy measured in the CVN tests. The same fundamental properties for elastic analyses, i.e. the elastic modulus, poisson’s ratio, were used for elastic analyses. Inelastic analyses were
based upon the von Mises yield criterion with shear failure model. Equivalent plastic strain (PEEQ) was a main parameter to control brittle/ductile properties of weld and base metal.

As shown in Figure 4-7, an energy-based parametric study was successfully conducted of the CVN test simulations that could provide appropriate PEEQ values of shear failure model (0.2 for base metal and 0.1 for weldment) which can define brittleness of the base metal and weldment materials.

(a) Undeformed and Deformed Finite Element Analysis Model for Charpy V-Notch

(b) Energy Dissipated by Rate-Dependent Plastic Deformation

Figure 4-7 FE Model and Simulation Results for E70T-4 Weld Metal and Base Metal
With the established models from quasi-static analysis and material properties determined from CVN tests, the computational simulations of the connection subjected to horizontal and vertical blast loads were carried out. The blast load was idealized as a triangular pulse. Peak force designated higher value than the maximum load applied during the quasi-static simulation in order to observe plastic behavior of the connection under the blast loads with different impulses. After that, the positive phase duration, \( t_d \), was gradually increased to apply greater impulses, as illustrated in Figure 4-8. The vertical and horizontal pressures were applied to the top and side surfaces of the girder respectively.

![Figure 4-8 Triangular Blast Load Pulses](image)

The assumed blast loading conditions shown in Figure 4-8 can be determined using the computer codes SHOCK (Naval Civil Engineering Laboratory 1988) and FRANG (Wager and Connett 1989). SHOCK is a blast load analysis program used to
calculate shock pressures, and impulses associated with shock waves. It can be used to obtain the peak shock pressures, and associated durations on all surface in the containing structure. FRANG can be used to calculate the gas pressure time-history and impulses that result from an internal explosion. By using SHOCK and FRANG, one of the simplified blast pulses in Figure 4-8 was estimated in the vertical and horizontal directions respectively. Figure 4-9 illustrates a perspective view of a schematic cubical chamber with an internal explosion event. The room sizes were based on the column and girder lengths in the test and numerical models (Figure 4-1). The explosive charge was assumed to detonate on the center of floor. The required input parameters for both programs are summarized in Table 4-7. Since the connection were attempted to be characterized in the vertical and horizontal directions independently and the room is not a cube but rectangular hexahedron, different charge weights were used for same horizontal and vertical line loads.

![Figure 4-9 Schematic Internal Explosion](image)
### Table 4-7 Input Parameters of SHOCK and FRANG

<table>
<thead>
<tr>
<th>SHOCK (shock pressure)</th>
<th>Vertical</th>
<th>Horizontal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Charge weight (lbs)</td>
<td>12</td>
<td>19</td>
</tr>
<tr>
<td>Distance to blast surface (ft)</td>
<td>11.33</td>
<td>11.33</td>
</tr>
<tr>
<td>Width of blast surface (ft)</td>
<td>22.67</td>
<td>22.67</td>
</tr>
<tr>
<td>Height of blast surface (ft)</td>
<td>22.67</td>
<td>11.33</td>
</tr>
<tr>
<td>Horizontal distance to charge from reflecting surface (ft)</td>
<td>11.33</td>
<td>11.33</td>
</tr>
<tr>
<td>Vertical distance to charge from reflecting surface (ft)</td>
<td>11.33</td>
<td>11.33</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>FRANG (gas pressure)</th>
<th>Vertical</th>
<th>Horizontal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume of the room (ft³)</td>
<td>5822.81</td>
<td>5822.81</td>
</tr>
<tr>
<td>Vent area (ft²)</td>
<td>1027.40</td>
<td>1027.40</td>
</tr>
<tr>
<td>Covered vent perimeter (ft)</td>
<td>136.0</td>
<td>136.0</td>
</tr>
<tr>
<td>Unit surface weight of the frangible panel (psf)</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>The shock load impulse (psi-msec)</td>
<td>94.2</td>
<td>124.1</td>
</tr>
</tbody>
</table>

One can establish many possible blast load scenarios by selecting the parameters defined in Table 4-7. Figure 4-10 showed a specific vertical and horizontal blast load as obtained with SHOCK and FRANG. In order to compare to an assumed blast pulses of Figure 4-8, the shock and gas pressures in the internal blast loads were simplified as a single triangular pressures including same impulse. Therefore, the blast pressures were simply assumed to be triangular pulses in the vertical and horizontal directions and utilized for understanding connection behaviors under the blast loads.
(a) Vertical Loading Cases (Ceiling)

(b) Horizontal Loading Cases (Front Wall)

Figure 4-10 Triangular Blast Load Pulses
Graphics produced by the simulation results clearly displayed the failure phenomenon of the connection members, as shown in Figure 4-11.

![Figure 4-11 Connection Failure under Blast Load (t_d=0.015)](image)

Rotation time-histories for each blast load pulse were obtained from the simulations in Figure 4-12. According to the vertical responses, the ultimate (failure) rotation was approximately assessed as 0.006 radians (0.344 degrees), and the failure appeared when t_d was 0.015 seconds. Thus, the maximum impulse that this connection can resist is \( \frac{1}{2} \times (4000 \text{ lb/in} \times (135.96 \text{ in})^2 / 2) \times 0.015 \text{ sec.} = 277.28 \text{ kips-in-sec.} \) In the case of the horizontal blast resistance, the maximum impulse was same as the vertical one, i.e. the connection was failed horizontally when the blast duration was 0.015 seconds. Therefore, it can be said that this connection demonstrated more ductile behavior in the horizontal direction than those in the vertical direction although the vertical strength is greater than horizontal one.
Figure 4-12 Rotation Time-History Results of Blast Analyses

(a) Vertical Direction

(b) Horizontal Direction
4.4 Simplified Frame Analysis

4.4.1 Quasi-Static Analysis

A finite element frame analysis was conducted with simpler beam elements and connector elements as opposed to beam, column, and connection members modeled by 3D elements. All of the same geometrical and material properties, boundary conditions, and loading history, which had been adopted in the previous simulations for the detailed model, were applied into the simple frame analyses, as shown in Figure 4-13.

(a) Vertical Loading                                       (b) Horizontal Loading

Figure 4-13 Simplified Frame Analysis

The key property of the simplified connection model is the moment-rotation relationship. The strain-rate independent curves (moment-rotation relationships in the quasi-static state) illustrated in Figure 4-5 were utilized first as a mechanical property of the connector element in the simplified frame analyses. The quasi-static analysis results from detailed connection models and simplified connector element models were compared in Figure 4-14 through 4-16. As seen in Figure 4-14, under the quasi-static
loading conditions, vertical and horizontal moment-rotation relationships were preserved in the analysis results of the simplified models.

(a) Vertical Direction

(b) Horizontal Direction

Figure 4-14 Moment-Rotation Relationships in the Quasi-Static Analyses
Especially, the moment-tip displacement relationships (Figure 4-15) could support the validity of the characterization methods for connection rotations explained in Section 3.4. Two different methods were utilized to define moment-rotation relationships. In Figure 4-15 (a) and (c) the relationships suggested by present study were shown as solid lines (connection) and those by TM5-1300 were plotted as dotted lines (midspan). Since the moment-rotation relationships by TM5-1300 were defined in terms of the displacements at midspan, they were less stiff than those by Equation 3-2 and Figure 3-1 in this study. With both relationships as connection element properties, the simplified connection models were simulated. According to the moment-tip displacement relationships in Figure 4-15 (b) and (d), there were large discrepancies between the outputs of detailed models and those of simple models using $M - \theta$ property of TM5-1300. Furthermore, in the case of horizontal displacement the result difference reached over 20 inches due to more severe bending of the girder. It means that the connection became more ductile than real property since the bending of the girder increased rotation values. On the other hand, the simple models using connection property proposed in this study could produce the girder tip responses which show good agreements with detailed model responses. These comparisons indicated that the TM5-1300 procedure in term of midspan deflection and member length mischaracterizes connection and cannot be used in the simplified models.
(a) Vertical Moment-Rotation Relationships based on Connection and Midspan

(b) Vertical Moment-Displacement Relationships

Figure 4-15 Quasi-Static Response Based on Connection and Beam Midspan (Continued)
(c) Horizontal Moment-Rotation Relationships based on Connection and Midspan

(d) Horizontal Moment-Displacement Relationships

Figure 4-15 Quasi-Static Responses Based on Connection and Beam Midspan
This simplified connection model could be extensively validated by the cyclic test introduced in Section 4-2. Using similar validation process for the detailed connection model, a cyclic test simulation was conducted in the simplified connection model. Table 4-8 and Figure 4-16 showed the comparisons between the test, the detailed model, and the simplified model simulation results. The displacements from the simplified model were around 15 percents of cyclic test results, with considerable savings in computation time and output file size (i.e., 48 hours running time and 3.75 GB for the detailed model but 3 minutes running time and 19 MB for the simplified model). Hence, as well as the case of detailed model, the simplified connection model was able to provide a good approximation of connection cyclic responses with huge cost savings.

Table 4-8 Load-Displacement Result Comparisons

<table>
<thead>
<tr>
<th>Cyclic Loads (kips)</th>
<th>Cyclic Test</th>
<th>Simulation (Detailed Model)</th>
<th>Simulation (Simplified Model)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Displacement (in.)</td>
<td>result</td>
<td>difference ratio (%)</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>75</td>
<td>0.375</td>
<td>0.382</td>
<td>-0.375</td>
</tr>
<tr>
<td>-75</td>
<td>-0.375</td>
<td>-0.390</td>
<td>0.492</td>
</tr>
<tr>
<td>150</td>
<td>0.75</td>
<td>0.737</td>
<td>0.737</td>
</tr>
<tr>
<td>-150</td>
<td>-0.75</td>
<td>-0.778</td>
<td>-0.778</td>
</tr>
<tr>
<td>175</td>
<td>1.0</td>
<td>0.975</td>
<td>0.975</td>
</tr>
<tr>
<td>-175</td>
<td>-1.0</td>
<td>-0.985</td>
<td>-0.985</td>
</tr>
<tr>
<td>195</td>
<td>1.5</td>
<td>1.51</td>
<td>1.51</td>
</tr>
<tr>
<td>-198</td>
<td>-1.4</td>
<td>-1.55</td>
<td>-1.55</td>
</tr>
</tbody>
</table>
Figure 4-16 Result Comparisons (Test, Simulation with Detailed and Simplified Models)
In addition, the behaviors of the simplified connection models subjected to inclined loads were analyzed using vertical and horizontal properties. The results were then compared with the results of the detailed models under the same load inclinations in terms with vertical and horizontal moment-rotation relationships, as shown in Figure 4-17. The comparative simulations exhibited high correlations between the results from detailed models and those from the simplified models. Therefore, it can be concluded that a simple connection model can be applicable to any inclined quasi-static loading situations if they are properly characterized in terms of vertical and horizontal moment-rotation curves from a detailed connection model.

![Diagram showing moment-rotation relationships under inclined loading.](image-url)

(a) 30 degrees

Figure 4-17 Moment-Rotation Relationships under Inclined Loading (Continued)
(b) 45 degrees

(c) 60 degrees

Figure 4-17 Moment-Rotation Relationships under Inclined Loading
4.4.2 Blast Analysis

As a following step, blast dynamic analyses were carried out and compared with the results from detailed models to evaluate performance of the simplified models. The vertical and horizontal line loads were applied to the girder in order to represent pressures on the top and side surfaces of the girder in the detailed models respectively. Unlike the quasi-static analysis results, the high-strain-rate connection responses were not effectively embodied in the strain rate independent connector element model, as shown in Figure 4-18. As the load duration was close to zero (became more instantaneous load), the high-strain-rate rotation time-history of the simplified frame analyses showed a little difference from that of full frame analysis with detailed model (i.e., \( t_d = 0.005 \)). However, the comparisons indicated larger discrepancies between the simplified and detailed results as the durations increased. The connector element behaviors with a quasi-static moment-rotation property were shown to be weaker than the behaviors obtained from the analyses with the detailed connection models. These discrepancies are probably due to strain-rate effect of materials in the connection details. Consequently, the moment-rotation curves obtained from quasi-static analysis need to be modified for the blast analyses, and therefore rate dependent connection properties are required for more feasible simplified frame models. Based on the moment-rotation relationships of blast analyses results in terms of detailed connection models and comparisons with the results of the simplified frame models, rate-dependent moment-rotation characteristics were established by multiplying by rotation based dynamic increase factors (Figure 4-19). It should be underlined that vertical behaviors became more brittle as higher impulsive loads were applied and horizontal behaviors were more ductile than vertical one under blast loads.
Figure 4-18 Result Comparisons between Detailed and Simplified Models

(a) Vertical Direction

(b) Horizontal Direction
Figure 4-19 Rate-Dependent Moment-Rotation Curves

(a) Vertical Direction

(b) Horizontal Direction
The simplified frames were reanalyzed using the modified established moment-rotation properties and the results were compared to those of detailed models (Figure 4-20). These comparisons showed that the simplified frame models considering rate-dependent connector element properties were capable of maintaining good accuracy while effectively reducing simulation time. Hence, it should be concluded that frame analyses could be carried out readily and accurately using simplified beam and connector elements instead of complex combinations of the beam, column, and connection members.

The rotations of the simplified connections subjected to inclined loads were compared with those of the detailed connections using vertical and horizontal rotation time-histories in Figure 4-21. Both results from the detailed and simplified models were corresponded with each other. Therefore, these results enable to conclude that the simple connection models can reasonably reflect structural behaviors of the detailed connection models in any inclined high-speed loading situations if they are properly characterized from detailed connection models in terms of rate-dependent vertical and horizontal moment-rotation relationships.
(a) Vertical Rotation Time-Histories

(b) Horizontal Rotation Time-Histories

Figure 4-20 Result Comparisons between Detailed and Modified Simplified Models
Figure 4-21 Comparisons of Result from Detailed and Simplified Models under Inclined and Different Load Durations (Continued)
Figure 4-21 Comparisons of Result from Detailed and Simplified Models under Inclined and Different Load Durations (Continued)

(c) 30 degrees of inclined loading, duration $t_d=0.015$

(d) 30 degrees of inclined loading, duration $t_d=0.02$
(e) 45 degrees of inclined loading, duration $t_d=0.005$

(f) 45 degrees of inclined loading, duration $t_d=0.01$

Figure 4-21 Comparisons of Result from Detailed and Simplified Models under Inclined and Different Load Durations (Continued)
(g) 45 degrees of inclined loading, duration $t_d=0.015$

(h) 45 degrees of inclined loading, duration $t_d=0.02$

Figure 4-21 Comparisons of Result from Detailed and Simplified Models under Inclined and Different Load Durations (Continued)
(i) 60 degrees of inclined loading, duration $t_d=0.005$

(j) 60 degrees of inclined loading, duration $t_d=0.01$

Figure 4-21 Comparisons of Result from Detailed and Simplified Models under Inclined and Different Load Durations (Continued)
(k) 60 degrees of inclined loading, duration \( t_d = 0.015 \)

(l) 60 degrees of inclined loading, duration \( t_d = 0.02 \)

Figure 4-21 Comparisons of Result from Detailed and Simplified Models under Inclined and Different Load Durations
4.5 Data Report

It was observed in the preceding sections that finite element analysis simulations could characterize the behaviors of steel moment connections subjected to quasi-static or blast-rate impulsive loads. A moment-rotation curve could effectively represent the structural character and it could be utilized as a property of a connector element in simplified frame analysis. The changes of the curve shapes with respect to the blast loadings demonstrated the need of a rate dependent property for the simple connector elements. Accordingly, not only a static moment-rotation relationship but also blast responses should be recorded in response data for one specific connection as described in Section 3.5. In order to understand the rotation-rate connection behaviors, blast analyses were conducted with more various blast loading conditions (Figure 4-22).

Figure 4-22 Various Blast Loads with Different Magnitude of Peak Loads and Durations
Figure 4-23 and 4-24 show connection responses with respect to the various blast loads. Peak rotation values were recorded with respect to each loading case in Table 4-9. And then, the simulations through simplified connection models were implemented to determine rate-dependent moment and rotation relationships. The increase factors extracted from the comparisons between the detailed and simplified model simulations were tabulated in Table 4-9. The increase factors define how the moment-rotation curves need to be adjusted in the simplified connection properties with respect to various loads. The factors were confirmed after comparisons with the results from detailed models as shown in Figure 4-23 and 4-24. According to the Table 4-9 and 4-10, the increase factors were proportionally related with the peak rotations unless the connection structures reached failure. That is, an increase factor may be anticipated if we acknowledge peak rotation and failure rotation. Using these factors, the moment-rotation curves of the connections subjected to various blast rate loads are approachable and usable to analyze simplified frames under quasi-static and blast loads.

Strain levels under different impulsive loads were identified through strain contours provided in the numerical model. Equivalent plastic strains were used to examine the responses. The maximum strain values and the location of the responses in all cases are summarized in Tables 4-11. Both vertical and horizontal results showed that the strain levels became higher and finally reached ultimate levels with increasing impulsive loads. As explained the high-strain-rate material properties in Section 4.3.2, the base metal and weldment material of values (failure strains) were 0.2 and 0.1 respectively, when the initial equivalent plastic strain value is equal to zero until yield point.
(a) Peak load = 1500 k/in

(b) Peak load = 2500 k/in

Figure 4-23 Vertical Rotation Responses to the Various Blast Loads (Continued)
Figure 4-23 Vertical Rotation Responses to the Various Blast Loads

(c) Peak load = 4000 k/in

(d) Peak load = 5000 k/in
Figure 4-24 Horizontal Rotation Responses to the Various Blast Loads (Continued)
(c) Peak load = 4000 k/in

(d) Peak load = 5000 k/in

Figure 4-24 Horizontal Rotation Responses to the Various Blast Loads
Table 4-9 Rate-Dependent Vertical and Horizontal Responses under Various Blast Loads

<table>
<thead>
<tr>
<th>Duration (sec)</th>
<th>Response</th>
<th>Vertical Peak Load (lb/in)</th>
<th>1500</th>
<th>2500</th>
<th>4000</th>
<th>5000</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.005</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00036 (0.021)</td>
<td>0.00063 (0.036)</td>
<td>0.00114 (0.065)</td>
<td>0.00215 (0.123)</td>
<td>1500</td>
</tr>
<tr>
<td></td>
<td>Increase Factor</td>
<td>1.0</td>
<td>1.0</td>
<td>1.20</td>
<td>1.35</td>
<td></td>
</tr>
<tr>
<td>0.01</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00057 (0.033)</td>
<td>0.00107 (0.061)</td>
<td>0.00364 (0.209)</td>
<td>0.02745 (1.573)</td>
<td>0.00068 (0.039)</td>
</tr>
<tr>
<td></td>
<td>Increase Factor</td>
<td>1.0</td>
<td>1.20</td>
<td>1.40</td>
<td>failure</td>
<td></td>
</tr>
<tr>
<td>0.015</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00068 (0.039)</td>
<td>0.00107 (0.061)</td>
<td>0.00364 (0.209)</td>
<td>0.02745 (1.573)</td>
<td>0.00068 (0.039)</td>
</tr>
<tr>
<td></td>
<td>Increase Factor</td>
<td>1.0</td>
<td>1.30</td>
<td>failure</td>
<td>failure</td>
<td></td>
</tr>
<tr>
<td>0.02</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00078 (0.045)</td>
<td>0.00219 (0.125)</td>
<td>0.10937 (6.266)</td>
<td>0.07679 (4.40)</td>
<td>0.00078 (0.045)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Duration (sec)</th>
<th>Response</th>
<th>Horizontal Peak Load (lb/in)</th>
<th>1500</th>
<th>2500</th>
<th>4000</th>
<th>5000</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.005</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00135 (0.077)</td>
<td>0.00232 (0.133)</td>
<td>0.00325 (0.186)</td>
<td>0.00334 (0.191)</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Increase Factor</td>
<td>1.0</td>
<td>1.0</td>
<td>1.30</td>
<td>1.30</td>
<td></td>
</tr>
<tr>
<td>0.01</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00221 (0.127)</td>
<td>0.00323 (0.185)</td>
<td>0.00511 (0.293)</td>
<td>0.00699 (0.40)</td>
<td>0.00221 (0.127)</td>
</tr>
<tr>
<td></td>
<td>Increase Factor</td>
<td>1.10</td>
<td>1.30</td>
<td>1.30</td>
<td>1.30</td>
<td></td>
</tr>
<tr>
<td>0.015</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00272 (0.156)</td>
<td>0.00423 (0.242)</td>
<td>0.00913 (0.523)</td>
<td>0.01646 (0.943)</td>
<td>0.00272 (0.156)</td>
</tr>
<tr>
<td></td>
<td>Increase Factor</td>
<td>1.30</td>
<td>1.30</td>
<td>failure</td>
<td>failure</td>
<td></td>
</tr>
<tr>
<td>0.02</td>
<td>Maximum Rotation [rad] (deg)</td>
<td>0.00331 (0.190)</td>
<td>0.00563 (0.323)</td>
<td>0.0159 (0.911)</td>
<td>0.06749 (3.867)</td>
<td>0.00331 (0.190)</td>
</tr>
</tbody>
</table>
### Table 4-10 Rotation-Based Increase Factors Dynamic Increase Factors

<table>
<thead>
<tr>
<th>Vertical Direction</th>
<th>Horizontal Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Rotation [rad] (deg)</td>
<td>DIF</td>
</tr>
<tr>
<td>0.0008 (0.046)</td>
<td>1.10</td>
</tr>
<tr>
<td>0.001 (0.057)</td>
<td>1.20</td>
</tr>
<tr>
<td>0.0012 (0.069)</td>
<td>1.25</td>
</tr>
<tr>
<td>0.0016 (0.092)</td>
<td>1.30</td>
</tr>
<tr>
<td>0.002 (0.115)</td>
<td>1.35</td>
</tr>
<tr>
<td>0.0035 (0.201)</td>
<td>1.40</td>
</tr>
<tr>
<td>0.006 (0.344)</td>
<td>Failure (midspan displacement: 2.79 in.)</td>
</tr>
</tbody>
</table>

### Table 4-11 Maximum Strains under Various Vertical and Horizontal Blast Loads

<table>
<thead>
<tr>
<th>Duration (sec)</th>
<th>Response</th>
<th>Peak Load (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1500</td>
</tr>
<tr>
<td>0.005</td>
<td>*PEEQ</td>
<td>0.0238</td>
</tr>
<tr>
<td>Location</td>
<td>fillet welds around shear tab</td>
<td></td>
</tr>
<tr>
<td>Vertical</td>
<td></td>
<td>0.1214</td>
</tr>
<tr>
<td></td>
<td>PEEQ</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Location</td>
<td>shear tab</td>
</tr>
<tr>
<td>0.01</td>
<td>PEEQ</td>
<td>0.20</td>
</tr>
<tr>
<td>Location</td>
<td>shear tab</td>
<td>shear tab</td>
</tr>
<tr>
<td>0.015</td>
<td>PEEQ</td>
<td>0.2001</td>
</tr>
<tr>
<td>Location</td>
<td>shear tab</td>
<td>shear tab</td>
</tr>
<tr>
<td>0.02</td>
<td>PEEQ</td>
<td>0.2001</td>
</tr>
<tr>
<td>Location</td>
<td>shear tab</td>
<td>shear tab</td>
</tr>
<tr>
<td>0.005</td>
<td>PEEQ</td>
<td>0.0017</td>
</tr>
<tr>
<td>Location</td>
<td>groove weld</td>
<td>groove weld</td>
</tr>
<tr>
<td>0.01</td>
<td>PEEQ</td>
<td>0.0060</td>
</tr>
<tr>
<td>Location</td>
<td>groove weld</td>
<td>groove weld</td>
</tr>
<tr>
<td>0.015</td>
<td>PEEQ</td>
<td>0.02366</td>
</tr>
<tr>
<td>Location</td>
<td>shear tab</td>
<td>beam flange</td>
</tr>
<tr>
<td>0.02</td>
<td>PEEQ</td>
<td>0.05353</td>
</tr>
<tr>
<td>Location</td>
<td>beam flange</td>
<td>fillet welds on column</td>
</tr>
</tbody>
</table>

*PEEQ: Equivalent Plastic Strain
It should be noted that the maximum strains were mainly located on the components around girder web with respect to the vertical loadings while those occurred around girder top and bottom flange parts with respect to the horizontal loadings. It means that the parts of the connection cross sections mainly resist against loads perpendicular to the each reference axis of moment of inertia.

In addition, Table 4-10 and 4-11 shows that this connection were failed when its rotation reached 0.006 radians (0.344 degrees) in vertical direction and 0.012 radians (0.688 degrees) in horizontal direction under blast loads. Considering the connection response and girder tip displacement in only vertical direction, in order to parallel with TM5-1300 damage assessments, the failure rotation was much less than 2 degrees which the blast design defined as no or light damage, as explained in Section 2.5.5. Even though we determine the failure rotation dividing the tip displacement by girder length, the failure rotation was 1.17 degrees. With quasi-static analysis results in Figure 4-15, these blast analysis results also indicate that connection design criteria in TM5-1300 need to be modified, confining connection behaviors except girder bending effect.

Also, this study proposed that a limit pressure is expressed as a limit moment, and a limit impulse is defined as the area of blast load pulse time-history. In the case of the given connection, for instance, the limit moment was computed as approximately 27000 kip·in at the vertical directions according to the quasi-static simulation results. A corresponding impulsive failure point was given by the area of the blast load pulse time-history, whose rotation reached the failure in the high-strain-rate simulations, i.e., 566.90 lb·in·sec. A repeating search of failure impulse with respect to different pressure enabled pressure-impulse diagram estimation, as shown in Figure 4-25.
Figure 4-25 Moment-Impulse Diagrams

(a) Vertical Direction

(b) Horizontal Direction
Since this diagram includes quasi-static and impulsive damage limits, it is expected to be an effective method of characterizing the connection structure in both loading situations. As explained in detail in Chapter 2, the P-I diagram is the combination of peak load and impulse that define damage threshold. If an arbitrary combination of pressure and impulse exceeds the curve, it will incur damage to the connection structure. Therefore, these diagrams will be useful to determine a connection structure property with respect to blast dynamic events.

In addition, the moment-impulse diagrams determined from detailed and simple connector element models were compared in Figure 4-25. Overall, the result comparisons in the vertical and horizontal directions were sufficient to demonstrate and maintain the equivalence between detailed and simplified models under blast load responses. The asymptotes derived from the detailed and simplified models were very close to each other within less than 10 percent. In the dynamic regimes, data points from the detailed models were located inside of the data line determined from the simplified models. The running time of the simulations to obtain single data point by detailed model was approximately 24 hours while those by simplified model were several minutes. Therefore, the simple model is expect to be more useful in the blast response characterization process since the finite element analyses in terms of dynamic explicit method are computationally more expensive than static analyses.

4.6 Summary

By following the listed steps and using an effective finite element code, preliminary analyses were conducted. Cyclic test data and FE analysis result were
compared in order to validate the established numerical model. The comparison indicated that the numerical model were able to accurately predict the connection behavior. After the validation, moment-rotation curves could be extracted from a quasi-static analysis. Mathematical equation presented in previous study was evaluated as an accurate expression for quasi-static connection property. In blast analysis it was investigated how the connection structure performed when various blast load were applied. The blast analyses, especially brittle failure, provided data points in the P-I diagram. Based upon the results of the detailed model, a simplified frame analysis was performed applying rate-dependent moment-rotation relationships and failure rotation for the connector element of the simple model. Comparisons of the rotation results between simple and detailed frame analyses showed that the connector elements were able to represent the complex connection component behaviors.
Chapter 5

DATABASE CONSTRUCTIONS

5.1 Introduction

So far, the characterization procedure and results have been described for one specific WUF-B type steel moment connection under quasi-static and high-strain-rate states. One of the goals of this study is to construct a database of structural properties of WUF-B connections under quasi-static and high-strain-rate loads. These data should be applicable to connector element properties in the simplified frame analysis. The design and property database construction processes for selected connection candidates are to be investigated in this chapter.

5.2 Connection Candidates

As mentioned before, previously computed results were developed in reference to only one type of beam-column combination and connection assembly as shown in Figure 2-3. Since the number of possible loading situations is very large, reasonable loading cases were selected and then girder-column and connection combinations were designed using the LRFD code. Table 5-1 lists girder and column candidates in this study. The flexural and axial strength capacity levels ($P_{eq}$ and $M_{eq}$ in Table 5-1) were designated based on the practical building design procedures. Statically indeterminate frames and moment connections are usually placed around the building exterior resist lateral loads. Most girders and columns of the statically indeterminate frame are subjected to combined axial compression and flexure generated from vertical (dead, live, snow loads, etc) and
lateral loads (wind, earthquake loads, etc). The girders are generally selected by using the equivalent moment loads and beam design charts (American Institute of Steel Construction, 2003). The required flexural capabilities of the girders in a fully restrained frame are less than those in an unrestrained frame due to load redistributions by redundant supports. Therefore, girders with relatively small flexural capacities of 100 to 600 k-ft were selected for this study. The columns were also treated as beam-columns. Bending moments are transmitted from girders through the moment resistant connections and combined with axial compressions. The influence of the combined loads causes an increase of moment in the columns (second order effect). Hence, equivalent axial loads should be used to determine column sizes from the column design tables (American Institute of Steel Construction, 2003). In this study the database table contained the columns that resist 400 to 3000 kips axial design loads.
Table 5-1 Girder and Column Candidates

### Roof

<table>
<thead>
<tr>
<th>NO</th>
<th>Girder</th>
<th>Equivalent Moment Range Meq (k-ft)</th>
<th>NO</th>
<th>Column</th>
<th>Equivalent Axial Pressure Range Peq (kips)</th>
<th>Connection Shear Force V (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14x22 (123)</td>
<td>100 ~ 150</td>
<td>1</td>
<td>W12x53 (474)</td>
<td>400 ~ 800</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>W12x26 (140)</td>
<td></td>
<td>2</td>
<td>W14x68 (604)</td>
<td></td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>W14x30 (176)</td>
<td></td>
<td>3</td>
<td>W14x82 (729)</td>
<td></td>
<td>20</td>
</tr>
</tbody>
</table>

### Floor

<table>
<thead>
<tr>
<th>NO</th>
<th>Girder</th>
<th>Meq (k-ft)</th>
<th>NO</th>
<th>Column</th>
<th>Peq (kips)</th>
<th>V (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W21x44 (359)</td>
<td>300 ~ 700</td>
<td>1</td>
<td>W12x53 (474)</td>
<td>400 ~ 800</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>W14x68 (604)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>W14x82 (729)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>W24x62 (578)</td>
<td></td>
<td>4</td>
<td>W12x87 (874)</td>
<td>800 ~ 1400</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5</td>
<td>W14x120 (1290)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6</td>
<td>W12x136 (1380)</td>
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<td></td>
</tr>
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<td></td>
<td></td>
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<td>7</td>
<td>W14x132 (1430)</td>
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<td></td>
</tr>
<tr>
<td>3</td>
<td>W24x68 (664)</td>
<td></td>
<td>8</td>
<td>W12x170 (1740)</td>
<td>1400 ~ 2200</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9</td>
<td>W12x210 (2170)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10</td>
<td>W14x211 (2330)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>11</td>
<td>W12x252 (2610)</td>
<td>2200 ~ 3000</td>
<td>60</td>
</tr>
</tbody>
</table>

floor girder and column assembly

roof girder and column assembly
5.3 Design of WUF-B Connections

WUF-B moment connections are subsequently designed based upon shear and moment loads. The shear force is resisted by a shear tab welded to the column flange and bolted to the pre-drilled beam webs with supplemental weld. Moment is transmitted through complete-joint-penetration groove welds between top and bottom flanges of the girder and supporting column. Stiffeners and doubler plates were added when compression or tension forces in girder flanges are greater than column capacities against local flange bending, local web yielding, web crippling, and compression buckling.

Therefore, the shear loads were defined based upon shear force diagrams obtainable from the rigidly connected girders subjected to AISC load combinations (American Institute of Steel Construction, 2003). The moment loads are designated from the design flexural strength. These connections were designed to support shear loads up to 60 kips and moment up to 600 k-ft. The component details for floor and roof connection designs were tabulated in Table 5-2 and 5-3 respectively.
### Table 5-2 Design Table (Floor)

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(unit: inch)
### Table 5-3 Design Table (Roof)

(unit: inch)

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- d<sub>g</sub>, d<sub>c</sub>: depth of girder or column
- t<sub>wg</sub>, t<sub>wc</sub>: web thickness of girder or column
- b<sub>g</sub>, b<sub>c</sub>: width of girder or column
- t<sub>b</sub>: flange thickness of girder or column
- b<sub>pl</sub>: width of shear tab plate
- h<sub>pl</sub>: height of shear tab plate
- t<sub>pl</sub>: thickness of shear tab plate
- L<sub>ev</sub>: length from edge to first bolt hole
- ϕ: bolt diameter
- N: number of bolts
- t<sub>w pl-cflange</sub>: thickness of weld between plate and column flange
- t<sub>w pl-gweb</sub>: thickness of weld between plate and girder web
- l<sub>st</sub>: length of stiffener
- b<sub>st</sub>: width of stiffener
- t<sub>st</sub>: thickness of stiffener
- t<sub>d</sub>: thickness of doubler plate
5.4 Database Construction

5.4.1 Quasi-Static Properties

The total number of design cases in this study is 45, which include 12 cases from roof members and 33 cases from floor members. First, the characterizing process described in the previous chapters was carried out through numerical simulations on the quasi-static basis. Rate-independent moment-rotation relationship database were collected as shown in Table 5-4, Table 5-5, and Figure 5-1.

First of all, as clearly shown in the moment and rotation curves, moment capacities depend mainly on the girders. The greater flexural capacities by means of thicker flange/web and larger moment of inertia, the higher yield and ultimate moments could be determined. This is due to the connection design concept in which the minimum capacity is required to resist the maximum flexural capacities through a girder. Therefore, flexural girder (beam) yielding would occur prior to the connection yielding if they are adequately designed and constructed. And also, stiffness generally depends on the kinds of column and configurations of panel zones. For example, when considering the cases W12 columns were used, case 2-6 and 3-6 exhibited lower stiffness than case 2-4 and 3-4 although the column sizes of the latters were higher grades than the formers. One of the reasons is that stiffeners were utilized in the case 2-4 and 3-4 while did not exist in case 2-6 and 3-6. Also, thicker doubler plates could support the structural capacities in panel zones of the case 2-4 and 3-4. These phenomena were observed in the comparisons between case 2-5 (3-5) and case 2-7 (3-7) that W14 column were assembled.
From previous strength and stiffness results, we would expect deformational features. In the categories where same sorts of girders were used, i.e., each case 1-, 2-, or 3-series, the yield and ultimate rotation values decreased as column members grew to large sizes although they had similar moment capacities. Namely, stronger columns made the connection structure somewhat brittle. It should be noted that excessive stiffening or strengthening to column panel zone may undermine connection ductility. The vertical strength and stiffness were much higher than the horizontal ones, as shown in preliminary results in Chapter 4. However, ductility was better in the horizontal direction.

As explained in Section 2.3 and applied in Section 4.3.1, moment-rotation curves could be represented mathematically. Several connections which can represent properties of the case studies were selected to express the $M - \theta$ relationships in the mathematical form using Equation 2-1 and 2-2 and equation parameters. The parameters for the equations were tabulated in Table 5-6. The mathematical representations for vertical and horizontal properties were solved and compared with finite element analysis results (Figure 5-2). As the comparisons in Chapter 4, the Power model (Richard-Abbott equation) with the proper parameters successfully represented the moment-rotation curves within 2 percents.
Table 5-4 Connection Property Database (Floor)

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### Table 5-5 Connection Property Database (Roof)

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<td>2.90E+05</td>
<td>8.72E-04</td>
</tr>
<tr>
<td>2-2</td>
<td>1.48E+09</td>
<td>9.45E+07</td>
<td>1.10E+06</td>
<td>3.28E+05</td>
<td>3.34E-03</td>
</tr>
<tr>
<td>2-3</td>
<td>1.28E+09</td>
<td>1.31E+08</td>
<td>1.52E+06</td>
<td>3.06E+05</td>
<td>2.30E-03</td>
</tr>
<tr>
<td>3-1</td>
<td>2.18E+09</td>
<td>3.19E+08</td>
<td>1.79E+06</td>
<td>4.00E+05</td>
<td>5.35E-04</td>
</tr>
<tr>
<td>3-2</td>
<td>2.06E+09</td>
<td>9.79E+07</td>
<td>1.81E+06</td>
<td>4.17E+05</td>
<td>1.34E-03</td>
</tr>
<tr>
<td>3-3</td>
<td>2.08E+09</td>
<td>1.45E+08</td>
<td>1.81E+06</td>
<td>3.95E+05</td>
<td>2.96E-03</td>
</tr>
</tbody>
</table>

### Table 5-6 Parameters of Mathematical Representation in Connection Property Database

<table>
<thead>
<tr>
<th>Case</th>
<th>Vertical Direction</th>
<th>Horizontal Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K_i</td>
<td>K_p</td>
</tr>
<tr>
<td>f 1-1</td>
<td>5.44E+9</td>
<td>2.21E+6</td>
</tr>
<tr>
<td>f 2-5</td>
<td>8.50E+9</td>
<td>2.12E+7</td>
</tr>
<tr>
<td>f 3-11</td>
<td>1.32E+9</td>
<td>4.94E+7</td>
</tr>
<tr>
<td>r 1-1</td>
<td>1.88E+9</td>
<td>1.04E+6</td>
</tr>
<tr>
<td>r 2-2</td>
<td>1.78E+9</td>
<td>2.87E+7</td>
</tr>
<tr>
<td>r 3-3</td>
<td>2.41E+9</td>
<td>6.99E+7</td>
</tr>
</tbody>
</table>
(a) Vertical Direction of Floor Connections

Figure 5-1 Moment-Rotation Relationship Database (Continued)
(b) Vertical Direction of Roof Connections Vertical Roof

Figure 5-1 Moment-Rotation Relationship Database (Continued)
(c) Horizontal Direction of Floor Connections

Figure 5-1 Moment-Rotation Relationship Database (Continued)
(d) Horizontal Direction of Roof Connections

Figure 5-1 Moment-Rotation Relationship Database
(a) Vertical Direction

Figure 5-2 Mathematical Representations for Moment-Rotation Relationship (Continued)
(b) Horizontal Direction

Figure 5-2 Mathematical Representations for Moment-Rotation Relationship
5.4.2 High-Strain-Rate Properties

As seen in the quasi-static results, moment-rotation curves as the characteristics of specific connection could be categorized with respect to girder and column combinations. Several connections which can represent properties in the case studies were selected for the high-strain-rate analyses instead of entire cases. Following the procedures, as explained in the Chapter 4, we could determine the blast dynamic properties of the connections in terms of rate-dependent increase factors and failure rotations as shown in Table 5-7 and Figure 5-3 to 5-8. The dynamic rotational increase factors in vertical direction increased as the maximum rotations increased under higher impulsive loads. However, in horizontal direction, the increase factors did not affect the blast-rate connection behavior. It can be said that horizontal bending and torsional stiffness of the selected girder and column were quite low; therefore, the girder and column deformations dominated the whole structural horizontal behaviors. On the other hand, the horizontal failure occurred at rotations three to five times larger than the vertical ones. It means that the horizontal behaviors were much more ductile than vertical ones as observed in the preliminary results in Chapter 4.

Failure rotations were also depicted in Table 5-7 and Figure 5-3 to 5-8. The failure rotations depended on the configurations of connection components. All vertical failure rotations were less than 2 degrees specified as no or light damage in TM5-1300. Thus, these results remark that the damage criteria should be more related with the connection behaviors.
Table 5-7 Rotation-Based Increase Factors and Equivalent Plastic Strains

<table>
<thead>
<tr>
<th>Case</th>
<th>Vertical Direction</th>
<th>Horizontal Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Rotation [rad] (deg)</td>
<td>Failure Rotation [rad] (deg)</td>
</tr>
<tr>
<td>Floor 1-1</td>
<td>0.003 (0.172)</td>
<td>0.008 (0.458)</td>
</tr>
<tr>
<td></td>
<td>0.003 (0.172)</td>
<td>0.008 (0.458)</td>
</tr>
<tr>
<td>Floor 2-5</td>
<td>0.001 (0.057)</td>
<td>0.003 (0.172)</td>
</tr>
<tr>
<td></td>
<td>0.001 (0.057)</td>
<td>0.003 (0.172)</td>
</tr>
<tr>
<td>Roof 1-1</td>
<td>0.006 (0.344)</td>
<td>0.003 (0.172)</td>
</tr>
<tr>
<td></td>
<td>0.002 (0.115)</td>
<td>0.007 (0.401)</td>
</tr>
<tr>
<td>Roof 2-3</td>
<td>0.007 (0.401)</td>
<td>0.001 (0.057)</td>
</tr>
<tr>
<td></td>
<td>0.007 (0.401)</td>
<td>0.001 (0.057)</td>
</tr>
</tbody>
</table>

*PEEQ: Equivalent Plastic Strain
(a) Vertical Rotation Time-Histories

(b) Horizontal Rotation Time-Histories

Figure 5-3 Rotation Time Histories from Detailed and Simplified Models (Floor 1-1)
Figure 5-4 Rotation Time Histories from Detailed and Simplified Models (Floor 2-5)
Figure 5-5 Rotation Time Histories from Detailed and Simplified Models (Floor 3-11)
(a) Vertical Rotation Time-Histories

(b) Horizontal Rotation Time-Histories

Figure 5-6 Rotation Time Histories from Detailed and Simplified Models (Roof 1-1)
(a) Vertical Rotation Time-Histories

(b) Horizontal Rotation Time-Histories

Figure 5-7 Rotation Time Histories from Detailed and Simplified Models (Roof 2-3)
(a) Vertical Rotation Time-Histories

(b) Horizontal Rotation Time-Histories

Figure 5-8 Rotation Time Histories from Detailed and Simplified Models (Roof 3-2)
The strain values and the locations where failure initiated were shown in Table 5-7. As explained in Chapter 4, failure strains (i.e., equivalent plastic strains are equal to 0.2 for base metals and 0.1 for weldment) were observed in the components around girder web and flange for the vertical and horizontal impulsive loads, respectively. Especially, in the horizontal loading cases of roof connections, the failure initiated at the bottom flange of girder and lower part of shear tab. These failures could be due to the twist at the column support as shown in Figure 5-9.

Based on the information from these blast dynamic analyses, moment-impulse diagrams were produced using various impulsive loads. Figure 5-10 are the moment-impulse diagrams for given connection cases. As explained in the preliminary results, the diagrams were made by tracing failure rotations with respect to possible blast loads with various impulses. Each diagram clearly shows the moment-impulse asymptotes and dynamic curved lines as characteristics about the blast-rate loads. The limit moment and
impulse were tabulated in Table 5-8. The statically strongest connection case (Floor 3-11) could resist the highest impulsive load combinations, and the weakest connection (Roof 1-1) failed under the lowest impulse and force. The limit values in the vertical direction were also greater than those in the horizontal direction. As a result, the limit moment and impulses increased in proportion to the quasi-static capacities.

Table 5-8 Limit Moment and Impulse

<table>
<thead>
<tr>
<th>Case</th>
<th>Vertical Direction</th>
<th></th>
<th>Horizontal Direction</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moment Asymptote (k·in)</td>
<td>Impulsive Asymptote (k·in·sec)</td>
<td>Moment Asymptote (k·in)</td>
<td>Impulsive Asymptote (k·in·sec)</td>
</tr>
<tr>
<td>Floor 1-1</td>
<td>2344.48</td>
<td>35.17</td>
<td>279.98</td>
<td>26.13</td>
</tr>
<tr>
<td>Floor 2-5</td>
<td>5049.64</td>
<td>55.23</td>
<td>641.12</td>
<td>26.71</td>
</tr>
<tr>
<td>Floor 3-11</td>
<td>5680.84</td>
<td>71.01</td>
<td>854.83</td>
<td>49.69</td>
</tr>
<tr>
<td>Roof 1-1</td>
<td>1036.98</td>
<td>18.94</td>
<td>92.65</td>
<td>21.00</td>
</tr>
<tr>
<td>Roof 2-3</td>
<td>1170.43</td>
<td>20.19</td>
<td>220.02</td>
<td>36.85</td>
</tr>
<tr>
<td>Roof 3-2</td>
<td>1517.14</td>
<td>25.49</td>
<td>186.65</td>
<td>34.22</td>
</tr>
</tbody>
</table>

5.5 Summary

Numerical investigations were conducted to assess the quasi-static and high-strain-rate behaviors with respect to various configurations of unreinforced welded-flange-bolted-web (WUF-B) steel moment connection. The connection cases were established based on LRFD design procedure considering realistic loading conditions of a typical building structure. Mechanical properties were collected from the quasi-static and high-strain-rate analyses through computational simulations. The present study provided database in which static and dynamic properties of other possible cases are stored. All results from WUF-B connection responses are expected to be an important part of a fast running algorithm of frame analysis.
(c) Vertical Direction

Figure 5-10 Moment-Impulse Diagrams (Continued)
(b) Horizontal Direction

Figure 5-10 Moment-Impulse Diagrams
Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Introduction

The conclusions in the present study are summarized in three categories, as follows: the quasi-static and blast-rate connection property definitions, the connection simplifications, and the property database constructions. Also, the limitations of this study and recommendations for future study are presented.

6.2 Conclusions

6.2.1 Connection Properties

With an efficient numerical model validation, the main objectives of this study were achieved and are presented as follows:

1. Three-dimensional finite element models using ABAQUS were preceded by comparing their predictions against experimental data obtained from AISC Northridge Test Program. The load and displacement relationships from the simulations and the experiments were very close to each other. It was demonstrated that the modeling procedure employed in this study is valid to reflect substantial connection behaviors.

2. In order to establish a steel connection model applicable to the blast-rate loading environment, dynamic increase factors (DIF) and brittle/ductile properties of weld and base metal the fracture toughness of the steel material were considered. A parametric study was conducted comparing energy dissipated by plastic
deformation with the absorbed energy measured in the Charpy V-Notch tests. The energy-based parametric study was successfully conducted of the CVN test simulations that could provide appropriate shear failure models.

3. From the validated model, push-over quasi-static analyses were conducted to derive the quasi-static connection properties. Vertical and horizontal moment-rotation relationships indicated that strength and stiffness of the connection were higher in the vertical way than in the horizontal way, although it exhibited high ductile behavior in the horizontal direction.

4. With the established models from quasi-static analysis and material properties determined from Charpy V-Notch tests, the computational simulations were carried out with respect to idealized blast loads with various durations. According to the dynamic analysis results, the connections showed more ductile behaviors in the horizontal direction than those in the vertical direction, although the vertical strength is greater that horizontal one.

5. The strain levels were investigated using strain contours in the numerical simulations. The maximum strains and the locations indicated that the strain levels became higher and finally reached ultimate levels with increasing impulsive loads. Under the vertical loading cases the maximum strains were mainly located on the components around the girder web. On the other hand, the maximum strains occurred around girder top and bottom flange parts with respect to the horizontal loadings. In conclusion, the parts of the connection mainly resist against loads perpendicular to the each reference axis of moment of inertia.
6. This study proposed moment-impulse diagram as a dynamic connection property. The diagram includes limit moment and a limit impulse defined as the area of blast load pulse time-history. Since this diagram includes quasi-static and impulsive damage limits, it is expected to be an effective method of characterizing the connection structure in both loading situations.

6.2.2 Simplified Connection Model

1. A finite element frame analysis was conducted with beam elements and connector elements. The moment-rotation relationships were utilized as mechanical properties of the connector elements. Under the quasi-static loading conditions, vertical and horizontal moment-rotation relationships were preserved in the analysis results of the simplified models. The moment-tip displacement relationships could support the validity of the characterization methods suggested in this study. According to the comparisons with cyclic test results, this simplified model was also able to simulate properly in the seismic loading condition.

2. In the present study, the connection responses with moment-rotation properties were compared against the properties by TM5-1300. The connection properties by TM5-1300 were defined in terms of the displacements at midspan containing bending effect of girder, so that the connection exhibits more ductile behaviors. With connection properties defined in this study, the simple models showed more accurate girder tip responses when compared with detailed model responses. These results indicated that the characterization method using TM5-1300 is inappropriate to apply to the connector element property of the simplified models.
3. The moment-rotation curves obtained from quasi-static analysis need to be modified for the high-strain-rate analyses as strain rate-dependent connection properties. Rate-dependent moment-rotation characteristics could be established by multiplying by rotation based dynamic increase factors. Similar to the quasi-static analysis results, the vertical behaviors under high-strain-rate loads became more brittle than horizontal ones as higher impulsive loads were applied although horizontal behaviors were more ductile than vertical one.

4. The increase factors for the rate-dependent moment-rotation relationships were proportionally related with the peak rotation magnitudes until the connection structures reached failure. Hence, the increase factor could be determined based on peak and failure rotation. Using these facts, the moment-rotation curves of the connections subjected to various blast rate loads were approachable.

5. The behaviors of the simplified connection models subjected to inclined quasi-static and impulsive loads were analyzed using vertical and horizontal properties. The comparative simulations showed high correlation between the results from detailed models and those from the simplified connection model. Therefore, this study showed that the simple model can be applicable to any inclined quasi-static and blast-rate loading situations if they are accurately characterized from the detailed connection model in terms of vertical and horizontal moment-rotation curves.

6. The simplified frame models considering connector element properties were capable of maintaining good accuracy in the quasi-static and blast-rate loading simulations while effectively reducing simulation time. It was concluded that
frame analyses could be carried out readily and accurately using simplified beam and connector elements instead of modeling detailed and complicated combinations of the beam, column, and connection members.

7. The failure rotations were not well within the blast design criteria for connections defined by TM5-1300. The design criteria for steel connections under blast loads need to be modified to consider only the connection behaviors without examining girder bending effects.

8. The moment-impulse diagrams obtained from detailed and simplified connection models were compared. The comparisons in the vertical and horizontal directions showed that the simple model adopted in this study was sufficient to describe the behaviors obtained from the original detailed model under blast load responses. Therefore, this study verified that the simple connection model is computationally efficient and reliable not only in progressive collapse rate quasi-static analyses but in the blast-rate dynamic analyses considering high velocity nonlinear and failure behaviors.

6.2.3 Database Construction

1. Database of structural properties of WUF-B connections under quasi-static and high-strain-rate loads were constructed. Girder-column and connection combinations were selected and designed using the LRFD code. The database showed that moment capacities depend mainly on the girders in the quasi-static properties. The greater flexural capacities by means of thicker flange/web and larger moment of inertia, the higher yield and ultimate moments could be
determined. And also, stiffness generally depends on the kinds of column and configurations of panel zones. Stiffeners and doubler plates contributed to the panel zones strength and had an effect on the connection stiffness.

2. Stronger column and panel zone made the connection structure somewhat brittle. It should be noted that excessive stiffening or strengthening to column panel zone may undermine connection ductility. The vertical strength and stiffness were much higher than horizontal ones, but ductility was better in the horizontal way.

3. The moment-rotation curves obtained by quasi-static push-over simulations were successfully represented in the mathematical form with high accuracy. Parameters were investigated for vertical and horizontal curves of each connection case. Therefore, the study showed that the Power model can be used as an accurate mathematical representation for connection property through proper comparisons.

4. Based on the information from these blast-rate analyses, rate-dependent moment-rotation relationships and failure rotations, moment-impulse diagrams were produced using various impulsive loads. The comparisons of the diagrams in different connections showed that the greater quasi-static capacities the connections have the higher limit moment and impulses. The limit moment and impulse in the vertical direction were also greater than those in the horizontal direction.

5. The present study provided database in which quasi-static and blast-rate dynamic properties of other possible cases are stored. All results from WUF-B connection responses are expected to be an important part of a fast running algorithm of frame analysis.
6.3 Recommendations

These numerical work and efforts provided greater insight into the structural static and dynamic behavior of WUF-B steel moment connection subjected to progressive collapse and blast event type loading conditions. In a future study, it is recommend that further research will be carried out on the following subjects:

1. More cases (i.e. more girder-column combinations and connection configurations) will enrich the property database.

2. In this study only WUF-B type moment connection was investigated. The research approach will be extensively able to apply into many kinds of connection assembly, such as different types of moment connections, shear connections, semi-rigid connections, and base connections.

3. Blast design criteria in existing steel connection design needs to be reestablished focusing on the structural capacities of connection configurations themselves, rather than girder or beam bending.

4. Test output data under short-duration loadings should support the development of a more accurate numerical simulation, i.e., more experimental work should include higher speed loading.

5. This study is a part of ongoing efforts to develop a fast running algorithm that will enable the judgment of survivability of a building frame under blast event and progressive collapse. Since the connection failure will be one of the significant causes of the building collapse, sufficient property data are essential to cover a large range of possible connection configurations. In parallel with the database construction, it will be necessary to develop data link system with the fast running
program considering occurrence of any loading incidence, determination of damaged state, and embodiment of structural performance after a loading incident.
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Impact Behaviors of Lightweight and Reinforced Concrete Structures
Steel Connection Analysis and Design under Quasi-Static, Seismic, and High-Strain-Rate Loads
Progressive Collapse Analysis of Building Structures
Protective Technology

Publications

Military
Radar Operator, Frigate Ship

Computer Skill
Structural Analysis Tools: ABAQUS/Standard & Explicit, ANSYS, LS-DYNA, STAAD, SAP2000
Programming: MATLAB, FORTRAN, EXCEL-VISUAL BASIC APPLICATIONS