Wind Drift Design of Steel Framed Buildings: An Analytical Study and a Survey of the Practice

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Abstract

The design of steel framed buildings must take into consideration the lateral drift of the structure due to wind loading and any serviceability issues that may arise from this lateral movement. This thesis focuses on one of these issues, damage to nonstructural components.

Although there are no specific requirements in the United States governing the effects of wind drift, it is an important issue which may significantly impact the buildings structural performance and economy. Furthermore, because these serviceability issues are not codified, there is a wide variation among design firms in how they are dealt with, leading to a greater economic disparity.

This thesis begins with a comprehensive review of the literature that covers all pertinent aspects of wind drift in steel framed buildings. Next an analytical study of the variations in modeling parameters is performed to demonstrate how simple assumptions can affect the overall buildings stiffness and lateral displacements. A study is then carried out to illustrate the different sources of elastic deformation in a variety of laterally loaded steel frames. The different modeling variables demonstrate how deformation sources vary with bay width, the number of bays and the number of stories, providing a useful set of comparisons.

To ascertain how serviceability issues are dealt with from firm to firm, a survey of the practice is developed to update the one conducted in 1988 (ASCE). In effect, the thesis is presented with the intention of suggesting and establishing a comprehensive, performance based approach to the wind drift design of steel framed buildings.

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"I get by with a little help from my friends."
- *John Lennon*

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"Don't let schooling interfere with your education. "
- *Mark Twain*

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Chapter 1

Introduction

"Lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems due to wind effects shall not impair the serviceability of the structure" (ASCE 7-05 App B.1.2).

1.1 Philosophy of Design for Drift

On the most basic level, structures are designed for strength (safety) and serviceability (performance). Adequate strength is obtained by designing structural members against buckling, yielding, instability and fracture in accordance with the applicable building code specifications. Serviceability issues include deflection, vibration and corrosion but with respect to wind the issues of concern are deformation (drift) and motion perception (acceleration). This thesis focuses on damage to nonstructural components as a result of drift.

Serviceability issues are dependent on the type of building and the needs of the owner or occupants. For these reasons, and because they are not life-safety related, current U.S. building codes do not regulate serviceability issues. Because these issues are not codified, choices on appropriate designs are left to the engineer's judgment. Designing for drift is important for both strength design, where second order (P-Delta) effects can create instability, and for serviceability. With respect to serviceability, designing for drift is done to prevent or limit unacceptable damage to nonstructural building components such as interior cladding and partitions as well as ensure the functionality of mechanical systems such as elevators.

Adequate building stiffness is obtained by designing a building to be within reasonable drift limits. However, there are three major sources of discrepancies in design that result in differences in economy and performance among design firms (Charney 1990[1]):

[1] References are located at the end of each Chapter

- definition of drift and variations in drift limits
- variations in methods of analysis used to predict drift
- variations in wind loads used for drift calculations

If a building is to be optimized to within a few percentage points of the acceptable limit for serviceability damage criterion, proper modeling is essential. As a quick example consider the following scenario from Charney (1990): Company A performs a building design using a drift limit of h/200 for a 10 year service wind but neglects shear and panel zone (joint) deformations in analysis. Company B designs the same building using a drift limit of h/600 for a 25 year service wind and includes all sources of deformation in analysis. Given that all other variables are equal and that wind drift is the controlling factor, which is often the case especially in non-seismic regions, Company A has most likely designed a less expensive building. The allowable drift is greater and certain sources of deformation are being ignored resulting in a building that is not nearly as stiff as the one designed by Company B. Although one building design is significantly less expensive, which one is adequate? It is hard to say given the amount of uncertainties involved, especially the contributions from nonstructural components which may well offset the apparently incautious design of Company A.

1.2 A Brief History of Wind and Structures

Engineers have always realized that wind can affect structures. The French structural engineer Alexandre Gustave Eiffel recognized the effects of wind when he designed the Eiffel Tower. At 986 feet, the Eiffel Tower was the tallest structure in the world from 1889 until 1931, when it was surpassed by the Empire State Building. In the design of the Eiffel Tower the curve of the base pylons was precisely calculated for an assumed wind loading distribution so that the bending and shearing forces of the wind were progressively transformed into forces of compression, which the bents could withstand more effectively (Mills 1999).

For advancements to come about in any field, it is usually true that there must be some sort of impetus for change; factors that spur new ideas and solutions. Economic factors drive many facets of our everyday life and this is especially true for the field of Structural Engineering. The need to build higher, particularly in dense urban areas, brought about advancements in engineering and construction techniques that saw the skyscraper boom of the 1920's and 30's and the revival in the 1960's. Building big meant spending big and subsequent advancements were made in the form of lighter, stronger materials. In turn, building large and light led to lightly damped and more flexible structures. Consequently wind was suddenly an important issue in the design of structures.

There are three important structural failures involving wind that deserve mention here. They are important milestones in the advancing art of designing for wind and will be presented in the order in which they occurred. Attention to wind was first brought to the forefront of the field in 1940 when Washington State's Tacoma Narrows Bridge collapsed under moderate, 40 mph winds. This is quite possibly the most well-known example of the effects of wind on a large structure. Failure was caused by inattention to the vibratory nature of the structure; the low yet sustained winds caused the bridge to oscillate at its natural frequency, increasing in amplitude until collapse. Wind tunnel tests were suggested and implemented for the subsequent bridge design (Scott 2001).

The second failure involved the 1965 failure of three, out of a total of eight, 400-foot reinforced concrete cooling towers. Located in England, the failure of the Ferrybridge cooling towers demonstrated the dynamic effects of wind at a time when most designs considered wind loading as quasi-static (Richards 1966). However, wind is gusty and these peaks in the flow must be designed for, not simply the average, especially when the structure is inherently flexible. The towers failed under the strong wind gusts when the wind load tension overcame the dead load compression. It has also been suggested by Armitt (1980) that the wind loading was magnified by the interference effects of the surrounding towers.

The third example involves Boston's John Hancock Tower. In early 1973 the John Hancock tower experienced 75 mph winds that were believed to cause over 65,000 pounds of double plane windows to crash to the sidewalks below. Due to an agreement between the involved parties nobody knows the exact reason why the windows failed, although it is widely speculated that the problems were due to a window design defect (Campbell 1996). In addition to the cladding issues the Tower swayed excessively in moderate winds, causing discomfort for occupants of the upper floors. The unacceptable motion was solved by installing two 300 ton tuned mass dampers, which had just been invented for the Citicorp Tower in New York (LeMessurier 1993). Additional lateral bracing was also added in the central core (at cost of $5 million) after it was determined that the building was susceptible to failure under heavy winds (Campbell 1996, Sutro 2000).

It is interesting to note that prior to construction of the John Hancock Tower, wind tunnel tests on the design were conducted in a less expensive aeronautical wind tunnel, as opposed to a boundary layer wind tunnel, and the results did not indicate any problems. The importance of modeling for the boundary layer, in which terrain, gustiness and surrounding structures all come into play, was suddenly obvious; the overall behavior and interaction of wind and structures was becoming apparent to the structural engineering community. With proper wind tunnel testing, the John Hancock Tower may have avoided costly retrofitting.

New York City's World Trade Center towers and Chicago's Sears Tower were among the first to fully exploit the developing wind tunnel technology. Built during the second skyscraper boom these buildings and others fully exploited all available resources and technological advancements. Boundary layer tests were conducted that allowed the designers to optimize the structural system for displacements, accelerations and to design the cladding for wind pressures as well. Technology continues to charge ahead and today's wind tunnel tests are more accurate and less expensive than ever before. For example, the pressure transducers used in wind tunnel tests are much less expensive; they have dropped in price from over a thousand dollars a piece in the 1970's to thirty or forty dollars today (Sutro 2000). Thanks to lower prices and faster computers, wind tunnel experts now get real-time wind tunnel data from 500 or more transducers, a vast improvement over the 8 or 16 typical in the 1970's.

Throughout the years innovations have been made in how structures are designed for the effects of winds loads, how wind loads are determined and applied, and how the limits of wind loads are defined and utilized. In a way, technology has both created and has helped to solve the problems related to wind effects on structures. As new materials, both stronger and lighter than predecessors, have been developed new problems have been encountered. The use of lighter concretes, composite floors and stronger structural steel has resulted in less damping and less stiffness. Less damping results in more motion (acceleration) and less stiffness results in greater lateral displacements. The importance of designing for wind has never been more apparent or more important.

1.3 Project Scope

In 1988 the ASCE Journal of Structural Engineering published a paper titled “Wind Drift Design of Steel-Framed Buildings: State-of-the-Art Report” (ASCE 1988). It was the culmination of four years of work by a task committee created by the ASCE Committee on Design of Steel Building Structures. The committee developed and conducted a survey to assess the state of the art of drift design for steel buildings. One-hundred and thirty-two firms were sent questionnaires and thirty-five responded. After the survey was conducted the committee published the above mentioned paper which summarizes the survey responses as well as opinions and comments provided by the task committee. This thesis aims to address the various issues affecting the design of steel framed buildings for drift and to design and conduct an updated version of the survey.

Chapter 2 of this thesis presents a comprehensive review of the literature since the last ASCE survey (1988) on pertinent topics related to drift design of steel frame structures. The main issues discussed here are Drift Limits and Damageability (2.2), Modeling and Analysis for Drift Design (2.3) and Wind Loads (2.4). Furthermore, the sub-topics address modeling procedures/techniques, methods of analysis, least weight/cost optimization, wind loading, code formulations, quantifying drift damage, damage definitions and cases of recorded building response. With this being said, many of the articles and discussions contained within are not directly related to steel framed buildings and may be applied to concrete and composite buildings as well. This is because all structures are subjected to wind loading and much of the work done here is tied directly to just that, wind loading.

Chapter 3 of the thesis can be viewed as an analytical investigation into many of the topics discussed in Chapter 2. A 10 story analytical “test building” is used to demonstrate the effects of different modeling practices on the lateral flexibility of the structure. The discussed issues include sources of deformation, P-Delta effects, joint region modeling, slab girder interaction, reduced live loads and nonstructural components. This chapter also serves to illustrate both the power and limitations of commercially available analysis software. Building diagrams/details can be found in Appendix A.

Chapter 4 involves the modeling and analysis of 27 planar steel frames and 18 framed tube structures to determine the various sources of deformation under lateral loads. Axial, shear and flexural deformations are compared with respect to the modeling variables of bay width, building height, and the number of bays. Also compared are the deformation contributions from the different structural members, which can be broken down into columns, beams and the beam-column joint, which is modeled several different ways for comparison purposes.

Chapter 5 is reserved for overall conclusions and recommendations for further work.

Appendix A addresses the newly developed ASCE sponsored Wind Drift Survey. The ASCE Journal of Structural Engineering first published a report on the state of the art in designing for wind drift of steel framed buildings in 1988. There have been significant advancements in the field of structural engineering since this last survey; the most important one being the widespread use of computers to assist in the design process. In addition to computer hardware and software advances that allow nearly limitless design possibilities and almost instantaneous analysis, the issue of serviceability has become more important than ever. Economically speaking, the serviceability considerations are commonly the controlling factor for many building designs. Furthermore, because these serviceability limits states are not codified in the United States and scarcely mentioned in design guides, it is of the utmost importance to gain an understanding of how the structural engineering community approaches these issues and feels about the guidelines currently in place. Appendix A addresses these issues and presents the survey in its entirety.

Appendix B contains the full results of the analytical test building models which are presented in Chapter 3.

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Chapter 2

Literature Review

2.1 Introduction

This review of the literature covers three main topics: Drift Limits and Damageability, Modeling and Analysis for Drift Design and Wind Loads.

The purpose of the literature review is to cover material related to wind drift from the perspective of damage of nonstructural components, modeling and analysis and the appropriate wind loads. Covering the issues in this way is crucial to establish that wind drift is a multi-dimensional issue that is dependent on many variables. In effect, the literature review is conducted with the intention of suggesting and establishing a comprehensive, performance based approach to the wind drift design of steel framed buildings.

2.2 Drift Limits and Damageability

Drift limits are imposed for two reasons: to limit second order effects and to control damage to nonstructural components. Although drift may also be limited to ensure human comfort, this thesis does not cover this issue. Limiting second order effects is necessary from a strength perspective while controlling damage to nonstructural components is a serviceability consideration. For serviceability issues several topics need to be discussed: the definition of damage, drift/damage limits to be imposed and the appropriate return interval to use when calculating wind loads.

2.2.1 Definition of Damageability

Traditionally drift has been defined in terms of *total drift* (the total lateral displacement at the top of the building) and *interstory drift* (the relative lateral displacement occurring between two consecutive building levels). Drift itself is not very useful in defining damageability as a total roof drift of 15 in. may be acceptable in a 40 story building but certainly not in a 10 story building and likewise an acceptable interstory drift is dependent on the story height. But when drifts are divided by heights the result is a drift ratio or *drift index*. The drift index is a simple estimate of the lateral stiffness of the building and is used almost exclusively to limit damage to nonstructural components. Equation 2.1 defines the drift index.

$$\textit{drift index} = \textit{displacement/height} \tag{2.1}$$

Referring to Figure 2.1, a total drift index (Equation 2.2) and an interstory drift index (Equation 2.3) can be defined as such:

$$\textit{total drift index} = \textit{total drift/building height} = \Delta/H \tag{2.2}$$

$$\textit{interstory drift index} = \textit{interstory drift/story height} = \delta/h \tag{2.3}$$

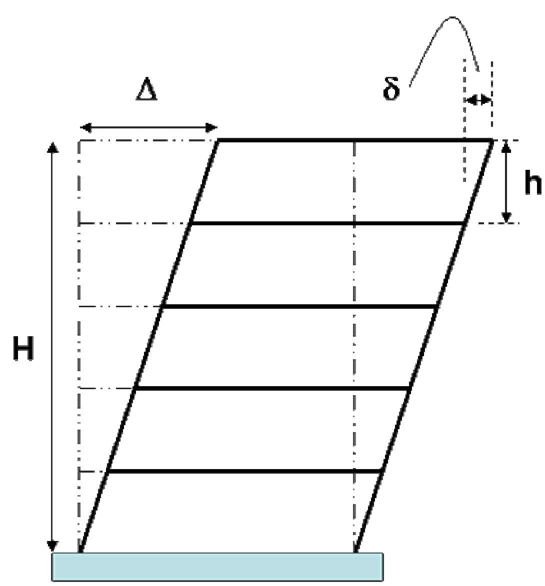

Figure 2.1: Drift Measurements

To limit nonstructural damage, these drift indices are limited to certain values to be discussed in the next section. Using drift indices is a straightforward, simple way to limit damage. However, three shortcomings are apparent in using drift indices as a measure of building damageability: One, it oversimplifies the structural performance by judging the entire building on a single value of lateral drift. Two, any torsional component of deflection and material damage is ignored. Three, drift as traditionally defined only accounts for horizontal movement or horizontal racking and vertical racking is ignored. The true measure of damage in a material is the shear strain which is a combination of horizontal and vertical racking. If one considers that the shear strain in the damageable material is the realistic parameter to limit, then it is seen that drift indices are not always sufficient.

Charney (1990c) and Griffis (1993) have proposed quantifying damage in terms of a drift damage index (DDI) that takes into account horizontal and vertical racking while excluding any rigid body rotation. The DDI is simply the average strain in a rectangular element. To calculate the DDI a planar, drift damageable zone (DDZ), with height H and width L, is defined where the local X,Y displaced coordinates at each corner are known (from analysis). This drift damageable zone is typically bounded by columns on both sides and floor diaphragms at the top and bottom. If this panel is defined by corner nodes A,B,C and D (Figure 2.2) the average shear strain or DDI can be defined by Equation 2.4 (Charney 1990c):

$$DDI = 0.5 * \left[\frac{X_A - X_C}{H} + \frac{X_B - X_D}{H} + \frac{Y_D - Y_C}{L} + \frac{Y_B - Y_A}{L} \right] \qquad (2.4)$$

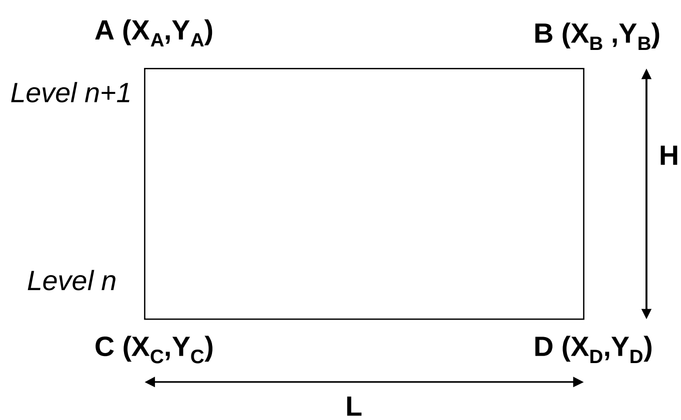

Figure 2.2: Drift Damageable Zone

Equation 2.4 is the average of the horizontal and vertical components of racking drift and provides the average shear strain in the panel while the commonly used interstory drift only accounts for horizontal racking. If the DDI is multiplied by the panel height, the result is the *tangential interstory drift* (Bertero *et al.* 1991). This value, which like the DDI takes into account the vertical component, can be compared to the *interstory drift*. Both the DDI and the tangential interstory drift provide better estimates of damage than the conventional interstory drift ratio or interstory drift.

Examples have been performed to highlight the shortcomings of using interstory drift as a measurement of damageability and can be found in the papers by Charney and Griffis. One example (Charney 1990c) is repeated here for demonstration purposes. Figure 2.3 shows two 10 story frames, one braced and one with moment connections, and Table 2.1 shows the comparisons between the interstory drift and the drift damage index. Bays a, b, c refer to the uppermost bays of the frames.

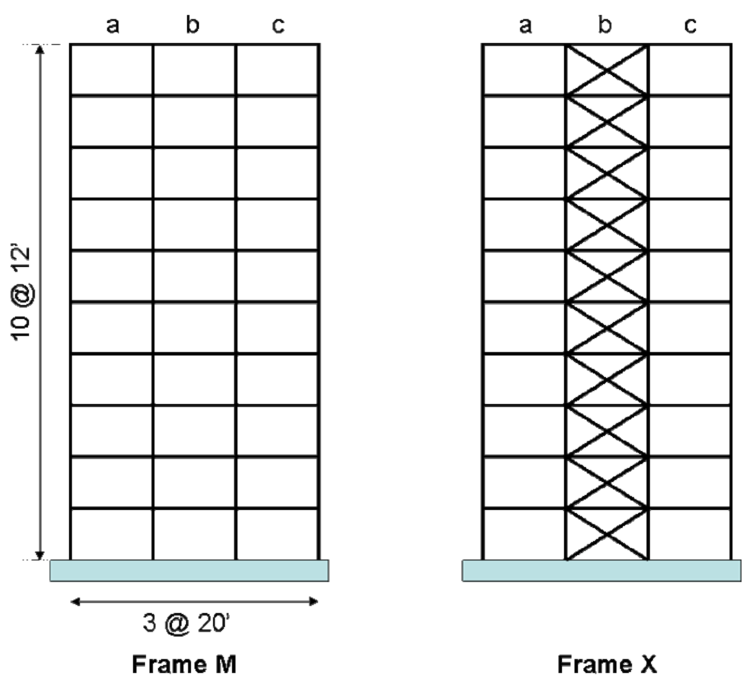

Figure 2.3: Drift Indices and Drift Damage Indices

Table 2.1: Drift Indices and Drift Damage Indices

		Interstory Drift Index (1)	Drift Damage Index (2)	Shear Distortion (%)	(2)/(1)
Frame M (unbraced)	**Bay a**	0.00267	0.00219	0.219	0.820
	Bay b	0.00267	0.00267	0.267	1.000
	Bay c	0.00267	0.00219	0.219	0.820
Frame X (braced)	**Bay a**	0.00267	0.00358	0.358	1.341
	Bay b	0.00267	0.00083	0.083	0.311
	Bay c	0.00267	0.00358	0.358	1.341

The two frames were subjected to identical lateral wind loads. The drift damage index in Table 2.1 was computed using Equation 2.4. For the unbraced frame (Frame M), drift damage is overestimated in the two outer bays by using the traditional interstory drift index. Conversely, damage in the outer bays of the braced frame (Frame X) is severely underestimated using the drift index (which doesn't account for the vertical component of racking) while the damage in the middle braced bay is overestimated. This overestimation is due to the rigid body rotation which causes little damage, in and of itself.

Although the example provided is planar, drift damageable zones can easily be extended for use in three dimensional models. Using a software modeling program such as SAP2000 (Computers and Structures, Inc. 2006), a membrane element with near zero stiffness can be inserted at any desired location in the structure; the analysis is then run, as normal, and a shear stress (τ, ksi) is reported for the diaphragm. Easily converted to shear strain (γ, in./in.) by dividing by the shear modulus G ($\gamma = \tau$/G), this value is compared against experimentally derived values to evaluate the performance of the material. To demonstrate this, an example using SAP2000 is now presented. In the program a material is defined that has virtually no stiffness and is defined as such:

E = 2.6 ksi (modulus of elasticity)

α = 0.3 (Poisson's ratio)

$$G = \frac{E}{2*(1+\alpha)} = \frac{29000}{2*(1+0.3)} = 1.0\ ksi \text{ (shear modulus of elasticity)}$$

A modulus of elasticity of 2.6 ksi and a Poisson's ratio value of 0.3 gives a shear modulus of 1.0 ksi which means shear stress = shear strain. Given that SAP2000 is able to show stress contours for areas, this is a convenient way to view the shear strain damage in the membrane element without doing any conversions. Once the material is defined, membranes can be inserted at any location in the building. Figure 2.4 shows the stress contours for membranes in a 3 story, 3 bay moment frame subjected to lateral loads.

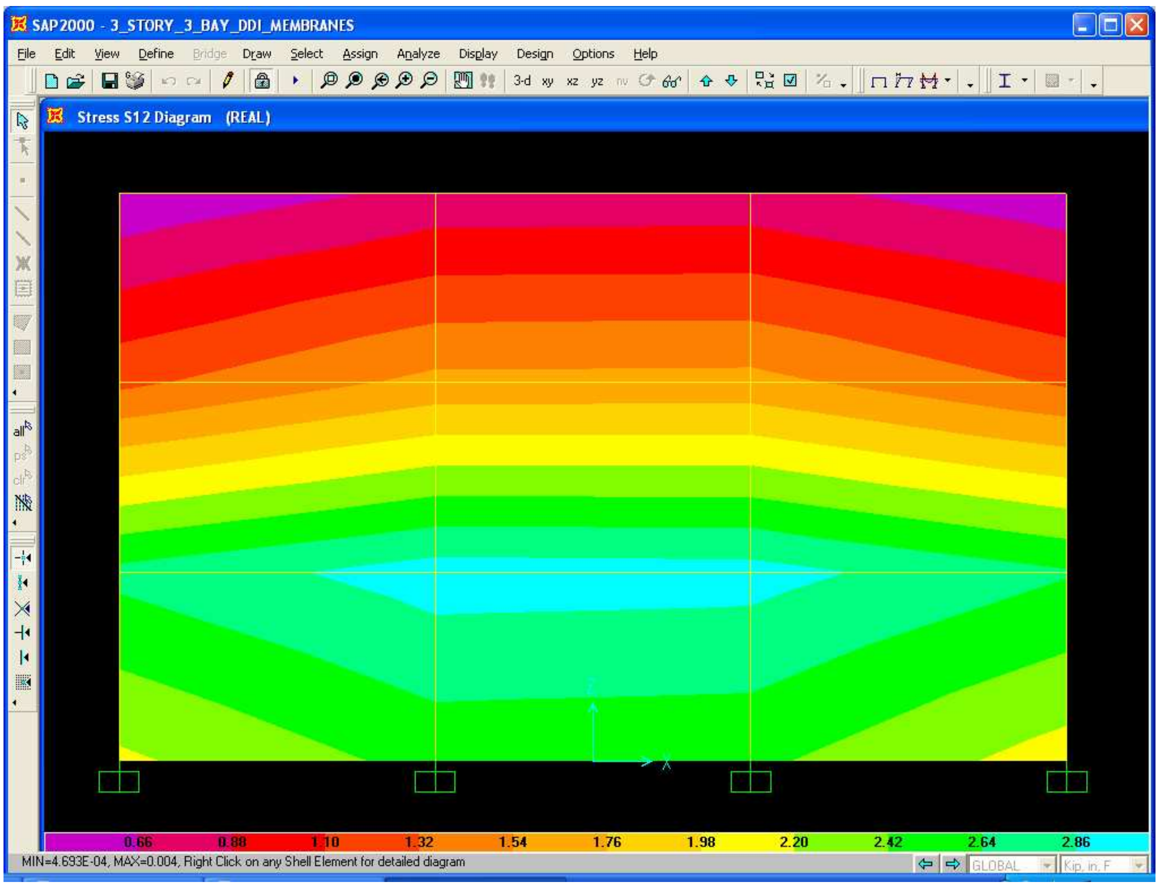

Figure 2.4: Shear Strain Contours

Horizontal loads are applied at the left most nodes and are directed to the left. Maximum strain is observed near the top of the bottom, middle bay. Minimum strain is observed in the top corners of the upper bays. Using strain membranes is convenient and allows the designer to easily observe areas of high strain.

2.2.2 Drift and Damage Limits

In the context of serviceability, drift limits are imposed to limit damage of nonstructural components to an acceptable level. This acceptable level is a function of several factors including the material in question, wind recurrence interval, building function, cost of repair vs. cost of prevention and the needs of the owner and/or occupants. Results of the 1988 ASCE (ASCE 1988) survey assessing the state of the art in designing for drift of steel frame structures shows that there is no consistency in these drift limits among designers, leading to variations in structural economy and performance (Charney 1990c).

First and foremost the damage limit is dependent on the material in question's damage threshold: the amount of strain that can be sustained before damage is visible. Damage thresholds are typically obtained through racking tests in which damage limits are obtained for common nonstructural building materials. Commonly used materials include brick, composite materials, concrete block, drywall, plaster, plywood and tile. A comprehensive listing of thresholds for various materials was compiled by Algan (1982) who basically assembled all the previous research results, encompassing over 700 racking tests and 30 sources. Damage was defined in terms of *damage intensity*, a value ranging from 0.1 (minor damage) to 1.0 (complete damage). Figure 2.5 serves to illustrate the differences in damage thresholds among common partition materials and is taken as a series of best-fit lines through the data gathered by Algan.

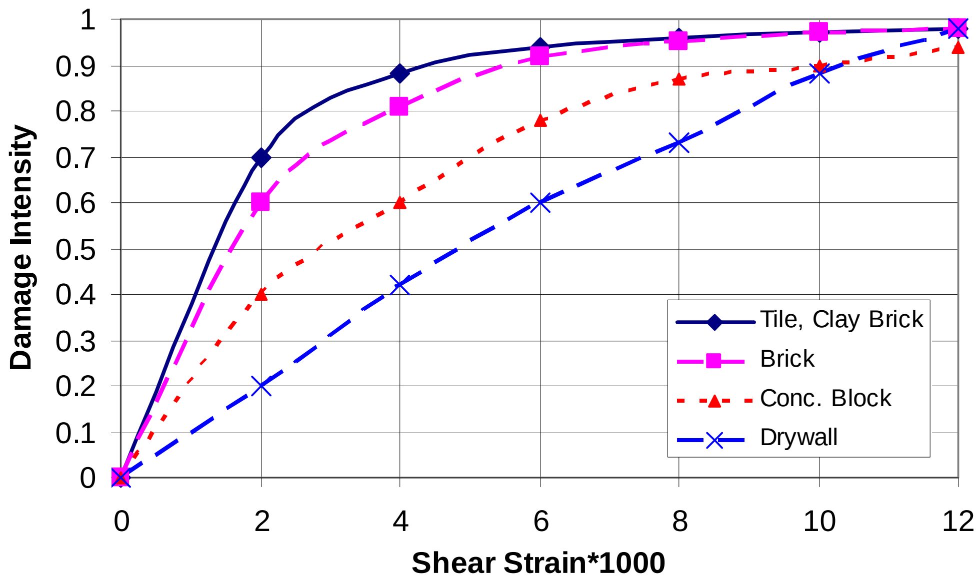

Figure 2.5: Shear Strain vs. Damage Intensity

Damage Intensity: 0.1 - 0.3…….minor damage

0.4 - 0.5…….moderate damage

0.6 - 0.7…....substantial damage

0.8 - 1.0…….major damage

A damage intensity value of 0.3 is considered the point at which damage is apparent and visible. At this point, minor repairs are probably necessary. Algan is also quick to point out that damage to many of the specimens was observed mainly in the corners and at the base where the specimen was anchored. Table 2.2 lists strain limits for several types of partition walls. The data is from Algan (1982) and represents the average values of multiple tests.

Table 2.2: Damage Limits for Several Types of Partition Walls

Wall Material	Strain Limit
Concrete Block	0.005
Brick	0.0025
Hollow-Clay Brick or Tile	0.002
Veneer (drywall, plywood, etc.)	0.007

The strain limit given in column 2 of Table 2.2 corresponds to substantial damage; that is, the point at which the material must be repaired or possibly replaced. A review of the literature reveals that additional research has been conducted regarding damage limits of pre-cast cladding and other materials. A recent paper (Carpenter 2004) has summarized tests on glass systems and pre-cast panel systems and also explains the typical testing for high-rise cladding systems. Current research by the Applied Technology Council into performance based seismic design has focused on nonstructural testing as part of the project scope (ATC 2006). This information should add to the database of knowledge concerning damage limits of common building materials.

Choosing a damage limit is based on material damage thresholds but is also based on building usage and the owner's needs. Consulting with the owner early on in the design stage to establish how often and how much damage is acceptable may be appropriate. Finally, the damage limit is also dependent on the wind recurrence intervals being designed for, design experience and engineering judgment.

2.2.3 Codification of Serviceability Limit States

Because serviceability limit states are not safety related and therefore not codified, there are wide variations in drift limits, as indicated in the 1988 ASCE Survey. The majority of the respondents agreed that drift should not be codified although the vast majority thought that more guidance should be provided. The ambivalence about codifying serviceability issues arises from the scarcity of valid data to define the serviceability limit states, the adverse economic consequences of using unjustifiable serviceability guidelines, and the tendency to view any codified standards as absolute (ASCE 1986). Regardless, more is needed than the short paragraph found in ASCE 7-05 addressing drift of walls and frames: "*Lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems due to wind effects shall not impair the serviceability of the structure*" (ASCE 7-05 App C). National building codes such as IBC do not offer any guidelines either. Numerical values for serviceability limits are not given by the European design code (EN 1993 1-1) although suggested serviceability lateral deflection limits are given in the designers guide series (Gardner and Nethercot 2005) and are shown in Table 2.3.

Table 2.3: Suggested Lateral Deflection Limits given in the Eurocode Designers Guide Series (2005)

Design Situation	Deflection Limit
Tops of columns in single story buildings, except portal frames	Height/300
Columns in portal frame buildings, not supporting crane runways	To suit cladding
In each story of a building with more than one story	Height of story/300

The National Building Code of Canada (NBCC 2005) stipulates that "the total drift per storey under specified wind and gravity loads shall not exceed 1/500 of the storey height" for serviceability considerations. Besides the story drift limit, little additional guidance is given.

Given the fact that wind drift is very often a controlling aspect in design, especially in areas of low seismic activity, it makes sense to offer some basic guidelines and requirements. This can and should be done without limiting engineering judgment and allowing for leeway based on building usage, owner needs, etc.

2.3 Modeling and Analysis for Drift Design

A 1983 study by Bouwkamp *et al.*, performed to access the influence of various structural and nonstructural modeling aspects on dynamic seismic response, serves to illustrate the effect of building stiffness with respect to lateral loads. Five buildings of primarily steel construction, ranging in height from 15 to 60 stories, whose in-situ properties were previously determined by low-amplitude dynamic testing, were modeled to compare analytical results with experimental results.

Between four and seven analytical models were performed for each structure, each model with increasing levels of refinement. The first model typically consisted of the lateral load resisting system modeled as planar frames for each direction (N-S and E-W) with lumped masses on each story. The second model contained variables such as the inclusion of fully rigid beam-column joint regions (not the beam-to-column connection, see Section 2.3.2 for details), three-dimensional modeling, participation of secondary framing systems, composite beam properties and distributed floor masses. Subsequent models involved adding the contribution of concrete block infill to the stiffness of the moment resisting frames, changing the floor weights to better reflect as-built conditions, including interior core frames (non-lateral load resisting), including the effects of foundation flexibility and increasing the effective slab width for composite girders. Lateral stiffness was judged by the modal periods for each model. With the exception of the foundation flexibility, the analytical model gained lateral stiffness with each change. The bare, lateral load resisting frames consistently produced modal periods that indicated they were too flexible as compared to the in-situ properties. Results show that several of the building models match quite well with experimental data, but one should be cautious about drawing conclusions. Modeling decisions, especially the decision to use fully rigid joints, are questionable and just because an analytical model matches well with data does not mean it is correct. The notion of comparing different analytical models to experimental data is an excellent idea but it must be done correctly. As a result of the many variables involved, there could be dozens of different models and therefore care should be taken in modeling.

The study by Bouwkamp *et al.* (1983) is a good example of all of the uncertainties involved in analytically modeling actual building behavior. With respect to the lateral stiffness of steel framed buildings under wind loads, the two important modeling issues that demand attention are sources of deformation and sources of lateral stiffness. These issues are closely related and it could be argued that they are in fact the same thing. For example, consider a girder in a rigid frame. It provides lateral stiffness to the frame but it also accounts for some of the lateral deformation that the frame experiences. To clarify the issue, any material deformations (axial, shear, flexure) will be considered as sources of deformation while all other aspects will be considered as sources of stiffness. An additional concern is second order or P-Delta effects. P-Delta effects will be discussed as a separate issue, apart from sources of deformation and sources of stiffness.

2.3.1 Sources of Deformation

To predict structural response under loading conditions it is necessary to accurately capture all sources of deformation occurring in the structure. Under wind loads this means including all sources of deformation that contribute to the total drift: flexural, axial and shear elastic deformations in the beams and columns as well as deformations in the beam-column joint (panel zone) region. Joint or panel zone deformations are shear and flexural deformations occurring within the region bounded by the column and beam flanges. Figure 2.6 shows a typical interior panel zone region.

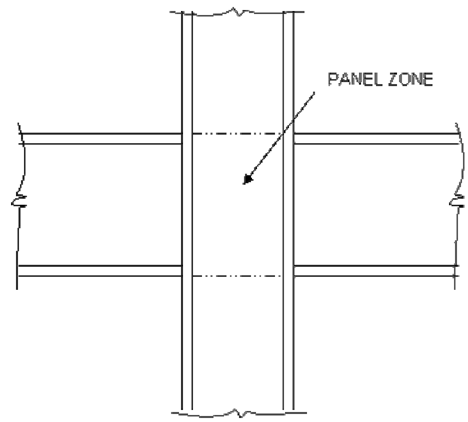

Figure 2.6: Typical Interior Sub-assemblage

Following the first wind drift survey (ASCE 1988), it was recommended by committee members that in addition to flexural deformations, "that lateral load calculations for steel buildings incorporate the following: beam and column shear deformations, column axial deformations, panel zone deformations, width of beam to column joints, and second order effects." Using sound engineering judgment to accurately and correctly include important sources of deformations is not an unreasonable expectation for engineers given the advanced state of the art and the powerful computing tools available today.

The 1988 survey (ASCE) found a wide variation among designers concerning the sources of deformation included in design. Half of the respondents indicated that they always include shear deformations, 88 percent include axial deformations of columns, 85 percent consider width of beam-column joints, 37 percent consider panel zone deformations and 72 percent perform second order drift analysis. Despite the seemingly simple percentages, it is difficult to reach definitive conclusions because many respondents offered comments and explanations that are not as simple as a yes/no answer. For example, consider the answers to the question of “Do you include panel zone deformations?”: 63 percent of the respondents said they do not explicitly include panel zone deformations while the remaining 37 percent consider panel zone deformations “when panel zone is over 10 percent of the center-to-center distance”, “in tube-type structures with close column spacing (5-10 ft)”, “when large panels exist”, where the contribution can be significant” or “when drift appears to be controlling”. Quantifying their responses in this way indicates a significant amount of engineering judgment among respondents based on a variety of factors such as building height, type of connections and others. In other words, the answer is not as simple as YES or NO and the survey questions were not posed properly to take this into account. The explanations are vague and without knowing further information, it is difficult to surmise why these answers have been given. This error will be avoided in the new survey.

An analytical study (Charney 1990b, see also Chapter 4) on the sources of elastic deformation under lateral loads demonstrates the potential pitfalls in modeling assumptions. In the study, lateral load analysis were carried out on more than 45 steel frames and tubes using the commercially available computer program SAP-90 (Wilson and Habibullah 1989) and an independently developed postprocessor DISPAR (Charney 1990a). The program DISPAR (acronym for DISplacement PARticipation factor) uses the principle of virtual work to quantify the contribution of each element (beam, column or joint region) to the total building drift and is also able to break down the contributing deformations into flexural, axial and shear. The analytical models ranged in height from 10 to 50 stories and differed in the number and the width of bays, from 5 10-foot bays to 13 20-foot bays. Results indicate the following:

- Shear deformations can be significant, contributing up to 26 percent (in a 40 story building with 10 ft bays) of the total lateral drift. They are especially important in tall (40-50 stories), slender tube structures where they can be as important as flexural deformations. The minimum contribution was 8.2 percent (in a 10 story building with 20 ft bays), still a significant amount.

- The contribution of axial deformations, which ranged from 26 to 59 percent of the total deformation, was shown to be very important in tall, slender frames with flexure being more of the controlling factor in the wider, shorter frames. It should be noted that the 1988 survey comments indicate that a considerable percent of respondents do not include axial deformations for buildings less than 10-stories. However, it is the height-to-width ratio of the wind bent, not the building height, which is the most important factor when considering axial deformations (ASCE 1988).
- Finally, beam to column joint (panel zone) deformations were considerable in all models. The contributions ranged from 15.6 to 41.3 percent.

(A study similar to the one discussed immediately above, but involving various joint models, is presented in Chapter 4 of this thesis. The reader is directed to Chapter 4 for further information on the various sources of deformation as well as effects of different joint models.)

From this study (and the one presented in Chapter 4) it is clear that all major sources of deformation should be included in the design and analysis of steel framed buildings. This is not unreasonable given the powerful computing tools available today. The two most commonly neglected sources of lateral deformations in steel buildings are shear deformations in the clear span region of beams and column and deformations within the beam-column joint region (ASCE 1988). Shear deformations will be discussed first.

When analyzing frame structures shear deformations are computed based on the effective shear area, typically taken as the gross area divided by a form factor, κ. A form factor of 1.2 is commonly used for rectangular sections. For wide flange shapes subjected to major axis shear, the shear area is commonly taken as the center-of-flange to center-of-flange depth of the member multiplied by the web thickness. The flanges are ignored due to negligible shear resistance. Although this approximation is accurate for deep, slender sections (e.g. W36x135) the flanges begin to contribute significantly for heavy, compact sections (e.g. W14x730). Detailed finite element analysis has confirmed this (Charney *et* *al.* 2005) and the following recommendations were made for calculating major axis shear deformations of wide flange sections:

- A form factor based on an effective shear area of $(d - t_f)t_w$ is reasonably accurate for all but the stockiest sections, such as heavy W14's. Note that for all wide flange sections SAP2000 (Computers and Structures, Inc. 2006) uses an effective shear area equal to the total depth times the web thickness.
- For stocky sections, the simplified Cowper expression (Cowper 1966) is recommended for calculating the section form factor and is given by Equation 2.5

$$\kappa = \frac{\left[\left(12 + 72m + 150m^2 + 90m^3\right) + 30n^2\left(m + m^2\right)\right]}{10(1 + 3m)^2} \tag{2.5}$$

where:

$$m = \frac{2b_f t_f}{dt_w} \qquad n = \frac{b_f}{d}$$

d = distance between center of flanges

For minor axis shear the wide-flange member behaves like two rectangular sections separated by a thin web and a form factor of close to 1.2 is adequate. With that being said, detailed finite element analysis has refined this idea and has shown that there is some shear resistance provided by the web. Consequently, the following empirical expression is recommended for the form factor (Iyer 2005):

$$\kappa = 1.2\frac{A_{gross}}{A_{flanges}} \tag{2.6}$$

Using the correct shear area is important in predicting shear deformations and consequently affects the computed lateral drift.

2.3.2 Modeling the Beam-Column Joint Region

As previously mentioned, deformations occurring in the joint region can be significant and should always be included in any drift calculations. Knowledge of the impact that beam-column joints behavior have on moment resisting steel frames is not new (Bertero *et al*. 1972, Becker 1975). However, as indicated in the 1988 ASCE Survey there is a general lack of understanding in the design field regarding modeling of the beam-column joint region. Joint deformations can be thought of as being composed of beam and column joint flexure and column joint shear. There are basically four different ways to model the panel zone region:

- clearspan or rigid model (no joint flexibility)
- partial rigid model (some joint flexibility)
- centerline model
- mechanical joint models: Krawinkler, Scissors

An in-depth study (Charney and Johnson 1986) comparing centerline, rigid and mechanical models has shown that the clearspan model, which ignores joint deformations, drastically underestimates building drift and is not recommended for design applications. As mentioned previously joint deformations contribute significantly to the total lateral drift and cannot rationally be neglected in design. The partially rigid model is common in design, possibly due to the fact that many popular design programs have built in capabilities for this model. Typically, through engineering judgment or recommendations in the literature, a rigid endzone factor, commonly called *Z*, is applied to the joint width. A *Z* value of zero is effectively a centerline model (no offset); a *Z* value of one is effectively a clearspan model while values in between provide some degree of rigidity, as Figure 2.7 shows. Figure 2.8 shows three of the four common analytical joint models.

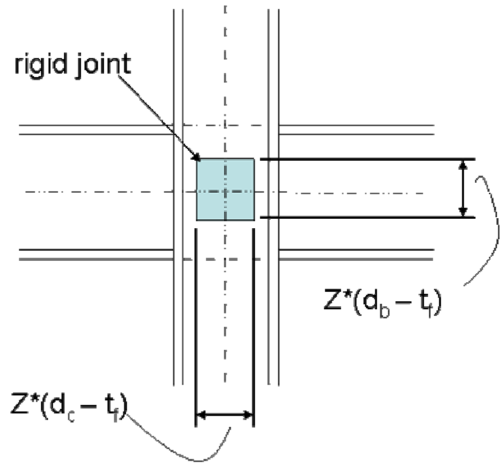

Figure 2.7: Joint Rigidity

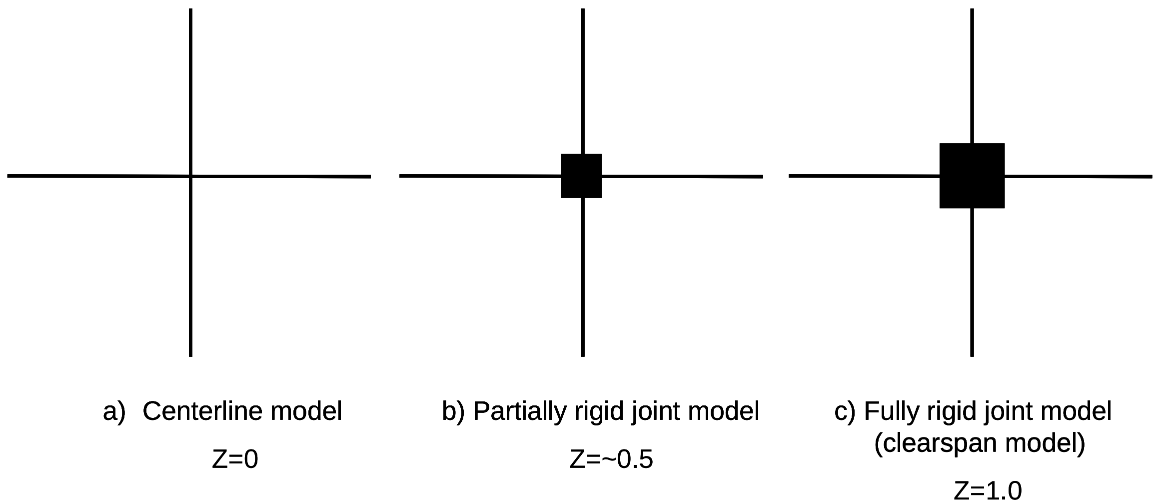

Figure 2.8: Use of the Rigid Endzone Factor

The area that is included in the joint region is considered rigid with respect to shear and flexure but not axial forces. A review of the literature reveals that a *Z* value of 0.5 is commonly recommended (Légera et al. 1991). However, this assumption is incorrect and underestimates true deformations in the panel zone.

The centerline model, shown in Figure 2.8(a), is the easiest way to represent the panel zone because frame members are idealized to meet at one, infinitely rigid point; the joint region is not explicitly modeled and there are no relative changes in the angle of rotation between the beam and column centerlines. The centerline model tends to overestimate flexural deformations and underestimate shear deformation within the panel zone. These sources of error tend to cancel each other out, resulting in reasonably accurate joint deformations. Further explanation on the reasons for these "self-correcting" errors is beyond the scope of this discussion. Interested readers are directed to the following references: Charney and Johnson 1986, Charney 1990b, Downs 2002. Two-thirds of the 1988 ASCE Survey respondents utilized the centerline model. This may be a result of the simplicity of the modeling technique and not the knowledge of exactly how the centerline model works. Caution should be applied when using the centerline model because joint deformations can be underestimated when shear strains occurring within the panel zone are very high and overestimated when flexural strains in the panel zone are large (Charney 1990b).

Mechanical joint models are another way to represent the behavior in the panel zone region. One of the first mechanical joint models was developed by Helmut Krawinkler (1978) and is referred to as the Krawinkler model. It is shown in Figure 2.9.

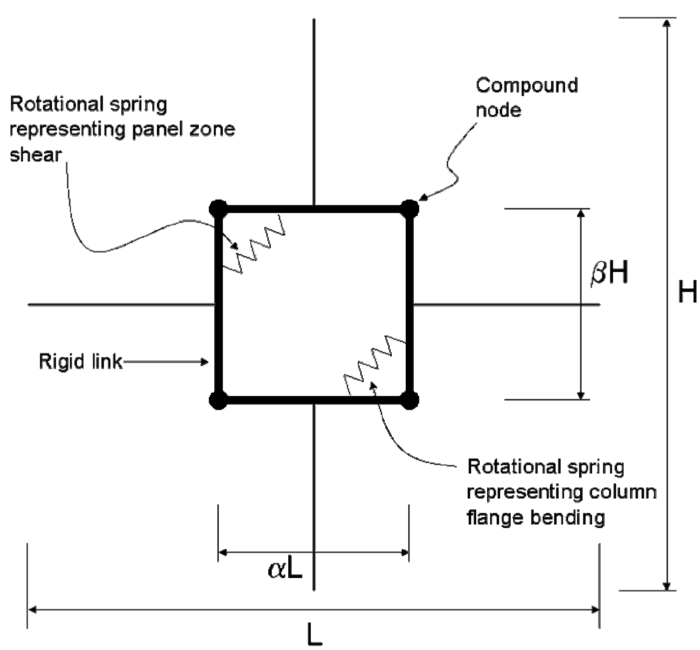

Figure 2.9: Krawinkler Model

The model treats the boundary of the panel zone as infinitely rigid links connected at the corners by compound nodes. Rotational springs are placed in two opposite corners to represent the resistance to panel zone shear and column flange shear. The so-called “Krawinkler model” physically represents panel zone distortion and is independent of the structural geometry outside the joint region. The one disadvantage is that a computationally taxing twenty-eight degrees of freedom are needed to explicitly model the panel zone using the Krawinkler representation. However, this can be decreased to four independent degrees of freedom in a planar structure, if the proper constraints are used (Downs 2002).

A second mechanical joint model, the “Scissors model”, (Krawinkler and Mohasseb 1987, Kim and Engelhardt 1995 and 2002, Schneider and Amidi 1998, Foutch and Yun 2002) is similar to the Krawinkler model in that it also uses rotational springs to represent panel strength and stiffness. The Scissors model has the advantage of only requiring four degrees of freedom per node (as opposed to twenty-eight for the Krawinkler model) but is hampered by its dependence on the geometry outside the joint region. Additionally, the Scissors model doesn’t represent true panel zone behavior as well as the Krawinkler model and is appropriate only for elastic structures or for systems where ductility demands are low (Charney and Marshall 2006). Figure 2.10 shows a schematic of the Scissors model. The panel spring, whose rotational stiffness is given by Equation 2.7, represents the shear resistance of the panel zone.

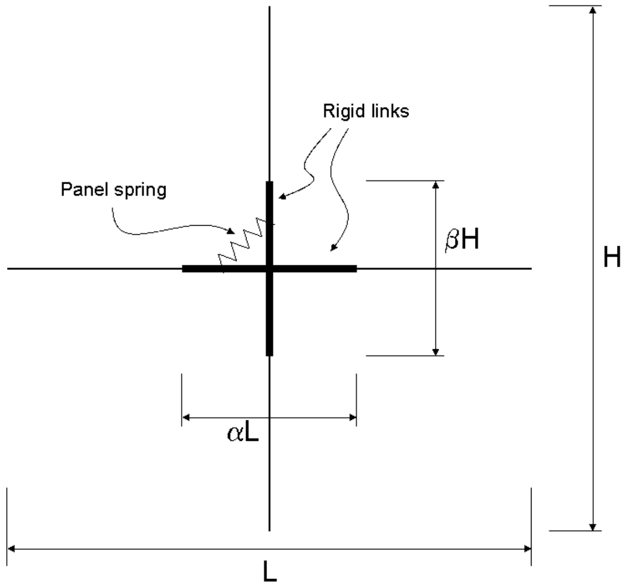

Figure 2.10: Scissors Model

$$K = \frac{\beta H \alpha L t_P G}{(1-\alpha-\beta)^2} \tag{2.7}$$

where

K = rotational stiffness of the panel spring (inch-kip/radian)

βH = center of flange to center of flange of beam (inches)

αL = center of flange to center of flange of column (inches)

H = mid-story to mid-story dimension (inches)

L = mid-span to mid-span dimension (inches)

t_p = thickness of the panel zone (inches) (t_p = t_w if no doubler plates)

G = shear modulus (ksi)

It should be noted that neither the Krawinkler nor the Scissors model includes the effects of axial or flexural deformations in the panel zone. The flexural behavior inside the panel zone is complex and difficult to model although recent work by Downs (2002) has provided some preliminary models that include the flexural component of joint deformation. These flexural and axial components of deformation are relatively small and the current model, which includes only shear deformations, gives accurate results.

2.3.3 Connection Flexibility

The lateral stiffness of a steel-framed building is composed of the lateral load resisting system (braced and unbraced frames), the non-lateral load resisting system (gravity frames, load bearing walls), nonstructural elements (walls, infill, cladding, etc.) and the interaction, intentional or otherwise, between these components. From the previously mentioned study by Bouwkamp *et al.* (1983) it is apparent that the bare frame alone does not provide all of the lateral stiffness in a structure. Modeling all contributing components is therefore essential to produce accurate and reliable results.

Modeling connections as either simple (pinned) or fully restrained (fixed) greatly simplifies the design and analysis process but is not entirely correct. In reality all pinned connections offer some degree of moment and rotational resistance and fall somewhere in between fixed and pinned. When it comes to performance based engineering there are many benefits of including PR (partially restrained) connections in design. For example, beams can be optimized for moments and economy can be realized through construction savings. However, the main focus here is on how the serviceability of frames is affected by connection flexibility.

Simple connections are assumed to rotate unrestrained and to transmit a negligible moment. Fully restrained connections transfer moments with a negligible amount of rotation. Partially restrained (commonly called semi-rigid) connections fall in between, allowing a rotation that is not negligible while transmitting some moment. Several national codes allow the consideration of partially restrained connections (AISC 2005, British Standards Institute, EN Eurocode 3 2003). The 2005 AISC Specification permits the use of PR connections “upon evidence that the connections to be used are capable of furnishing, as a minimum, a predictable percentage of full end restraint” (AISC 2005; [B3.6b]).

When utilizing the advantages of partially restrained connections, the rigidity of the connections affects frame stiffness and may shift the governing design from strength to serviceability (Gao and Halder 1995). Frames designed for strength may not be adequate when serviceability is considered. Although much research has been performed regarding the strength and reliability of frames with PR connections there is a lack of knowledge regarding the serviceability characteristics. Experimental analysis (Chui and Chan 1997) confirms that the type of connection employed in design affects lateral displacement. Chui and Chan measured the stiffness of two common beam to column connections (welded and bolted) in the laboratory, developed non-linear moment-rotation relationships and then input these into an analysis program. The modeled structure was a one bay, three story steel frame subjected to lateral and gravity loads and was modeled as both a braced and unbraced frame. The connections were modeled four different ways for a total of eight trials (four for the braced frame and four for the unbraced frame). The different connection types were analyzed to determine how lateral drift at the top floor as well as the vibration frequencies of the first two modes was affected by the following connections: Rigid (infinitely stiff), Pinned (no rotational resistance), Welded and Bolted (both with experimentally derived non-linear moment rotation curves). As one might expect the computed drifts and frequencies were insensitive to connection flexibility for the braced frame with the reverse holding true for the unbraced frame. These results were expected: connection flexibility affects lateral deflection and frame behavior. However, the question of how to implement the advantages of PR construction in design is still a topic of discussion.

Current AISC Specifications (2005) also provide a simplified alternative to PR connections: Flexible Moment Connections (FMC). Flexible moment connections are both simpler and more conservative than PR connections (Geschwindner and Disque 2005). For gravity loads, the connection is assumed to provide no rotational restraint and the connection is modeled as pinned. For lateral loads, the connection is assumed to provide full restraint and act as a FR moment connection. Various types of FMC and the design procedure are found in Part 11 of AISC (2005).

Two-thirds of the respondents of the 1988 survey (ASCE 1988) did not account for connection flexibility when modeling for drift. Accounting for the inherent rigidity in beam to column connections that are typically modeled as pinned can provide economic benefits in that mid-span beam moments can be reduced and beam sizes can be optimized. In relation to wind loading PR connections are becoming especially popular for low rise steel frames where fully rigid connections are usually not necessary for resisting lateral loads due to wind (Sakurai *et al.* 2001). In addition, PR connections in low rise braced frames can be used to replace bracing members. Using semi-rigid connections in design provides clear advantages; the problem is developing adequate and reliable models for use in design. With the art advancing toward performance based engineering, more research and work is necessary in this area.

2.3.4 Composite Action

Besides acting as a force collecting diaphragm, the concrete floor slab provides substantial benefits when designed to act compositely with the beams. Primarily used in the design of beams for gravity loads, composite action can provide significant lateral stiffness to rigid frames controlled by wind loads while also increasing overall building economy. Under lateral loads there are basically four moment regions in the girders of rigid frames that can be defined as shown in Figure 2.11 (Schaffhausen and Wegmuller 1977).

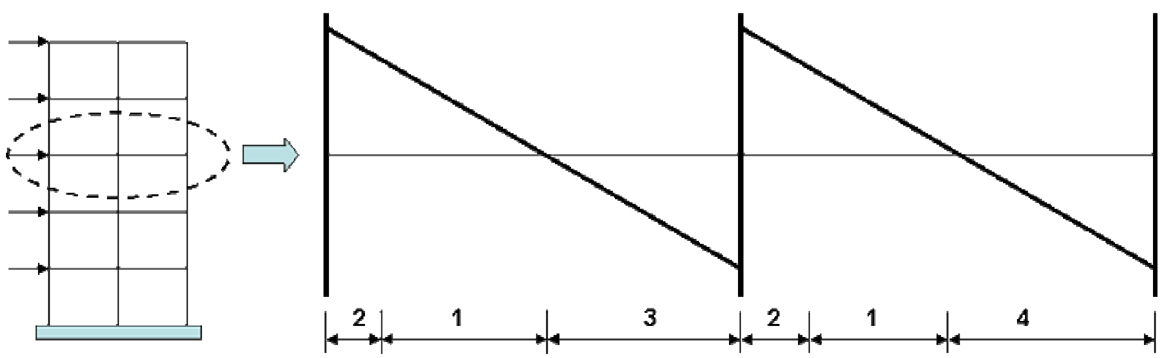

Figure 2.11: Girder Moment Regions

Region 1: Girder is subjected to positive bending moment (concrete slab is in compression) and the full effective slab width can be considered as providing flexural resistance.

Region 2: Adjacent to an exterior windward column or an interior column, the girder is subjected to a positive bending moment but a reduced slab width is used due to the flexural force distribution in the slab. Although the effective slab width must decrease to the width of the column at exterior columns, using the full effective slab width provides nearly identical results (Schaffhausen and Wegmuller 1977). Comparing a full effective width to a width that decreases to the width of the face of the column, results show member forces only differ by 1-2 percent (Vallenilla and Bjorhovde 1985).

Region 3: Girder is subjected to a negative bending moment; slab is generally ignored in calculations of composite flexural strength. Negative slab steel may be present in this region, which differentiates it from region 4. The girder alone provides flexural resistance.

Region 4: The girder is subjected to a negative bending moment and there is typically no negative slab steel at this location (exterior column). Girder alone provides flexural resistance.

The effective slab width, the concrete compressive strength and the number of shear connectors (percent composite) are the main components involved in calculating the composite flexural strength of the member. In regions 1 and 2 the composite moment of inertia can be used in the analytical model while in regions 3 and 4 the girder properties alone should be used, although the slab in tension may be included for low level loads. Under serviceability loads the slab may act fully composite even though it may not be designed that way. Additionally, the slab width used for strength design may be overly conservative for serviceability design. The AISC Design Guide on Vibrations (Murray *et al.* 2003) recommends a slab width of up to 40 percent of the beam span, as opposed to the 25 percent limit specified in the AISC Specification for Structural Steel Buildings (2005). It has been suggested (Leon 2001; Leon and Hajjar 2003) that revisions to the provisions for composite flexural members in the AISC Specification (1999, 2005) are necessary to allow for more risk consistent serviceability design. Once such revision suggested by Leon (2001) is revising the lower bound moment of inertia (LBMI) approach in the Specification. Currently the LBMI approach is based on stresses that are seen at strength design loads. A more accurate approach for serviceability is to use a

LBMI approach which is based on stresses seen at service loads. It is clear that more research into this area should be conducted.

2.3.4.1 Floor Diaphragms

In many cases, lateral loads on a structure are transmitted to the lateral load resisting system (shear walls, moment-resisting frames, etc) by using the floor deck as a collector diaphragm. Concrete filled metal decks make excellent diaphragms and are very common in design. The use of composite design, in which the concrete slab is made integral with the supporting beam through shear studs, is commonplace and provides several well-known advantages: reductions in the weight of steel, shallower steel beams, increased beam stiffness and increased span length for a given beam shape (Salmon and Johnson 1996). Considerable cost reductions due to the overall reduction of floor depth and cumulative savings in nonstructural materials and mechanical/electrical/plumbing systems provide another incentive for the use of composite construction.

How the diaphragm is modeled also affects the computed lateral drift of the building. The simplest way to model a concrete floor slab is as a rigid diaphragm, as multiple degrees of freedom are eliminated and the behavior is simplified. In the past this method of modeling was preferred, especially for tall buildings, as it is less taxing on the computer's resources. Presently a computer's hardware and software capabilities are usually not of concern and the diaphragm can be modeled with some degree of flexibility, although for low-rise buildings this degree of sophistication may not be necessary. For structures where the floor slab acts to transfer a large amount of load, accurate modeling of the floor slab becomes more essential. Consider Figure 2.12, a plan view of a floor diaphragm that is transferring horizontal wind forces to three vertical shear walls.

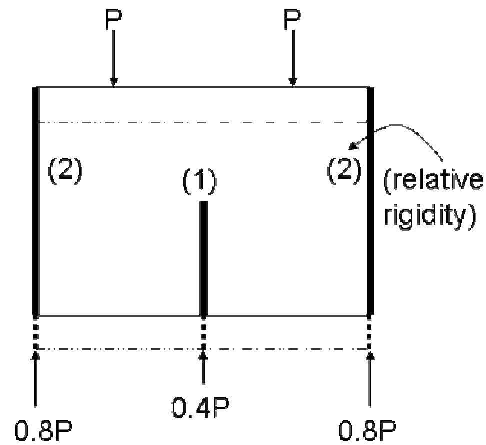

Figure 2.12: Relative Diaphragm Stiffness

If the diaphragm is modeled as fully rigid, then the amount of force picked up by each shear wall is directly proportional to its stiffness. The two outer walls are stiffer than the inner wall by a factor of two and therefore take twice the load. On the other hand, if the diaphragm is modeled with some amount of flexibility then the support (shear walls) reactions are directly related to the stiffness of the diaphragm. It can be important, especially in shear wall structures where the concrete shear walls are very rigid as compared to the floor slab, to model the concrete floor slabs as flexible diaphragms (Su *et al.* 2005). If the lateral system is moment resisting frames, which are relatively flexible as compared to the concrete floor slab, the rigid diaphragm assumption is very reasonable. Without accounting for the flexibility of the diaphragm the forces in the lateral load resisting system may be incorrect leading to incorrect lateral drift values and incorrect damage calculations.

This is another area where engineering judgment is required, as the decision on how to model floor slabs depends upon a number of parameters, the most important being the rigidity of the diaphragm as compared to the shear walls or frames that provide lateral resistance. However, including flexible diaphragms in design is simplified by powerful computers and software.

2.3.5 Nonstructural Components

In view of the complex nature of building behavior and to simplify the design, all of the lateral stiffness required to resist wind loading for a typical building design is attributed to the lateral load resisting system. Under strength (ultimate) loading conditions this simplification is justifiable. At this point the structure is assumed to have lost the contributions due to nonstructural components (NSC). Conversely, under serviceability loading conditions neglecting NSC can result in overly conservative designs. Commonly neglected NSC include frame infill (brick, masonry, etc.), interior and exterior walls (cladding), stairwells and elevator shafts. There are several reasons why including the contributions of NSC is important:

- Because damage limits (drift limits) are commonly controlling factors in design, particularly in tall (over 10 stories) or slender buildings, considerable economic gains are possible through accounting for all sources of lateral stiffness.
- Accounting for NSC can help match measured responses with predicted responses.
- Under earthquake loading the additional stiffness provided by NSC may "amplify the building response by shifting key structural frequencies into critical ground motion spectral ranges." (Pinelli *et al.* 1995)

An extensive study by Su *et al*. (2005) combined detailed FE modeling with field measured experimental data from three buildings to determine the stiffening contributions of NSC. The test buildings were all of concrete construction with different lateral load resisting systems: a 15 story moment resistant frame, a 14 story frame-wall and a 41 story shear-wall. The buildings were first modeled as bare frames with rigid diaphragms and then modifications were made, one at a time, to account for: internal and external nonstructural walls (including parapet walls, precast façade walls), secondary beams and flexible floor diaphragms. In all buildings studied the bare frame model significantly overestimated the experimentally found periods of vibration in both translational (x and y) and torsional modes, indicating large contributions from the commonly ignore NSC.

In contrast to the study above, a study by Brownjohn *et al.* (1998) of a 67 story building in Singapore found remarkable agreement between the predicted and measured dynamic properties of the building. The building, Republic Plaza, is a 920 foot tall structure composed of a reinforced concrete central core wall with perimeter steel columns and was completed in 1995. Modal properties were obtained for the building during construction (bare frame only) and for the completed building using accelerometers. Detailed finite element analysis was performed using SAP90 (Computers and Structures, Inc. 2006) to model the building as accurately as possible, including nonstructural components. The predicted translational periods (5.41, 5.18 sec) matched strongly with the measured response (5.44, 5.15 sec). These results demonstrate the influence of NSC and the ability to create a detailed and accurate analytical model. Also, the results indicated that the as-built building was stiffer than anticipated (due to nonstructural components) although is was concluded that the curtain wall had no noticeable effect on the structural behavior, possibly due to the nature of the connections.

Most of the work that has been done regarding NSC involves nonstructural walls and cladding material. Nonstructural walls, which can be broadly classified as either masonry or veneer supported by studs, will be discussed first.

2.3.5.1 Nonstructural Walls

Masonry walls are commonly constructed of concrete block, brick and hollow-clay brick or tile (Algan 1982). The most detailed work regarding NSC has dealt with brick and masonry infill, whose lateral stiffness contributions have long been recognized. Although buildings designed today are much different than those 60 years ago, experiments conducted as long ago as the late 1940's revealed that the presence of infilled frames in 14 story steel buildings increased the lateral stiffness by a factor of 10-20 (Polyakov *et al.* 1952). Analytically, the idea to replace the infill by equivalent diagonal compression struts was first proposed in 1961 (Holmes) and has been used by many researchers in subsequent work. Research, both analytical and experimental, and measured building behavior under design loads has consistently shown that NSC (especially infilled frames) contribute significantly to a buildings overall lateral stiffness.

So if the contribution has been consistently verified, why isn't masonry infill included in all building design? Mainly, the major problem with a design procedure in which infills are explicitly included in the analysis model is that masonry infills, being traditionally non-engineered, have as-built properties which, at the design stage are almost impossible to estimate reliably and/or to specify, and at the construction stage are hard to control (Fardis *et al.* 1999). Additionally the infills may be subject to alterations during the service life of the structure.

Although masonry infill has been studied in some detail, the results show large variations in the stiffness ratio of the bare frame to the infilled frame (Su *et al.* 2005). Several experiments summarized by Su show a range of lateral stiffness values for infilled concrete frames. Although the experiments involve concrete frames the results demonstrate the stiffness contributions of NSC. Keeping in mind the differences in test specimens (from one-bay one-story to multi-story and multi-bay) and the methods of analysis (FE, numerical analysis, monotonic loading, quasi-static cyclic loading, earthquake simulation) the ratio of the infilled frame stiffness to the bare frame range from 1.2 (Lu 2002) to 50 (Shing *et al*. 1994; Mehrabi *et al*. 1996). These experiments are limited in that most were performed on reduced scale structures, some are frames only and some are full buildings with or without infilled frames throughout.

A forensic structural investigation into steel buildings damaged in Iran by a large earthquake (Moghaddam 2004), demonstrated that the presence of infill helped to preserve the buildings structural integrity. Cracks were observed in the infill, especially in the corners and along the diagonal. Although the infills were not considered in the initial design, the observed seismic behavior led engineers to conclude that their stiffening effect was significant and that repairs should be made. Repairs were made to ensure the infill panels could be depended on in a future earthquake. This is a case of NSC being effective for structural strength but not being quantified in any design work.

The other subtopic of nonstructural walls is veneer walls. Veneer walls refer to drywall, plywood, plaster board or other similar materials supported by wood or metal studs. The majority of the experimental research done in this area has sought to define damage thresholds and focuses on the behavior of individual wall panels. Algan (1982) summarizes the results of nearly all of these efforts prior to 1982, with a great deal of work being done by Freeman (1966; 1968; 1971; 1974; 1977). More recently and moving in the direction of modeling applications, Adham *et al.* (1990) experimentally derived the load deformation response characteristics of six light gage steel stud partition walls for implementation in structural analysis programs.

Concerning the effect of partition walls on overall building behavior, experimental testing by Yanev and McNiven (1985) compared two types of masonry walls and several types of stud supported veneer partitions. It was found that the stiffer masonry walls provided more lateral stiffness but were destroyed under strong ground motion. The more flexible veneer partitions suffered less damage but did not provide as much lateral stiffness. Smith and Vance (1996) developed analytical models of several types of veneer walls to be incorporated in analysis programs. The work involved a FE element model, load deformation characteristics and experimental testing to verify the model behavior. Strong correlation was established between the analytical and experimental models: the ratio of energy dissipated by the analytical model versus the experimental results ranged from 0.9 to 1.1.

2.3.5.2 Cladding

Although designed only to carry their own weight and resist out of plane wind loads, the connections between the frame and the cladding transfer in-plane lateral forces to the cladding panel. As a result, the cladding provides some degree of lateral stiffening, depending on the nature of the connection, to the structure. Generally, the connections are detailed so that the cladding is not damaged by the movement of the structural frame although Pinelli *et al.* (1995) have suggested employing this interaction to provide damping to the structure as well as stiffness.

A paper by Goodno *et al*. (1984) details their efforts to account for the stiffening effects of precast concrete cladding. Full scale tests were performed to determine the modal frequencies of a 25 story building. Then arbitrary cladding stiffness values were added to the bare frame analytical model until mode periods matched the in situ results. The cladding was found to affect the modal responses (especially the torsional response) by decreasing the period by anywhere from 11 to 95 percents. The investigation did consider composite effects but did not account for any other nonstructural components that may have contributed to the buildings lateral stiffness. A similar study by Mahendran and Moor (1995) on a full scale portal frame also verified the stiffening effects of cladding and additionally illustrated the need for better three dimensional modeling for serviceability considerations.

Due to the degree of uncertainty involved, a simplified procedure would be ideal for designing for drift, a serviceability issue that does not demand the rigorous modeling necessary for seismic strength design issues. There are three ways to deal with the presence of NSC in a building:

- Account for explicitly and use damageability based drift limits
- Reduce the computed drift by a factor to account for the additional stiffness
- Increase the allowable drift by a factor to account for the additional stiffness

Explicitly accounting for the nonstructural components in a building is difficult but it is the most desirable of the three options listed. Reliable analytical models must be developed to implement in modeling software.

2.3.6 Foundation Stiffness Flexibility

Another factor to consider when modeling a structure is the foundation flexibility. A flexible foundation will reduce the overall lateral stiffness of the structure, increase horizontal deflections and hence increase the second order effects. The decrease in lateral stiffness will also increase the natural periods of the system. These factors are important for reliability analysis under strong winds, especially in locations of low seismic activities where wind is the controlling factor, and in determining dynamic characteristics of the system to control motion induced discomfort. The choice of whether or not to include the effects of foundation flexibility is dependent on the soil type along with the substructure and is consequently an area where engineering judgment should be exercised. For example, a mat foundation may simply rotate rigidly with the structure, not causing any damage but certainly increasing second order effects. On the other hand, a building with columns supported by piles may experience differential settlement which could result in damage to building components.

2.3.7 Second Order (P-Delta) Effects

Under lateral loads, steel structures, particularly moment resisting frames, may experience significant displacements. The action of gravity loading on such laterally deformed structures may lead to significant amplification of lateral displacements and internal forces. These second order effects of members (P-δ) and the frame (P- Δ) should be limited to prevent excessive deflections and to ensure structural stability. The AISC Design Specifications for Structural Steel Buildings (2005) allows 2 methods of analysis: General second order analysis (computer methods) and second order analysis by amplified first order analysis (amplification factors). Amplified first order analysis (the B_1/B_2 approach) is straightforward for relatively simple planar frames but has several clear disadvantages:

- It can be cumbersome to apply to asymmetric structures.
- Many of the P-Δ techniques are dependent upon the idealization of the floors as rigid diaphragms.

Given the computational resources available today, and because for practical purposes multi-story frame design/analysis is done using computer methods which generally incorporate matrix formulation using stiffness and/or flexibility coefficients (Salmon and Johnson 1996), this discussion will be limited to matrix based evaluation of second order effects.

Typically when designing a building, a first order elastic analysis is performed first and then it is decided whether to consider second order effects. This decision is based on a calculated stability ratio satisfying a certain criteria which is dictated by code bodies. If the criterion is not satisfied or if the engineer desires, a second-order elastic analysis is performed next.

Two methods are available for the matrix formulation of second-order elastic frame elements: the stability function approach and the geometric stiffness approach. Both approaches can account for member (P-δ) and overall (P-Δ) second order effects. As described by White and Hajjar (1991) the stability function approach is based directly on the governing differential equation of an initially straight, elastic beam-column while the geometric stiffness approach is commonly based on an assumed cubic polynomial variation of the transverse displacements along the element length (consistent geometric stiffness). The stability function approach, while being more accurate, is not conducive to computer programming. It requires twelve separate beam-column stability functions to describe one frame elements. The geometric stiffness matrix is dependent only on the members force and length, is conducive to programming and is easily extended to general three-dimensional analysis. It can be implemented on a member-by-member basis (requires iteration, member axial forces change) or a floor-by-floor basis (no iteration, total floor axial force doesn't change). Because the geometric stiffness approach involves some approximation it can produce errors when the member's axial force is very large compared to Euler's elastic buckling load, specifically when $P>0.4P_e$ (member axial force is greater than 40 percent of Euler's buckling load). In this instance the member must be broken up into several (typically no more than three) pieces (White and Hajjar 1991). It should be noted that in an unbraced frame the axial forces are rarely large enough to make this step necessary and, if so, the errors produced are small; roughly 5 percent.

In SAP2000 (Computers and Structures, Inc. 2006) the P-Δ effect is implemented on an element by element basis using a three-dimensional, consistent (deflected shape is assumed to be a cubic polynomial function) geometric stiffness matrix for each frame element. Other software programs may form the stiffness on a floor by floor basis (such as ETABS) which requires no iteration as the total axial load at each level is constant. This approach commonly employs the linearized geometric stiffness (approximating the deformed shape by one or more straight lines as opposed to a cubic function). Whichever method is employed directly affects the accuracy of the solution. For example, the consistent geometric stiffness method may be more accurate, especially for structures with large P-Delta effects, but this accuracy comes at the expense of increased solution time. The engineer should be aware of the limitations and relative advantages of the different approaches and make decisions accordingly.

2.3.8 Structural Optimization

For given building drift constraints, the economic sizing of members for the lateral load resisting system is a task that has historically been highly iterative, time intensive and based on trial and error. The engineer uses his/her intuition and past experience to select appropriate member sizes, the analysis is run, lateral drift limits are evaluated and necessary changes are made. The process continues until the drift limit is satisfied in an economic manner. Of course the taller the building becomes, the more difficult the member sizing process.

A technique for solving these issues is based on the principle of virtual work, an idea first described by Velivasakis and DeScenza (1983). (The principle of virtual work is described in further detail in Chapter 4 of this thesis). Through this process, the engineer is able to easily and quickly identify members which are too flexible (or too stiff) and economically resize the members. The method is still iterative, as member forces are dependent on the section properties, but the overall design process is greatly improved. Research in this area includes work and real world application examples by Baker (1990, 1991) Charney (1993), Grierson (1984, 1989), Park (1997, 2002, 2003) and others. However, if member sizes are chosen simply to satisfy certain drift constraints, one would end up with a very uneconomical distribution of members. Economy is also dependent on member distribution. Chan *et al.* (1995) have presented a virtual work based resizing technique for the "least weight design of steel building frameworks of fixed topology subject to multiple interstory drift, member strength, and sizing constraints."

Additional research in this area should focus on constraints such as damageability in the nonstructural walls. Using shear strain in drift damageable zones (DDZ) as a member sizing constraint instead of interstory drift will result in even more optimal member selection for the lateral load resisting system. Nonstructural damage due to torsion as well as lateral movement is more rationally measured by the shear strain. The appropriate constraint to use in any structural optimization algorithm is therefore the shear strain in the nonstructural components and not the interstory drift.

2.4 Wind Loads

Because serviceability limits are not codified the wind loads used for drift and perception of motion calculations are inconsistent from firm to firm, leading to variations in structural performance and economy. Economy plays an important role in building design and that is why properly designing for serviceability issues is a crucial part of the design process. To take full advantage of properly modeling the sources of lateral stiffness and accurately capturing all sources of deformation it is imperative to use appropriate wind loads.

There are several methods, each with relative advantages and disadvantages, currently available to determine wind loads on a structure:

- appropriate codes and specifications
- boundary layer wind tunnel testing
- database assisted design (DAD)
- *computational aerodynamics*

The use of computational fluid dynamics to calculate wind loads on structures is still in the research phase and is therefore listed in italics. The first two methods are generally accepted and widely used while the use of DAD, permitted by ASCE 7-05 and the subject of much research, is still being developed for widespread use. Each one of these methods will now be discussed separately, with respect to the following factors (Charney 1990) which affect design wind loads:

1. the mean recurrence interval (MRI)
2. the wind velocity, which is a function of the recurrence interval and the geographic location
3. topography and roughness of the surrounding terrain
4. variation in wind speed with the wind direction (directionality factors)

5. the buildings dynamic characteristics
6. the buildings shape
7. shielding effects from adjacent buildings

2.4.1 Factors Affecting Design Wind Loads

2.4.1.1 Mean Recurrence Interval

Generally speaking it is economically prohibitive to design a building with such a degree of lateral stiffness as to confidently ensure that no nonstructural components will be damaged during the buildings service life. So the question becomes what amount of damage can be tolerated before repair and how often can the damage reoccur? The answers to these questions depend on the building usage and the building owner.

Under strength or ultimate loading conditions a 50 year wind mean return interval is typically used for most building designs. For serviceability criterion, it is commonly accepted today that an appropriate return interval is 10 years, a value that is now recommended in the National Building Code of Canada (NBCC 2005) and was previously suggested by others (Galambos and Ellingwood 1986, Charney 1990, Griffis 1993). This is based on the average occupancy time for a building tenant and the fact that it is not reasonable to base a serviceability criterion on a 50 year building life expectancy (Galambos and Ellingwood 1986).

However, the choice of an MRI is heavily dependent on the occurrence of damage that is acceptable to the owner and may vary drastically depending on the building/owner needs. A life-cycle analysis, comparing cost of repair vs. cost of avoidance, can provide valuable information regarding an appropriate choice. The owner should provide input into how much damage is acceptable and how often. Once a MRI is chosen a reference source must be consulted to determine the wind velocity associated with the chosen MRI. The commonly used ASCE 7-05 Specifications provides a wind map of 50 year velocities, based on 3 second gusts. A table (Table C6-7 in Chapter 6 of ASCE 7-05) of conversion factors is provided to convert from a 50 year MRI velocity to a 500, 200, 100, 25, 10 or 5 year velocities. These factors are based on data fitting of a Type I Extreme Value

Distribution to measured wind speed data. A formula for calculating the conversion factor for any interval period for non-hurricane regions (as defined in ASCE 7-05) has been given by Peterka and Shahid (1998):

$$f_R = 0.36 + 0.10 \ln(12 * R) \quad (2.8)$$

where

f_R = conversion factor from MRI = 50 to MRI = R
R = mean recurrence interval period of interest in years (e.g., 10 years)

Research done by Thom (1954, 1960 and 1968) resulted in published papers that provided wind velocity maps of 2, 50 and 100 year MRI's along with an easy method of determining wind velocities for any other MRI. Comparing Thom (1960) with ASCE 7-05 reveals extremely similar factors (1.08 from Thom vs. 1.07 from ASCE 7-05) for converting a 50 year wind velocity to a 100 year wind velocity indicating similar probabilistic assumptions and data fitting techniques. Mehta (1983) presents wind speed conversion factors for 5, 10, 20 and 25 year MRI's. A statistical analysis by Rosowsky (1995) on raw wind speed data that was used in the development of ASCE 7 determined conversion factors for reduced reference periods on the basis of maintaining comparable load exceedence probabilities. Rosowsky provides a table (Table 2.4) for converting from the base 50 year 3 second wind gust velocity to 0.5, 1, 2, 5, 25 or 100 year mean wind speeds and a logarithmic scale plot (Figure 2.13) for determining other reference periods. This data is useful as the shortest MRI that ASCE provides is 5 years, although Equation 2.8 (which is not given in ASCE 7-05) can be used to calculate other values.

Table 2.4: Factors for Reduced MRI's

Factors for Reduced MRI's, non-hurricane regions		
MRI	**ASCE 7-05**	**Rosowsky**
0.5	-	0.62
1	-	0.67
2	-	0.72
5	0.78	0.80
10	0.84	-
25	0.93	0.94
50	1.00	1.00
100	1.07	1.07
200	1.14	-
500	1.23	-

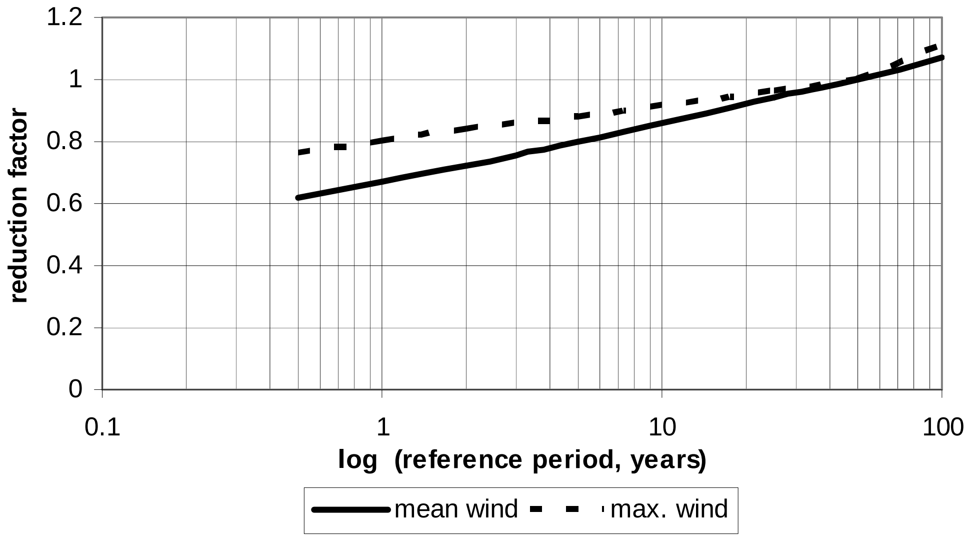

Figure 2.13: Factors for Reduced MRI's [2]

Note that the conversion factors given by Rosowsky are for non-hurricane regions only. Hurricane regions were not considered in the study which brings up the issue of site dependence. Conversion factors are dependent on the recurrence interval of the wind speed in question and are therefore site specific. For example, the calculated modification factors for wind speeds measured in central Kansas may be very different than those calculated for wind speeds measured in northern California which are very different than those based on wind speeds in southern Florida. However, to provide

[2] Reprinted from Structural Safety, Vol. 17, Rosowsky, D.V., Estimation of Design Loads for Reduced Reference Periods, pp. 17-32, Copyright (1995), with permission from Elsevier

simplified design wind speed values, the continental United States is broken down into two wind regions: hurricane and non-hurricane regions.

Providing a method to find wind velocities associated with any MRI is an important and necessary reference for any design engineer, especially with the rising popularity of limit state design and performance based engineering. As an example, if a structure is going to be obsolete or demolished after 3 years then it makes sense to design for a reduced MRI. Another example is the calculation of construction loads. If the probability of exceeding a limit in a certain period of time is determined, from the needs of the owner or based on the building usage, the design MRI can be determined from the following equation:

$$R = \frac{1}{1-(1-P_n)^{1/n}} \tag{2.9}$$

where

R = design MRI

P_n = probability of failure during n years (0 to 1.0)

n = period in question, years

Similarly, ASCE 7-05 gives the following information in the Chapter 6 Commentary (ASCE 2005):

> The probability P_n that the wind speed associated with a certain annual probability P_a will be equaled or exceeded at least once during an exposure period of n years is given by
>
> $$P_n = 1-(1-P_a)^n \tag{2.10}$$
>
> and values of P_n for various values of P_a and n are listed in Table C6-6. As an example, if a design wind speed is based upon P_a = 0.02 (50 year mean recurrence interval), there exists a probability of 0.40 that this speed will be equaled or exceeded during a 25-year period, and a 0.64 probability of being equaled or exceeded in a 50-year period.

While the above equation may be useful, the author feels that the MRI should be directly related to the acceptable damage threshold – i.e. the serviceability wind velocity should be a function of acceptable damage (depending on use), cost of repair (material) and recurrence interval.

2.4.1.2 Wind Velocity

There are several methods of measuring average wind speed including the fastest mile (the time it takes for one mile of air to pass), the mean hourly (average wind speed over one hour), the 3 second gust (average wind speed over a 3 second period) and others. Prior to ASCE 7-95 the wind velocities were based on the fastest mile wind speed, a measurement that the National Weather Service discontinued in favor of the 3 second peak gusts. To convert from the ASCE 7-93 wind map, which provided fastest-mile speeds, to the new peak 3 second gusts map, a study was undertaken in which a conversion factor of 1.2 was deemed reasonable (CPWE, 1994, p. 7). The study which produced this conversion factor has been called into question by Simiu *et al.* (2003) who points out several reasons why the new 3 second gust speeds can cause overestimation or underestimation of the wind load, depending on the location. It is pointed out that the study was not widespread enough to produce reliable data, especially for hurricane prone areas.

In the United States wind velocities (3 second peak gusts) are gathered through National Weather Service anemometer readings, typically at a distance 10 m (33 feet) above the ground. These readings, whose accuracy is dependent on several factors such as anemometer type, measuring/gathering techniques, are then analyzed and an appropriate probabilistic distribution is assumed and used to fit the data. The result of this work, for the designing structural engineer, is simplified wind contour maps such as the ones in ASCE 7-05. A great deal of debate and research has been conducted regarding which methods of extreme wind distribution are the most accurate, the scope of which is beyond this discussion.

2.4.1.3 Topography and Roughness of the Surrounding Terrain

The influence of terrain topography is site dependent and requires engineering judgment. Most analytical and simplified techniques employ the use of a topographic exposure factor which is applied to the wind pressure to account for the effects of surrounding terrain. Wind tunnels, through scale modeling of the surroundings, are able to better account for these effects and in turn produce more accurate results.

2.4.1.4 Wind Directionality

Wind loads are calculated based on the assumption that the wind is blowing at a right angle to the building face, regardless of the site specific wind characteristics. This conservative approach has led to the development of the wind directionality factor (Davenport 1977, Ellingwood *et al*. 1980). This factor accounts for two effects; (1) The reduced probability of maximum winds coming from any given direction (2) the reduced probability of the maximum pressure coefficient occurring for any given wind direction (ASCE 7-05).

It is important to note when the wind directionality factor is applicable in calculating wind loads and the following discussion pertains to ASCE 7. There has always been a wind directionality factor (designated as K_d) but prior to ASCE 7-98 it was included in the load factor of 1.3 that is applied to wind in the strength loading combination. Currently the wind directionality factor, which can only be used in the strength loading combinations, has been separated from the load factor of 1.3 which is why the load factor is now 1.6. For the great majority of buildings the wind directionality factor is 0.85 and 0.85*1.6=1.36, which is nearly the same load factor as before.

Several researchers (Rigato *et al.* 2001, Heckert and Simiu 1998) have pointed out that the wind directionality factor is dependent on the mean return interval. The study by Rigato *et al.* makes use of database assisted design (refer to Section 2.3.5 of this thesis) to investigate wind directionality effects. It is shown that for a 50 year MRI wind load, the wind directionality factor is approximately 0.86 (very close to the code prescribe 0.85) while for a 500 year MRI the factor is approximately 0.95. The writers "ascribe the increase of the wind directionality reduction factor with mean recurrence interval to the greater chance that a directionally unfavorable intense wind would affect the structure in, say, 2,000 years than in, say, 25 years." By this same line of reasoning, would the factor decrease for a short, say 10 year, MRI? The authors do not investigate this question.

If the wind directionality factor is meant to account for the reduced probability of maximum winds coming from the most unfavorable direction for building response, the factor should also be applicable to serviceability wind loads, and possibly with a greater reduction than 0.85. The current use of the wind directionality factor is confusing and should be revised to account for its dependence on the winds MRI.

2.4.1.5 The Buildings Dynamic Characteristics

The rigidity of a building in the along-wind direction affects the loads that it experiences. A very rigid building will not move much in the wind and the effect of wind gusts magnifying the building motion is negligible, leading to a simplified analytical expression for wind pressures. For flexible structures the load magnification effect caused by gusts in resonance with along-wind vibrations is more apparent and needs to be taken into account when calculating wind pressures. Again, analytical techniques tend to be conservative and wind tunnel testing, depending on the model used, can provide more accurate results.

2.4.1.6 Building Shape

The physical shape of a building greatly affects the structural-wind interaction, especially the torsional component of response. Specifications tend to be very conservative regarding the influence of building shape and for irregular, tall or slender buildings wind tunnel testing is highly recommended. For low-rise and commonly constructed buildings the most significant effect of the building shape is points of high cladding pressures and possible channeling effects on pedestrians.

2.4.1.7 Shielding (Interference) Effects

In a heavily built-up urban environment the wind loads a building experience are heavily dependent on the surrounding buildings. These surrounding buildings may either shield the building completely or channel wind directly onto the building. The influence can be substantial, as demonstrated in a lawsuit filed in the 1970's by the owners of several buildings in the vicinity of the World Trade Center Towers in New York who claimed their buildings were experiencing "unusual, increased and unnatural wind pressures" (Kwok 1989) due to the newly constructed Towers.

Wind tunnel testing is a valuable resource for considering the effects of surrounding structures and the differences between code loads and wind tunnel loads are most heavily dependent on the shielding or interference effects. However, wind tunnel loads below 80 percent of the code loads may not be used for strength design, unless it is shown that it's the shape of the structure, not the shielding effect, which is giving the results (ASCE 7-05). For drift considerations, there are no guidelines. It may be reasonable to use the lower wind tunnel loads for drift calculations if the shielding can be justified and expected to stay in place. Regardless, considerable engineering judgment is required with respect to the issue of shielding.

2.4.2 Code Determined Wind Loads

For the great majority of buildings designed the code defined wind loads are adequate. Advanced testing in a wind tunnel is not viable for several reasons including the cost, time, required resources and the fact that in the preliminary design stage a number of building shapes may be considered. Building codes are generally able to account for items 1 through 5 below and using the code method is a relatively straightforward and familiar process for most designers.

1. the mean recurrence interval (MRI)
2. the wind velocity, which is a function of the recurrence interval and the geographic location
3. topography and roughness of the surrounding terrain
4. variation in wind speed with the wind direction (directionality factors)
5. the buildings dynamic characteristics
6. *the buildings shape*
7. *shielding effects from adjacent buildings*

(Building codes are not able to account for items 6 and 7 of the aforementioned list, which is why they are listed in italics).

The first step in using the code approach is to determine the wind velocity or pressure associated with the selected MRI. Velocities, depending on the code being used, can be based on 3 second gust wind speeds, 10 minute mean wind speeds, mean hourly wind speeds or fastest mile wind speeds.

Most international codes and standards make use of the GLF, or gust loading factor, (Davenport 1967) to determine the equivalent static along-wind loading on a structure. It should be noted that no matter what averaging technique is used the wind pressure at a given location for a given return interval is the same. In other words the calculated wind pressure on a building is independent of the averaging technique used to determine the wind speed. Although the traditional GLF method ensures an accurate estimation of the displacement response, it may fall short in providing a reliable estimate of dynamic response components which is why a more consistent procedure for determining design loads on tall structures has been proposed (Zhou and Kareem 2001). Interested readers are directed to the reference.

Aside from how code specified wind velocities are derived, there are other factors that affect the calculated wind load. All major codes deal with these factors in similar ways: through the use of modification factors which are applied to the wind velocity. Focusing on the analytical approach defined in ASCE 7-05 reveals the presence of a gust effect factor, a wind directionality factor, an importance factor and a topographic factor.

2.4.3 Code Comparisons

Several studies have been carried out to compare how codes differ in treating wind loads on structures. A comparative study (Zhou *et al.* 2002) among the major codes employing gust loading factors, ASCE 7-98 (United States), AS1170.2-89 (Australia), NBC-1995 (Canada), RLB-AIJ-1993 (Japan), and Eurocode-1993 (Europe) shows considerable differences in predicted along-wind loads. These differences are attributed to variations in the definition of wind field characteristics in the respective codes and standards. To examine the validity of the different codes a study (Kijewski and Kareem, 1998) was carried out to compare different code derived along-wind acceleration response with wind tunnel derived response. Comparisons were made between peak and RMS along-wind accelerations, found from codes and from a wind tunnel test. ASCE 7-95 and AIJ (the Japanese code) data matched well with the wind tunnel results. The above mentioned study also compared how different codes deal with across-wind and torsional effects. The national codes of Australia, Canada and Japan all contain procedures for determining across wind accelerations while the Japanese code is the only one to address torsional-induced lateral accelerations. For the across-wind RMS (root mean square) acceleration the Australian Standard was the closest to the wind tunnel results and for the torsional-induced acceleration the Japanese Recommendations proved quite accurate. These results should not come as a surprise as the empirical expressions are based on wind tunnel experiments. If anything the study confirms the effectiveness of using wind tunnel experiments to derive empirical formulas, at least for regularly shaped buildings.

It should be noted that it is difficult to validate the actual wind loads a structure experiences. It is possible to obtain actual building cladding pressures due to wind but the most common way of validation is by comparing measured response (from a wind tunnel, as in the above study, or in situ testing) with predicted response. However, because empirical formulas are developed from wind tunnel tests the only real method of validation is through field measurements. If the responses are close then the loads must be correct; but comparing responses is much different than comparing the actual loads the building feels. The response is dependent, in different amounts, on the mass, damping and stiffness of the structure while the wind load felt by the structure is relatively independent of the structure's dynamic properties. These limitations must be considered when one makes comparisons.

2.4.4 Wind Tunnel Testing

The turbulent nature of the wind near the surface of the earth, due to friction between the air and the earth's surface, requires special attention in wind tunnels. For that reason the wind tunnels used in structural applications differ tremendously from those used in aeronautical applications. They must take into account the effects of this interaction in the boundary layer, leading to the name "boundary layer wind tunnel" or BLWT. These tunnels, by having a roughened floor and turbulence generators, are able to correctly simulate the wind profile and turbulence. The boundary layer wind tunnel has facilitated the design of many structures and the development of a generation of codes and specifications. It is a powerful tool for both researchers and professional engineers in the design field, and truly is the State of the Art in tall building design. Additionally, the price of testing has decreased over the years, meaning the wind tunnel isn't just for super-tall buildings anymore. Wind tunnel testing can provide more realistic loads than traditionally conservative code loads and the costs they save through this can make tunnel testing a viable option for 10 (in hurricane prone regions) and 22 (in non-hurricane prone regions) story buildings and up (Gamble 2003). Figure 2.14 demonstrates the disparity in wind loads for a regular shaped, medium height building. (Figure 2.14 is shown for illustrative purposes and is only one example of a wind tunnel test).

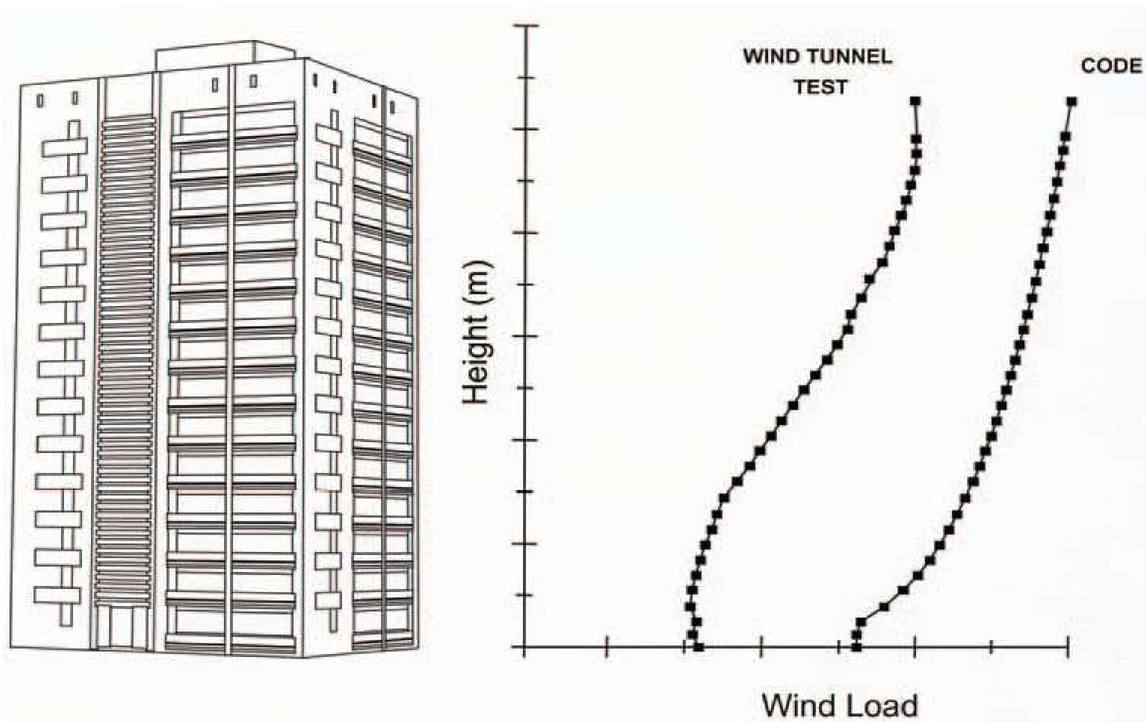

Figure 2.14 Wind Tunnel vs. Code Derived Wind Loads [3] (Gamble)

There are three basic types of wind tunnel models: (1) rigid pressure model; (2) rigid high frequency force balance model; (3) aeroelastic model. One or more of these models may be used in building design depending on the needs of the engineer. ASCE publishes a

[3] Reprinted with permission by STRUCTURE® magazine • November 2003

manual titled *Wind Tunnel Model Studies of Buildings and Structures* (ASCE 1998) that provides useful information on wind tunnel testing.

2.4.4.1 Rigid Pressure Model

The rigid pressure model is the simplest wind tunnel model to build as it just needs to match the physical features, typically at a scale of 1:400 or 1:500, of the prototype building. There is no need to model the mass, stiffness or damping of the building. This model is commonly constructed of Plexiglas with hundreds of pressure taps to measure the wind induced pressure, which can then be converted, through probabilistic methods and data fitting, to find peak loads on the cladding. As it is necessary to simulate the near field characteristics, surrounding buildings are generally modeled using polystyrene foam.

Although the rigid pressure model is primarily used to obtain local pressure distributions to aid in glass/cladding design, for dynamically insensitive buildings the data can be used to obtain overall structural loads. Given that it is rigid, the dynamic response characteristics are not directly obtained nor are effects from wind gusts in resonance with the building motion. In tall or flexible buildings the dynamic component of the loading can be dominant and the rigid pressure model may severely underestimate the actual loading. This method is considered adequate for buildings with a height-to-weight ratio of less than five (Taranath 1998).

2.4.4.2 Rigid High Frequency Force Balance Models

The high frequency force balance (HFFB) model can be used to directly obtain wind induced dynamic forces acting on the two fundamental sway and the fundamental torsional modes of vibration. Floor shears, overturning and torsional moments due to the wind loads and the acceleration at the top of the building can be determined through this method. Tip displacements of the model can be determined through the use of strain gauges. An advantage of the HFFB model is that it can account for the dynamic component of the wind load without reference to the prototype buildings stiffness and mass. If desired, the designer can then use the data from the test to determine the dynamic response of alternate structural systems. This benefit along with the short time period needed to model and test the structure (typically two days) makes the high frequency force balance model a valuable design aid.

2.4.4.3 Aeroelastic Models

Aeroelastic models are the most complex in that they must correctly represent the stiffness, damping and mass of the prototype building. Accelerations are obtained as are lateral forces and moments due to the wind loading. These models are difficult to construct and complete testing can take from ten to twelve weeks, making aeroelastic models the most time consuming and typically the most expensive of all wind tunnel tests. However, these models are the best at capturing the dynamic behavior of a structure, particularly when vortex shedding and resulting across-wind accelerations are a design concern. It should be noted that aeroelastic models cannot provide wind pressures for cladding design and do not directly provide tip displacements. Tip displacements can be obtained through double integration of the recorded acceleration, which can be prone to error when the signals are small. A laser measurement technique (Balendra *et al.* 2005) has been developed that allows direct displacement measurement and provides results consistent with the conventional strain gauge derived displacements.

The choice of if a wind tunnel test is a necessary step in the design of a building, and the ensuing choice of an appropriate model requires knowledge of what is desired by the engineer. This may simply be the exterior pressures for cladding design or could be the complete dynamic behavior of the structure in interaction with the surroundings to determine maximum accelerations. Time, cost and the required information all play a role in determining if, and if so, what type of wind tunnel testing is necessary.

2.4.5 Database Assisted Design

Codes and specifications are by default based on simplified formulas, tables, and plots leading to designs which may be inconsistent with respect to serviceability and risk considerations. A study by Simiu and Stathopoulos (1997) has shown that these inconsistencies include gust response factors for rigid buildings, across-wind response of tall buildings, wind load factors, directional wind effects, dependence of aerodynamic coefficients on building geometry, and the estimation of wind effects on non-linear structures. As a result, the accuracy of wind loads applied to the structure is not up to par with the accuracy of currently available structural design software.

This discrepancy between the accuracy of loads and the accuracy of analysis techniques can be reduced by utilizing the computational resources available today. One method is Database Assisted Design or DAD for short. For a history of DAD the interested reader is referred to Fenves *et al.* (1995). DAD relies on powerful computers and software applications to define wind loads on a structure through an extensive library of wind tunnel test results and a known building size and geometry. The DAD approach entails the use of large databases of aerodynamic pressures, the optional use of databases of directional extreme wind speeds, and the use of structural information needed for the description of linear or nonlinear structural behavior (Whalen *et al.* 2002). The outcome of this approach is increased economy and safer, more risk-consistent structures. A study by Seokkwon *et al.* (2002) comparing the "estimated wind load capacities of low-rise steel building frames based on loading patterns (magnitude and distribution) established from aerodynamic databases on the one hand, and on patterns specified in the ASCE 7 Standard on the other demonstrates that DAD can lead to safer designs at lower costs." The research on DAD has focused on reliability analysis and design of low rise steel frames. However, DAD has the capability of becoming a powerful design aid with respect to wind loads, especially for commonly constructed building geometries such as low-rise, gable-roof steel structures.

2.4.6 Computational Fluid Dynamics

Possibly the future of wind engineering, computational fluid dynamics, or CFD for short, refers to how one treats a continuous fluid in a discretized way using computer software. Currently CFD can be used to model air flow in a building, in an environment or to investigate how air moves around an object. Providing accurate pressure loads on buildings is another issue. Wright (2004) points out that several major drawbacks of CFD: it can only solve steady state flow, the choice of a turbulence model is crucial and that modeling the atmospheric boundary layer is difficult. Recent work by Hajj (2004) and many others has sought to address these issues. An overview of the current state of CFD in wind engineering applications is presented by Murakami (2004).

Despite the challenges that lie ahead in the field of computation fluid dynamics, it is envisioned that this technology will one day be employed to analytically model wind loads on structures. Today an actual historical earthquake record can be applied to an analytical or experimental building model. In the future, will a historical hurricane or tornado be simulated to assist in design? It is not unreasonable to think so.

2.5 Building Response

Measuring the full-scale in situ response of a structure to actual loads is beneficial in that the collected data may be used to validate (or disprove) design procedures and design methods and reveal unsafe or uneconomical design decisions. The measured response can then be compared to the predicted response, from both finite element software and wind tunnel methods, to provide valuable information for the engineering community.

A pioneer in many respects, Gustave Eiffel was perhaps the earliest to research building action under the dynamic effects of wind. In one of the first instances of field measurements, Eiffel found the sway of his 986 foot tower to be roughly 2.5 inches (Taranath 1998). This corresponds to a drift ratio of H/5000, a ratio that would be economically prohibitive to design for today when acceptable conservative limits hover around H/500. Around the same time as Eiffel's measurements, the sway of Chicago's 16 story Monadnock Building was being measured through the use of a plumb bob suspended in the stair shaft from the top floor. According to the "plumb bob data" and the use of a survey transit the building was determined to sway roughly 1.5 inches (Taranath 1998). This corresponds to a likewise conservative total drift ratio of H/1600, not too surprising for a building with 12 foot thick load-bearing walls. The action of the Empire State Building under wind loads was observed by Rathbun (1940), who wrote an ASCE published paper in which he compared the building oscillations to the tines of a tuning fork. The re-emergence of super-tall buildings in the 1960's and the development of new technology, such as boundary layer wind tunnel testing, accelerometers and computers, has spurred the instrumentation of many buildings, a trend that continues to this day.

2.5.1 Methods of Measuring Response

There are many uncertainties involved in the design of a structure: damping values are at best an educated guess, wind loads are approximated and the overall stiffness may be greater than estimated due to contributions from the nonstructural components. To avoid compounding these sources of uncertainty it is desirable to use accurate, precise and dependable tools to measure actual building response. A popular method used to capture the dynamic response of a structure is installing accelerometers at specific building locations. These devices capture the buildings acceleration (at the accelerometer location) and allow dynamic characteristics of the building to be determined. Accelerometers have been used for some time but the future of building monitoring may be in satellites.

2.5.1.1 Real Time Kinematic Global Positioning System (GPS)

The main appeal of using GPS to record response is that the total displacement, both static and dynamic components, can be measured. Accelerometers can only capture the dynamic component of displacement. However, there are several distinct disadvantages to using GPS (Tamura *et al.* 2000): the natural frequency of the building must be less than 2 Hz and the tip displacement must be greater than 2 cm. As more accurate systems are developed better results can be expected but the one obstacle that is unavoidable is that GPS relies on satellite communication to function. When the signals are blocked or not enough satellites are present, the measurements can be disrupted. This can be a problem in dense urban environments. Despite this, GPS accuracy has shown to compare very well with accelerometers in a recent study (Kijewski-Correa *et al.* 2005): GPS RMS acceleration errors (found through differentiating the displacements twice) were roughly +5.5 percent of the accelerometer errors. The use of GPS to monitor building movement shows potential for future instrumentation and is currently being used in the Chicago Full-Scale Monitoring Program (discussed in Section 2.5.2.1).

2.5.2 Boundary Layer Wind Tunnel vs. Full Scale Comparisons

For the majority of buildings designed a wind tunnel study is simply not necessary. In these cases the designer will most likely resort to the code specified procedure for determining appropriate wind loads. It is important to validate the effectiveness of code loads and design assumptions by comparing wind tunnel data, in-situ data and code derived data for a wide variety of building geometries, heights and locations. When this is done refinements to the codes and to wind tunnel methods can be properly accomplished.

The evaluation of wind loads on buildings is carried out mainly by using codes and standards, whose specifications are generally based on wind tunnel tests performed on isolated structures in an open terrain. However, it has been shown by several researchers that wind loads on buildings in realistic environments may be considerably different from those measured on isolated buildings, because of the so-called interference effects from nearby structures (Khandari 1998). The disparities between code loads and wind tunnel derived loads are mostly due to the interference effects but also due to the geometry differences, as code defined loads are based on relatively simple building plans. A study (Stathopoulos 1984) on low rise buildings in the presence of larger buildings, comparing the National Building Code of Canada and the ANSI Standard to wind tunnel loads, shows underestimations as great as 46 percent or overestimation as large as 525 percent due to necessary generalizations in the code specifications.

In a separate study (Taranath 1998) comparisons between code values and wind tunnel values were made for twenty-four buildings located in several major North American cities. Building shapes varied from simple box like geometries to complex, highly irregular forms and ranged in height from 460 to 1113 feet. Using base overturning moments as points of comparison, ASCE 7-88 and the National Building Code of Canada (NBCC) values were compared with wind tunnel data. On average the wind tunnel data resulted in base overturning moments 13 percent less than the ASCE 7-88 values and 17 percent less than the NBCC values. In these cases the codes were found to be conservative, as expected, when compared with the wind tunnel studies. However, roughly 25 percent of the buildings, as compared to ASCE 7-88 and 15 percent as compared to NBCC, exceeded the code values. In these cases the code was unconservative. The complex interaction of wind loading, location topography and building dynamic characteristics indicate that wind tunnel studies are advisable when dealing with unique conditions. Unique conditions can be considered as the following: a flexible structure (natural frequencies below 1 Hz), an irregularly shaped building or a building that may be affected by the channeling or buffeting of wind due to nearby structures.

2.5.2.1 High-rise Buildings

Although high-rise building design is the exception rather than the norm for the typical design firm, tall buildings are more sensitive to wind loading and are therefore commonly studied in the wind tunnel. Validating wind tunnel methods for tall building design is integral to satisfying serviceability considerations. Given that wind tunnel data is already available the buildings are prime candidates for building instrumentation, as the following examples (far from a comprehensive review, which is beyond the scope of this discussion) demonstrate.

A 47 Story Building, Houston Texas

An excellent example of a wind sensitive building that was highly under-designed due to the prescribed code loads and technology at the time has been presented in a conference paper by Griffis (1996). The general findings and conclusions will be summarized here, as this building demonstrates the importance of designing to control drift.

The building, a 628 foot tall, 47 story unbraced steel framed skyscraper located in downtown Houston, exhibited poor performance under lateral loads from the beginning. It was designed in 1971 according to the 1969 *AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* and for the wind loads prescribed by the local building code which required 20 PSF up to sixty feet and 30 PSF for the remaining height. In addition to the gross underestimation of the equivalent static loads by the code the following statements about the design can be made:

- Serviceability was not considered and a drift limit was not imposed
- Second order effects were neglected for the leaner columns and all girders
- Shear deformations were excluded

- Panel zone deformations were excluded
- Center-to-center dimensions were used

Complaints from the occupants and owners described noise generated by building components such as the glass curtainwall, partitions, ceilings, etc. during common wind events. Motion perception was also reported, though at a far lower frequency. Cracking and damage to nonstructural components was a common problem. During Hurricane Alicia, a category 3 storm that made landfall in August 1983, the building experienced an estimated gradient wind speed of 82 mph. It is estimated that interstory drift ratios of 0.035 (h/29) in the upper stories and 0.018 (h/56) in the lower stories were experienced (Griffis 1996).

After the storm a force balance model of the building was evaluated in a boundary layer wind tunnel. The base moments obtained from this study indicate that the building came close to its ultimate strength and in fact the analysis results in failure if a damping value less than 4 percent is used. A field investigation of the building found no cracking in susceptible welds at beam to column joints and the building was determined to be vertically plumb within acceptable erection limits. The structure remained elastic during this major event. Although the building withstood a design event the assumptions made by the code and in the analysis resulted in a structure that was inadequate with respect to serviceability. In 1994 the building was retrofit to satisfy current codes and to better serve the tenants. On each side of the building two of the exterior columns were made composite and 9 story tall diagonal steel X-bracing was added between these newly composite columns (Colaco *et al.* 2000).

Allied Bank Plaza, Houston Texas

(Note that this building is now known by the name "Wells Fargo Bank Plaza"). Completed in late 1982, Allied Bank Plaza (as it was known at the time) is a 71 story steel framed office building located in downtown Houston. Due to its height and location in a hurricane prone area, a wind tunnel model study was conducted at the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario to aid in designing for the wind loads, including hurricanes. The model was an aeroelastic one, performed for a variety of wind directions and speeds. The completed building was instrumented with accelerometers on the top floor, oriented in the general direction of the two fundamental sway modes, to monitor the dynamic response characteristics of the structure. On August 18, 1983 the building experienced what could be considered a major wind event when Hurricane Alicia made landfall in Houston. The recorded fastest-mile winds came very close to exceeding the code-specified 90 mph winds for a 50 year mean recurrence interval. Accelerometers had to be manually activated as the storm made landfall and the technician responsible for this risky job reported trouble walking, (Dalgliesh and Surry 2003) an observation that was later confirmed by the measured data.

Comparisons between the predicted and actual accelerations showed strong agreement and the following conclusions were drawn (Powell and Georgiou 1987, Taranath 1998):

- Lateral loads were estimated to be close to 75 percent of the design loads (Isyumov and Halvorson 1986).
- Building accelerations reached at least 50 milli-g's.
- The magnitude of the loads matched well with the aeroelastic wind tunnel model while the Houston code wind loads overestimated the mean loads by roughly 100 percent.
- The mean wind loads represented only 20-30 percent of the total structural loads, indicating the importance of the dynamic effects of wind gusts.
- Predicted total drift in the two fundamental sway directions (2.5 and 2 feet) compared somewhat well with the measured values (2.58 and 1.25 feet).
- No evidence of structural distress was evident in the storms aftermath.

In this case the wind tunnel testing proved to be quite accurate as compared to measured response.

Di Wang Tower, Shenzhen China

Di Wang Tower is a 1066 foot tall building with a height to width ratio of about nine, located in a typhoon prone region of China. The 79 story building is composed of a reinforced concrete core wall coupled with perimeter steel frames. Di Wang Tower was modeled using a high frequency force balance model and tested at the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario to determine overall wind loads and the dynamic properties. Accelerometers were installed in the completed building to capture actual building performance; the passage of Typhoon Sally in 1996 provided excellent data to compare with the wind tunnel data. Comparisons (Li *et al.* 2004) with respect to accelerations on the top floor show excellent agreement between the force balance model and measured response. The differences between the field measurements and the wind tunnel data were in the range of 4.0–12.5 percent.

Chicago Full-Scale Monitoring Project

This ongoing project (Kijewski-Correa *et al.* 2005) is the result of a joint collaboration between members of the faculty at the University of Notre Dame, the design firm Skidmore, Owings & Merrill LLP and the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario. It involves building instrumentation, finite element computer modeling and wind tunnel modeling of 3 tall buildings in Chicago, whose names have not been revealed, to validate the wind tunnel methods commonly used in design against observed full-scale performance. Two of the buildings are steel tube buildings and the other is a concrete shear wall/outrigger system. Such validation, through determining in-situ periods of vibration and damping ratios, is a valuable piece of information considering the amount of trust designers place in scale model wind tunnel tests. Each one of the buildings has been instrumented with four accelerometers to capture translational and torsional movement, roof level anemometers to provide a more complete description of wind field characteristics above downtown Chicago, and high-precision GPS to capture static and dynamic displacements under wind. To date over 2.5 years of data has been logged with results comparing well to FE models. The results are not publicly available nor are the building names at this point in the project. As the project continues, more thorough and definitive conclusions are expected with respect to the validation of wind tunnel methods.

2.5.2.2 Low-rise Buildings

Given that the great majority of constructed steel buildings are not super-tall and are therefore designed using the code approach, it is desirable to validate the code defined winds, through the use of wind tunnel testing and full scale monitoring of these buildings. With the powerful capabilities of computer hardware/software for modeling and design, using risk consistent and economically realistic wind loads is extremely desirable.

For low-rise buildings perception of motion is likely not a problem and the main benefit of validating code values is increased economy.

Silsoe Research: Agricultural Buildings

Field measured pressure coefficients on several large agricultural buildings located in England were gathered from 1974 onward (Richardson *et al.* 1997). These buildings were subsequently modeled, on a 1:100 scale, in 1991 at the BLWT at the University of Western Ontario to compare mean pressure coefficients. Four buildings were tested, all with sloping roofs from 10 to 15 degrees and reaching an average height of approximately 20 feet. External pressure coefficients for the windward side walls, with the wind normal to the long wall, show good agreement. BLWT values range from 64 percent to 125 percent of full-scale values, with disparities contributed to the absence of an upwind boundary layer profile for two of the buildings and also to Reynolds number effects.

Texas Tech University Test Facility

A full scale low-rise building located at Texas Tech University with pressure taps on the buildings sides and roof, has been a valuable research tool ever since its inception. The building was built in 1989 and has dimensions of 30’ by 45’ by 13’ high. An article by Levitan and Mehta (1992) provides full details of the structural system and the pressure monitoring system. The building has been used to collect data for use in Database Assisted Design (Ho *et al.* 2005), and for use in comparing wind tunnel tests to in-situ measured wind pressures (Dalgliesh and Surry 2003) among many other research applications.

2.6 Summary

The review of the literature began with drift limits and the definition of damageability. It was shown that the commonly used interstory drift ratio is not the best measurement of damage due to lateral wind loads. The shear strain in the material is the true measure of damage. The drift damage index or DDI was presented as an alternative and its use was explained through several examples. It was seen that, depending on the frame (braced or unbraced), the interstory drift index may severely miscalculate the actual damage in the buildings partition walls.

Next, the issue of modeling and analysis for wind drift serviceability was discussed, with the first topic being sources of stiffness. Though sources of stiffness and sources of deformation can be considered as basically the same thing, the distinction was made that sources of deformation refer only to material deformations. With regards to sources of stiffness, several studies were presented which indicate the stiffening effect provided by nonstructural components. However, more research is needed into the issue of how to incorporate nonstructural components in the analytical model. The issue of the beam to column connection flexibility was also discussed, which may provide additional unaccounted for stiffness to braced frames and simple connections. As more experimental work is done, the use of partially restrained connections will grow due to the added benefits. Next, the effect of the composite slab was discussed with regards to wind loads. Additional research in this area should focus on how much of the slab is effective at relatively low serviceability wind loads and the appropriate moment of inertia (based on percent cracked) to use for the concrete. The action of the slab as a diaphragm was also discussed.

Sources of deformation were discussed next. The varying contributions of axial, shear and flexure to the total lateral drift was illustrated and are discussed in more detail in Chapter 4. From the 1988 ASCE Survey, it was determined that shear deformation and the panel zone region are two major sources of modeling uncertainties. Shear area was discussed with respect to wide flange members and several formulas were presented to calculate form factors. The panel zone region was discussed next. Two mechanical joint models, the Krawinkler and the Scissors, were presented and the Scissors model was discussed in detail. Shortcomings of present joint modeling techniques were shown. Finally P-Delta effects were brought up and how different structural software programs handle this issue was shown.

Wind loads were discussed next. The mean recurrence interval of loads used for wind drifts can range from 10 to 50 years depending on the firm. Wind loads can be determined from the appropriate code, wind tunnel testing, or database assisted design. The code approach, although simple and convenient, may provide unrealistic loads depending on the terrain and building type and especially when there is significant shielding. More research is needed into the wind directionality factor (K_d in ASCE 7-05) and its dependence on the wind's mean return interval. Additional research should also be done to make sure that the conversion from fastest mile to 3 second gust velocities in ASCE 7 was done correctly. Wind tunnel testing was discussed next. Several types of wind tunnel models were presented as well as comparison between the wind tunnel and in situ testing. The question of whether wind tunnel loads should be used for serviceability loads, even if they are below 80 percent of the code determined loads (the limit for strength loads) was brought up.

The future of wind loading may very well be in computational fluid dynamics. Research into this field has really just begun, but a few examples were cited. With technology evolving at the current pace, it is not unrealistic to think that one day wind tunnels will be obsolete, replaced by powerful computer modeling software that replicates the structure, surroundings and the winds dynamic characteristics.

Several cases of measured response were reported. Monitoring building behavior is a necessary tool, one that should be used more often. GPS is gaining ground and is able to capture both the static and dynamic components of the buildings movement. Currently, GPS technology is being used for the ongoing Chicago Full Scale Monitoring Project and may prove a simpler and less expensive alternative to accelerometers. If this is the case, more buildings should be monitored to add to our understanding of real world building behavior.

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Chapter 3

Test Building Modeling and Analysis

3.1 Overview

Present day computers and software are powerful tools for the design engineer but require accurate input to produce reliable results. For a given structure there are a number of assumptions regarding structural modeling that affect the building's lateral stiffness. Many of these assumptions, such as included sources of deformation, beam-column joint modeling, composite action, nonstructural components and second order effects were discussed in the literature review. This chapter aims to illustrate some of these assumptions and their resulting effects on a given building's lateral response under a ten year MRI wind load.

The structural system of the analytical building is discussed first, along with the design of the gravity and lateral load resisting system. Next the lateral loads are calculated based on Method 2 of ASCE 7-05, the Analytical Method. The wind loads are determined for both strength (a 50 year MRI wind, with applicable load factor) and serviceability (a 10 year MRI wind, with no load factor). Finally the analytical models are presented. Points of comparison between the models are made based on displacement vs. height and the periods of the first six modes. Observations are made and the relative merits of each model are examined.

3.2 Test Building: Structural System

Located in Jersey City, NJ the hypothetical building that was modeled is a rectangular (100 ft by 150 ft plan dimensions) ten-story steel building with four braced frames in the short direction and two moment frames in the long direction. (It is the same building used in the Wind Drift Survey and the reader is directed to Appendix A for complete building details, including site location, member sizes and gravity loads.) Beam and brace connections in the braced frames and gravity frames were modeled as pinned while beam connections in the moment frame were modeled as fully fixed. The floors are all modeled as rigid diaphragms, an assumption that may not be valid when stiff shear walls are used (see Section 2.3.4.1 of this thesis). The building supports were modeled as fully fixed in the moment frame direction and pinned in the braced frame direction. The braces are rectangular HSS sections and all other members are typical rolled W-sections. The LRFD design philosophy was employed and members were designed to satisfy current ASCE 7 (ASCE 2005) load combinations.

3.2.1 Computer Software

SAP2000 Version 9 (Computer and Structures 2005) was used to perform all of the building modeling and analysis. Modeling was done in three-dimensions and analysis cases were linear elastic, with the exception of the P-Delta effects. KeySolver (Pathak 2004), a finite element analysis program that is able to read SAP data files, was also used. Through the principle of virtual work, KeySolver is able to quantify the different sources of deformation contributing to the displacement at a specific building location. Chapter 4 of this thesis also utilizes KeySolver, and more information on this program and how it utilizes the principle of virtual work is available there.

3.3 Lateral Loads

The determination of all of the wind loads was based on ASCE 7 (ASCE 2005), which uses average 3 second gusts at 33 ft above the ground as the standard of measurement. The Analytical Method (ASCE 7-05 Section 6.5) was used to go from the map obtained wind velocities to building velocity pressures at each level of the building. The velocity pressure is given by ASCE 7-05 Equation 6-15 and is defined as follows: "Velocity pressure q_z evaluated at height z shall be calculated by the following equation:"

$$q_z = 0.00256 K_Z K_{zt} K_d V^2 I \quad (psf) \tag{3.1}$$

where

K_z = velocity pressure exposure coefficient (ASCE 7-05 Section 6.5.6.6)

K_{zt} = topographic factor (ASCE 7-05 Section 6.5.7.2)

K_d = wind directionality factor (ASCE 7-05 Section 6.5.4.4)

V = basic wind speed, mph (ASCE 7-05 Figure 6-1)

I = importance factor (ASCE 7-05 Section 6.5.5)

The windward and leeward pressures are combined to obtain the total wind pressure acting in any given direction.

3.3.1 Wind Loads: Strength Design

For the strength design of the members a 50 year mean return interval was used for the wind loads. An Exposure Category C was selected based on the buildings location and surrounding terrain (see Appendix A, Figure 1). The choice of an Exposure Category is based on subjective opinion, and is not always clear but the engineer must use his/her best judgment to make this decision. Using ASCE 7-05 the following values were obtained for use in Equation 3.1:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} \tag{3.2}$$

where

z = height at which pressure is to be evaluated (ft)

z_g = terrain exposure constant, based on Exposure Category C (ASCE 7-05 Table 6-2)

= 900

α = terrain exposure constant based on Exposure Category C (ASCE 7-05 Table 6-2)

= 9.5

K_{zt} = 1.0

K_d = 0.85 (must be used in conjunction with strength based load combination Factors)

V = 110 mph (based on Exposure Category C)

I = 1.0

Substituting these values into Equation 3.1 yields the following equation for the velocity pressure, q_z as a function of the height, z:

$$q_z = 52.92\left(\frac{z}{900}\right)^{0.21} \tag{3.3}$$

This value of q_z must now be multiplied by a gust effect factor and an external pressure coefficient to obtain the pressures on the windward and leeward faces of the structure.

G = gust effect factor (ASCE 7-05 Section 6.5.8)
= 0.85
C_p = external pressure coefficient (ASCE 7-05 Section 6.5.11.2)
= 0.8 for the windward faces
= -0.5 for the leeward face in the E-W direction
= -0.4 for the leeward face in the N-S direction

Note that the negative sign on the external pressure coefficients indicates that the pressure is acting away from the face of the building and is in effect a suction pressure. Also C_p is based on the ratio of B (horizontal dimension of the building normal to the wind) to L (horizontal dimension of the building parallel to the wind), which is why the value differs for the E-W and N-S directions. The velocity pressure as a function of height can now be defined as q_zGC_p. Multiplying Equation 3.3 by GC_p yields the following equation for pressure on the building's windward face only:

$$q_zGC_p = 35.99\left(\frac{z}{900}\right)^{0.21} \tag{3.4}$$

Wind pressure acting on the leeward face of the building does not vary with height and is taken as the pressure evaluated at the mean roof height. For the leeward face in the N-S direction the following is obtained by combining Equation 3.3 with the appropriate gust effect factor and external pressure coefficient and using a mean roof height of 135 ft.

$$q_h G C_p = -12.09 \text{ psf} \tag{3.5}$$

Similarly for the leeward face in the E-W direction the following is obtained with the only difference between Equations 3.5 and 3.6 being the external pressure coefficients.

$$q_h G C_p = -15.11 \text{ psf} \tag{3.6}$$

Now the total pressure can be determined and converted to a point load at each level by multiplying the pressure by the tributary area. For the N-S direction (150 ft wide) there are four braced frames so each frame has a tributary width of 37.5 ft and a tributary height equal to the story height. Table 3.1 summarizes the calculations.

Table 3.1: 50 Year Wind Loads in the N-S Direction

		PRESSURE (psf)			Area	Point load on each braced
LEVEL	Height (ft)	Windward	Leeward	TOTAL	(ft^2)	frame (kips)
2	18	15.56	12.09	27.65	581.25	16.07
3	31	17.70	12.09	29.79	487.5	14.52
4	44	19.06	12.09	31.15	487.5	15.18
5	57	20.13	12.09	32.22	487.5	15.71
6	70	21.02	12.09	33.11	487.5	16.14
7	83	21.78	12.09	33.87	487.5	16.51
8	96	22.46	12.09	34.55	487.5	16.84
9	109	23.07	12.09	35.16	487.5	17.14
10	122	23.62	12.09	35.71	487.5	17.41
R	135	24.13	12.09	36.22	243.75	8.83
TOTAL BASE SHEAR PER BRACED FRAME:						154.36
TOTAL BASE SHEAR = 4 frames*154.36 = 617.44 kips						

For the E-W direction (100 ft wide) the calculations are similar except the tributary width is different and the leeward pressure is different. There are two moment frames, each with a tributary width of 50 ft and a tributary height equal to the story height. Table 3.2 summarizes the calculations.

Table 3.2: 50 Year Wind Loads in the E-W Direction

LEVEL	Height (ft)	PRESSURE (psf) Windward	Leeward	TOTAL	Area (ft^2)	Point load on each moment frame (kips)
2	18	15.56	15.11	30.67	775	23.77
3	31	17.70	15.11	32.81	650	21.33
4	44	19.06	15.11	34.17	650	22.21
5	57	20.13	15.11	35.24	650	22.90
6	70	21.02	15.11	36.13	650	23.48
7	83	21.78	15.11	36.89	650	23.98
8	96	22.46	15.11	37.57	650	24.42
9	109	23.07	15.11	38.18	650	24.82
10	122	23.62	15.11	38.73	650	25.18
R	135	24.13	15.11	39.24	325	12.75
TOTAL BASE SHEAR PER MOMENT FRAME:						224.84
TOTAL BASE SHEAR = 2 frames*224.84 = 449.68 kips						

3.3.2 Wind Loads: Serviceability Design

For the serviceability issue of wind drift a different set of lateral loads were used. Because 50 years is not a reasonable return interval for serviceability a reduced MRI of 10 years, the average occupancy of a building tenant, was chosen (Galambos and Ellingwood 1986).

To convert from the 50 year map obtained wind velocity to a 10 year wind velocity a reduction factor was used. ASCE 7-05 Table C6-7 provides reduction factors for other mean recurrence intervals. Consulting this table, a reduction factor of 0.74 was selected. Multiplying by the map obtained 50 year wind velocity yields a velocity of V_{10year} = 0.74(110mph) = 81.4 mph

The only other difference for the 10 year wind loads is the wind directionality factor, K_d which now drops out of Equation 3.1. According to section 6.5.4.4 of ASCE 7-05, the wind directionality factor is only applied when used in conjunction with load combinations specified in ASCE 7-05 Sections 2.4 and 2.5. (For more information on the wind directionality factor, please refer to 2.3.1.4 of the literature review). With everything else remaining the same, Equation 3.1 and 3.2 are used to obtain the following equation for velocity pressure as a function of the height z.

$$q_z = 34.09\left(\frac{z}{900}\right)^{0.21} \tag{3.7}$$

Once again, this value of q_z must now be multiplied by a gust effect factor and an external pressure coefficient to obtain the pressures on the windward and leeward faces of the structure.

G = gust effect factor (ASCE 7-05 Section 6.5.8)
= 0.85
C_p = external pressure coefficient (ASCE 7-05 Section 6.5.11.2)
= 0.8 for the windward faces
= -0.5 for the leeward face in the E-W direction
= -0.4 for the leeward face in the N-S direction

The velocity pressure as a function of height can now be defined as q_zGC_p. Multiplying Equation 3.7 by GC_p yields the following for pressure in the windward direction.

$$q_zGC_p = 23.18\left(\frac{z}{900}\right)^{0.21} \tag{3.8}$$

For the leeward face in the N-S direction the following is obtained by combining Equation 3.7 with the appropriate gust effect factor and external pressure coefficient and using a mean roof height of 135 ft.

$$q_hGC_p = -7.79 \text{ psf} \tag{3.9}$$

Similarly for the leeward face in the E-W direction the following is obtained with the only difference between Equations 3.9 and 3.10 being the external pressure coefficients.

$$q_hGC_p = -9.73 \text{ psf.} \tag{3.10}$$

Now the total pressure can be determined and converted to a point load at each level by multiplying by the tributary area. Table 3.3 summarizes the calculations.

Table 3.3: 10 Year Wind Loads in the N-S Direction

		PRESSURE (psf)			Area	Point load on each braced
LEVEL	Height (ft)	Windward	Leeward	TOTAL	(ft^2)	frame (kips)
2	18	10.00	7.79	17.79	581.25	10.34
3	31	11.40	7.79	19.19	487.50	9.36
4	44	12.28	7.79	20.07	487.50	9.78
5	57	12.96	7.79	20.75	487.50	10.12
6	70	13.54	7.79	21.33	487.50	10.40
7	83	14.03	7.79	21.82	487.50	10.64
8	96	14.47	7.79	22.26	487.50	10.85
9	109	14.86	7.79	22.65	487.50	11.04
10	122	15.22	7.79	23.01	487.50	11.22
R	135	15.54	7.79	23.33	243.75	5.69
TOTAL BASE SHEAR PER BRACED FRAME:						99.43
TOTAL BASE SHEAR = 4 frames*99.43 = 397.72 kips						

For the E-W direction the calculations are similar except the tributary width is different. Table 3.4 summarizes the calculations.

Table 3.4: 10 Year Wind Loads in the E-W Direction

		PRESSURE (psf)			Area	Point load on each moment
LEVEL	Height (ft)	Windward	Leeward	TOTAL	(ft^2)	frame (kips)
2	18	10.00	9.73	19.73	775.00	15.29
3	31	11.40	9.73	21.13	650.00	13.74
4	44	12.28	9.73	22.01	650.00	14.30
5	57	12.96	9.73	22.69	650.00	14.75
6	70	13.54	9.73	23.27	650.00	15.12
7	83	14.03	9.73	23.76	650.00	15.44
8	96	14.47	9.73	24.20	650.00	15.73
9	109	14.86	9.73	24.59	650.00	15.98
10	122	15.22	9.73	24.95	650.00	16.22
R	135	15.54	9.73	25.27	325.00	8.21
TOTAL BASE SHEAR PER MOMENT FRAME:						144.79
TOTAL BASE SHEAR = 2 frames*144.79 = 289.58 kips						

3.3.3 Loading Combinations

For the drift calculations the loads applied to the structure were unfactored. All of the building models were subjected to the same unfactored 10 year wind loads calculated in Section 3.3.2 and gravity loads based on the information given in Appendix A. The full live load was reduced according to Section 4.8 of ASCE 7-05. Equation 3.11 shows the loading combination used for drift calculations.

$$1.0D + 1.0L_{reduced} + 1.0W_{10year} \tag{3.11}$$

Only ASCE Wind Loading Case 1, in which 100 percent of the wind load is applied in each direction independently, is studied.

3.4 Analytical Building Models

Each of the individual sections in Section 3.4 focuses on the following unique modeling parameters and how the model is affected by the modeling assumptions:

- Sources of Deformation
- Second Order P-Delta Effects
- Beam-Column Joint Modeling
- Influence of the Composite Floor Slab
- Survey Live Loads
- Contributing Stiffness of Nonstructural Components

Comparisons are made based on lateral displacement vs. height and mode periods. Tables of program output displacements for all of the models can be found in Appendix B along with periods of the first six modes for each model.

3.4.1 Sources of Deformation

The purpose of this section is to investigate the effect of ignoring certain sources of deformation in the analytical model. The following points must first be made:

- Flexural deformations are always included in all members.
- Axial deformations are always included in the diagonal brace elements (otherwise the building would not move laterally in the braced frame direction).
- Axial deformations are excluded in beams/columns by artificially increasing the axial area of the elements (applying a section property modifier).

- Shear deformations are excluded by artificially increasing the shear area of the elements (applying a section property modifier).
- Joint deformations are not explicitly accounted for but the following can be said:
  - When the joint is considered fully rigid (Z = 1.0; see Section 2.3.3 and Figure 2.3 for details) there are no deformations in the joint region.
  - When the joint is considered fully flexible (Z = 0) the joint has no analytical dimensions but additional deformations are picked up by the beams and columns framing into that joint (refer to Figure 2.4).
- P-Delta effects are not included for these models.

Table 3.5 shows the progression of the models, from the stiffest to the most flexible. Figure 3.1 illustrates the results for loads applied in the braced frame direction.

Table 3.5: Modeling Parameters

		Shear Deformations			Axial Deformations		
Model #	Z value	Beams	Columns	Braces	Beams	Columns	Braces
1A	1	no	no	no	no	no	yes
1B	1	no	no	no	yes	yes	yes
1C	1	yes	yes	yes	yes	yes	yes
1D	0	yes	yes	yes	yes	yes	yes

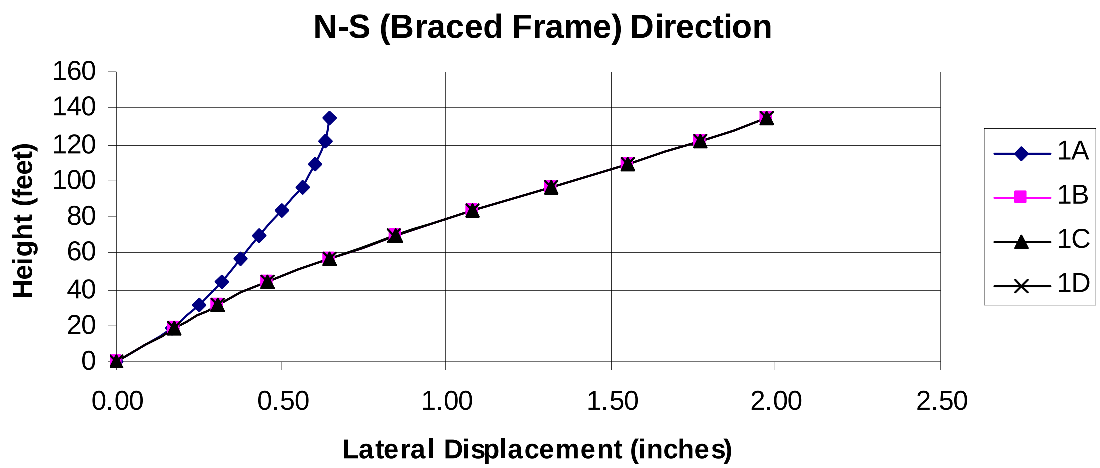

Figure 3.1: Lateral Flexibility in the N-S Direction with Respect to Included Sources of Deformation

Data from models 1B, 1C and 1D are nearly identical, which is why it appear that there are only two trend lines. From Figure 3.1 several observations can be made:

- The stiffest model (1A) neglected axial and shear deformations.
- The subsequent three models produced nearly identical lateral displacements leading to the following corollaries:
  - Shear deformations are relatively unimportant in braced frames
  - Axial deformations in columns and braces are the dominant source of lateral flexibility in this braced frame configuration.

For loads applied in the E-W direction, Figure 3.2 illustrates the results.

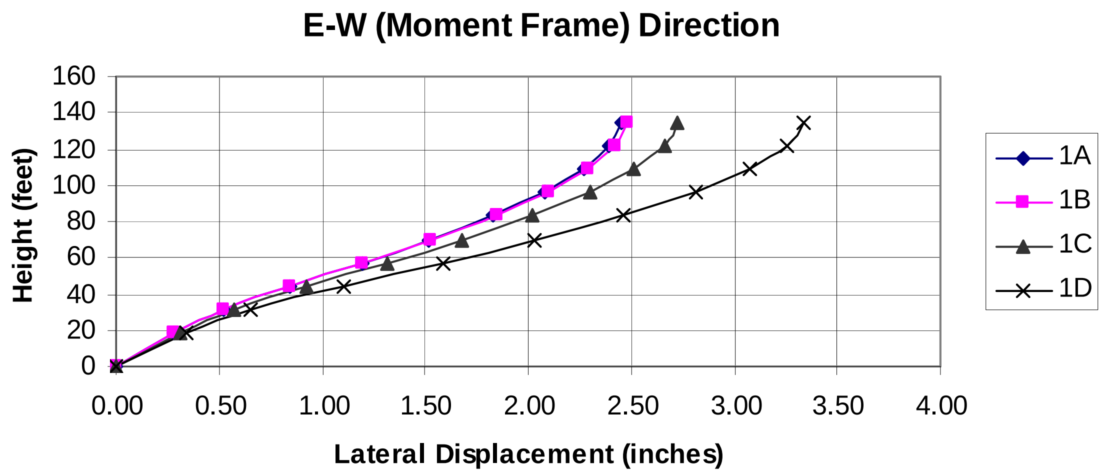

Figure 3.2: Lateral Flexibility in the E-W Direction with Respect to Included Sources of Deformation

From Figure 3.2 the following can be observed:

- The stiffest model (1A) neglected axial and shear deformations.
- Including axial deformations (1B) made less of a difference than including shear deformations (1C).
- The largest change was observed by using a Z value of zero (1D)

So for this moment frame configuration, shear deformations are more important than axial deformations and the choice of joint modeling, which will be investigated further in Section 3.4.3, is very important as well.

3.4.1.1 Displacement Participation Factors

To more fully investigate the different sources of deformation, a virtual work based program has been used to quantify the contributions of flexural, shear, axial and joint deformations to the total lateral drift at the building's roof level. The program KeySolver (Pathak 2004) uses the principles of virtual work to quantify the contribution of each element (beam, column or joint region) to the total building drift and is also able to break down the contributing deformations into flexure, axial and shear. KeySolver is able to analyze the structure given the appropriate input file, which is from SAP2000 v. 7.

(Chapter 4 goes into more detail on the topic of virtual work principles and displacement participation factors or *DPF's*). KeySolver was run for Model 1D, the case in which all sources of deformation are included and centerline analysis is used. Table 3.6 shows the contributions for wind loading in the N-S direction.

Table 3.6: *DPF's* for the N-S (Braced Frame) Direction

	Girders	Columns	Braces	TOTAL
Axial	0	1.327	0.631	1.958
Shear-major	0	0.001	0	0.001
Shear-minor	0	0	0	0
Torsion	0	0	0	0
Flexure-major	0	0.005	0	0.005
Flexure-minor	0.008	0.003	0	0.012
TOTAL	0.008	1.336	0.631	1.976

From the table it is clear that axial deformations are dominant, accounting for 68 percent of the total drift at the roof level. Referring back to Figure 3.1 it is easy to see why the displaced shape did not change from models 1B to 1D. Including shear and joint deformations makes virtually no difference and even flexural deformations are negligible in this case. It should be noted that there is a stiffening component due to flexure (as truly pinned connections do not exist) and the actual observed in situ building displacement may be less than the 1.958 in. calculated above.

In the moment frame direction the effects of the different sources of deformation is much more apparent. Table 3.7 shows the results from KeySolver.

Table 3.7: *DPF's* for the E-W (Moment Frame) Direction

	Girders	Columns	Braces	TOTAL
Axial	0	0.029	0	0.029
Shear-major	0.086	0.206	0	0.261
Shear-minor	0	0	0	0
Torsion	0	0	0	0
Flexure-major	1.960	1.090	0	3.051
Flexure-minor	0.050	-0.049	0	0
TOTAL	2.096	1.276	0	3.341

Flexural deformations in the girders and joint deformations account for over half of the total roof level displacement. In this case axial deformations are negligible and shear deformations are quite small. Chapter 4 demonstrates that shear deformations are highly dependent on the width of the bays, with the smaller bay width resulting in larger shear contributions.

3.4.2 P-Delta Effects

The next modeling parameter to be studied is the P-Delta effect (refer to Section 2.3.7 for details). To simplify the analysis and single out P-Delta as the point of comparison, four models were analyzed and as Table 3.8 shows, all sources of deformation are included. The two variables are P-Delta effects and the joint rigidity *Z* value (refer to Section 2.3.2 for details). Because the computer program SAP2000 forms the geometric stiffness matrix on an element basis, both member (P-δ) and overall (P-Δ) second order effects are accounted for. Figure 3.3 illustrates the results for loads applied in the N-S direction only.

Table 3.8: Modeling Parameters

			Shear Deformations			Axial Deformations		
Model #	Z value	P-Delta	Beams	Columns	Braces	Beams	Columns	Braces
2A	1	no	yes	yes	yes	yes	yes	yes
2B	0	no	yes	yes	yes	yes	yes	yes
2C	1	yes	yes	yes	yes	yes	yes	yes
2D	0	yes	yes	yes	yes	yes	yes	yes

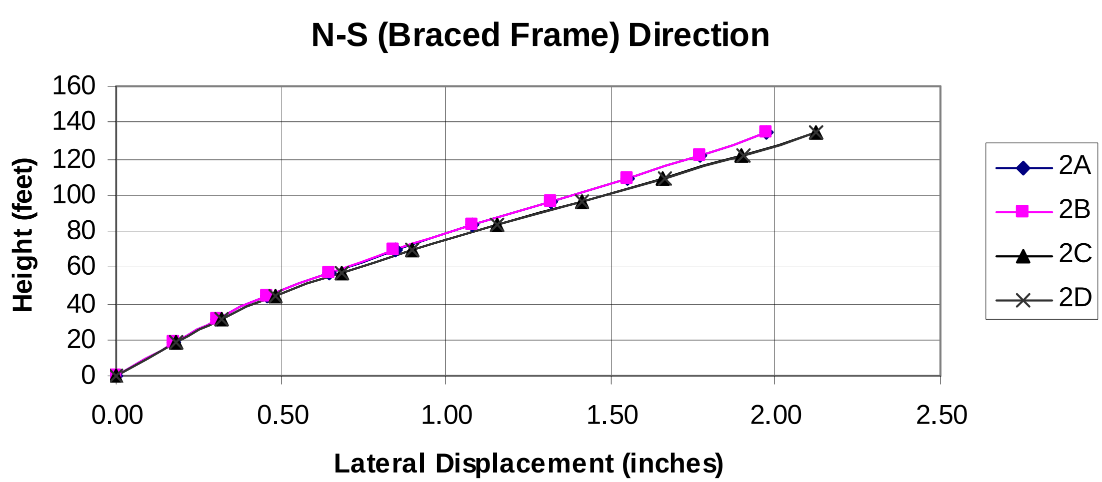

Figure 3.3: Lateral Flexibility in the N-S Direction with Respect to P-Delta Effects

It can be seen that the pairs 2A and 2B along with 2C and 2D are virtually the same. This is to be expected as the only difference from A to B and from C to D is the joint rigidity Z value which does not affect the braced frames. In this case, including P-Delta effects increases the lateral displacements by up to 7 percent. Figure 3.4 illustrates the results for loads applied in the E-W direction only.

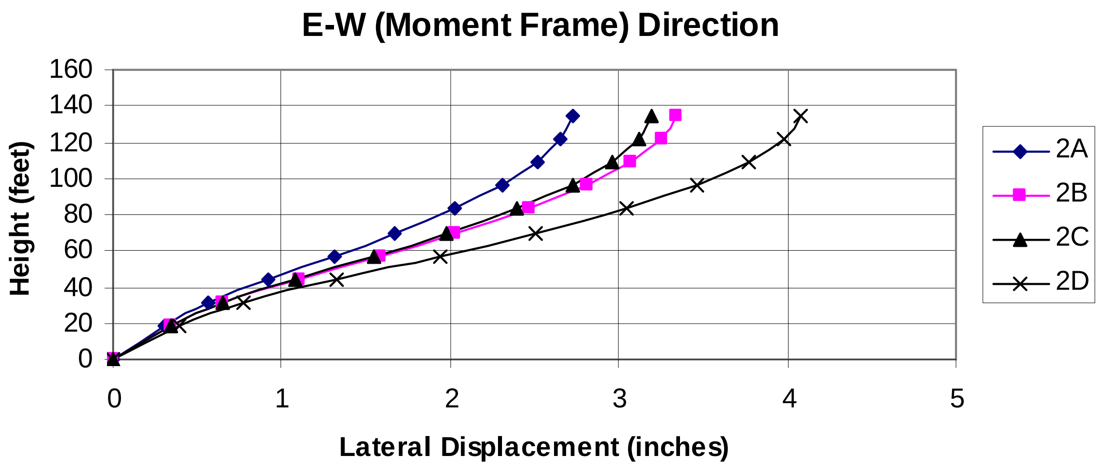

Figure 3.4: Lateral Flexibility in the E-W Direction with Respect to P-Delta Effects

From Figure 3.4 the following observations can be made:

- The stiffest model, 2A, has completely rigid joints (Z=1.0) and ignores P-Delta effects
    - Adding P-Delta effects increases the displacements by up to 17 percent but the model is still relatively stiff.
- Model 2B has completely flexible joints (Z=0) and ignores P-Delta effects.
    - Adding P-Delta effects results in the most flexible model, 2D.
    - Displacements are increased by up to 22 percent.

Steel moment frames are generally more laterally flexible than braced frames and that was observed in these models. This difference resulted in P-Delta effects being more pronounced in the moment frames. It should also be noted that joint modeling has a large effect on displacements and this parameter will be studied next.

3.4.3 Beam Column Joint Modeling

Beam column joints in moment resisting frames can contribute significantly to the building's overall lateral flexibility. The purpose of this section is to illustrate several ways in which the joint region may be modeled using commercially available software. Section 2.3.2 of this thesis contains more explicit details on the joint models used within and the reader is referred to this section for more information. Both Figure 2.7, which shows the use of the *Z* factor, and Figure 2.8 which shows commonly used joint representations, are repeated here for convenience.

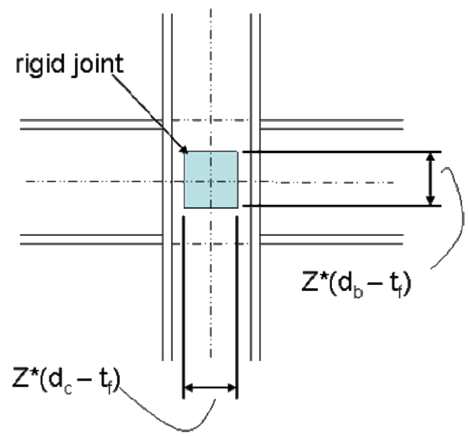

Figure 2.7: Joint Rigidity

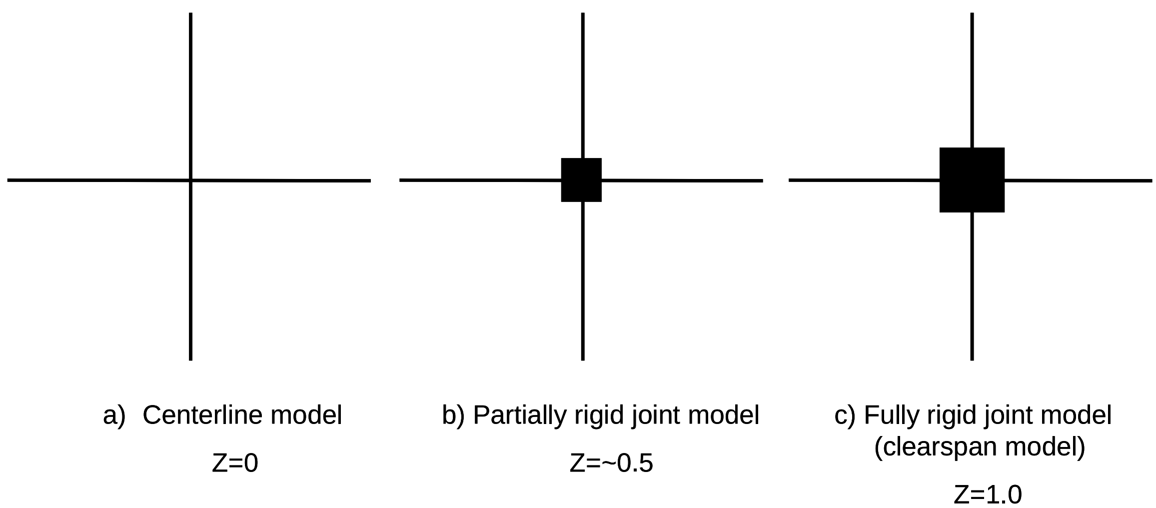

Figure 2.8: Use of the Rigid Endzone Factor

Instead of the simplified models shown in Figure 2.8, a mechanical model of the joint can be used. One such model is the Scissors model. (Details on the development of the

Scissors model are discussed in Section 2.3.2 to which the reader is referred for more information). Both Figure 2.10 and Equation 2.7 are repeated here for convenience. A sample calculation will be carried out to illustrate how the Scissors spring values were obtained for the level 5 moment connections (Appendix A, Figure 3 shows building details) of this particular building.

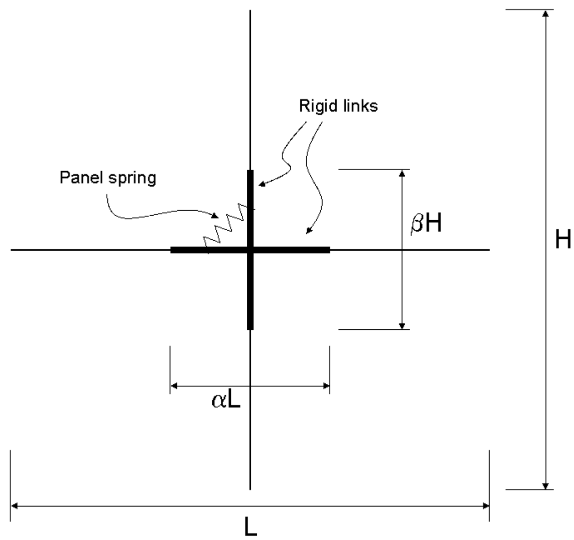

Figure 2.10: The Scissors Model

$$K = \frac{\beta H \alpha L t_p G}{(1-\alpha-\beta)^2} \qquad (2.7)$$

where

K = rotational stiffness of the panel spring (inch-kip/radian)

βH = center of flange to center of flange of beam (inches)

αL = center of flange to center of flange of column (inches)

H = mid-story to mid-story dimension (inches)

L = mid-span to mid-span dimension (inches)

t_p = thickness of the panel zone (inches) (t_p = t_w if no doubler plates)

G = shear modulus (ksi)

Level 5 Properties	**Beams: W24x62**	**Columns: W14x132**
H = 156”	d = 23.7”	d = 14.7”
L = 360”	t_f = 0.59”	t_f = 1.03”
G = 11,200 ksi		t_w = 0.645”

Scissors Spring Calculations

$\beta H = d_b - t_f$ = 23.7” – 0.59” = 23.11”

$\beta = 0.148$

$\alpha L = d_c - t_f$ = 14.7” – 1.03” = 13.67”

$\alpha = 0.038$

$$K = \frac{\beta H \alpha L t_P G}{(1-\alpha-\beta)^2} = \frac{(23.11'')(13.67'')(0.645'')(11{,}200 ksi)}{(1-0.038-0.148)} = 2{,}803{,}640\ \frac{inch \cdot kip}{radian}$$

Rotational springs are placed at each joint in the moment frame and this value was used for the level 5 Scissors joints. Other values were calculated similarly.

To investigate joint modeling, four different models were analyzed. All models include all sources of deformation and P-Delta effects so that the only variable is the way the joint is modeled. Table 3.9 lists the different modeling parameters.

Table 3.9: Modeling Parameters

			Shear Deformations			Axial Deformations		
Model #	**Joint Modeling**	**P-Delta**	**Beams**	**Columns**	**Braces**	**Beams**	**Columns**	**Braces**
3A	Z=1.0	yes	yes	yes	yes	yes	yes	yes
3B	Z=0.5	yes	yes	yes	yes	yes	yes	yes
3C	Z=0.0	yes	yes	yes	yes	yes	yes	yes
3D	Scissors	yes	yes	yes	yes	yes	yes	yes

Because the way the joints are modeled mainly affects the moment frames, only the displacements in the E-W direction will be presented. Displacements in the N-S (braced frame) direction differ slightly from model to model because of the three dimensional interaction but not enough to demand attention. Once again, displacements for all models can be found in Appendix B. Figure 3.5 illustrates the lateral displacements in the E-W direction.

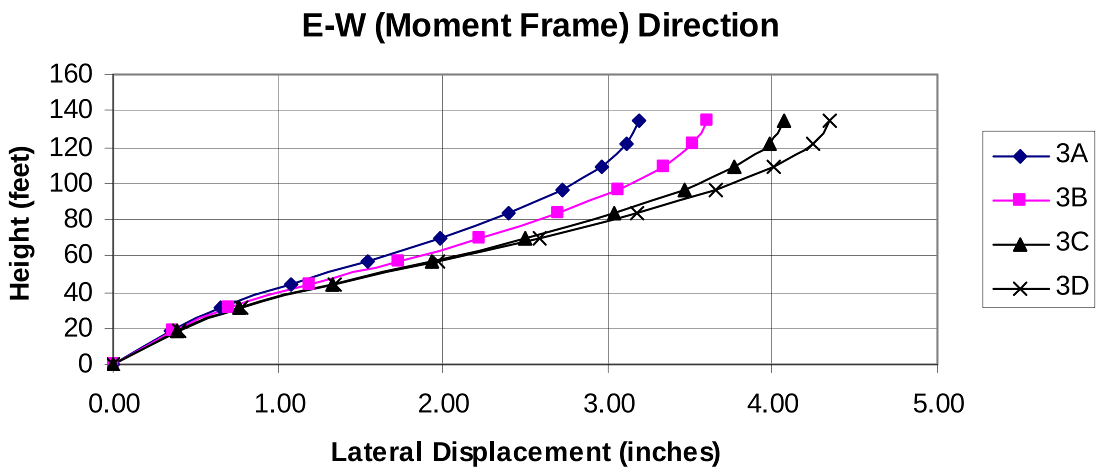

Figure 3.5: Lateral Flexibility in the E-W Direction with Respect to Joint Modeling

Of the modeling parameters looked at so far, it can be seen that the way the beam column joint region is modeled can be very influential.

- With respect to 3A, the fully rigid joint model (Z=1.0), the subsequent models have the following effects
    - Model 3B (Z=0.5) increases lateral displacement by up to 13 percent.
    - Model 3C (Z=0) increases lateral displacements by up to 28 percent.
    - Model 3D (Scissors) increases lateral displacements by up to 36 percent.

3.4.4 Slab-Girder Interaction

Other than allowing the use of a rigid diaphragm constraint at each level, all of the analytical models so far have not considered any effects of the concrete floor slabs on the buildings lateral stiffness. For beam design the slab acts compositely with the steel member in accordance with the number of shear connectors. For this building most beams were 50 to 70 percent composite. The effect of the slab is to increase the positive moment of inertia of the member, thereby providing more flexural resistance. A more detailed discussion of the effects of composite action can be found in Section 2.3.4. Figure 2.11 is repeated here for convenience and the various regions are defined following the figure.

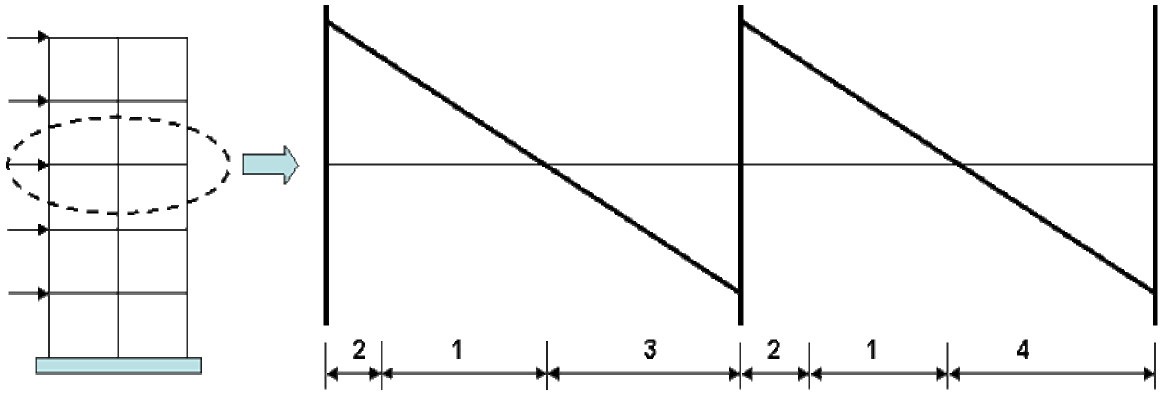

Figure 2.11: Girder Moment Regions

Region 1: Girder is subjected to positive bending moment (concrete slab is in compression) and the full effective slab width can be considered as providing flexural resistance.

Region 2: Adjacent to an exterior windward column or an interior column, the girder is subjected to a positive bending moment but a reduced slab width is used due to the flexural force distribution in the slab. Although the effective slab width must decrease to the width of the column at exterior columns, using the full effective slab width provides nearly identical results (Schaffhausen and Wegmuller 1977). Comparing a full effective width to a width that decreases to the width of the face of the column, results show member forces only differ by 1-2 percent (Vallenilla and Bjorhovde 1985).

Region 3: Girder is subjected to a negative bending moment; slab is generally ignored in calculations of composite flexural strength. Negative slab steel may be present in this region, which differentiates it from region 4. The girder alone provides flexural resistance.

Region 4: The girder is subjected to a negative bending moment and there is typically no negative slab steel at this location (exterior column). Girder alone provides flexural resistance.

The moment frames in the analytical model are exterior frames and three different girder sizes were used: W27x84, W24x62 and W24x55. Figure 3.6 illustrates the typical cross-sectional properties used to calculate the composite moments of inertia for use in regions 1 and 2 of Figure 2.5.

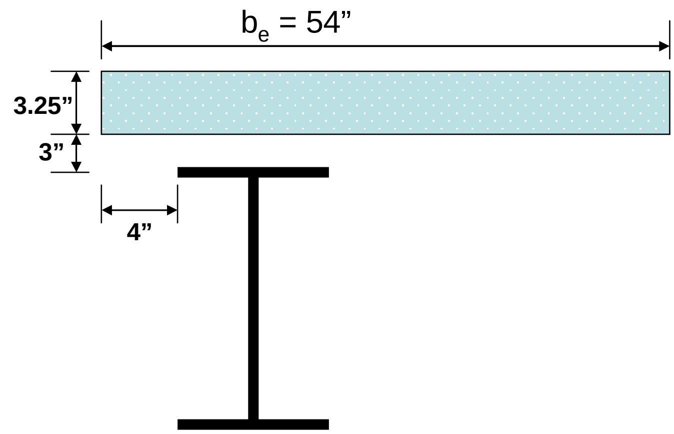

Figure 3.6: Typical Composite Girder Cross-section

Composite section properties were calculated based on the AISC Specifications for Structural Steel Buildings 13th edition (AISC 2005). The section properties were calculated based on the number of shear studs present (percent composite) in lieu of the simplified design tables. Table 3.10 shows the results.

Table 3.10: Composite Moments of Inertia

	Moment of Inertia (inches^4)		
Section	**Beam Alone**	**100% Composite**	**As Designed**
W27x84	2850	5744	4897
W24x62	1560	3719	3087
W24x55	1360	3398	2801

To investigate the effect of the slab acting compositely with the girder, four models were analyzed; two as bare frames and two with the composite moments of inertia applied to the regions of positive bending moments. Although the effective slab width must decrease to the column width at exterior columns, experiments have shown that the differences between accounting for this or simply using the whole effective slab width are negligible (Schaffhausen and Wegmuller 1977) so the whole width was used at all locations. All sources of deformation and P-Delta effects are considered so that the only variable is the composite moment of inertia and the joint modeling used. As with all previous models, the reduced live load value was used. Table 3.11 summarizes the modeling parameters.

Table 3.11: Modeling Parameters

Model #	Joint Modeling	Composite Girder Properties
4A	Z=0.0	no
4B	Scissors	no
4C	Z=0.0	yes
4D	Scissors	yes

Figure 3.7 shows the bare frames (model 4A) moments in the moment resisting frames under the combined gravity and lateral loads. These moment diagrams are used to delineate the various regions in which composite section properties are used.

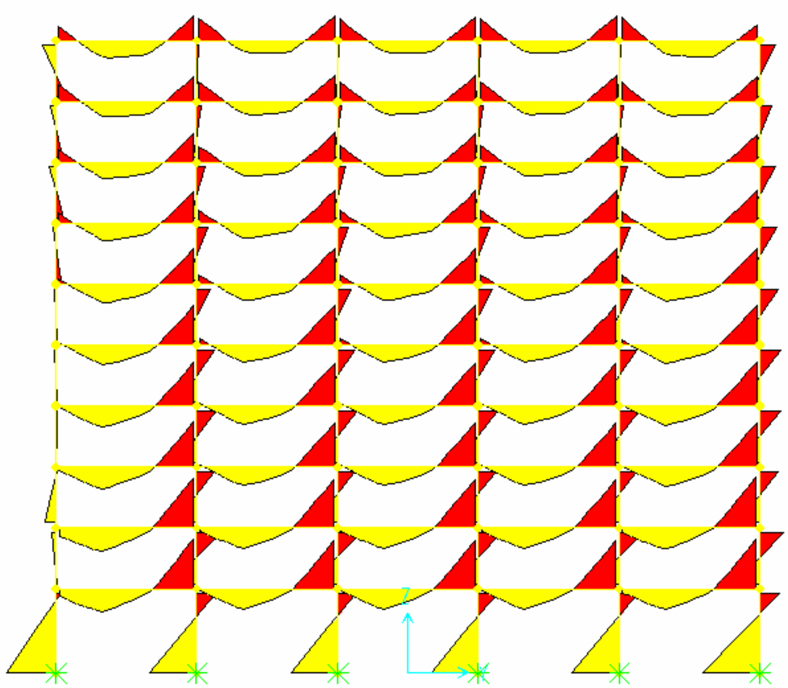

Figure 3.7: Model 4A Moment Diagrams (SAP 2000)

In Figure 3.7 the light colored regions indicate positive bending moments (slab is in compression, therefore composite section properties can be used) while the dark regions indicate negative bending moments (slab is in tension, non-composite properties used). The analysis was performed for loads acting in both the N-S and E-W directions. However, because the effect of the composite girders was predominantly to stiffen the moment frames only the results in that direction will be presented (Figure 3.8). Refer to Appendix B for the results of all analysis cases

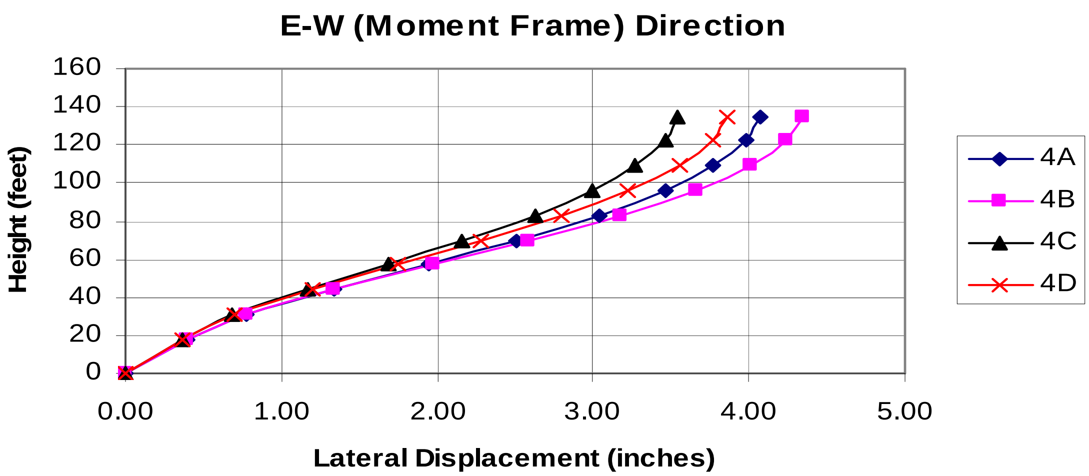

Figure 3.8: Lateral Flexibility in the E-W Direction with Respect to Composite Action

As expected, the increased moments of inertia in the girders of the moment resisting frames increased the overall lateral stiffness of the structure. Lateral displacements decreased by up to 13 percent for the case of Z = 0 and by up to 11 percent for the Scissors models. The composite section properties affected the Scissors model slightly less because to begin with the model was very flexible and more sensitive to P-Delta effects.

3.4.5 Live Loads for Analysis

A variation on the model was to use a floor live load that is more consistent with loads normally seen for the occupancy. For strength design a floor live load of 50 psf was used. This value is based on the maximum expected load for a 50 year reference period.

Additionally a partition live load of 20 psf was also applied to the floors. These loads, reduced as permitted by ASCE 7-05 Section 4.8, have been used in all of the models up to this point.

It is well known that the floor loads measured in a live-load survey usually are well below present design values (Peir and Cornell 1973, McGuire and Cornell 1974, Ellingwood and Culver 1977, Sentler 1975). Table C4-2 in ASCE 7-05 lists mean survey live load values for different occupancy types and for offices the mean value is 10.9 psf with a standard deviation of 5.9 psf. So for the serviceability limit state of wind drift under 10 year wind loads, a floor live load of 10.9 psf was used instead of the 70 psf (reducible per ASCE 7-05 section 4.8) used for the strength design. This difference in loads will impact the lateral displacements due to the second order P-Delta effects.

Four models were considered with the only variables being the joint modeling and the floor live load used. All sources of deformation were included and P-Delta effects were considered. Table 3.12 lists the different models.

Table 3.12: Modeling Parameters

Model #	Joint Modeling	Live Loads Applied (psf)
5A	Z=0.0	70 (reducable)
5B	Scissors	70 (reducable)
5C	Z=0.0	10.9 (survey load)
5D	Scissors	10.9 (survey load)

The resulting lateral displacements in the N-S and E-W directions are presented in Figures 3.9 and 3.10 respectively.

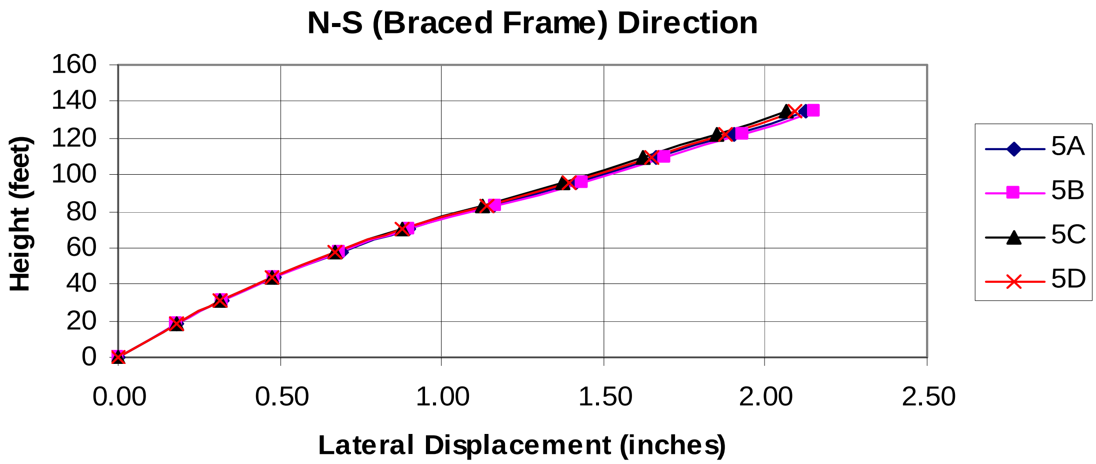

Figure 3.9: Lateral Flexibility in the N-S Direction with Respect to Survey-based Live Loads

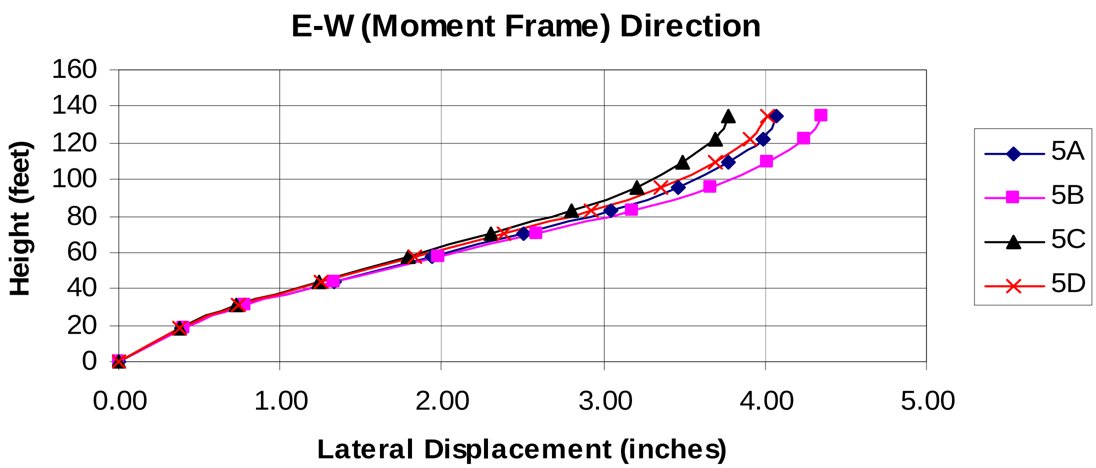

Figure 3.10: Lateral Flexibility in the E-W Direction with Respect to Survey-based Live Loads

Looking at the above figures it is easy to see that the much stiffer braced frames were not very sensitive to the change in loads. This is consistent with Section 3.4.2 in which it was shown that P-Delta effects made much less of a difference on the braced frames than on the moment frames. This is not to say that in general braced frames are less sensitive to P-Delta effects or to reduced loads but it merely demonstrates that braced frames are typically more laterally stiff than their moment resisting counterparts. Given that the braced frames did not deflect laterally very much to begin with, it is to be expected that P-Delta effects and the reduced live loads would not influence the frame significantly.

On the other hand, the much more flexible moment resisting frames were noticeably affected by the reduced live loads. For the case of Z=1.0 (models 5A and 5C), lateral displacements decreased by up to 7.5 percent. For the more flexible case using the Scissor joint representations (models 5B and 5D), lateral displacement decreased by up to 8 percent. In a more flexible structure the effects of using reduced live loads would be even more apparent.

3.4.6 Nonstructural Components

The as-built lateral stiffness of a building can be much greater than anticipated because of the stiffening effect of the nonstructural components (NSC). NSC which are generally ignored in analysis include building façades, masonry infill, and non-load bearing walls/partitions. For more information on NSC the reader is referred to Section 2.3.5 of this thesis.

To account for the stiffening effects of interior walls, membrane elements were added around the interior core of the test building. In the E-W direction membrane elements were placed in every other interior bay. In the N-S direction membrane elements were placed in every interior bay. Several bays were skipped to account for doorways, openings, etc and out of a total of one hundred interior bays, membrane elements were placed in seventy. As the elements were meant to represent drywall, a modulus of elasticity of 290 ksi and a Poisson's ratio of 0.15 was used (Su *et al.* 2005). Walls were assumed to extend from floor to ceiling and the thickness used was 1 inch. All sources of deformation and P-Delta effects were included so that the only variables were joint modeling and whether or not wall elements were modeled, as Table 3.13 shows.

Table 3.13: Modeling Parameters

Model #	Joint Modeling	Non-Stuctural Components
6A	Z=0.0	None
6B	Scissors	None
6C	Z=0.0	Drywall in building core
6D	Scissors	Drywall in building core

Figures 3.11 and 3.12 show the stiffening effect that the membrane elements had in the N-S and E-W directions respectively.

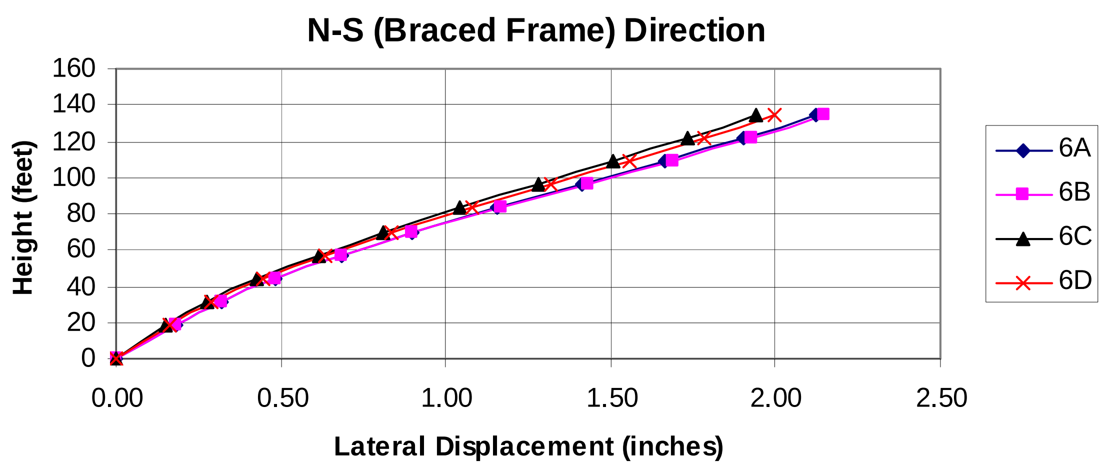

Figure 3.11: Lateral Flexibility in the N-S Direction with Respect to Nonstructural Components

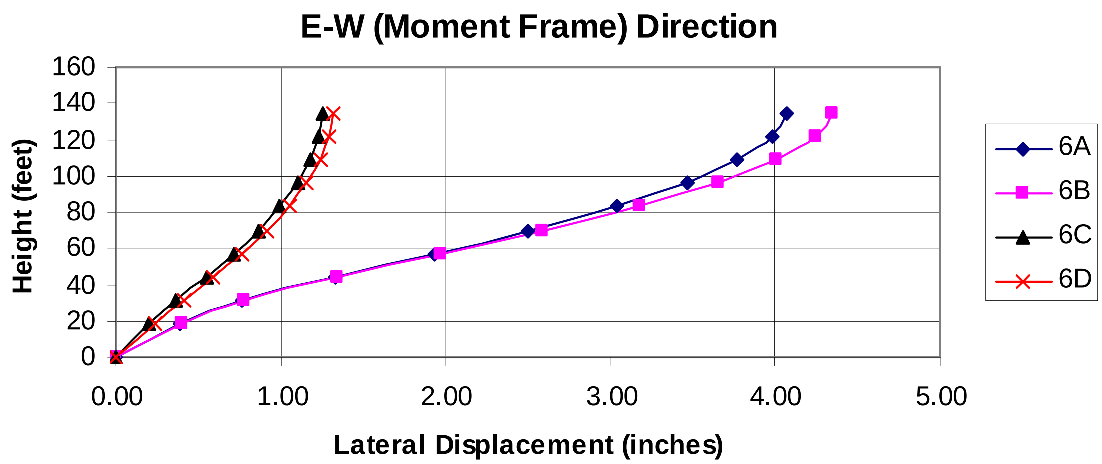

Figure 3.12: Lateral Flexibility in the E-W Direction with Respect to Nonstructural Components

The membrane elements acted as a large shear wall extending the height of the building and provided a considerable amount of lateral stiffness. It can been seen from the above figures that the NSC affected the stiffness in the moment frame direction much more than in the braced frame direction, despite the fact that there were more membrane elements added in the N-S direction (40) as compared to the E-W direction (30). There are two reasons why this was observed:

- Rigid body rotation: because a major source of deformations in braced frames is axial shortening of the columns, bays tend to rotate rigidly.
  - As a result of this, the membrane elements were not subjected to high stresses and did not stiffen the braced frames considerably.
  - Membranes in the E-W direction served to constrain flexural bending and joint rotations, thereby providing a stiffening effect.
- The membrane element's force-deformation relationship. Drywall is a brittle material that loses much of its strength after a certain strain is reached.
  - The model failed to account for this behavior, leading to an overly stiff building.

- Connection details: the walls were assumed to extend from floor to ceiling and were modeled as an integral part of the structural system
    - The way the NSC are detailed is a major factor in how they contribute to the building's lateral stiffness.

To illustrate the rigid body rotation typical in braced frames subjected to lateral loads, a drift damage index or DDI, (Section 2.2.1) as well as an interstory drift index (Section 2.2.1) is calculated for several bays of the braced frame and moment frame of model 6A. Recall that the DDI, through including the vertical component of racking drift, is a more accurate measure of damageability than the interstory drift index. Figure 3.13 shows the bays that were compared for model 6A and Table 3.14 shows the results.

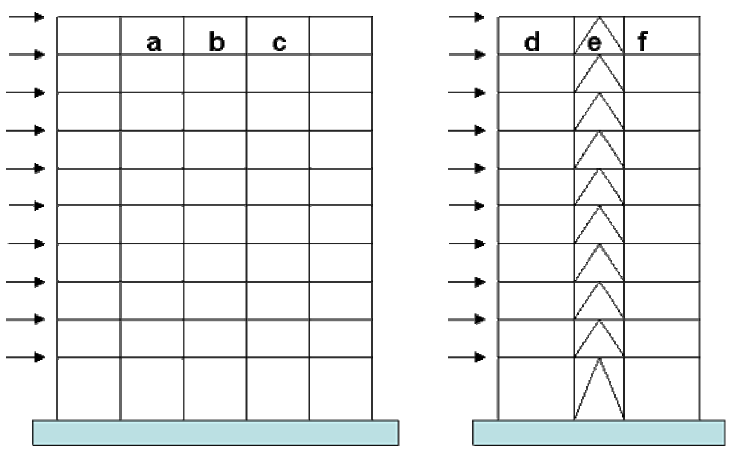

Figure 3.13: DDI vs. Interstory Drift Indices

Table 3.14: DDI vs. Interstory Drift Indices

BAY	Interstory Drift Index (1)	DDI (2)	(2)/(1)
a	0.000571	0.000179	0.31
b	0.000571	0.000983	1.72
c	0.000571	0.000561	0.98
d	0.001423	0.000596	0.42
e	0.001423	0.000767	0.54
f	0.001423	0.003563	2.50

Looking at column 4 of Table 3.14, the ratio of the DDI to the Interstory Drift Index, the following is observed:

- The interstory drift index considerably underestimates drift damage in bays *b* and *f*.
- The interstory drift index overestimates drift damage in bays *a*, *d* and *e*.
- Bay *e* deforms mainly by rotating rigidly which causes little damage in and of itself.

From this example the shortcomings of using an interstory drift index as a measure of drift damageability is apparent. Further details on drift damage indices as well as another example can be found in Section 2.2.1.

It is interesting to compare the interstory drift values for models 6A and 6C to observe the stiffening effects of the NSC in the two different directions. Figures 3.14 and 3.15 below illustrate the effects for the case of Z=0 (models 6A and 6C).

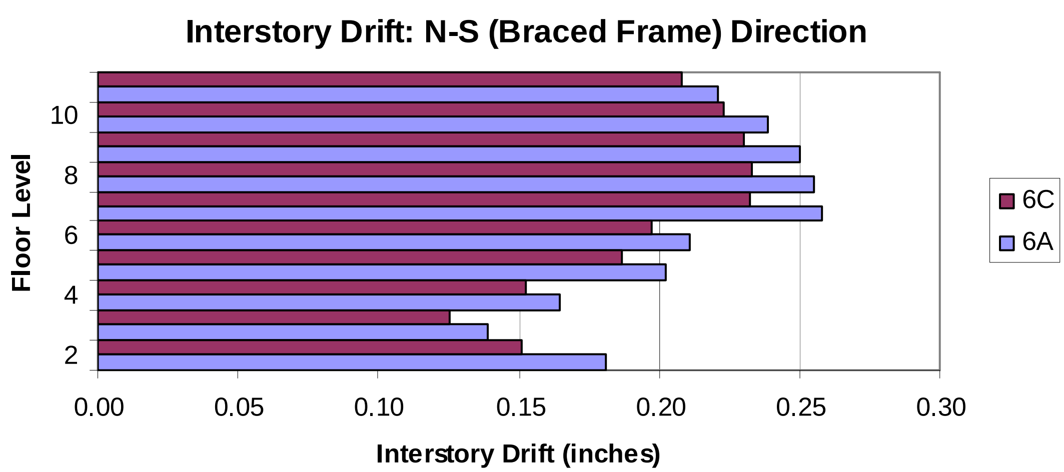

Figure 3.14: Interstory Drift in the N-S Direction

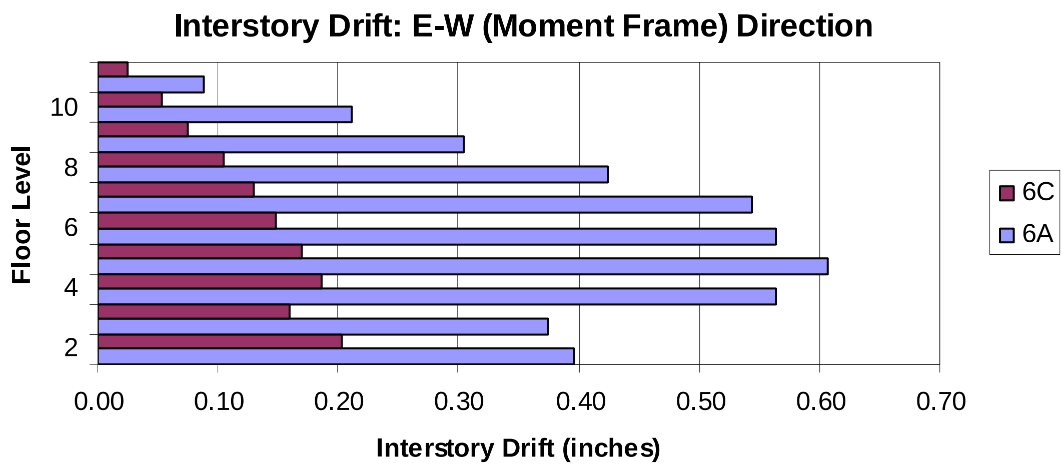

Figure 3.15: Interstory Drift in the E-W Direction

For both joint representations, drift in the N-S direction was reduced by up to 9 percent while drift in the E-W direction was reduced by a surprisingly large 70 percent. This large stiffening effect and the overall reliability of the model will be discussed further in Section 3.5.

3.4.7 Recommended Model

In this section, the recommended model will be presented. This recommended model is based on the author's opinions and reflects the work researched in the literature review. The recommended model accounts for all sources of deformation along with several sources of stiffness and is compared to the most laterally stiff and flexible of the aforementioned models. Table 3.15 summarizes the modeling parameters for these three cases.

Table 3.15: Modeling Parameters

		Shear Deformations			Axial Deformations						
Model #	**Comments**	**Beams**	**Columns**	**Braces**	**Beams**	**Columns**	**Braces**	**P-Delta**	**Joint**	**Live Loads**	**Composite**
1A	Stiffest model	no	no	no	no	no	yes	no	Z=1.0 (rigid)	-	no
3D	Most flexible	yes	yes	yes	yes	yes	yes	yes	Scissors	Full load	no
7A	Recommended	yes	yes	yes	yes	yes	yes	yes	Scissors	Survey load	yes

As observed from the above table the recommended model (model 7A) includes all sources of material deformations in all members, includes P-Delta effects by forming the consistent geometric stiffness matrix for each element, employs the Scissors model to closely replicate actual joint behavior, uses the survey live loads and considers composite action of the girders. Nonstructural components were not included. This is due to the lack of available data to accurately model the force-deformation characteristics of these materials as well as other uncertainties such as connection details. Figure 3.16 illustrates the lateral displacements in the N-S direction.

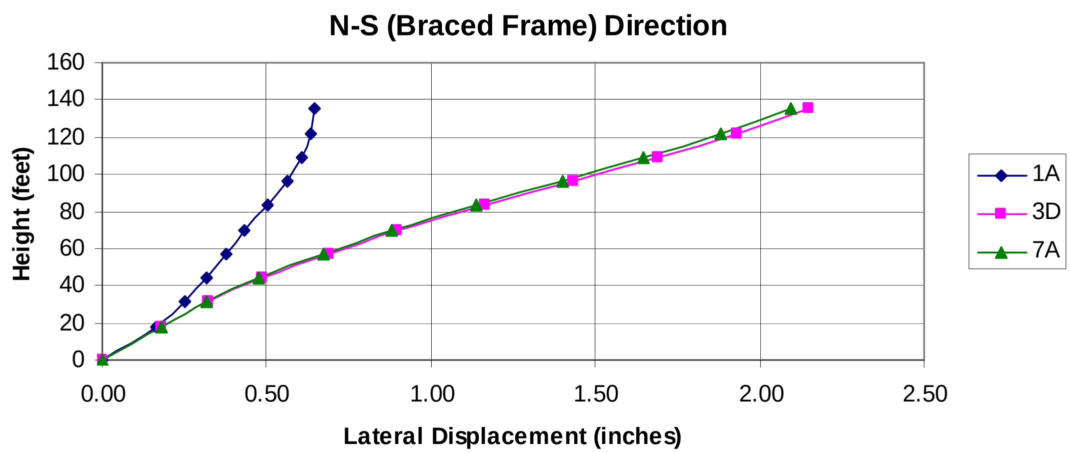

Figure 3.16: Lateral Flexibility in the N-S Direction with Respect to the Recommended Model

It is seen that the recommended model does not deviant significantly from the most flexile model but is much different than the stiffest model. As mentioned earlier, P-Delta effects are not very large for this stiff, braced frame configuration and composite section properties mainly affect the flexural resistance in the girders of the moment frames. The results in the E-W direction are shown in Figure 3.17.

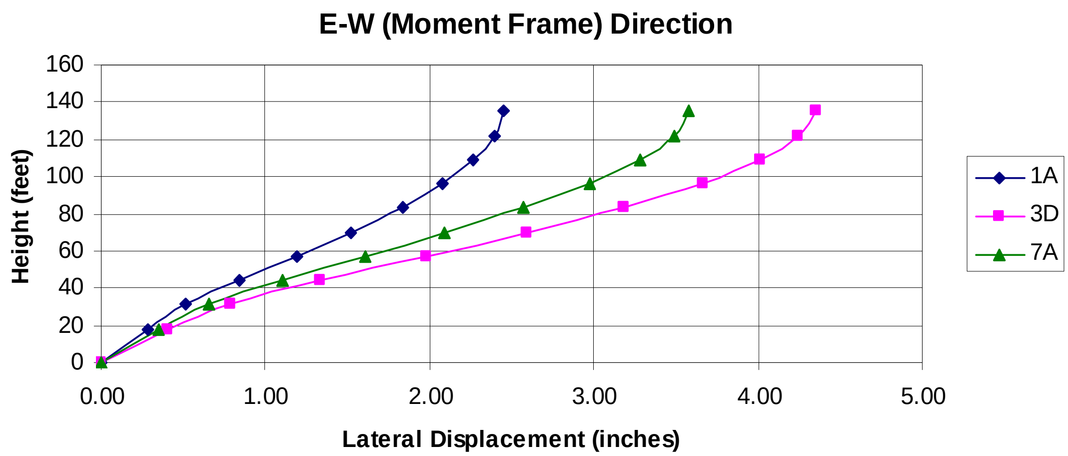

Figure 3.17: Lateral Flexibility in the E-W Direction with Respect to the Recommended Model

In the E-W direction, the recommended model falls right in between the two extremes. P-Delta effects are large in the moment frames but by taking advantage of the composite girders and using the smaller, more risk consistent survey live loads, the lateral displacements are decreased.

To evaluate the recommended building's overall performance the drift damage index or DDI is calculated for several bays. Recall that the DDI, which is simply a measure of the shearing distortion, provides a more accurate measure of building damage due to lateral displacements than the traditionally used interstory drift ratio. First, for the lateral load resisting system, a moment frame and an X-braced frame are presented. Figure 3.18 illustrates the 18 bays whose DDI's and interstory drift ratios are shown in Table 3.16.

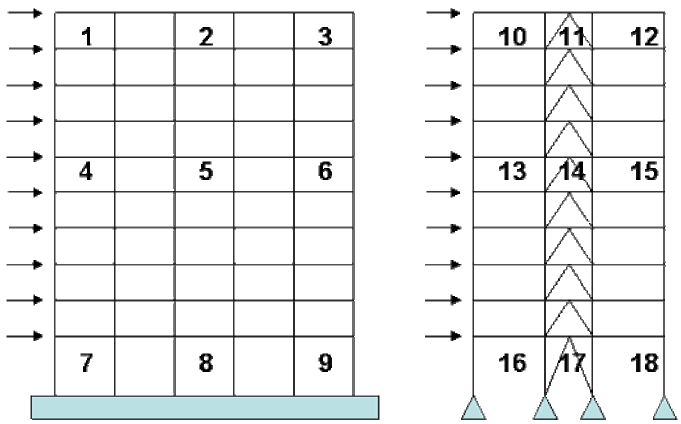

Figure 3.18: Moment Frame and X-Braced Frame

Table 3.16: DDI vs. Interstory Drift Index for the Bays of Figure 3.19

Bay	DDI* (a)	Interstory Drift Index* (b)	Ratio, (a)/(b)
1	0.09	0.55	0.16
2	0.67	0.55	1.20
3	1.15	0.55	2.08
4	2.70	3.02	0.89
5	3.08	3.02	1.02
6	3.42	3.02	1.14
7	1.56	2.22	0.70
8	1.61	2.22	0.72
9	1.67	2.22	0.75

*Values are x 10^{-3}

Bay	DDI* (a)	Interstory Drift Index* (b)	Ratio, (a)/(b)
10	0.35	1.25	0.28
11	0.04	1.25	0.03
12	0.24	1.25	0.19
13	0.25	1.58	0.16
14	0.01	1.58	0.00
15	0.21	1.58	0.13
16	0.04	0.82	0.05
17	0.01	0.82	0.01
18	0.05	0.82	0.06

*Values are x 10^{-3}

The fourth column of each table provides a ratio of the DDI to the interstory drift index. For the moment frame it is observed that the interstory drift index has overestimated damage in four of the nine bays and underestimated damage in four of the nine bays. Only for bay 5 did the interstory drift index provide adequate results. The most serious discrepancy is in bay 3 where the interstory drift index has underestimated damage by a factor of two. Results for the X-braced frame are quite different. The interstory drift index has seriously overestimated damage in all nine of the bays. This is especially true for bays 11, 14 and 17 (the X-braced bays) whose movement is mostly a rigid body rotation, causing no damage in and of itself.

To determine if the DDI's are acceptable Table 2.2, which provides strain limits for various types of partition walls, is consulted. The table is re-printed here for convenience.

Table 2.2: Damage Limits for Several Types of Partition Walls

Wall Material	Strain Limit
Concrete Block	0.005
Brick	0.0025
Hollow-Clay Brick or Tile	0.002
Veneer (drywall, plywood, etc.)	0.007

All bays of the X-braced frame can be considered adequate. For the moment frame, bays 4, 5 and 6 may cause problems depending on the wall material. If a brick material is used then some stiffening of the frame may be in order but if drywall is used, than the bays can be considered adequate.

3.5 Summary and Conclusion

It is now useful to bring all of these preceding models together and make some overall observations regarding modeling assumptions. This will begin with sources of deformation and continue through to nonstructural components.

First of all, given that capability of today's software programs all sources of deformation are generally included by default in the program. In SAP2000 v.9 the only way to "ignore" a deformation source was to use a property modifier to increase the appropriate physical property (shear area for shear deformations, axial area for axial deformations, etc.) and therefore minimize its deformation. As a result, there was no computational advantage to "ignoring" any sources of deformation as the full stiffness matrix was formed for each member. In fact, as a result of increasing the shear and/or axial areas by a factor of 1000, accuracy was actually lost in the solution. Of all of the modeling parameters discussed, it is suspected that sources of deformation, with the exception of those occurring within the joint region, is one that varies very little from firm to firm. In many cases the software dictates what choices a design engineer makes and this may be one of those cases: there is no explicit checkbox option regarding sources of deformation and so the default value of including everything is used. This is a good thing because, given the state of technology, all sources of deformation should be included in any reliable building analysis.

The second investigated parameter was P-Delta effects and it was shown that these effects can have significant consequences. The stiffer braced frames were less sensitive to P-Delta effects but the moment frames were significantly affected. Depending on the software being used, including P-Delta effects may or may not require iterations to reach an answer. SAP 2000 v.9 requires iterations because a geometric stiffness matrix is formed for each element. Programs such as ETABs (Computers and Structures, Inc. 2006) form the linearized geometric stiffness matrix on a story by story basis and therefore do not require iteration to solve (see Section 2.3.7 for more information on P-Delta effects). If equilibrium is considered about the deformed geometry (large displacement analysis) then even more iterations may be required. Once again, given the state of the art including P-Delta effects is not as difficult as it once was.

Joint modeling was investigated next. This is the modeling parameter that probably varies the most from firm to firm and is also a parameter that significantly affects a buildings lateral stiffness. Commercially available software does not currently have built in mechanical models for the joint region and although centerline modeling ($Z = 0$) does provide reasonable results (Section 2.3.2) it is not the correct way to model a joint. As a large source of deformation and of energy dissipation in seismic events, the panel zone region continues to be studied and understood. As a result, more refined joint models along with their integration in commercial software are to be expected in the near future.

Slab girder interaction was the next parameter. The increase in flexural resistance provided by the composite girders was significant. However, modifying members to account for this was not so simple using SAP 2000 v.9. Girders had to be split at inflection points to be able to apply a moment of inertia property modifier to only the sections subjected to positive bending (slab in compression). It would be convenient if conditional property modifiers could be incorporated into the software.

Survey based live loads were influential in reducing overall building drift. Because wind drift is a serviceability issue it is not unreasonable to use reduced live loads. If reduced live loads are used it is more reasonable to use a load that is based on field survey results, such as those provided in Table C4-2 of ASCE 7-05, than using a blanket reduction factor. Using reduced, survey based live loads is an engineering decision that may also relate to the buildings present as well as future functions in addition to the owner’s needs, and should therefore be carefully considered.

The last parameter that was investigated was the stiffening effect provided by nonstructural components. Although a simple example, the models in 3.4.6 along with the research that was discussed in 2.3.5 demonstrate both the large stiffening effect of NSC as well as analytical modeling difficulties. In model 6 for example, the amount of lateral stiffness provided by the drywall in the E-W direction was entirely unreasonable even though the material properties were correct, if not underestimated (such as a wall thickness of 1 in.). The best measure of the stiffening effects of the nonstructural components may very well entail forced vibration field measurements of modal periods. Although several of these have been conducted and are discussed in Chapter 2, more work is needed in this area as no amount of analytical modeling can replace full-scale testing.

Finally the author’s recommended model was presented. This model included all sources of material deformations, used composite girder section properties and employed survey based live loads. Seeing as damage due to wind drift is a serviceability limit state, it is desirable to model the building as realistically as possible. With the exception of the stiffening effects of nonstructural components and the rotational stiffness provided by “pinned” connections, it is felt that this model quite accurately represents true building behavior.

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Chapter 4

Sources of Deformation: An Analytical Study

4.1 Introduction

Chapter 2 of this thesis referenced a paper by Charney (1990) titled "Sources of Elastic Deformation in Laterally Loaded Steel Frame and Tube Structures" which was presented at the *Council on Tall Buildings and Urban Habitat, Fourth World Congress* in 1990. In addition to providing useful information on sources of deformation in steel frames, the results from the analyses presented within this chapter will be used to update and republish this paper in a more widespread and accessible structural engineering periodical.

4.2 Overview

In Chapter 2 of this thesis the various sources of deformation in steel framed building were discussed. These sources include flexural, axial and shear deformations in the columns, beams and beam-column joint. An additional component is torsional deformation, which is usually quite small and negligible for regular buildings. This chapter of the thesis involves the modeling and analysis of 27 planar steel frames and 18 framed steel tube structures to determine the various sources of deformation under lateral loads. Axial, shear and flexural deformations are compared with respect to the modeling variables of building height, bay width and the number of bays, and method of joint modeling. It is shown that the various contributions are heavily dependent upon the height of the building and the width of the bays.

To quantify the various sources of deformation, a virtual work method is used; this will be discussed first, in Section 4.3. Next, the frame and tube structures as well as the lateral loads will be discussed in Section 4.4, Description of Analysis. Then the results of the analyses will be presented and discussed in Section 4.5. An overall conclusion follows in Section 4.6.

4.3 Method of Analysis

In the study within, the principle of virtual work is employed to quantify the various sources of deformation. Virtual work is commonly taught as a method to calculate a displacement. Consider the simple, statically determinate truss of Figure 4.1 which resists lateral loads through axial deformations. Concerning node 12, the top right node of Figure 4.1, there are several ways to calculate it's displacement under the "real loads".

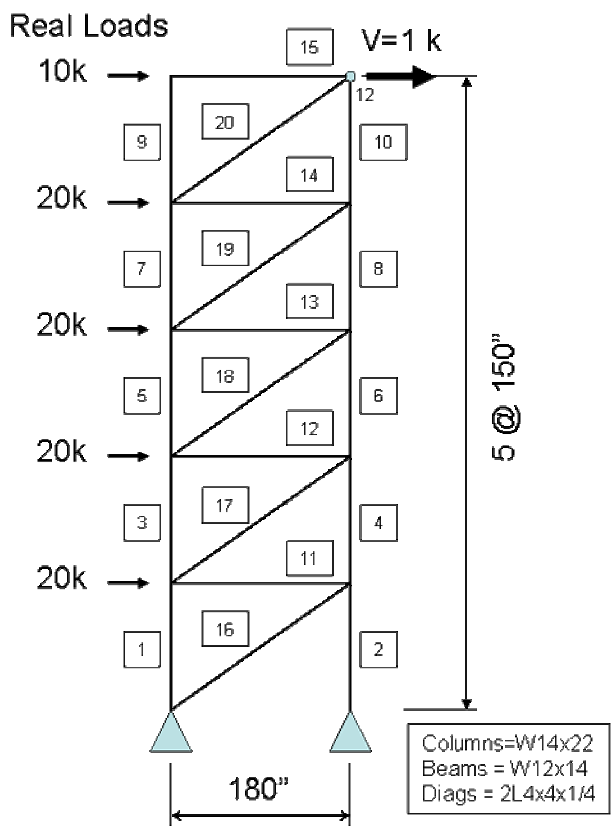


Figure 4.1: Five Story Planar Truss

Stiffness matrices for each element could be formed, assembled into the full structural stiffness matrix and displacements solved for that way. This is the way a computer analysis program would obtain the solution. Without a computer, the easiest hand method of solution would is to use the principle of virtual work. Because the truss is statically determinate, member forces are easily obtained through equilibrium under two loading cases, referred to as the *real loads* and the *virtual loads*. Referring to Figure 4.1 the *virtual load* is applied at node 12 and in the direction of the desired displacement (in this case the direction is obvious; if not, an assumption is made which is then verified or discredited by the sign of the calculated displacement). Virtual work is covered in most basic structural analysis texts; the basic formula for a planar truss, such as the one in Figure 4.1, is now presented in Equation 4.1.

$$Fv * \Delta r = \sum_{i=1}^{m} Fv_i * \delta r_i \tag{4.1}$$

where

Fv = the virtual force applied (in this case, the 1 k load at node 12)

Δr = the unknown real displacement which is desired (at node 12 in this case)

Fv_i = the axial force in member *i* due to the virtual load

$\delta r_i = \dfrac{Fr_i * L_i}{A_i * E}$ = the axial elongation in member *i* due to the real loads

m = total number of truss members

Each truss member contributes, through axial compression or elongation, to the total displacement at node 12 under the *real loads*. The term *displacement participation factor* is used to refer to an individual element's contribution to the total displacement at a specific point in a structure, under a specific load. The right hand side of Equation 4.1 is the summation of each member's *displacement participation factor* or *DPF* for short and can be rewritten as such:

$$Fv * \Delta r = \sum_{i=1}^{m} DPF_i \tag{4.2}$$

Table 4.1 summarizes the calculations for each member of the truss in Figure 4.1 using Equation 4.1.

Table 4.1: Virtual Work Calculations for the Truss of Figure 4.1

MEMBER	Fr (kips)	Fv (kips)	A (in.^{2})	L (in.)	δr (in.)	DPF (in.-kips)	volume (in.^{3})	SI*10,000
1	133.33	3.3333	6.49	150	0.0027	0.3542	973.50	3.64
2	-208.88	-4.1667	6.49	150	-0.0033	0.6936	973.50	7.13
3	75	2.5000	6.49	150	0.0020	0.1494	973.50	1.54
4	-133.33	-3.3333	6.49	150	-0.0027	0.3542	973.50	3.64
5	33.33	1.6677	6.49	150	0.0013	0.0443	973.50	0.46
6	-75	-2.5000	6.49	150	-0.0020	0.1494	973.50	1.54
7	8.33	0.8333	6.49	150	0.0007	0.0055	973.50	0.06
8	-33.33	-1.6667	6.49	150	-0.0013	0.0443	973.50	0.45
9	0	0.0000	6.49	150	0.0000	0.0000	973.50	0.00
10	-8.33	-0.8333	6.49	150	-0.0007	0.0055	973.50	0.06
11	-90	-1.0000	4.16	180	-0.0015	0.1343	748.80	1.79
12	-70	-1.0000	4.16	180	-0.0015	0.1044	748.80	1.39
13	-50	-1.0000	4.16	180	-0.0015	0.0746	748.80	1.00
14	-30	-1.0000	4.16	180	-0.0015	0.0448	748.80	0.60
15	-10	0.0000	4.16	180	0.0000	0.0000	748.80	0.00
16	117.15	1.3017	3.88	234.31	0.0027	0.3176	909.12	3.49
17	91.12	1.3017	3.88	234.31	0.0027	0.2470	909.12	2.72
18	65.09	1.3017	3.88	234.31	0.0027	0.1764	909.12	1.94
19	39.05	1.3017	3.88	234.31	0.0027	0.1059	909.12	1.16
20	13.02	1.3017	3.88	234.31	0.0027	0.0353	909.12	0.39
					SUM =	3.0408		
Node 12 Displacement = 3.04/1.0 = 3.04 inches								

Referring to Table 4.1 it is observed that the *DPF* for member 1 is 0.3542 in-kips. Dividing through by the virtual load of 1 k the *DPF* is 0.3542 in. and therefore member 1 accounts for $\frac{0.3542}{3.04} * 100\% = 11.65\%$ of the real load total roof drift at node 12. Another way to look at how much a member contributes is to calculate its *sensitivity index* or *SI* (Charney 1993). The *SI* of a member is simply its *DPF* divided by its volume. If it is desired to reduce the displacement at the virtual load, adding material to the element with the largest *SI* would be the most efficient use of material. In other words, the most efficient way to decrease the displacement at node 12 in the truss example is to add material to member 2 because this member has the largest *SI*. To optimize the entire structure, each members *SI* should be exactly the same. Given the constraints of available member sizes, alternate loading directions and load cases, this is obviously unrealistic for a real structure. However, the basic principles are commonly used in overall structural optimization applications, discussed in Chapter 2 of this thesis.

The truss example was presented for its simplicity as only axial forces are present in each element. However, *DPF's* can also be computed for any two or three dimensional frame element; Equation 4.3 presents the basic formula for calculating the *DPF* for any type of frame or truss member.

$$DPF_i = \int_{vol} \sigma v_i * \varepsilon r_i \quad dv \tag{4.3}$$

where

σv_i = stress in member *i* due to the *virtual loads*

εr_i = strain in member *i* due to the *real loads*

vol = volume of member *i*

The stresses and strains in a frame member are due to flexural, axial, shear and torsional forces. Equation 4.3 can be rewritten as the summation of these components:

$$DPF_i = Fmaj_i + Fmin_i + Vmaj_i + Vmin_i + P_i + T_i \tag{4.4}$$

where

$Fmaj_i$ = flexural component for major axis bending

$Fmin_i$ = flexural component for minor axis bending

$Vmaj_i$ = component of shear for forces parallel to the major axis

$Vmin_i$ = component of shear for forces parallel to the minor axis

P_i = axial component

T_i = torsional component

The structures analyzed within this chapter have horizontal nodal loads at each level and no member loads. The resulting member force distributions are linear, greatly simplifying the *DPF* calculations. As an example, consider the 3-story planar frame in Figure 4.2 which is subjected only to horizontal nodal loads.

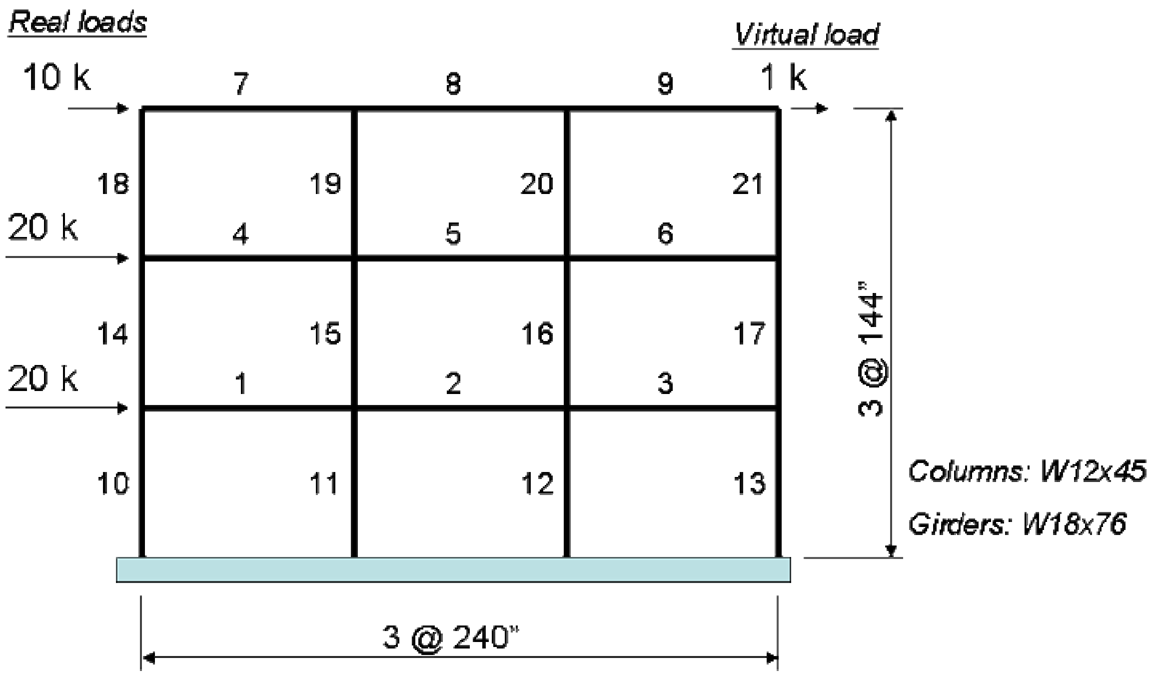


Figure 4.2: 3-Story Planar Frame

The moment and shear force distribution in any girder due to the real loads is similar to what is shown in Figure 4.3(b). A more accurate force distribution for the joint region is shown in Figure 4.3(a), which has been verified by detailed finite element models (Charney 1985).

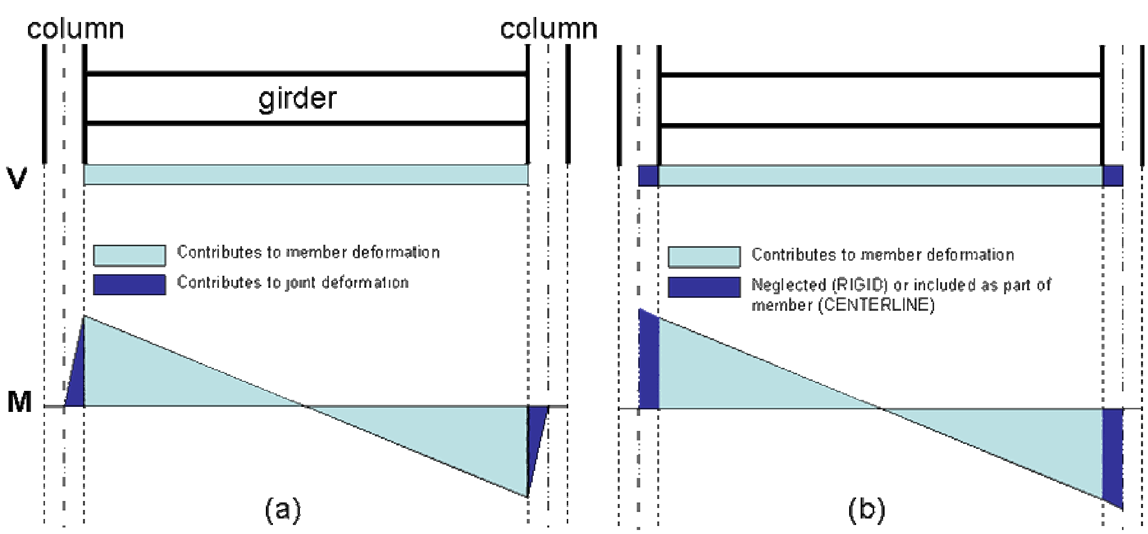


Figure 4.3: Modified (a) and Traditional (b) Girder Shear and Moment Distributions

The forces at the face of the joint are found by analyzing the model with the joints being considered as fully rigid (Z =1.0). Deformation in the joint region is due to a flexure component from the beam and girder and a single shear component from the column. Looking at Figure 4.3(a) it can be seen that there is no girder shear contribution in the joint region. This is accounted for by the shear from the column. Figure 4.4 shows the traditional and modified force distribution for a typical column in the frame of Figure 4.2.

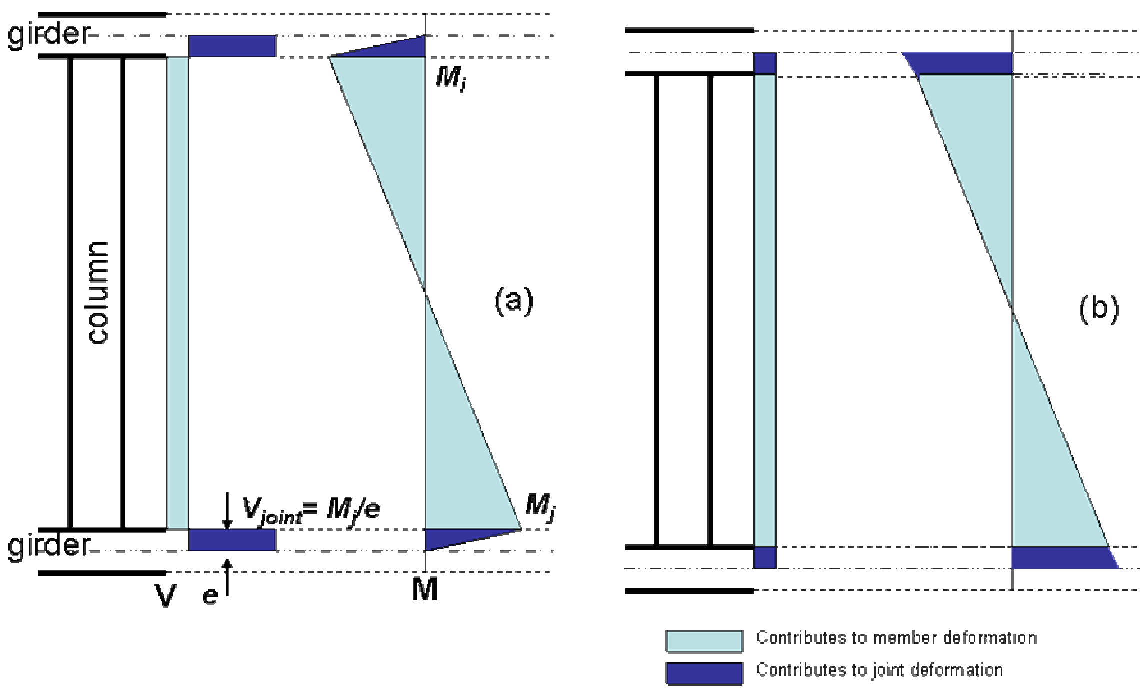


Figure 4.4: Modified (a) and Traditional (b) Column Shear and Moment Distributions

The column joint shear is calculated by equilibrium in the joint and is equal to the moment at the face of the joint divided by the distance e shown in Figure 4.4(a). In the following analyses, the beam-column joint deformation is composed of the beam and column joint flexure as well as the column joint shear. Axial deformations in the joint region are included in the columns and the beams.

4.3.1 3-Story Frame Example

The frame of Figure 4.2 will now be taken as an example of how *DPF*'s for a typical frame in the study are calculated. If the displacement at the top right corner of the frame under the real loads is desired, a virtual load of 1 k is placed at that point and in the suspected direction of displacement, which in this case is obvious. First, member forces are calculated under both the real and virtual loading cases assuming the joints are fully rigid (Z = 1.0). In the following discussion, member forces due to real loads will be shown by capitalized letters and member forces due to the 1 k virtual load will be represented by lower-case letters. Beginning with axial forces, the *DPF* due to axial forces in any member can be calculated using Equation 4.3. Because both the area and the member axial forces are constant over the length, with substitution the equation simplifies to the following expression for a single member:

$$DPF_{AXIAL} = \frac{P * p}{A * E} * L \qquad (4.5)$$

where

P = member axial force due to the real loads

p = member axial force due to the virtual load

A = cross-sectional area

L = length of the member, centerline dimensions

E = modulus of elasticity

Shear deformations in the members are due to the major and minor components. For a planar frame with member axes oriented in the strong direction for bending, there are only major components. The shears and shear areas are constant over the length of the member and Equation 4.3 can be simplified to the following expression for a single member:

$$DPF_{SHEAR} = \frac{V * v}{A_v * G} * L_{clear} \tag{4.6}$$

where

V = member shear due to the real loads

v = member shear due to the virtual load

A_v = shear area

L_{clear} = length of member, clearspan dimensions

G = shear modulus of elasticity

Flexural *DPF's* are calculated next. As with shear, there is both a major and minor component but in a planar frame with member axes oriented in the strong direction for bending, there is only the major component. Flexural *DPF*'*s* must be calculated by integrating Equation 4.3 because the moments are not constant over the length. Rewriting Equation 4.3 and assuming a constant moment of inertia along the length gives the following expression for a single member (note that the integration is over the clearspan length of the member and does not include the joint region):

$$DPF_{FLEXURE} = \int_0^{L_{clear}} \frac{M(x) * m(x)}{E * I} \, dx \tag{4.7}$$

where

$M(x)$ = moment due to the real loads

$m(x)$ = moment due to the virtual load

I = moment of inertia

E = modulus of elasiticity

For the joint region, the *DPF*'s are calculated similarly, using the modified member force distributions shown in Figure 4.3(a) and 4.4(a). A girder will have a joint flexure component at both ends but no joint shear component. A column framing into a girder at both ends will have a joint flexure and joint shear component at both ends. Table 4.2 summarizes the calculations for the 3-story frame of Figure 4.2.

Table 4.2: Virtual Work Calculations for the Frame of Figure 4.2

MEMBER DPF * 1000							
MEMBER	**AXIAL**	**SHEAR**	**FLEXURE**	**JOINT_F**	**JOINT_V**	**TOTAL**	**% of TOTAL**
1	-0.149	4.855	48.900	2.520	0.000	56.126	5.2
2	0.007	2.500	24.200	1.281	0.000	27.988	2.6
3	0.065	4.771	48.000	2.476	0.000	55.312	5.1
4	0.007	2.336	23.200	1.208	0.000	26.750	2.5
5	-0.050	1.560	15.100	0.799	0.000	17.409	1.6
6	-0.030	2.431	24.200	1.258	0.000	27.860	2.6
7	-0.605	0.339	3.477	0.177	0.000	3.388	0.3
8	-0.927	0.252	2.442	0.129	0.000	1.896	0.2
9	-0.536	0.381	3.943	0.200	0.000	3.987	0.4
10	2.804	7.764	55.800	2.539	20.700	89.606	8.2
11	0.154	11.300	77.000	4.482	36.600	129.536	11.9
12	0.169	11.200	76.100	4.429	36.100	127.998	11.8
13	2.866	7.536	54.200	2.463	20.100	87.165	8.0
14	0.649	3.126	18.400	2.651	21.600	46.426	4.3
15	0.019	7.918	46.300	6.706	54.700	115.642	10.6
16	0.026	7.905	46.300	6.695	54.600	115.525	10.6
17	0.690	3.091	18.100	2.619	21.400	45.900	4.2
18	0.048	0.920	5.842	0.801	6.540	14.152	1.3
19	0.000	2.613	15.500	2.224	18.100	38.438	3.5
20	0.001	2.752	16.300	2.341	19.100	40.494	3.7
21	0.054	1.082	6.772	0.938	7.652	16.497	1.5
SUM	5.263	86.631	630.075	48.935	317.191	1088.096	100.0
% of TOTAL	0.484	7.962	57.906	4.497	29.151	100.000	
Displacement at top right node = 1088.096/1000 = 1.088 inches							

Table 4.2 gives the *DPF's* for each member in terms of the five different components. Notice that there is no joint shear component for members 1 through 9. These members are girders and this is consistent with the modified forces shown in Figure 4.3(a). Also shown is the percent of the total for each member and for each source of deformation. Looking at the sources of deformation it is clear that flexure is dominant followed by joint deformations. It is important to note that a computer analysis program which considers the joint region as fully rigid (Z = 1.0) would neglect the columns labeled JOINT_F and JOINT_V in Table 4.2 and the reported displacement would be given as $\frac{1088.096 - 48.935 - 317.191}{1000} = 0.721\, in.$, a significant underestimation of the actual displacement. Considering the joint as fully flexible (Z = 0) results in a displacement of

0.891 in. which is still underestimating the displacement calculated using the modified force distributions. The studies presented within further demonstrate these errors.

4.3.2 Computer Software

The modified force distribution is employed in the program KeySolver (Pathak 2004) which was used to perform the analysis of the frames contained herein. KeySolver is a program that uses the principle of virtual work described above to output the various *DPF's* for each member of a two or three dimensional frame structure. KeySolver calculates member forces assuming the joints are fully rigid (Z = 1.0). The program is able to calculate the *DPF's* using the either the modified or the traditional force distributions shown in Figures 4.3 and 4.4. Element *DPF's* are reported for beams, columns and the joint region in terms of major and minor flexure and shear, axial, and torsional components. The program SMGP (Pathak 2004) was used to generate all of the models for input into KeySolver. SMGP is a simple program that generates a frame given the overall dimensions, number of bays, width of the bays and the story heights.

4.4 Description of Analysis

The analytical study involved modeling and analyzing 27 planar frames and 18 framed tube structures. Variables studied include the building height, the width and number of bays, and the method of joint modeling. All models have a consistent story height of 12.5 ft. Table 4.3 summarizes the models.

Table 4.3: Summary of the Models

Number of Bays	Bay Width	Number of Stories	Type
5	10 ft	10	frame
9	15 ft	20	frame
13	20 ft	30	frame
		40	tube
		50	tube

All members used are commonly available W sections. Columns were all W14's while the beams ranged from W24's to W40's. Member properties varied every five stories and for each series of analyses (constant number of stories and number of bays) the member property distribution remained the same and only the bay spacing was changed. This was done to simplify the design and analysis of so many frames. For the 40 and 50 story framed tubes, it was assumed that the number of bays on the flange side was equal to the number of bays on the web side.

The lateral loads applied were wind loads obtained using ASCE 7-05, Method 2 (ASCE 2005). A 10 year MRI wind, with a 3 second gust velocity of 76 mph (basically anywhere in the Midwest) and an Exposure Category C was used. Total building drift under the wind loads ranged from H/200 to H/1000 where H = total building height.

4.5 Results of the Analysis

Table 4.4 shows the results of all the models in terms of percent flexure, axial, shear and joint for each case. Recall that member forces were found assuming fully rigid joints (Z = 1.0) and the modified force distribution of Figure 4.3(a) and 4.4(a) was then used to determine the joint contributions.

Table 4.4: Analysis Summary

	10 STORY PLANE FRAMES								
	# of 10 ft bays			# of 15 ft bays			# of 20 ft bays		
	5	9	13	5	9	13	5	9	13
% Flexure	41.2	43.8	44.6	49.3	50.7	51.1	55.1	56.0	56.3
% Axial	10.3	4.8	2.8	4.7	2.2	1.0	2.6	1.1	0.1
% Shear	14.3	14.9	15.2	11.3	11.5	11.6	9.3	9.4	9.5
% Joint	34.2	36.4	37.4	34.7	35.6	36.3	32.9	33.5	34.1

	20 STORY PLANE FRAMES								
	# of 10 ft bays			# of 15 ft bays			# of 20 ft bays		
	5	9	13	5	9	13	5	9	13
% Flexure	31.3	35.7	39.1	42.8	42.9	45.7	50.4	48.1	50.9
% Axial	29.0	15.5	8.5	14.3	7.7	4.4	8.2	4.8	2.7
% Shear	12.1	14.2	14.9	10.7	12.1	11.8	9.2	10.5	9.9
% Joint	27.6	34.6	37.5	32.1	37.2	38.1	32.1	36.7	36.5

	30 STORY PLANE FRAMES								
	# of 10 ft bays			# of 15 ft bays			# of 20 ft bays		
	5	9	13	5	9	13	5	9	13
% Flexure	22.8	31.8	36.9	36.5	42.5	45.5	46.1	49.7	51.5
% Axial	46.0	23.9	15.1	25.4	12.0	7.5	15.2	7.3	4.6
% Shear	11.0	15.3	13.7	11.0	12.9	11.2	9.9	10.8	9.3
% Joint	20.2	29.0	34.2	27.1	32.6	35.8	28.9	32.2	34.5

	40 STORY FRAMED TUBES								
	# of 10 ft bays			# of 15 ft bays			# of 20 ft bays		
	5	9	13	5	9	13	5	9	13
% Flexure	20.5	29.7	36.8	30.7	34.9	42.7	37.6	38.0	47.0
% Axial	46.3	22.8	10.7	26.9	12.1	5.9	17.1	7.8	4.1
% Shear	13.0	14.8	16.1	13.9	14.4	13.4	13.3	13.6	11.5
% Joint	20.2	32.6	36.3	28.6	38.6	38.1	32.1	40.5	37.4

	50 STORY FRAMED TUBES								
	# of 10 ft bays			# of 15 ft bays			# of 20 ft bays		
	5	9	13	5	9	13	5	9	13
% Flexure	16.3	26.2	34.9	27.0	33.1	41.3	35.2	37.3	45.9
% Axial	57.6	31.1	14.9	36.3	16.8	8.0	23.7	10.7	5.4
% Shear	10.2	14.2	15.8	12.1	14.3	13.5	12.1	13.7	11.7
% Joint	15.9	28.4	34.4	24.6	35.7	37.2	29.0	38.3	37.0

To better illustrate the findings, the bar charts of Figure 4.5 are presented, which graphically show 20 of the 45 analysis cases. The total drift is broken down into percent flexural, axial, shear and joint (which include both joint shear and joint flexure) which all sum to 100 percent. The horizontal axis represents the number of stories so that the change in deformation sources with respect to building height can be observed for a constant number and width of bays. The bar charts of Figure 4.6 show the same data but in a different way. Flexural, axial, shear and joint are shown on the horizontal axis so that the various contributions can be observed. Once again, the vertical axis shows the percent of the total roof drift that that deformation source contributes.

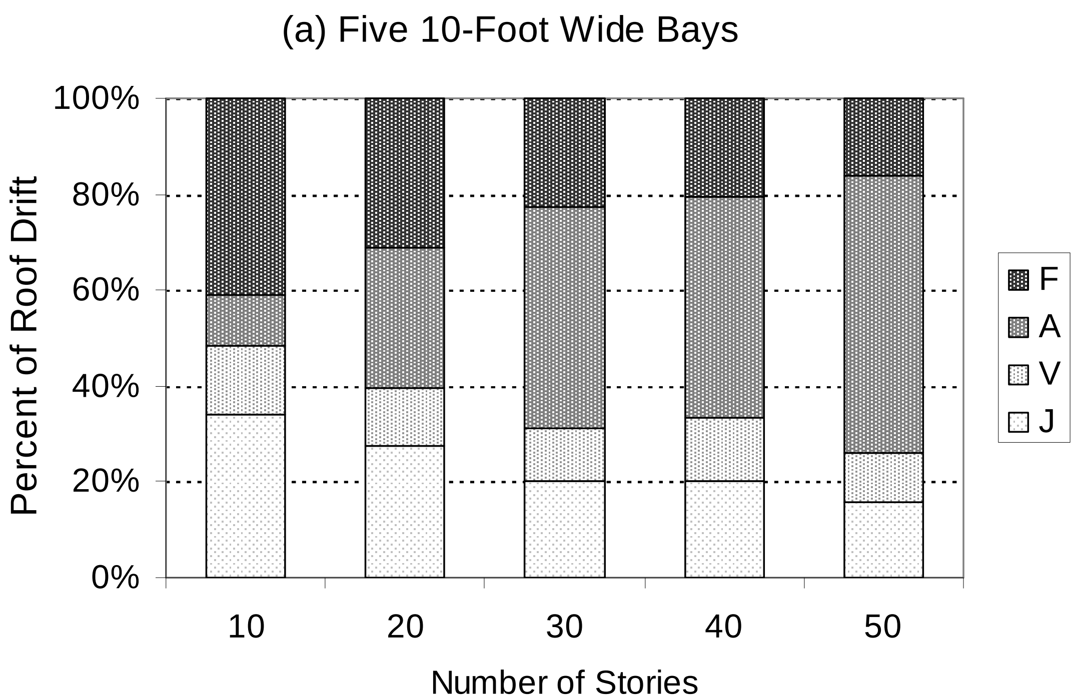
4.5.1 Axial and Flexural Deformations

As expected, the axial deformations are dominant for the narrow (5 bays at 10 ft) frames and nearly negligible for the wide (13 bays at 20 ft) frames. This is consistent with the recommendations given by the ASCE task committee following the 1988 Wind Drift Survey (ASCE 1988). In all cases there is a trend for the axial deformations to increase with building height. The maximum axial participation is 58 percent for the 50 story framed tube with five 10 ft bays (with a slenderness ratio of 12.5 it is the tallest and most narrow of the 45 models).

Flexural deformations are inversely proportional to the axial deformations. The wider, shorter frames experience significant flexural deformations which decrease with respect to increasing height. As expected, the maximum contribution of 56 percent was observed in the 10 story frame with ten 20 ft bays (the widest, shortest frame). This model also experienced the least percent axial at just 0.1 percent.

4.5.2 Shear Deformations

In general the shear deformations tend to increase with the number of bays but decrease with the width of the bays. The framed tubes show a higher percentage of shear deformation than the planar frames due to the shear lag effect. In any case, the shear deformations contribute significantly to the lateral displacement and cannot rationally be discounted in any of the models. The minimum contribution is 9 percent and the maximum is 16 percent.



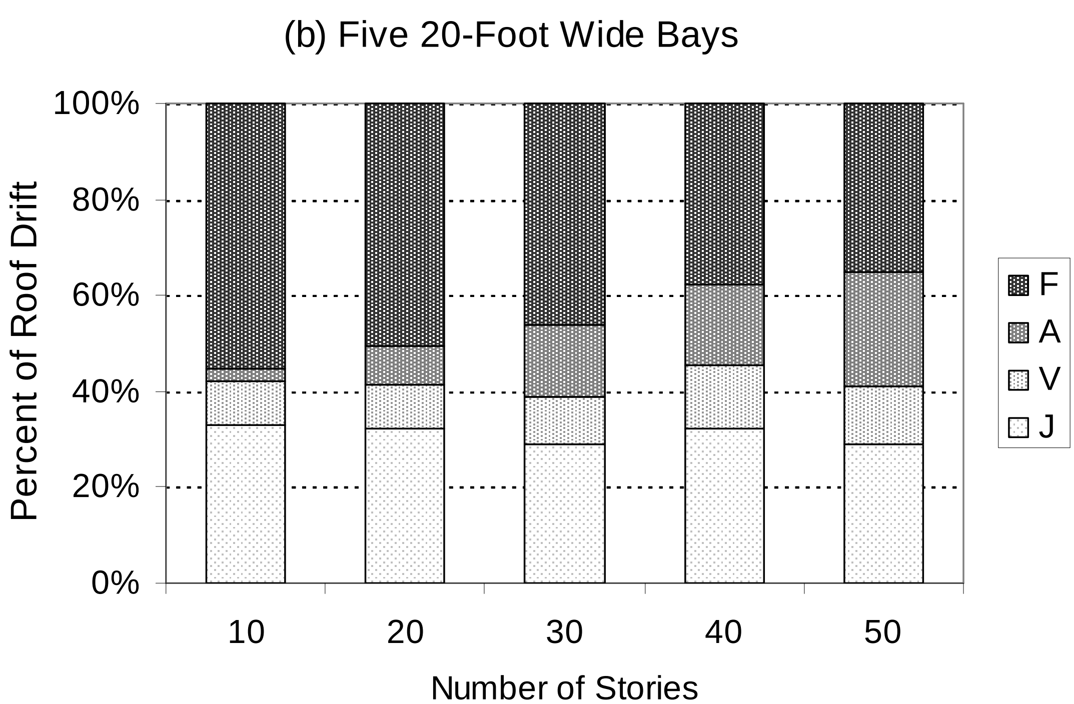
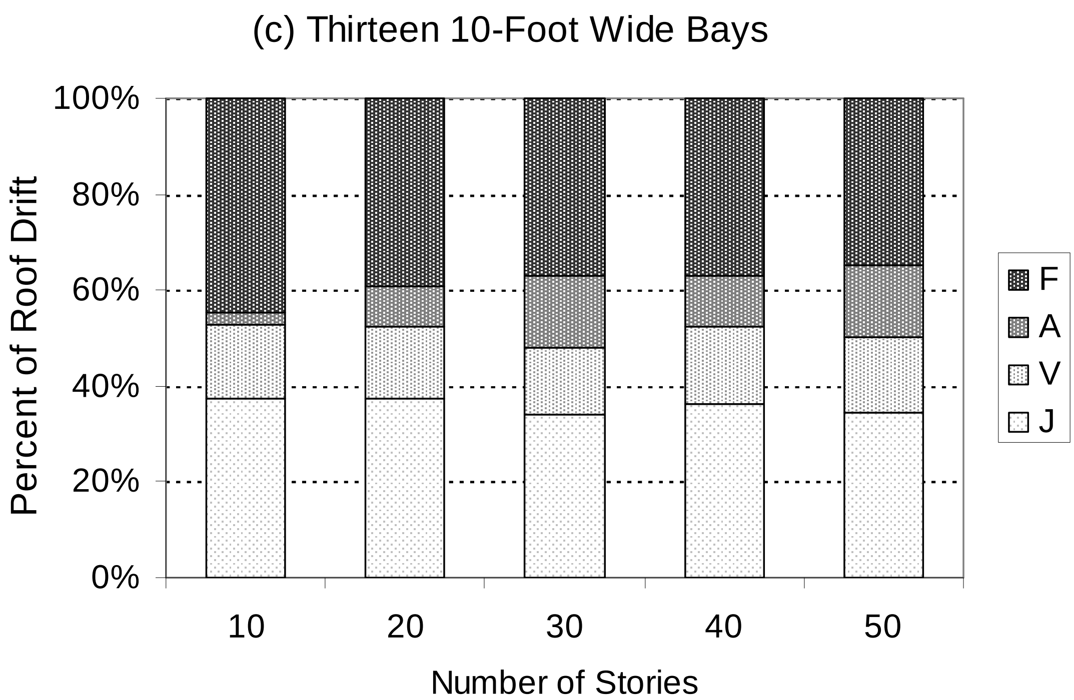


Figure 4.5: Results Grouped by Number of Stories



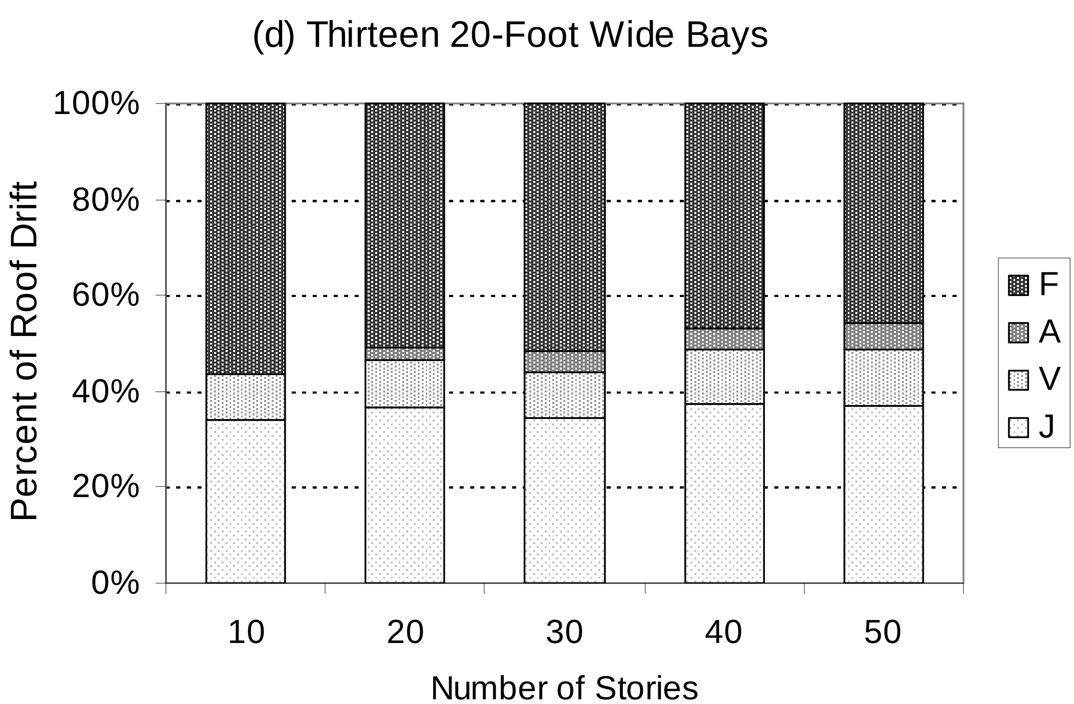


Figure 4.5 (cont'd): Results Grouped by Number of Stories

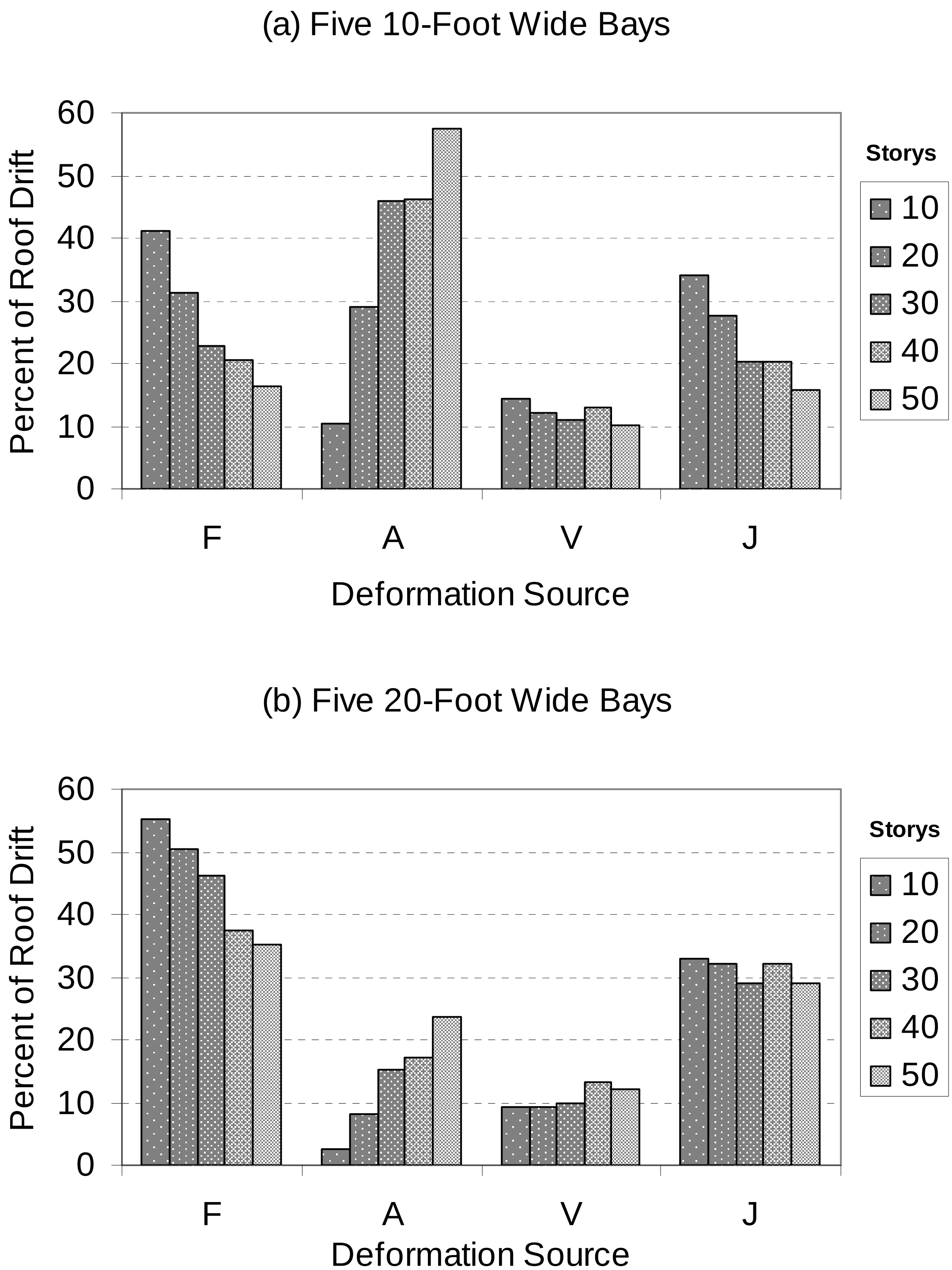
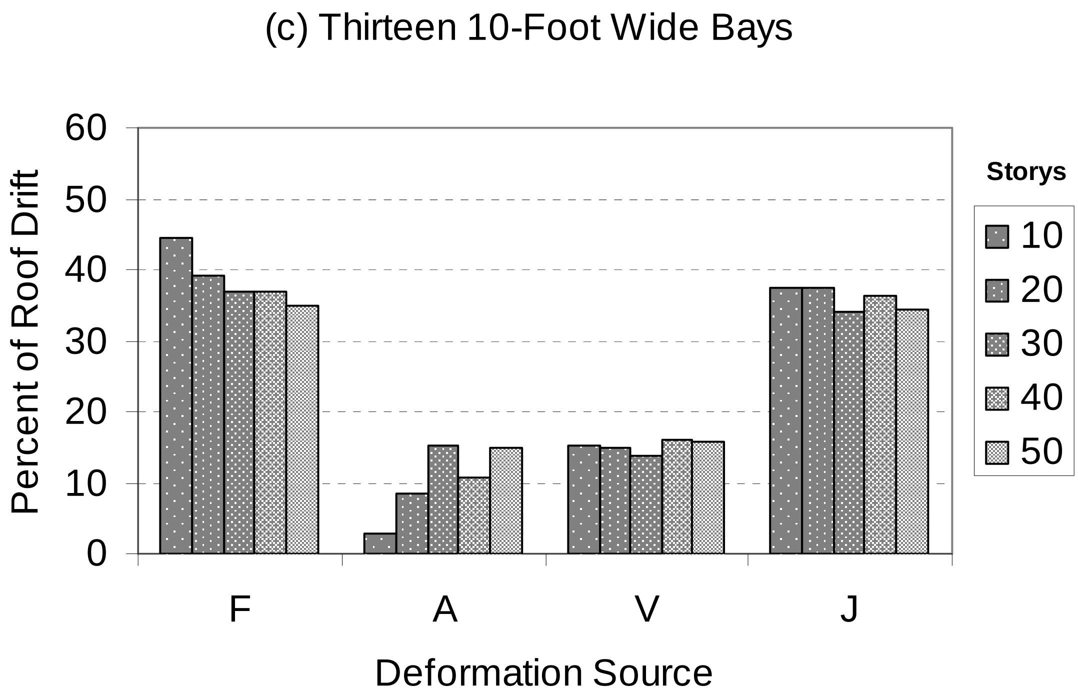


Figure 4.6: Results Grouped by Deformation Source



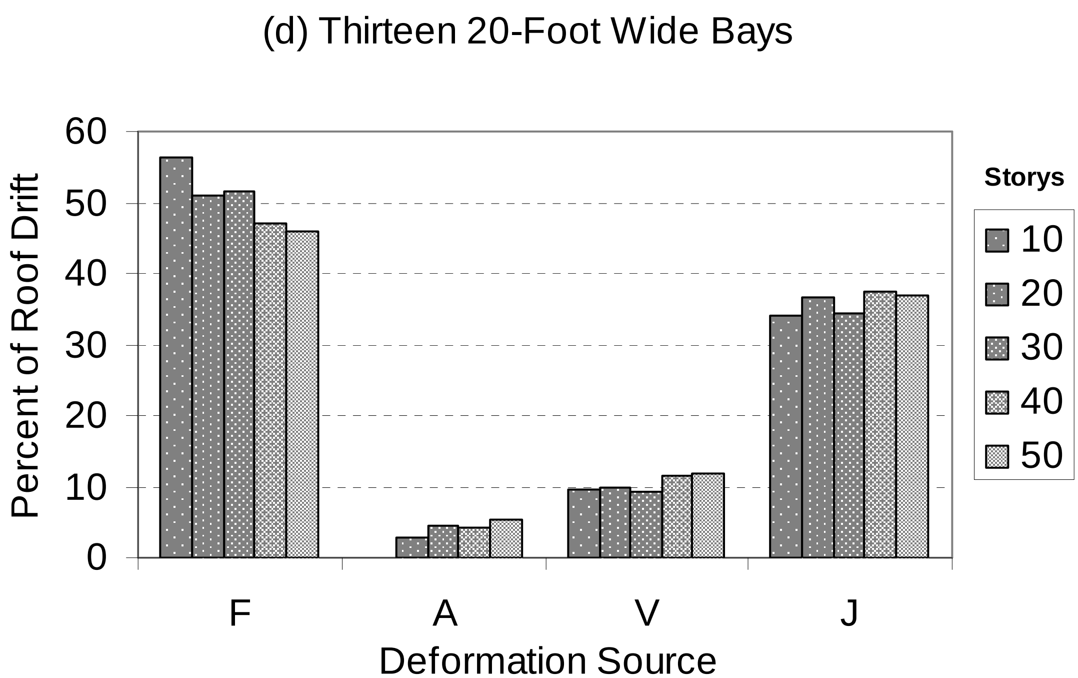


Figure 4.6 (cont'd): Results Grouped by Deformation Source

4.5.3 Joint Deformations

Joint deformations are important in all of the models and should never be neglected. The minimum observed participation was 16 percent in the 50-story tube with five 10 ft bays. The maximum was 41 percent in the 40-story tube with nine 20 ft bays, although many models had values close to this. It is important to note that the average joint contribution across all of the models was 33 percent.

The way the joint region is modeled varies widely from firm to firm as indicated by the 1988 Wind Drift Survey (ASCE 1988). These different modeling approaches were discussed in Chapter 2 and illustrated analytically in Chapter 3. To further demonstrate the shortcomings of the traditionally used rigid zone factor *Z*, the analysis of the 30-story 9-bay (15 ft wide bays) frame was repeated with *Z* taken as 0.0, 0.25, 0.50, 0.75 and 1.0 and also with the Scissors joint. The results are summarized in Table 4.5.

Table 4.5: Effect of Joint Modeling on Total Roof Drift

Deformation Source	Method of Joint Modeling						
	KeySolver*	Scissors	Z=0	Z=0.25	Z=0.50	Z=0.75	Z=1
% flexure	42.4	44.7	46.5	50.1	54.0	58.4	63.1
% axial	12.6	13.5	13.9	14.8	15.7	16.6	17.8
% shear	12.8	13.4	14.1	15.2	16.4	17.7	19.2
% joint	32.2	28.4	25.6	19.9	13.8	7.2	0.0
Total Drift (in.)	10.22	9.70	9.31	8.64	8.03	7.45	6.91

*KeySolver uses the modified force distributions of Figures 4.3(a) and 4.4(a)

As the rigid zone factor *Z* increases, the percent joint contribution decreases because less and less of the joint is considered as flexible. For the case of *Z*=1.0 the entire joint is considered rigid and does not contribute at all. The Scissors model does not have the ability to include a joint flexural deformation, which is why its displacement is lower than the KeySolver model, which includes both flexural and shear joint deformations. For the case of *Z*=0 (centerline modeling) the total displacement is less than both the Scissors and KeySolver analysis because, as seen in Figures 4.3(b) and 4.4(b), joint flexural deformations are overestimated while the more dominant joint shear deformations are underestimated. Figure 4.7 illustrates the results graphically.

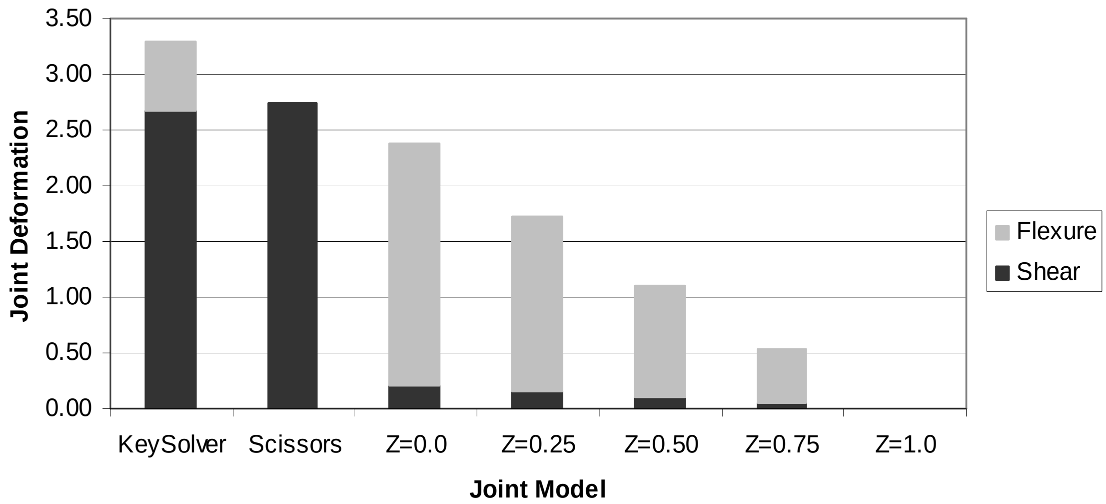


Figure 4.7: Comparison of Joint Deformation Sources

4.5.4 Member Contributions

Instead of breaking the sources of deformation in percent axial, shear and flexure, the results can be looked at in terms of the relative contributions from girders, columns and joints. The 30-story 9-bay (15 ft wide bays) which was reanalyzed in Section 4.5.3 is broken down into the percent of member contributions in Table 4.6 and the data is shown graphically in Figure 4.8.

Table 4.6: Member Contributions

	% Girders	% Columns	% Joint	TOTAL
% Axial	-0.03	12.00	0	11.97
% Shear	6.40	6.53	26.38	39.31
% Flexure	23.59	18.92	6.21	48.72
TOTAL	29.96	37.45	32.59	100.00

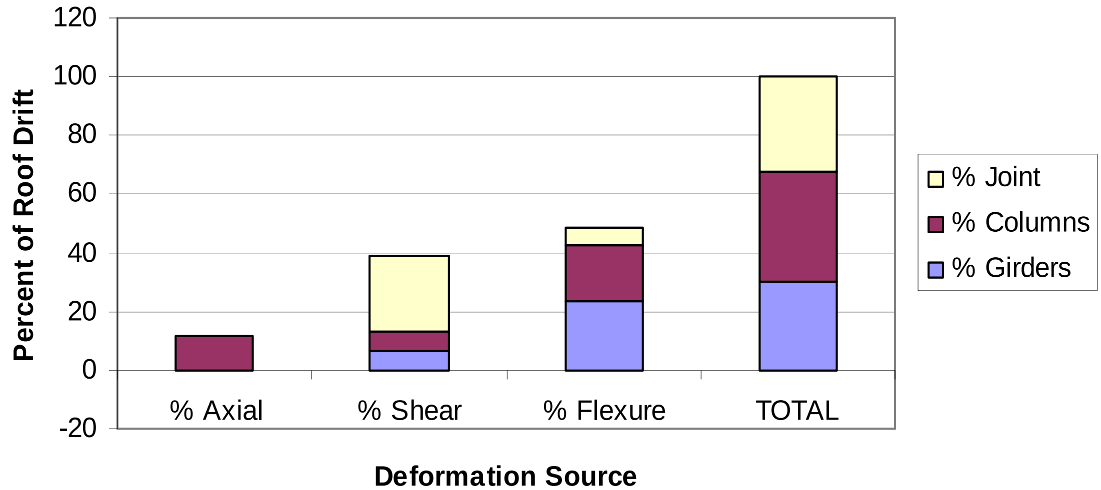


Figure 4.8: Comparison of Member Contributions

For this particular example flexure was the dominant source of deformation followed by shear. It is seen from Figure 4.8 that shear in the joint region is a major source of deformation, even more so than axial for this particular example. Once again it is seen that joint deformations are a very important component of lateral displacement and should always be included in the analytical model.

4.6 Summary

The study presented in this chapter shows that the various sources of deformation in laterally loaded steel frames and tubes vary with respect to bay width, number of bays, total height and joint modeling. In general, the taller and more slender the frame becomes, the greater the contribution from axial deformations. Flexural deformations are inversely proportional to the axial deformations and generally decrease with height and increase with bay width. Shear deformations were shown to always contribute significantly to the total drift and cannot rationally be neglected in any of the cases. Tube structures showed a slightly higher contribution from shear, as expected. Joint deformations were very important, and increased with the number and width of bays. By comparing the different joint models, it was shown that the commonly used rigid zone factor *Z* always underestimates the actual joint *DPF* and consequently the entire building drift. This in turn leads to underestimated P-Delta effects, further compounding the error, and possibly leading to both strength and serviceability issues.

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Chapter 5

Conclusions

5.1 Conclusions and Recommendations

With respect to serviceability, designing for drift is done to prevent or limit unacceptable damage to nonstructural building components such as interior cladding and partitions as well as to ensure the functionality of mechanical systems such as elevators.

Adequate building stiffness is obtained by designing a building to be within reasonable drift limits. However, there are three major sources of discrepancies in design that result in differences in economy and performance among design firms (Charney 1990):

- definition of drift and variations in drift limits
- variations in methods of analysis used to predict drift
- variations in wind loads use for drift calculations

This thesis investigated these sources of discrepancy through a thorough review of the literature (Chapter 2), an analytical study of a typical 10 story office building (Chapter 3), an analytical study on the sources of member deformations (Chapter 4) and by developing a survey to assess the current state of the professional practice (Appendix A). In other words, this thesis was undertaken and written with the intention of suggesting and establishing a comprehensive, performance based approach to the wind drift design of steel framed buildings.

5.1.1 Definition of Damage

Currently the interstory drift index is commonly used in practice to evaluate and limit damage to nonstructural components. This measure of damage does not take into account vertical racking and is therefore not sufficient. Shear strain in the material is the best measure of damage. A simple formula for calculating the average shear strain in a rectangular bay was presented in Chapter 2. Examples presented in Chapters 2 and 3 demonstrated the shortcomings of the interstory drift index. It was also suggested and shown through an example that membrane elements can easily be used in common structural analysis programs to graphically show damage in the building. Such graphical representations are much more informative than commonly used interstory drift indices.

Drift can and should be used to refer to lateral building movement but it should not be used as a measure of damage in nonstructural elements. Finally, damage limits should be based on the material in question as well as the needs of the owner/tenants. Tables and figures for various materials were presented in Chapter 2.

5.1.2 Basic Modeling and Analysis

Accurately modeling the building and accounting for all sources of deformation is essential to produce reliable results. Sources of material deformations were discussed in Chapter 2 and studied in Chapters 3 and 4. Chapter 4 utilized the principle of virtual work to quantify the sources of deformation for a series of 45 steel frames and framed tubes. The analyses illustrated that axial deformations are dominant in tall, slender frames and nearly negligible in wide, short frames. Flexural deformations were shown to be inversely proportional to axial deformations. Shear deformations were important in all analysis cases. Several formulas for calculating minor and major axis shear areas of wide flange sections, which have been verified by detailed finite element testing, were presented in Chapter 2 (Charney *et al.* 2005; Iyers 2005). It is suggested that these formulas for shear areas be implemented in common structural analysis programs.

Modeling of the beam-column joint region was discussed in detail and it was stated that deformations in the joint region should always be considered. Chapter 2 summarized the available models, from the commonly used rigid zone factor Z, to the Krawinkler and Scissors models. Multiple references were cited, and several analyses in Chapters 3 and 4 were performed, to illustrate the inadequacies of using the rigid zone factor.

Using centerline analysis ($Z = 0$) may provide adequate results but it is not correct. The reason it provides adequate results is that shear deformations in the panel zone are underestimated, while flexural deformations are overestimated, producing offsetting errors which give a false impression of a correct analysis. A modified joint force distribution, along with the principle of virtual work, was utilized in Chapter 4 to obtain more accurate lateral displacements. Studies to compare different Z values to the Scissors model were performed. It was 1978 when Professor Helmut Krawinkler developed an analytical model that accurately represents the strength, stiffness and deformation response behavior of the panel zone region. In spite of this, the most commonly used methods in design today are not correct and can be very inaccurate, at the cost of building economy and possibly building safety. Although the mechanical model of the joint region is the most accurate, it is also the most difficult method and many constraints limit its use in daily design. It should be noted that although commercial software does provide the tools to create mechanical joint models and implement them in daily design, most engineers use the rigid endzone factor to model the panel zone region. Given the capabilities of computer hardware/software, the adoption of mechanical joint modeling is entirely possible and should be encouraged in the engineering community. However, until the mechanical joint model is better provided by software packages and strongly recommended by the code bodies, it is unlikely that it will be widely used. Until that point in time, and in lieu of more accurate modeling techniques, the centerline model is recommended as it provides the best estimate of true joint deformations.

In structural design the assumption is often made that a connection is “pinned” or “fixed”. However, even when modeled and designed as pinned, the beam to column connection supplies a certain amount of moment and rotational resistance. Accounting for this resistance results in a partially restrained or PR connection. Several studies were discussed which examine the benefits of using PR connections in design. The major obstacle to widespread use of this type of connection is a reliable model that defines the moment-rotation characteristics. With the art advancing toward performance based engineering, more research and work is necessary in this area. It is noted that AISC (2005) permits the use of PR connections and as more research is done, it is envisioned that their use will become more common due to their benefits.

Concrete floor slabs are commonly designed to act compositely with supporting beams, allowing for lighter members and an economical gravity system. Under gravity loads the benefits are well known and commonly used. Using composite section properties to help resist lateral wind loads was investigated in Chapter 3 of the thesis. It was shown that the increased moment of inertia in the girders of the moment frames provided a significant amount of overall stiffness to the frame. Difficulties were noted in that the increased moment of inertia can only be used for the beam when the slab is in compression, which provided some challenges for use in the analysis program SAP2000 (Computers and Structures, Inc. 2006). When modeling for drift design, the composite action should be utilized and this was recommended in the final model of Chapter 3.

Nonstructural components, whose presence is almost always neglected in building design, have been shown to provide a considerable amount of lateral stiffness. Multiple studies were presented which demonstrate that the stiffness of the bare frame is often times quite different than that of the completed building. Although their contribution is recognized, a reliable way to incorporate the NSC stiffness into the building requires more research. In a major design event such as an earthquake the NSC should not be relied on but for wind drift serviceability design, it makes sense to account for this additional source of stiffness.

The literature review of Chapter 2 touched briefly upon the issues of modeling the floor diaphragm and the foundation flexibility. A rigid diaphragm assumption may not be appropriate for structures in which the lateral load resisting system is very stiff in comparison to the floor slab (such as a shear wall system). Both of these issues require considerable engineering judgment.

An additional issue in modeling is second order or P-Delta effects. There are various hand methods and simplified methods for accounting for these effects. The literature review focused on computer based methods. The two common ways matrix based programs include P-Delta effects are through forming a consistent geometric stiffness matrix for each element (requires iteration) or forming a linearized stiffness matrix on a floor by floor basis (does not require iteration). It is important for the design engineer to understand exactly how the structural analysis software he/she is employing is accounting for P-Delta effects. The inclusion of P-Delta effects was illustrated in the analysis of Chapter 3 where it was concluded that any reliable building design should include these effects. Incorporating P-Delta effects is not difficult given the capabilities of today's software packages.

Directly related to P-Delta effects is the choice of a gravity live load to use for wind drift calculations. It is not reasonable to use the full live load used for strength design. Survey based loads are provided in ASCE 7 (2005) and these should be used in place of those used for strength design. No load factors should be used in the appropriate wind drift loading combinations.

5.1.3 Wind Loads

Wind loads were discussed in detail as correctly modeling the structure and using appropriate damage measures relies on using correct wind loads. Choosing an appropriate mean return interval for calculating wind loads is a choice that must be made by the designing engineer as serviceability issues, not being of concern to life or safety, are generally not addressed in building codes. In the United States no guidance is offered on the subject of wind drift. It was found that both the Canadian and European building codes (NBCC 2005; EN 1993) offer a minimal amount of guidance in either the code or the accompanying design guide series (Gardner and Nethercot 2005). The design engineer is left to choose an appropriate wind serviceability MRI based on building usage, past experience and the needs of the owner/tenants. It was found that an MRI of 10 years is recommended in the literature and in the Canadian building code for serviceability design. The commonly used ASCE 7 (2005) document does not provide any recommendations but does provide conversion factors to convert from the map defined 50 year wind speeds to various other reference periods. It is recommended that an equation for calculating the conversion factor for any reference period be provided in ASCE 7.

The design wind pressure is directly proportional to the square of the wind velocity. ASCE 7 defines basic wind speeds in terms of the 3 second gust speed at 33 ft above the ground in Exposure Category C. It was found in the review of the literature that the method used to convert from the previously used fastest mile wind speeds (ASCE 7-93 and prior) to the current 3 second gust wind speeds (ASCE 7-95 to current) may have been flawed. More research should be conducted in this area.

A factor called the "wind directionality factor" is found in ASCE 7. Applied to the wind velocity in the equation to determine wind pressure, this factor accounts for the reduced probability of maximum winds coming from the most unfavorable direction for building response. For most buildings the factor is 0.85 regardless of site location, building orientation or the mean recurrence interval of the wind speed being used. However, it has been shown that this factor is directly related to the wind MRI being used, an issue not addressed in the current provisions. For MRI's greater than 50 years this leads to unsafe designs while for the shorter MRI's used in serviceability design, this issue results in overly conservative designs. Current provisions are therefore not adequate and more research should be conducted in this area.

Building shielding is common in heavily built up urban areas. However, no matter what degree of shielding is present, wind tunnel derived loads are never permitted to be lower than 80 percent of the code derived loads for strength design. As far as serviceability loads are concerned, no guidelines exist but it is felt that this restriction can be relaxed significantly for wind drift design. This is obviously an area where engineering judgment is necessary and more guidelines should be offered.

The literature review also provided information on other methods of obtaining wind loads. Different wind tunnel models were discussed and summarized. Database assisted design, which is useful for common building geometries is another option, although more work is necessary in this area. Computation fluid dynamics was touched upon as perhaps being the future of wind engineering. Although this field holds much promise, its everyday implementation is still years away.

5.1.4 Building Response

Measuring the full-scale in situ response of a structure to actual loads is beneficial in that the collected data may be used to validate (or disprove) design procedures and design methods and reveal unsafe or uneconomical design decisions. The measured response can then be compared to the predicted response, from both finite element software and wind tunnel methods, to provide valuable information for the engineering community.

The use of GPS to monitor building response has replaced the traditionally used accelerometers in many applications. GPS has the ability to capture both the static and dynamic components of building movement while accelerometers capture only the dynamic component. Chapter 2 presented several instances of recorded building response and mentioned a major ongoing project in Chicago (Kijewski-Correa *et al.* 2005). Measuring full-scale building response is a valuable tool that should be used more often, as it is the only true way to validate the design procedure currently in use.

5.1.5 Wind Drift Design Survey

A survey to assess the current state of the practice is given in Appendix A. Currently in progress, this survey will provide useful information on how companies nationwide address the issue of wind drift in steel framed buildings. It is envisioned that the results from this survey alongside the information provided in the literature review and learned from the analytical studies in this thesis will provide a motivation and basis for performance based wind drift design of steel framed buildings.

5.2 Final Recommendations

The author's overall recommendations are now presented regarding the design of steel framed buildings to satisfy the limit state of wind drift serviceability.

- Wind drift refers to the lateral movement of a building due to wind loading. Nonstructural damage due to this lateral movement is the result of shear strains exceeding elastic material limits. In the design stage, shear strain in drift damageable zones should be constrained to certain limits based on the material, the wind velocity return interval used to calculate wind loads and the building usage/owners needs. References should be consulted for damage limits for the nonstructural materials, such as the ones provided in Chapter 2 of this thesis.

- The MRI used for calculating wind loads for the limit state of wind drift serviceability should not be greater than 10 years. Where past experience has shown that a shorter MRI is adequate, the MRI may be reduced per engineering judgment.

- The wind directionality factor which is found in ASCE 7 (2005) should be applied to the wind velocity used to calculate wind loads for the limit state of wind drift serviceability.

- Shielding due to neighboring buildings may be considered in wind tunnel testing. Per engineering judgment, tunnel obtained wind pressures below 80 percent of the equivalent code obtained wind pressures may be used for the limit state of wind drift serviceability. In the absence of wind tunnel tests and site specific wind speed data, the code obtained wind pressures may be reduced by up to 20 percent per engineering judgment.

- In the modeling and analysis of buildings subjected to lateral wind loads the following guidelines are offered:

a) All sources of material deformations in the structural members should be considered.
    i) This includes deformations arising from major and minor axis flexural stresses, major and minor axis shear stresses, axial stresses and torsional stresses.
b) Material deformations occurring in the beam-column joint regions should be considered.
    i) In the absence of reliable mechanical models, material deformations in the joint may be accounted for by using a rigid zone factor of zero (Centerline model).
c) The effects of floor beams and girders acting compositely with the concrete floor slab should be considered.
d) The stiffness of connections not considered fully fixed may be considered.
    i) A reliable force-deformation response curve for the specific type of connection should be used.

e) Second order P-Delta effects should be included in the analysis.
    i) Survey based live loads should be used.
    ii) For the appropriate LRFD load combination, all loads should be unfactored.
f) Foundation flexibility may be considered based on the type of foundation and the condition of the substructure material.
g) Nonstructural building components may be included in the structural model.
    i) Properties used for the nonstructural materials should be based on experimental testing from reliable research sources.
    ii) The contributing stiffness from nonstructural elements should only be considered for the limit state of wind drift serviceability.

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Appendix A

Wind Drift Survey

A.1 Overview

This section of the thesis addresses the new survey of the practice to assess the current state of the art in wind drift design of steel buildings. A new survey was considered necessary given the significant advances that have transpired in the field since the last survey, nearly twenty years ago, when the use of computers was just becoming commonplace. In terms of structural engineering, computers are to the late 20th century what steel was to the late 19th century; a revolutionary force that has changed the face of the profession. When the first survey was conducted, computers were just beginning to be used in everyday design and with very limited applications. Computer capability has increased in three significant ways:

1. Computations per Second per $1000 has increased substantially in accordance with Moore's Law, The Fifth Paradigm
2. Program capacity places virtually no restrictions on what can be modeled, the number of degrees of freedom or the physical dimensions of the structure
3. Graphical user interaction provides a whole new level of control

Additionally, the following advancements have come about as a result of the constantly evolving technology:

- The formulation of matrix structural analysis of plane and space trusses and frames and their implementation in computer programs
- The development of the finite element and implementation in computer programs
- Direct second order analysis, including all sources of deformation, accounting for foundation flexibility and accurate modeling of the panel zone regions are all within reach for the typical design firm.

- Structural optimization algorithms that, through reducing overall building weight and/or increasing member redundancy, enable designers to achieve a level of structural performance and economy that was previously difficult if not impossible to attain.
- Wind tunnel testing continues to advance while becoming more widespread and affordable
- The adoption of the Load and Resistance Factor Design (LRFD) philosophy by many structural engineers

Furthermore, in addition to technological advancements the following changes in the structural engineering field have affected the way structures are designed for drift:

- The typical return interval used in design for serviceability issues of drift has changed dramatically, from a 50 year to a 10 year mean return interval.
- It has been shown that drift is not the most accurate measurement of structural damageability (Charney 1990, Griffis 1993).
- Performance based design, developed for earthquake engineering, shows much promise in redefining the practice of design against wind damage.

It is extremely important to gain an overview of the ways in which these advances have affected the state of the art and to bring these changes, positive and negative, to the attention of the community. Because serviceability limit states are not codified, the way they are handled may differ considerably from firm to firm; these differences will be brought to light.

A.2 Methods and Procedure

The survey was developed by the author and Dr. Finley Charney of Virginia Tech in collaboration with the ASCE Task Committee on Drift Control of Steel Buildings. Many of the questions posed in the original survey were used as well as additional questions that reflect the current state of the art. In an attempt to make the survey widely assessable, with the added benefit of convenient data recovery, the internet was used as a distribution tool.

As a “front-page” for the survey, a web page was created that contained instructions, a glossary of terms, as well as a link to both a paper and online copy of the survey. For the web-based version, the website *QuestionPro.com* was used to create a user-friendly online version of the survey for the respondents to fill out online. As an alternative, an identical paper copy of the survey was available from the “front-page” website for those who wished to print it out and mail in their responses. The survey consists of two parts:

PART 1: General Drift and Perception of Motion Issues
PART 2: Structural Analysis of Ten Story Building

Part 1 deals mainly with drift issues, with questions ranging from technical (How do you model the beam-column joint region?) and practical (How many years of design experience do you have?) to philosophical (Should wind drift criteria be codified?). Part 1 is split into six distinct sections:

I. Personal and Company Information
II. Wind Drift: General Issues
III. Structural Modeling Issues
IV. Wind Loads
V. Wind Drift Limits
VI. Perception of Motion (optional)

Section I, by asking about company size and experience, will enable correlation between other sections (such as modeling procedures) with company size. For example, do larger companies have more resources which enable advanced modeling? Do smaller companies generally follow a certain approach? Sections II through V contain specific questions regarding the topic at hand. Comment boxes are provided frequently as some questions can not simply be answered as yes or no. In addition, anonymity is guaranteed in an attempt to gather personal opinions/practices as opposed to making it feel like an engineering exam or a critique of the firm. Section VI addresses perception of motion issues. This section is optional and intended only for those with significant experience in the area. Motion perception is a design issue that is more common in tall and/or slender structures, which the majority of the practice does not deal with frequently. However, the issue of wind drift is very commonly dealt with among engineers. That is why wind drift is the main topic with motion perception included as an addendum.

Part 2 of the survey, optional and only to be filled out by those who completed Part 1, asks the respondent to conduct an analysis of a simple 10 story steel framed building. All member sizes, connection details, diaphragm assumptions, and lateral and gravity loads are provided. The respondents are asked to model the building as he/she normally would and provide data on the computed response in the form of lateral drift and modal periods. Respondents are then asked to use ASCE 7-05 to determine wind loads for the structure. Several assumptions must be made at this point: an exposure category must be selected based on the aerial photograph of the site and a recurrence interval for wind drift calculations must be selected, based on company practice, to calculate the wind velocity. Using the calculated wind loads, the analysis is carried out and tables of lateral drift are filled out. As an extra incentive to perform Part 2 of the survey, a structural design/analysis program (SAP2000 or ETABS) will be given away, through a random drawing, to one of the respondents. In order to solicit the responses from as many structural engineers as possible, the survey was widely advertised in several major publications (Journal of Modern Steel Construction, Structural Engineer, Structures) and through email advertisement.

A.3 Preliminary Results

As of the time of this printing, the survey has not garnered enough responses to provide a conclusive list of results and conclusions. After all of the survey results are collected, a comprehensive paper will be published in a major engineering journal. This will allow the results to be distributed to the profession, along with opinions and comments from the committee. It is hoped that through this survey and the subsequent publication of journal, the important issue of how wind drift in steel framed buildings is dealt with among design firms will be better understood.

However, some preliminary data is available and will now be presented. At this point there are 44 respondents to the survey. The following statistics are presented.

- 48 percent hold a Bachelors Degree
- 43 percent hold a Masters Degree
- 9 percent hold a Doctorate
- Average design experience of 17 years

In response to the question "At what building height does wind drift become a design issue?" the majority answered one story and felt that all buildings should consider the effects of wind drift.

The question of whether wind drift limits should be codified elicited strong responses. While the majority of the respondents felt that serviceability issues should not be strictly codified, there was a clear consensus that more guidance is necessary through design guides or a standard of care. Here are some of the comments offered.

"Since building codes are silent on allowable drift, the standard of care by which structural engineers are judged with respect to the adequacy of their designs is highly variable."

"There should be a common recommendation for drift. The temptation is to allow excessive drift to limit costs. Some building manufacturers have NO drift limits and, of course, their buildings are cheaper."

"I think it would be valuable for there to be a code defined standard of care for drift limits."

"There has been too much of a trend toward attempting to codify every aspect of structural engineering. Some issues, especially issues as subjective as drift perception, should be left to discretion."

The majority of respondents said that they include all sources of deformation in analysis; however, there were some surprising statistics:

- 42% never include shear deformations in beams
- 38% never include shear deformations in columns
- 23% never include axial deformations in columns

The large majority includes P-Delta effects and use 100 percent of the design gravity live loads for this calculation.

Regarding the panel zone region, the majority (73 percent) employ the centerline model.

A large range of drift limits were observed among the respondents. For a one story metal warehouse the drift limits ranged from h/40 to h/500. A 50 year wind MRI was the most commonly used wind load for drift calculations, but values provided ranged from 10 to 100 years.

A.4 ASCE Wind Drift Survey

The survey is now presented in its entirety, beginning with the introduction and concluding with the glossary.

Design of Steel-Framed Buildings for Wind Drift and Motion Control

Instructions

The survey is divided into two Parts, consisting of a total of seven Sections. Part 1 of the survey contains Sections I through VI. Sections I through V cover issues that relate primarily to wind drift. Section VI covers issues related exclusively to perception of motion. Section VI should be completed only if you have significant experience with perception of motion issues. To provide incentive for completing Part 1 of the survey, one free copy of ASCE 7-05 will be provided to five respondents. The respondents designated to receive the document will be determined by a drawing.

Part 2 of the survey contains only one section. This part of the survey is optional, and requires that the respondent perform drift related calculations for a hypothetical ten-story structure. To provide incentive for completing Part 2 of the survey, a copy of SAP2000 or ETABS will be awarded to the employer of one respondent. The firm designated to receive the software will be determined by a drawing.

Each part of the survey may be filled out by use of an on-line utility, or by completing a paper version. Both versions of the survey may be found at the following web address:

http://www.celes.ictas.vt.edu/drift/

The paper version should be mailed to the Committee **only** if the on-line version is not attempted. The mailing address is as follows:

Dr. Finley A. Charney
ASCE Drift Survey
PO Box 990
Blacksburg, VA 24063

Several specific guidelines for completing the survey are provided below:

- The Committee's desire that the survey be completed by individuals, and not by firms. Hence, several members of the same firm are encouraged to complete the survey.

- Part 1 of the survey requests the name of the respondent's employer. This information is required. It will be used for data reduction purposes only, and will not be made public.

- Part 1 of the survey asks you to enter an eight digit alphanumeric code (e.g FAC1952X). The same code must be entered in Part 2 of the survey. This code is used to link together the two parts of the survey. The code will also be used to notify the winners of the drawings for the ASCE 7 document and the software. This will be done by posting the winning codes on the Survey's Home Page. The winning codes and instructions for collecting the prizes will be posted on June 1, 2006.

- It is highly recommended that the paper version of each part of the survey be reviewed in its entirety before the on-line version is accessed, or before the paper version is completed.

- Each part of the on-line version of the survey must be completed at one sitting. Filling out the paper version ahead of time will facilitate this.

- The survey may contain technical terms that are not familiar to you. If this is the case, refer to the glossary document that is available on the Survey's Home Page.

- If you do not know the answer to a particular question, please do not guess. Simply leave the response for that question blank.

- The estimated time to complete the various parts of the survey are as follows:

Estimated Survey Completion Times

Survey Section	Estimated Time Required to Complete (Minutes)
I: Personal and Company Info	5
II: Wind Drift: General Issues	30
III: Structural Modeling	5
IV: Wind Loads	5
V: Drift Measures and Limits	5
VI: Perception of Motion	5
VII: Frame Calculations (optional)	120

Thank you for taking the time to fill out this survey.

Design of Steel-Framed Buildings for Wind: Drift and Motion Control

PART 1: General Drift and Perception of Motion Issues

Instructions

Instructions for filling out this survey may be found at the following web site:

http://www.celes.ictas.vt.edu/drift/

I) Personal and Company Information

I-1) The name of your design firm (please spell out). Please note that the name of your firm will not be made public.

____

I-2) To allow us to link Part 1 of the survey with Part 2, please create an eight digit alphanumeric code to identify yourself. Please remember the code for future reference. You will be asked for this code in Part 2 of the survey.

____

I-3) Your location (City and State):

____

I-4) Your level of college education:
______ B.S.
______ M.S.
______ Ph.D.

I-5) Number of years of experience in a design office:

______

I-6) Please indicate if you hold the following professional licenses:
______ P.E.
______ S.E.

Please provide *estimates* for questions I-6 through I-10.

I-7) Total number of structural engineers in your firm who design buildings (including all offices):

______

I-8) Average years of college education of individuals performing structural analysis and design calculations related to this questionnaire:

______

I-9) Average years of professional experience of individuals performing structural analysis and design calculations related to this questionnaire:

______

I-10) Average years of professional experience of persons in responsible charge of persons performing calculations related to this survey:

______

I-11) Complete the following table that indicates the volume of work in your entire firm over the last ten years. If you do not have information for your firm, enter the information for your office, and indicate in the appropriate space that the data is for your office only.

Enter the approximate number of steel building in the specified building use category. If your firm has been designing buildings for less than ten years, specify the appropriate number of years in the space provided below the table.

Building Use	**Number of Stories**			
	1 - 5	6-20	21-40	>40
Commercial				
Residential				
Institutional				
Industrial				
Other				
Total				

Number of years designing buildings if less than ten years:

______

The data in the above table is for:

______ The entire firm

______ My office

II) Wind Drift: General Issues

II-1) Is wind drift a common design issue for buildings designed by your firm**?**

______ Yes, very common (>75% of all steel buildings)
______ Yes, somewhat common (>50% of all steel buildings)
______ No, somewhat uncommon (<50% of all steel buildings)
______ No, very uncommon (<25% of all steel buildings)

II-2) At what building height (number of stories) does wind drift become a significant design issue?

______ Stories

II-3) Does your firm have written policies and procedures related to wind drift?

______ Yes
______ No

II-4) If you answered "yes" to question 3, would you be willing to share the guidelines with the Task Committee?

______ Yes (please email fcharney@vt.edu and you will be contacted further)
______ No

II-5) Why do you limit drift? Please rank the following reasons in terms of priority 1 to 5 with 1 being the highest priority and 5 being the lowest priority.

_____ To limit damage to structural elements
_____ To limit damage to nonstructural elements
_____ To limit lateral accelerations
_____ To minimize or reduce second-order (P-Delta) effects
_____ Other. (please specify): ____________________________________

II-6) Current building codes (e.g. 2006 IBC) do not specify drift limits for wind loads. Do you believe that wind drift limits should be specified in building codes?

______ Yes
______ No

II-7) Please explain your answer in (6) above and provide any comments in the space below.

II-8) Complete the following information for the *five* structural steel buildings that were most recently designed by your firm in which *wind controlled the design of the lateral load system* (seismic controlled buildings are ineligible).

Question II-8, Building # 1

Frame Type (more than one type may be selected for combined or dual systems)

______ Moment Resisting Frame with Rigid Connections
______ Moment resisting Frame with Flexible Connections
______ Concentrically Braced Frame
______ Eccentrically Braced Frame
______ Other

Building Dimensions

_____ Number of stories
_____ Height (ft)
_____ Length (ft)
_____ Width (ft)

Average building density

_____ pcf

Use (occupancy). Check all that apply if the building is multi use.

_____ Commercial
_____ Residential
_____ Institutional
_____ Industrial

Location (City, State)

____

Exposure (per ASCE-7 descriptions)

______ A
______ B
______ C
______ D

Basic wind speed and mean recurrence interval used for Strength

______ Speed (mph for 3 second gust)
______ MRI (years)

Basic wind speed and mean recurrence interval used for Serviceability

_____ Speed (mph for 3 second gust)
_____ MRI (years)

Was a wind tunnel used in the design of this building

_____ No
_____ Yes

Question II-8, **Building # 2**

Frame Type (more than one type may be selected for combined or dual systems)

______ Moment Resisting Frame with Rigid Connections
______ Moment resisting Frame with Flexible Connections
______ Concentrically Braced Frame
______ Eccentrically Braced Frame
______ Other

Building Dimensions

_____ Number of stories
_____ Height (ft)
_____ Length (ft)
_____ Width (ft)

Average building density

_____ pcf

Use (occupancy). Check all that apply if the building is multi use.

_____ Commercial
_____ Residential
_____ Institutional
_____ Industrial

Location (City, State)

____

Exposure (per ASCE-7 descriptions)

______ A
______ B
______ C
______ D

Basic wind speed and mean recurrence interval used for Strength

______ Speed (mph for 3 second gust)
______ MRI (years)

Basic wind speed and mean recurrence interval used for Serviceability

_____ Speed (mph for 3 second gust)
_____ MRI (years)

Was a wind tunnel used in the design of this building

_____ No
_____ Yes

Question II-8, Building # 3

Frame Type (more than one type may be selected for combined or dual systems)

______ Moment Resisting Frame with Rigid Connections
______ Moment resisting Frame with Flexible Connections
______ Concentrically Braced Frame
______ Eccentrically Braced Frame
______ Other

Building Dimensions

_____ Number of stories
_____ Height (ft)
_____ Length (ft)
_____ Width (ft)

Average building density

_____ pcf

Use (occupancy). Check all that apply if the building is multi use.

_____ Commercial
_____ Residential
_____ Institutional
_____ Industrial

Location (City, State)

____

Exposure (per ASCE-7 descriptions)

______ A
______ B
______ C
______ D

Basic wind speed and mean recurrence interval used for Strength

______ Speed (mph for 3 second gust)
______ MRI (years)

Basic wind speed and mean recurrence interval used for Serviceability

_____ Speed (mph for 3 second gust)
_____ MRI (years)

Was a wind tunnel used in the design of this building

_____ No
_____ Yes

Question II-8, Building # 4

Frame Type (more than one type may be selected for combined or dual systems)

______ Moment Resisting Frame with Rigid Connections
______ Moment resisting Frame with Flexible Connections
______ Concentrically Braced Frame
______ Eccentrically Braced Frame
______ Other

Building Dimensions

_____ Number of stories
_____ Height (ft)
_____ Length (ft)
_____ Width (ft)

Average building density

_____ pcf

Use (occupancy). Check all that apply if the building is multi use.

_____ Commercial
_____ Residential
_____ Institutional
_____ Industrial

Location (City, State)

____

Exposure (per ASCE-7 descriptions)

______ A
______ B
______ C
______ D

Basic wind speed and mean recurrence interval used for Strength

______ Speed (mph for 3 second gust)
______ MRI (years)

Basic wind speed and mean recurrence interval used for Serviceability

_____ Speed (mph for 3 second gust)
_____ MRI (years)

Was a wind tunnel used in the design of this building

_____ No
_____ Yes

Question II-8, Building # 5

Frame Type (more than one type may be selected for combined or dual systems)

______ Moment Resisting Frame with Rigid Connections
______ Moment resisting Frame with Flexible Connections
______ Concentrically Braced Frame
______ Eccentrically Braced Frame
______ Other

Building Dimensions

_____ Number of stories
_____ Height (ft)
_____ Length (ft)
_____ Width (ft)

Average building density

_____ pcf

Use (occupancy). Check all that apply if the building is multi use.

_____ Commercial
_____ Residential
_____ Institutional
_____ Industrial

Location (City, State)

____

Exposure (per ASCE-7 descriptions)

______ A
______ B
______ C
______ D

Basic wind speed and mean recurrence interval used for Strength

______ Speed (mph for 3 second gust)
______ MRI (years)

Basic wind speed and mean recurrence interval used for Serviceability

_____ Speed (mph for 3 second gust)
_____ MRI (years)

Was a wind tunnel used in the design of this building

_____ No
_____ Yes

Question II-8, All Buildings

In the following table please indicate which single item predominantly controlled the design of the lateral load resisting system for the buildings described above.

	Wind: Strength	Wind: Drift	Wind: Motion
Building #1	☐	☐	☐
Building #2	☐	☐	☐
Building #3	☐	☐	☐
Building #4	☐	☐	☐
Building #5	☐	☐	☐

III) Structural Modeling Issues

III-1) What software does your firm use with respect to *lateral load analysis* for steel structures? List the three programs most often used for each building category. Enter “in-house” if the software was developed by your firm:

a) Low rise buildings (1 - 5 stories)
1) ______________________
2) ______________________
3) ______________________

b) Mid Rise Buildings (6 - 20 stories)
1) ______________________
2) ______________________
3) ______________________

c) Tall buildings (21 stories and taller)
1) ______________________
2) ______________________
3) ______________________

III-2) When modeling buildings, the structure is <u>most often</u> modeled in….
______ two dimensions
______ three dimensions

III-3) Please explain your answer to question 2.

III-4) When modeling in three dimensions, a variety of floor diaphragm models may be used. How often do you use the approaches listed below?

	Usually	Sometimes	Rarely
Fully flexible	☐	☐	☐
Rigid in-plane, flexible out of plane	☐	☐	☐
Using membrane type finite elements	☐	☐	☐
Using plate type finite elements	☐	☐	☐
Using shell type finite elements	☐	☐	☐

III-5) Please explain you answer to question 4.

III-6) When modeling beams and girders, do you explicitly include the following effects?

	Usually	Sometimes	Rarely
Composite action	☐	☐	☐
Axial deformations	☐	☐	☐
Shear deformations	☐	☐	☐

III-7) When modeling columns, do you explicitly include the following effects?

	Usually	Sometimes	Rarely
Axial deformations	☐	☐	☐
Shear deformations	☐	☐	☐
Splice location and flexibility	☐	☐	☐

III-8) When modeling diagonal braces, do you explicitly include the following effects?

	Usually	Sometimes	Rarely
Axial deformation	☐	☐	☐
Shear deformation	☐	☐	☐
Tension only braces	☐	☐	☐
Stiffness of gusset plates	☐	☐	☐

III-9) Do you explicitly include beam-column (pane zone) deformations?

Usually	Sometimes	Rarely
☐	☐	☐

III-10) Briefly explain your answer to question 9.

III-11) When modeling the beam column joint (panel zone) region of moment resisting frames, a variety of approaches may be used. Indicate how often the following models are used. (See glossary for definitions).

	Usually	Sometimes	Rarely
Centerline analysis (no rigid end zones)	☐	☐	☐
Clear span analysis (fully rigid end zones)	☐	☐	☐
Partially rigid end zones	☐	☐	☐
Panel zone effects are modeled explicitly	☐	☐	☐

III-12 When beam-column joints are modeled explicitly, the following model is most often used (see glossary for definitions):

______ Scissors model
______ Krawinkler Model
______ Section property correction factors
______ Other approach (please specify) __________________

III-13) Do you design structures with semi-rigid (PR) connections?

Usually	Sometimes	Rarely
☐	☐	☐

III-14) Do you explicitly include non-structural elements in the model?

Usually	Sometimes	Rarely
☐	☐	☐

III-15) Briefly explain your answer to question 14.

III-16) Do you include foundation flexibility in your model?

Usually	Sometimes	Rarely
☐	☐	☐

III-17) Briefly explain your answer to question 16.

III-18) Do you include second order (P-Delta) effects?

Usually	Sometimes	Rarely
☐	☐	☐

III-19) Briefly explain your answer to question 18.

III-20) When modeling P-Delta effects, please indicate how often the following procedures are used.

	Usually	Sometimes	Rarely
Manually post-process computer results	☐	☐	☐
Use story geometric stiffness	☐	☐	☐
Use element geometric stiffness	☐	☐	☐
Use large displacement analysis	☐	☐	☐
Other:	☐	☐	☐

III-21) When modeling P-Delta effects, what percent of reduced live load do you use when computing gravity loads?

______ %

III-22) Do you use any procedures for optimum (least weight) distribution of materials?

______ None used
______ Virtual work based procedures
______ Formal optimization procedures
______ Other (please specify) ________________________________

III-23) For tall flexible buildings, the wind load is a function of the structure's damping. Please specify, in percent critical, the level of damping used in your design of steel buildings.

______ Moment resisting frames
______ Partially rigid frames
______ Braced frames

IV) Wind Loads

IV-1) Where do you typically obtain wind loads? Indicate in percent so that the total is 100.

______ Building code
______ Wind tunnel
______ Database assisted design
______ Other (please specify) ______________________________

IV-2) Do you use different wind loads for serviceability and strength?

______ Yes
______ No

IV-3) What return period do you use for wind drift?

______ 1 year
______ 10 year
______ 25 year
______ 50 year
______ 100 year
______ Other (please specify) ____

IV-4) Does the owner or architect play a role deciding what MRI (Mean recurrence interval) to use for serviceability?

Usually	Sometimes	Rarely
☐	☐	☐

IV-5) When designing for strength, ASCE-7 requires the consideration of unbalanced wind loads. Do you use unbalanced wind loads when evaluating drift?

______ Yes
______ No

IV-6) Do you believe the code procedures for determining wind loads are clear? (For example, would two engineers working independently calculate the same wind loads for a given structure, location and wind return interval?)

______ Yes
______ No

IV-7) What is the primary motivation for recommending the use of a wind tunnel? If you have more than one reason, please rank your responses by entering “1” for the most important, “2” for intermediate importance, and “3” for low importance.

______ to obtain realistic cladding pressures
______ to obtain more realistic pressures for design of structure
______ to obtain estimates of lateral accelerations
______ other (please specify) ______________________________

IV-8) For what building height do you generally recommend wind tunnel tests?

______ 10 stories or taller
______ 20 stories or taller
______ 40 stories or taller
______ taller than 60 stories
______ other (please specify) ______________________________

IV-9) When using a wind-tunnel, do you use different exposure models for drift and strength issues?

______Yes
______ No
______ Never used a wind tunnel

IV-10) When using code based loads, do you use different exposure models for drift and strength issues?

______Yes
______ No

V) Wind Drift Limits

V-1) Which of the following measures of wind drift is most often used in your firm?
______ total drift (or total drift ratio)
______ interstory drift (or inter-story drift ratio)
______ total drift and interstory drift (using different limits)
______ shear strain
______ other (please specify) ________________________________

V-2) At what location in the floor plates do you compute the drift?
______ at the center of mass
______ at the building extremities

V-3) When comparing the *allowable* wind drift to the quantity computed in (1) and (2) above, the allowable drift is a function of the following: (Indicate importance for each item using a scale of 1 to 5, where 1 is least important and 5 is most important)
______ type of structural system (moment frame, braced frame)
______ building use (residential, commercial, etc.)
______ building height
______ cladding type
______ interior partition type (masonry, dry wall, etc.)
______ mean recurrence interval of wind loads
______ method of calculating drift
______ none of the above. The same drift limit is always used.

V-4) Have you ever performed an analysis to determine whether it is more economical to add stiffness to avoid damage, or to accept damage and the associated costs?
______ Yes
______ No

V-5) Four hypothetical building are to be designed by your firm. On the basis of your current design procedures, fill in the last three columns of the table below. Please enter the drift limits as a ratio (e.g. H/500 would be entered as 0.0020). If you use strain as a drift measure, enter the strain directly.

Type	**Use**	**Height (stories)**	**Cladding**	**Interior Partitions**	**Wind MRI (years)**	**Drift Measure [a]**	**Drift Limit**
Moment Frame	Warehouse	1	Metal Cladding	None			
Braced Frame	Hospital	10	Brick	Drywall			
Moment Frame	Office Building	30	Glass and granite curtain wall	Drywall (service core only)			
Framed Tube	Hotel	80	Glass Curtain wall	Drywall (service core only)			

All buildings are rectangular with a plan dimension of 100 ft by 180 ft.
[a] See question V-1.

VI) Perception of Motion (optional)

This portion of the survey is optional. Please respond only if you have significant experience in designing buildings where perception of motion due to wind is a common design issue.

VI-1) Which factors are most likely to cause perception of motion to become a design issue (select all that apply):

_____ building height
_____ building slenderness
_____ building density
_____ structural system
_____ terrain, including existing construction in close proximity to subject building
_____ acceptance criteria specified by owner

VI-2) Does your firm have written policies and procedures related to perception of motion?

______ Yes
______ No

VI-3) If you answered "yes" to question 2, would you be willing to share the guidelines with the Task Committee?

______ Yes
______ No

VI-4) What wind mean recurrence interval do you use for perception of motion?

______ 1 year
______ 10 year
______ 25 year
______ 50 year
______ 100 year
______ Other (please specify) _____

VI-5) What level of damping do you assume in calculations related to perception of motion in steel buildings? (Enter values in percent critical)

______ moment resisting frames
______ partially rigid frames
______ braced frames

VI-6) What motion limits (in milli-g) do you use for the building types indicated below. The motion limit should correlate with the MRI selected in question 5. Please indicate if the motion limits are maximum or RMS.

Maximum ☐ *RMS* ☐

______ office buildings
______ residential buildings
______ mixed use facilities

VI-7) Are the motion limits specified in question 6 frequency dependent?

______ Yes
______ No

VI-8) Are the motion limits specified in question 6 part of a standard (e.g. ISO).

______ Yes (please specify) ____________________________
______ No

VI-9) Do you use approximate formulas for estimating motion in tall buildings? If so, please provide a reference. (Example: National Building Code of Canada).

______ Yes (please specify) ____________________________
______ No

VI-10) Please fill in the two tables below with respect to perception of motion issues in the past five steel building you designed where motion was a design issue.

#	Building Use	Structural System	Height (Stories)	Maximum Height/ width	Density (pcf)	Bldg. period T (seconds)
1						
2						
3						
4						
5						

#	Design MRI (years)	Wind Tunnel Method (Force Balance, Aeroelastic)	Maximum acceleration from wind tunnel (Milli-g)	Acceleration limit (Milli-g)	Are accelerations Absolute or RMS?
1					
2					
3					
4					
5					

VI-11) Have you ever used any of the following motion control devices in the design of your buildings? (*Check all that apply*)

______ Tuned mass dampers
______ Sloshing fluid dampers
______ Pendulum dampers
______ Viscoelastic dampers
______ Viscous fluid dampers
______ Other (please specify) ____

VI-12) Have you ever taken a "wait and see" approach to the use of controlling devices, where a building is completed without the device, and such devices are implemented only after building use indicates that such devices are necessary?

______ Yes
______ No

VI-13) Have you ever taken an "informational approach" to the problem of wind induced motion (i.e. inform the owner/occupants that the building is safe and the motion is to be expected on occasion)?

______ Yes
______ No

VI-14) Current building codes (e.g. 2006 IBC) do not specify lateral acceleration limits. Do you believe that codes should specify such limits?

______ Yes
______ No

VI-15) Briefly explain your answer to question 14.

Design of Steel-Framed Buildings for Wind: Drift and Motion Control

PART 2: Structural Analysis of Ten Story Building

Instructions for Completing PART 2
Please complete this part of the survey ONLY if you also completed part 1 of the survey.

It is recommended that you print out this survey and read through all of the questions before beginning any work. After performing the required calculations, enter your responses on the paper copy of the survey, and then transfer the information to the on-line survey, located at:

http://www.celes.ictas.vt.edu/drift/

If you prefer, you may mail the completed paper survey to:

Finley Charney
ASCE Wind Drift Survey
PO Box 990
Blacksburg, VA 24063

Introduction
It this part of the survey you are asked to perform a variety of calculations on a hypothetical 10-story office building located in Jersey City, New Jersey. The approximate site for the building is the corner of Washington Blvd. and Thomas Gangemi Dr. Figure 1 provides a site plan and an aerial photograph for the site. (Illustrations may be found at the end of this document.)

The building is rectangular in plan, as shown in Figure 2. The lateral load resisting system consists of moment resisting frames in the E-W direction and braced frames in the N-S direction. Figures 3 through 5 provide elevations for these frames. Figure 6 is an elevation of the end-frames (on grid lines A and F). The interior columns of these frames resist gravity loads only. The moment frames are fully welded, with typical connection details as shown in Figure 7. Connection details for the braced frame are illustrated in Figure 8. Gravity connections are typical shear-tab connections.

General information about the structure is provided below:

General Information

- Beams and columns: A992 steel
- Construction is shored.

- Braces, HSS: A500 steel
- Studs: ¾ in diameter, Fu = 60 ksi
- Concrete: 4 ksi lightweight concrete (110 pcf)
- Slab: 6.25" total depth on a 3" metal deck.

Design Gravity Loads

Typical Floor Dead Loads (not including steel framing)

Slab, deck and finish = 50 psf

Ceiling and mechanical = 15 psf

Typical Floor Live Loads

Floor = 50 psf

Partition = 20 psf

Roof Dead Loads

Slab and deck = 50 psf

Ceiling and mechanical = 15 psf

Roofing = 15 psf

Roof Live Loads

Minimum Live Load = 20 psf

Snow = 16 psf

Other Gravity Loads

Exterior Cladding = 15 psf (vertical surface)

Design Wind Loads

Use ASCE 7-05. See Figure 1 for exposure and terrain conditions.

Instructions for Structural Analysis

Only minimal instructions are provided for the analysis, as follows:

1. Assume the floor and roof diaphragms are rigid in-plane and flexible out of plane.
2. The bases of the columns of the moment frames are fully fixed.
3. The bases of the columns of the gravity and braced frames are pinned.
4. All structural analysis is to be performed as if you will be using the results to check the adequacy of the structure for wind drift.
5. When performing the structural analysis, it is very important that the building be analyzed in accordance with the current state of the practice in your firm. Please do not modify your calculations on the basis of what you infer from the questions asked in this survey.
6. Do not check the building for strength.

1) Enter the 8-Digit alphanumeric identification code you created when you completed Part 1 of the survey.

____

2) Compute the periods of vibration for the first six modes, and enter the results in the following table:

Mode	Period of Vibration (seconds)	Predomination Direction N-S, E-W, or Torsion
1		
2		
3		
4		
5		
6		

3) Compute the <u>total lateral displacement</u> that would occur at each level of the building if a 10 kip lateral force is applied at the center of mass of each level of the building (including the roof), as follows:

(a) Loads applied in the N-S direction only

(b) Loads applied in the E-W direction

Enter you results in the table below:

Level	Displacement (inches) for loads applied in the N-S direction only.	Displacement (inches) for loads applied in the N-S direction only.
R		
10		
9		
8		
7		
6		
5		
4		
3		
2		

4. Using ASCE 7-05 to determine wind loads, perform a structural analysis and complete the tables below. <u>Use the same analytical model as was used for question 4.</u>

Wind exposure used in determining wind loads (A, B, C, D)	
Wind recurrence interval used for drift calculations (years)	
Was the structure considered as Flexible or Rigid (for gust response)?	
Did you use a directionality factor in computing wind loads?	
Drift Measure Type (see below)	
Drift Limit Type (must be consistent with Drift Measure)	
Software used to analyze building	
Time it took you to determine the wind loads (minutes)	
Time it took you to set up the analytical model (minutes)	

Drift Measure types are as follows:
- Total drift (inches)
- Inter-story Drift (inches)
- Total Drift Ratio
- Inter-story Drift Ratio
- Local Shear Strain

For Winds Acting in the N-S Direction

Level	Total Wind Force (kips)	Total Drift at Center of Mass (inches)	Computed Drift Measure	Drift Limit
R				
10				
9				
8				
7				
6				
5				
4				
3				
2				

For Winds Acting in the E-W Direction

Story Level	Total Wind Force (kips)	Total Drift at Center of Mass (inches)	Computed Drift Measure	Drift Limit
R				
10				
9				
8				
7				
6				
5				
4				
3				
2				

5. Would you consider the building adequate with regards to wind drift serviceability? Please explain your answer.

6. Write a short (500 words or less) description of your analysis.

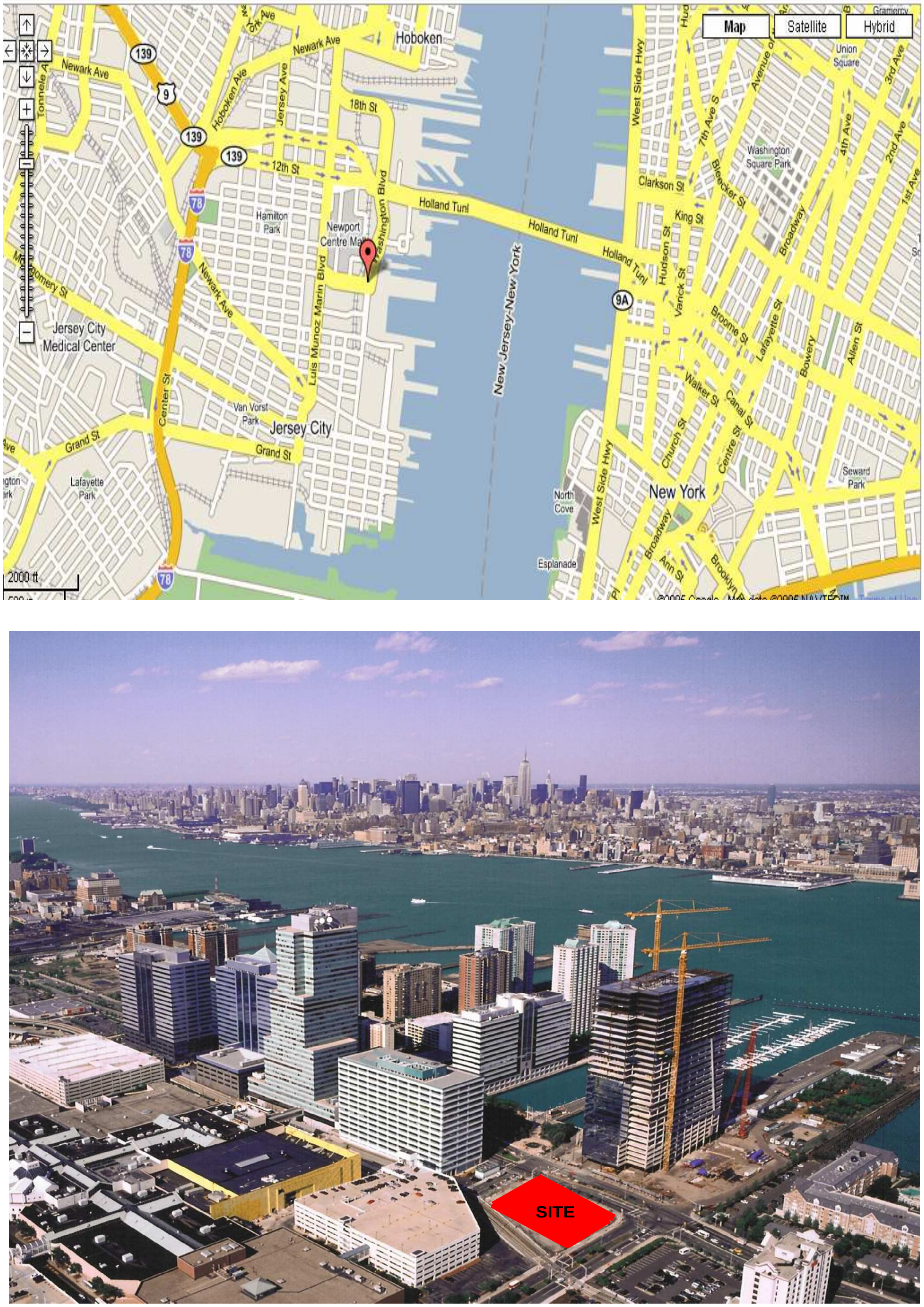

Figure A.1: Site Map and Aerial Photograph

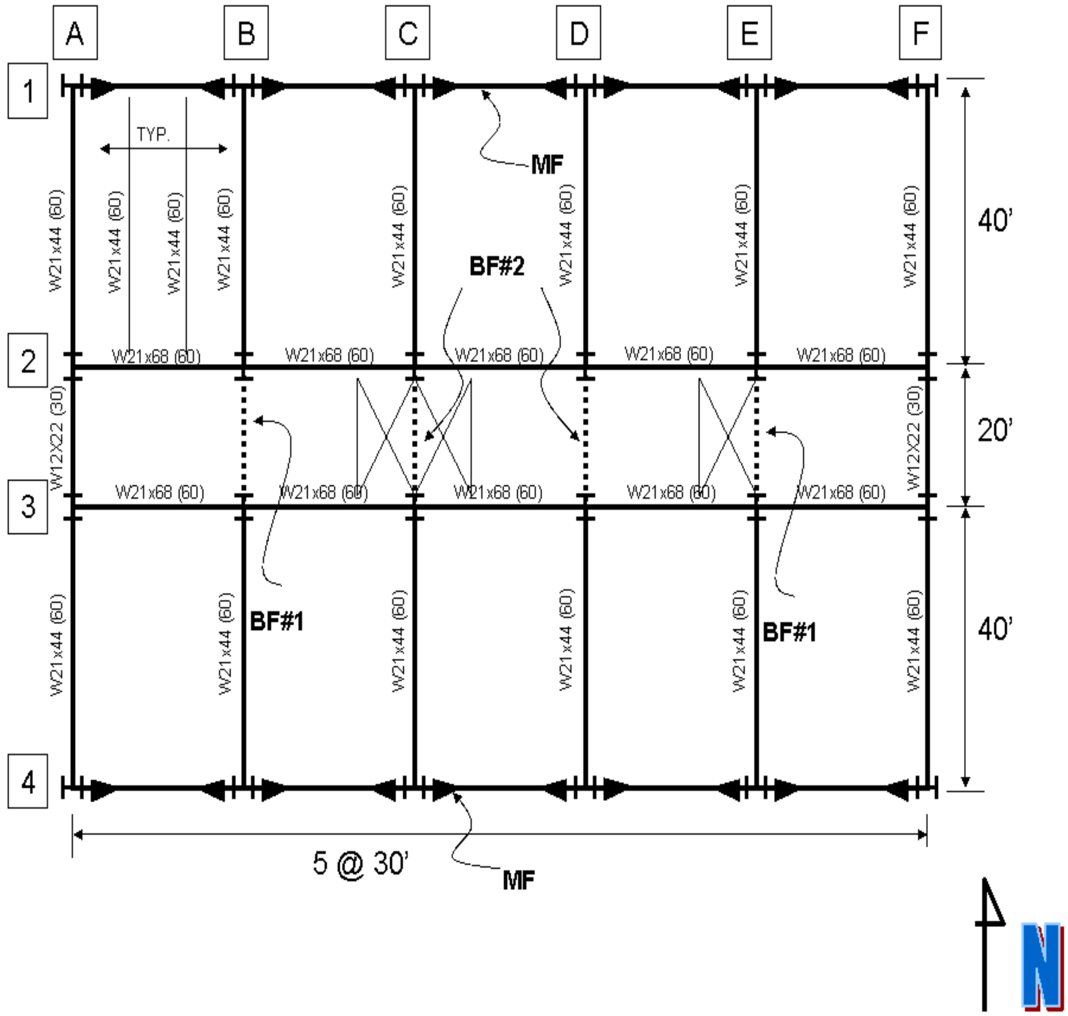

Figure A.2: Typical Floor Plan (Roof Plan is Similar)

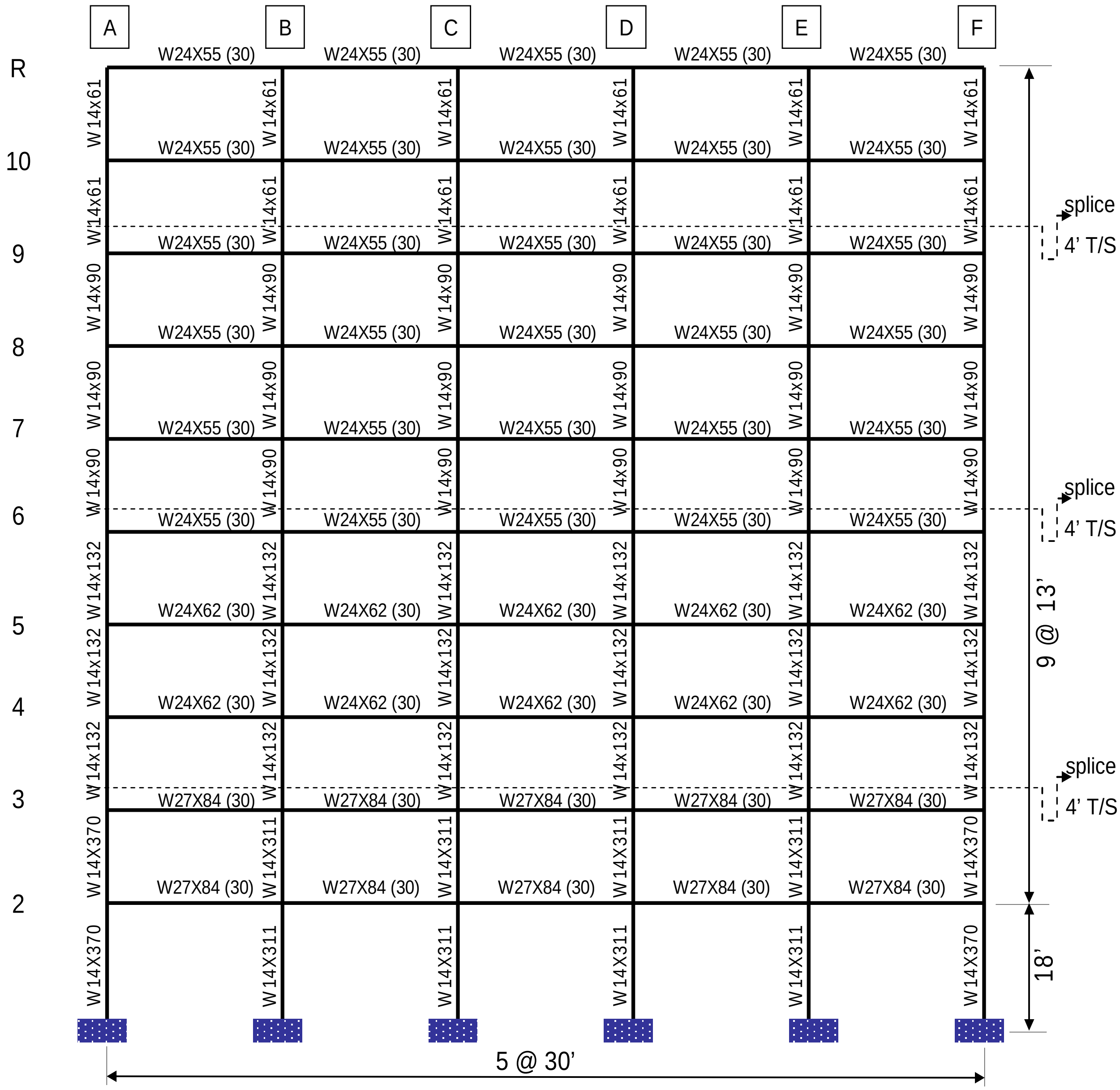

Figure A.3: Moment Frame
(Column lines 1 and 4)

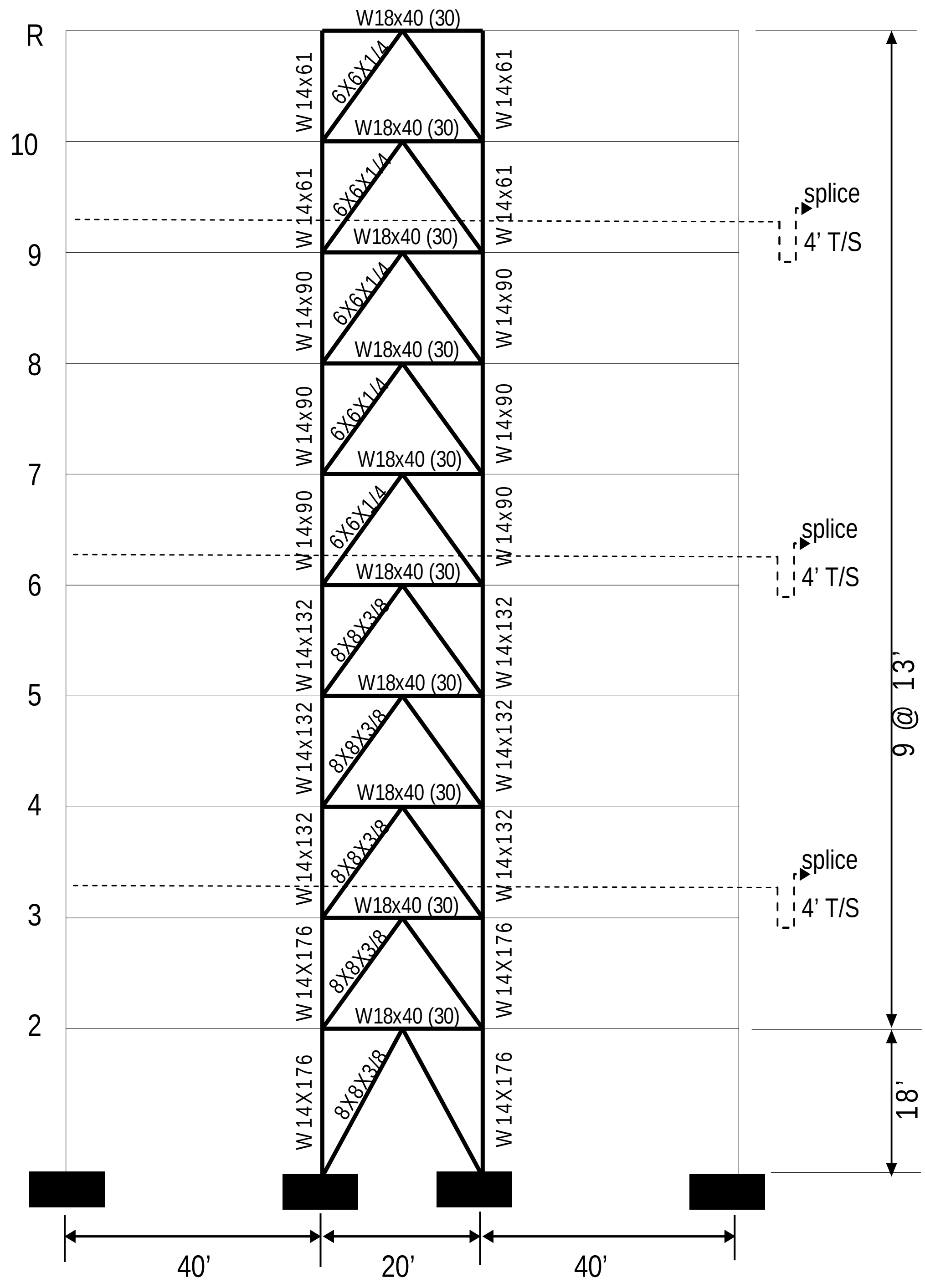

Figure A.4: Braced Frame #1
(Column Lines B and E)
*NOTE: all braces are HSS

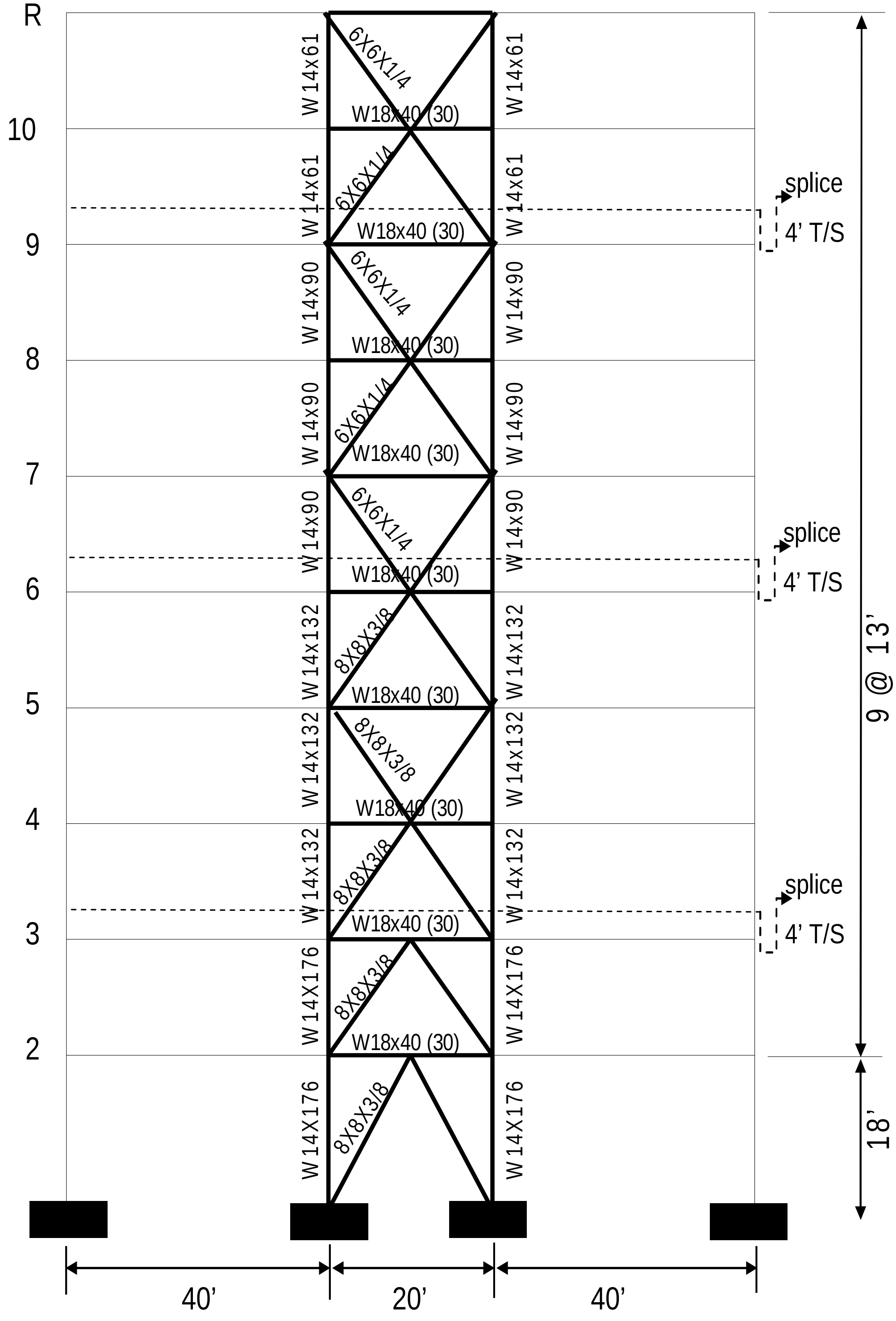

Figure A.5: Braced Frame #2
(Column lines C and D)
*NOTE: all braces are HSS

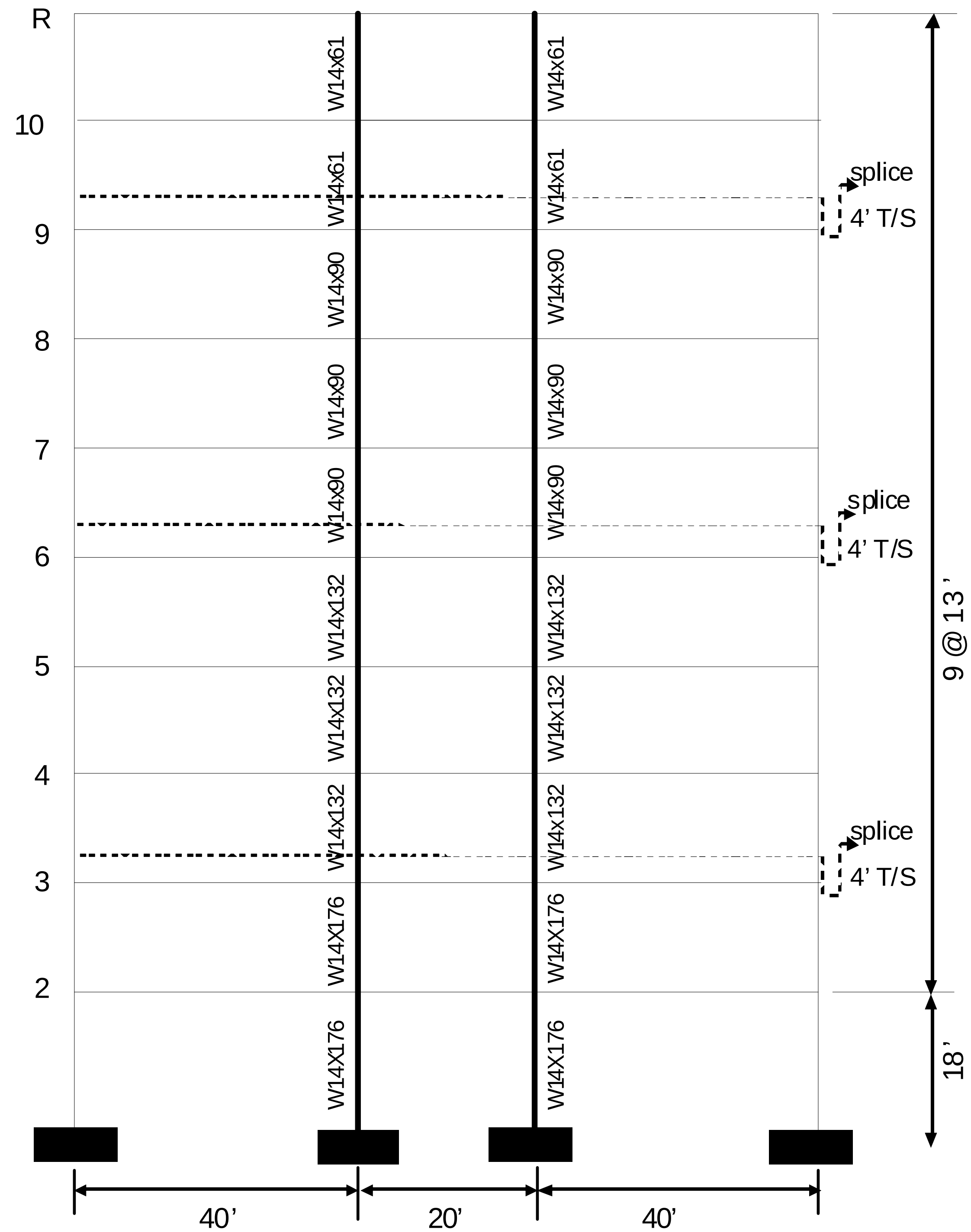

Figure A.6: Typical Gravity Frame
(Column lines A and F)

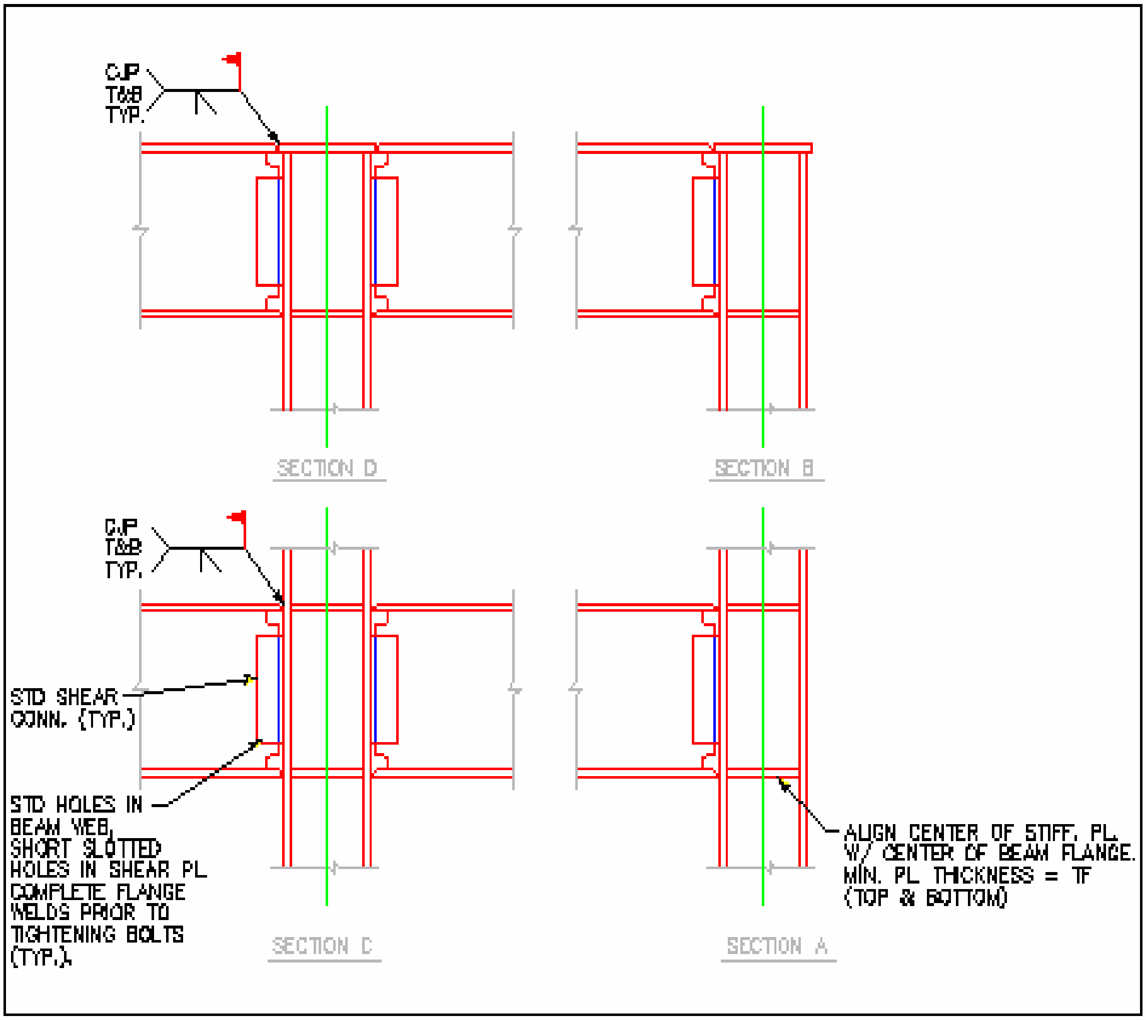

Figure A.7: Typical Moment Frame Connections

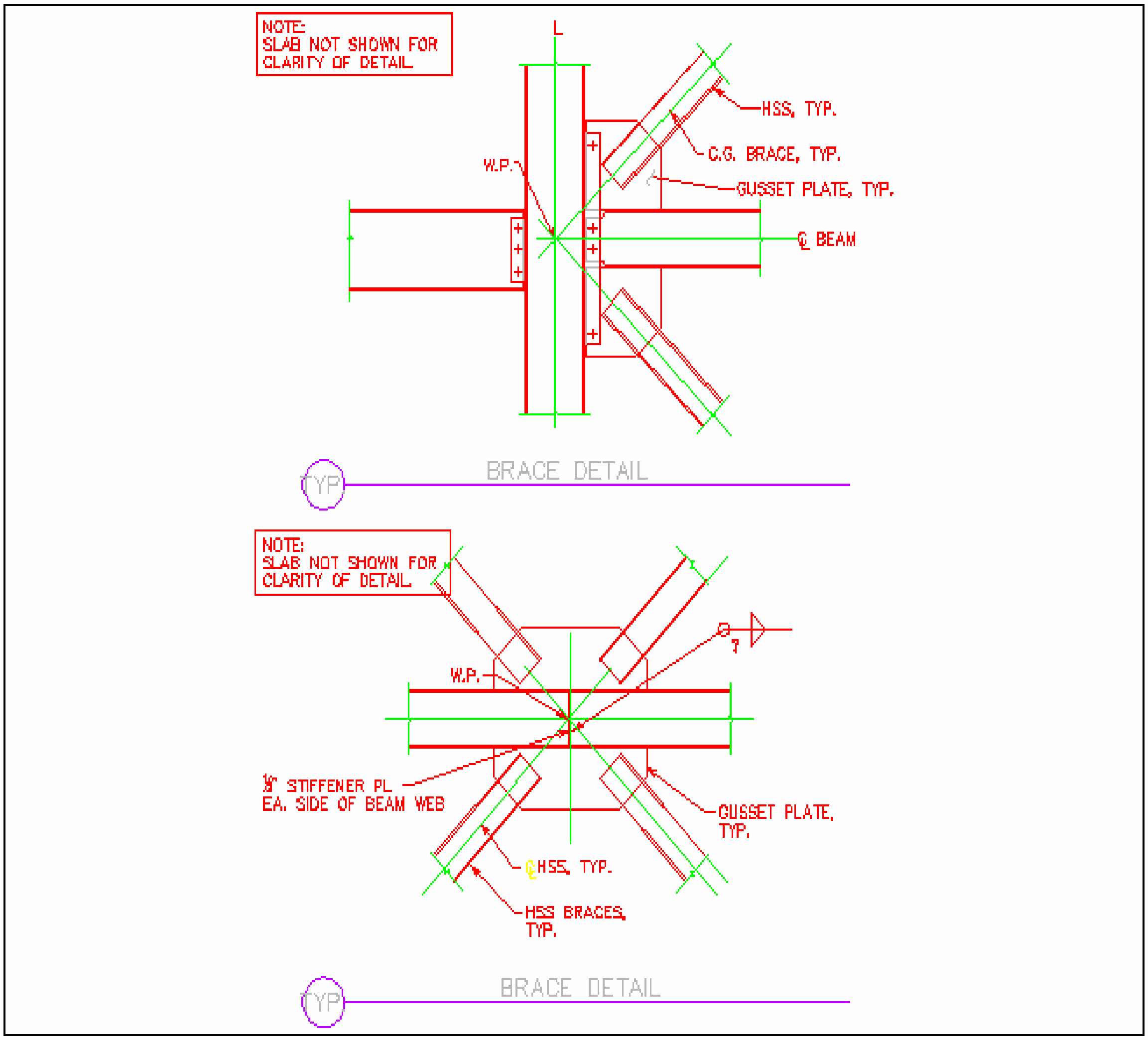

Figure A.8: Typical Braced Frame Connections

ASCE/SEI Wind Drift Survey Glossary

Centerline Analysis
Structural analysis which assumes that the beam-column joint region has zero width or depth. See Figure 9a.

Clear Span Analysis
Structural analysis in which the entire depth and width of the beam-column joint is considered to be infinitely rigid. See Figure 9c. Often referred to as Fully Rigid Joint analysis.

Consistent Geometric Stiffness
Geometric stiffness formulated on the basis of a cubic polynomial deformation pattern between member ends

Fully Restrained Connection
See the definition for Fully Rigid Connection

Fully Rigid Connection
A beam-to-column connection that provides complete flexural continuity between the beam and the column

Geometric Stiffness
That component of an elements stiffness that accounts for second-order (P-Delta) effects

Interstory Drift
The difference in lateral displacement between two adjacent levels of a building

Interstory Drift Index
The interstory drift divided by the vertical distance between the two adjacent stories for which the inter story drift is computed

Krawinkler Model
An analytical model of a beam-column joint as illustrated in Figure 10a

Linearized Geometric Stiffness
Geometric stiffness formulated on the basis of a straight-line deformation pattern between member ends

Mean Recurrence Interval (MRI)
Also called Return Interval. The MRI is the inverse of the annual probability. The 3-second gust speeds in ASCE 7-05 are based on a 50 MRI, or an annual probability of 0.02

Membrane Element
A planar finite element that resists in-plane forces only

Partially Restrained Connection
See the definition for Semi-Rigid Connection

Partially Rigid Joint Analysis
Analysis in which it is assumed that the beam column joint is rigid, but has reduced dimensions. See Figure 9b

Plate Bending Element
A planar finite element that resists moments about two mutually perpendicular axes that are situated in the plane of the element. The element has zero in-plane stiffness

Rigid Diaphragm
A floor or roof diaphragm that is assumed to be infinitely rigid in-plane, and infinitely flexible out-of-plane

Shell Element
A planar finite element that acts as a combined *membrane* and *shell* element

Semi-Rigid Connection
A beam-to-column connection that provides limited flexural resistance

Total Drift
The difference in lateral displacement between the roof of a building and the base of the building

Total Drift Index
The total drift divided by the height of the building

Scissors Model
An analytical model of a beam-column joint as shown in Figure 10b

Shear Strain
A dimensionless measure of damage in nonstructural elements

Root Mean Square (RMS)
The square root of the mean of the squares of a function. For example, the RMS of the function $f(t)=A\sin(\omega t)$ is $0.707A$

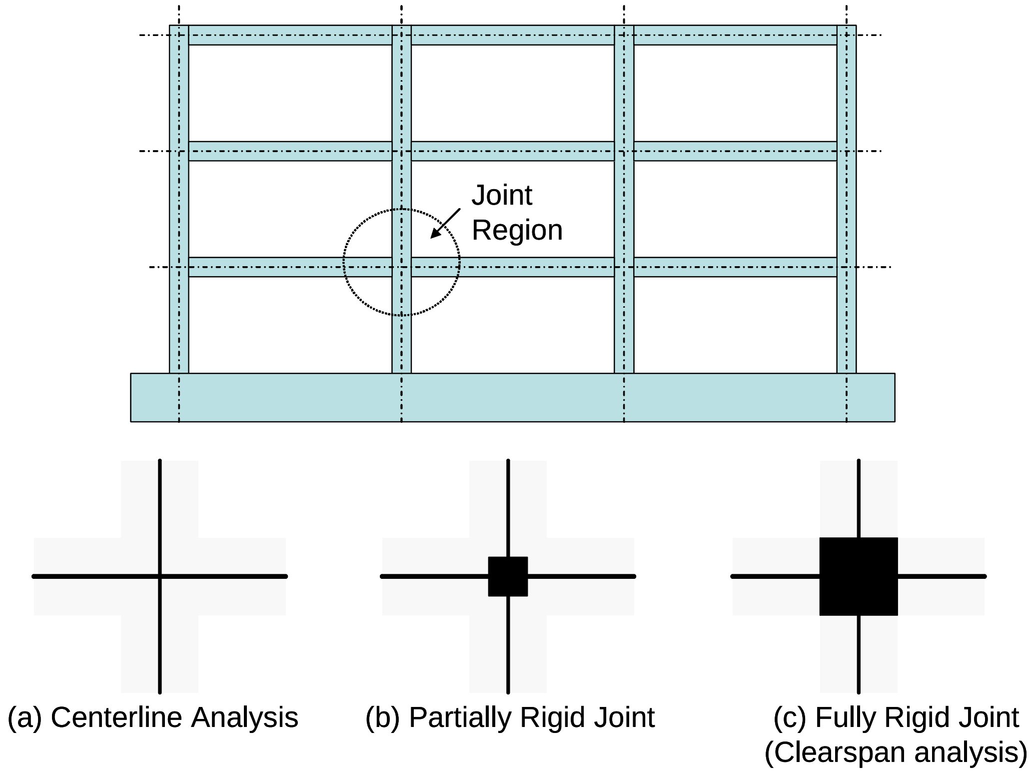

Figure A9. Various Simple Joint Models

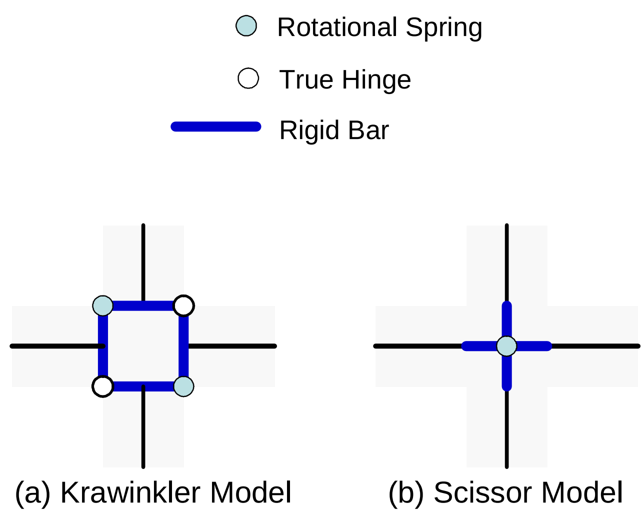

Figure A10. Various Mechanical Joint Models

Appendix B

Results from the Analytical Models in Chapter 3

B.1 Introduction

Tables of lateral displacements and modal periods are provided for all of the analytical models which were presented in Section 3.4.

B.2 Results from 3.4.1 (Sources of Deformation)

Table B.1: Lateral Displacements in the N-S Direction

	Model #			
Level	**1A (in)**	**1B (in)**	**1C (in)**	**1D (in)**
G	0.000	0.000	0.000	0.000
2	0.167	0.173	0.173	0.173
3	0.249	0.305	0.305	0.305
4	0.319	0.460	0.460	0.460
5	0.380	0.648	0.648	0.648
6	0.435	0.845	0.845	0.845
7	0.504	1.082	1.083	1.084
8	0.563	1.318	1.318	1.319
9	0.605	1.549	1.549	1.550
10	0.634	1.770	1.770	1.771
R	0.645	1.974	1.974	1.975

Table B.2: Lateral Displacements in the E-W Direction

	Model #			
Level	**1A (in)**	**1B (in)**	**1C (in)**	**1D (in)**
G	0.000	0.000	0.000	0.000
2	0.280	0.281	0.307	0.342
3	0.519	0.520	0.569	0.656
4	0.841	0.844	0.927	1.107
5	1.191	1.197	1.312	1.584
6	1.520	1.529	1.675	2.029
7	1.833	1.846	2.023	2.464
8	2.084	2.101	2.304	2.814
9	2.269	2.290	2.513	3.074
10	2.397	2.423	2.659	3.261
R	2.451	2.482	2.724	3.341

Table B.3: Modal Periods and Shapes

	Period (sec)			
Mode	**1A**	**1B**	**1C**	**1D**
E-W 1	0.784	0.788	0.825	0.913
Torsion 1	0.500	0.656	0.673	0.707
N-S 1	0.348	0.564	0.564	0.564
E-W 2	0.286	0.287	0.302	0.334
Torsion 2	0.199	0.232	0.267	0.247
N-S 2	0.172	0.183	0.183	0.201

B.3 Results from 3.4.2 (P-Delta Effects)

Table B.4: Lateral Displacements in the N-S Direction

	Model #			
Level	**2A (in)**	**2B (in)**	**2C (in)**	**2D (in)**
G	0.000	0.000	0.000	0.000
2	0.173	0.173	0.181	0.181
3	0.305	0.305	0.321	0.320
4	0.460	0.460	0.485	0.485
5	0.648	0.648	0.687	0.687
6	0.845	0.845	0.898	0.898
7	1.083	1.084	1.156	1.156
8	1.318	1.319	1.411	1.411
9	1.549	1.550	1.661	1.661
10	1.770	1.771	1.900	1.901
R	1.974	1.975	2.121	2.122

Table B.5: Lateral Displacements in the E-W Direction

	Model #			
Level	**2A (in)**	**2B (in)**	**2C (in)**	**2D (in)**
G	0.000	0.000	0.000	0.000
2	0.307	0.342	0.346	0.395
3	0.569	0.656	0.648	0.768
4	0.927	1.107	1.076	1.332
5	1.312	1.584	1.544	1.939
6	1.675	2.029	1.980	2.502
7	2.023	2.464	2.397	3.045
8	2.304	2.814	2.724	3.470
9	2.513	3.074	2.960	3.774
10	2.659	3.261	3.121	3.985
R	2.724	3.341	3.191	4.074

Table B.6: Modal Periods and Shapes

	Period (sec)			
Mode	**2A**	**2B**	**2C**	**2D**
E-W 1	0.825	0.913	0.826	0.913
Torsion 1	0.673	0.707	0.673	0.707
N-S 1	0.564	0.564	0.564	0.564
E-W 2	0.302	0.334	0.302	0.334
Torsion 2	0.267	0.247	0.237	0.247
N-S 2	0.183	0.201	0.183	0.201

B.4 Results from 3.4.3 (Beam Column Joint Modeling)

Table B.7: Lateral Displacements in the N-S Direction

	Model #			
Level	**3A (in)**	**3B (in)**	**3C (in)**	**3D (in)**
G	0.000	0.000	0.000	0.000
2	0.181	0.181	0.181	0.181
3	0.321	0.321	0.320	0.321
4	0.485	0.485	0.485	0.485
5	0.687	0.687	0.687	0.687
6	0.898	0.898	0.898	0.899
7	1.156	1.156	1.156	1.167
8	1.411	1.411	1.411	1.433
9	1.661	1.661	1.661	1.690
10	1.900	1.900	1.901	1.930
R	2.121	2.121	2.122	2.151

Table B.8: Lateral Displacements in the E-W Direction

	Model #			
Level	**3A (in)**	**3B (in)**	**3C (in)**	**3D (in)**
G	0.000	0.000	0.000	0.000
2	0.346	0.370	0.395	0.402
3	0.648	0.705	0.768	0.783
4	1.076	1.196	1.332	1.340
5	1.544	1.728	1.939	1.978
6	1.980	2.223	2.502	2.589
7	2.397	2.699	3.045	3.179
8	2.724	3.071	3.470	3.660
9	2.960	3.338	3.774	4.011
10	3.121	3.522	3.985	4.246
R	3.191	3.601	4.074	4.353

Table B.9: Modal Periods and Shapes

	Period (sec)			
Mode	**3A**	**3B**	**3C**	**3D**
E-W 1	0.826	0.868	0.913	0.941
Torsion 1	0.673	0.690	0.707	0.719
N-S 1	0.564	0.564	0.564	0.571
E-W 2	0.302	0.318	0.334	0.344
Torsion 2	0.237	0.242	0.247	0.250
N-S 2	0.183	0.191	0.201	0.202

B.5 Results from 3.4.4 (Slab-Girder Interaction)

Table B.10: Lateral Displacements in the N-S Direction

	Model #			
Level	**4A (in)**	**4B (in)**	**4C (in)**	**4D (in)**
G	0.000	0.000	0.000	0.000
2	0.181	0.181	0.181	0.181
3	0.320	0.321	0.321	0.321
4	0.485	0.485	0.485	0.485
5	0.687	0.687	0.688	0.687
6	0.898	0.899	0.899	0.900
7	1.156	1.167	1.158	1.168
8	1.411	1.433	1.413	1.434
9	1.661	1.690	1.663	1.691
10	1.901	1.930	1.903	1.931
R	2.122	2.151	2.125	2.153

Table B.11: Lateral Displacements in the E-W Direction

	Model #			
Level	**4A (in)**	**4B (in)**	**4C (in)**	**4D (in)**
G	0.000	0.000	0.000	0.000
2	0.395	0.402	0.362	0.370
3	0.768	0.783	0.684	0.704
4	1.332	1.340	1.172	1.194
5	1.939	1.978	1.686	1.749
6	2.502	2.589	2.159	2.277
7	3.045	3.179	2.627	2.797
8	3.470	3.660	3.000	3.229
9	3.774	4.011	3.273	3.549
10	3.985	4.246	3.466	3.764
R	4.074	4.353	3.544	3.857

Table B.12: Modal Periods and Shapes

	Period (sec)			
Mode	**4A**	**4B**	**4C**	**4D**
E-W 1	0.913	0.941	0.847	0.887
Torsion 1	0.707	0.719	0.684	0.701
N-S 1	0.564	0.571	0.565	0.573
E-W 2	0.334	0.344	0.319	0.332
Torsion 2	0.247	0.250	0.244	0.247
N-S 2	0.201	0.202	0.192	0.196

B.6 Results from 3.4.5 (Reduced Live Loads)

Table B.13: Lateral Displacements in the N-S Direction

	Model #			
Level	**5A (in)**	**5B (in)**	**5C (in)**	**5D (in)**
G	0.000	0.000	0.000	0.000
2	0.181	0.181	0.178	0.178
3	0.320	0.321	0.315	0.315
4	0.485	0.485	0.475	0.476
5	0.687	0.687	0.673	0.672
6	0.898	0.899	0.878	0.879
7	1.156	1.167	1.129	1.139
8	1.411	1.433	1.377	1.397
9	1.661	1.690	1.620	1.647
10	1.901	1.930	1.852	1.879
R	2.122	2.151	2.067	2.094

Table B.14: Lateral Displacements in the E-W Direction

Level	**5A (in)**	**5B (in)**	**5C (in)**	**5D (in)**
G	0.000	0.000	0.000	0.000
2	0.395	0.402	0.374	0.380
3	0.768	0.783	0.724	0.736
4	1.332	1.340	1.241	1.247
5	1.939	1.978	1.794	1.827
6	2.502	2.589	2.309	2.381
7	3.045	3.179	2.807	2.919
8	3.470	3.660	3.201	3.362
9	3.774	4.011	3.488	3.690
10	3.985	4.246	3.689	3.912
R	4.074	4.353	3.775	4.014

Table B.15: Modal Periods and Shapes

	Period (sec)			
Mode	**5A**	**5B**	**5C**	**5D**
E-W 1	0.913	0.941	0.913	0.941
Torsion 1	0.707	0.719	0.707	0.719
N-S 1	0.564	0.571	0.564	0.571
E-W 2	0.334	0.344	0.334	0.344
Torsion 2	0.247	0.250	0.247	0.250
N-S 2	0.201	0.202	0.201	0.202

B.7 Results from 3.4.6 (Nonstructural Components)

Table B.16: Lateral Displacements in the N-S Direction

	Model #			
Level	**6A (in)**	**6B (in)**	**6C (in)**	**6D (in)**
G	0.000	0.000	0.000	0.000
2	0.181	0.181	0.151	0.163
3	0.320	0.321	0.277	0.291
4	0.485	0.485	0.429	0.446
5	0.687	0.687	0.616	0.636
6	0.898	0.899	0.813	0.836
7	1.156	1.167	1.045	1.078
8	1.411	1.433	1.279	1.319
9	1.661	1.690	1.509	1.557
10	1.901	1.930	1.732	1.782
R	2.122	2.151	1.940	1.996

Table B.17: Lateral Displacements in the E-W Direction

	Model #			
Level	**6A (in)**	**6B (in)**	**6C (in)**	**6D (in)**
G	0.000	0.000	0.000	0.000
2	0.395	0.402	0.203	0.240
3	0.768	0.783	0.362	0.410
4	1.332	1.340	0.548	0.593
5	1.939	1.978	0.717	0.768
6	2.502	2.589	0.866	0.918
7	3.045	3.179	0.996	1.054
8	3.470	3.660	1.100	1.160
9	3.774	4.011	1.176	1.240
10	3.985	4.246	1.229	1.294
R	4.074	4.353	1.253	1.319

Table B.18: Modal Periods and Shapes

	Period (sec)				
Mode (A,B)	**6A**	**6B**	**Mode (C,D)**	**6C**	**6D**
E-W 1	0.913	0.941	Torsion 1	0.757	0.666
Torsion 1	0.707	0.719	E-W 1	0.748	0.566
N-S 1	0.564	0.571	N-S 1	0.741	0.550
E-W 2	0.334	0.344	Torsion 2	0.268	0.232
Torsion 2	0.247	0.250	E-W 2	0.255	0.207
N-S 2	0.201	0.202	N-S 2	0.227	0.173

B.8 Results from 3.4.7 (The Recommended Model)

Table B.19: Lateral Displacements in the N-S Direction

	Model #		
Level	**1A (in)**	**3D (in)**	**7A (in)**
G	0.000	0.000	0.000
2	0.167	0.181	0.178
3	0.249	0.321	0.315
4	0.319	0.485	0.476
5	0.380	0.687	0.673
6	0.435	0.899	0.879
7	0.504	1.167	1.139
8	0.563	1.433	1.398
9	0.605	1.690	1.647
10	0.634	1.930	1.880
R	0.645	2.151	2.095

Table B.20: Lateral Displacements in the E-W Direction

	Model #		
Level	**1A (in)**	**3D (in)**	**7A (in)**
G	0.000	0.000	0.000
2	0.280	0.402	0.348
3	0.519	0.783	0.659
4	0.841	1.340	1.106
5	1.191	1.978	1.608
6	1.520	2.589	2.090
7	1.833	3.179	2.570
8	2.084	3.660	2.975
9	2.269	4.011	3.278
10	2.397	4.246	3.485
R	2.451	4.353	3.577

Table B.21: Modal Periods and Shapes

	Period (sec)		
Mode	**1A**	**3D**	**7A**
E-W 1	0.784	0.941	0.887
Torsion 1	0.500	0.719	0.701
N-S 1	0.348	0.571	0.573
E-W 2	0.286	0.344	0.332
Torsion 2	0.199	0.250	0.247
N-S 2	0.172	0.202	0.196