Abstract

GRIFFIN, JOSHUA WAYNE. Experimental and Analytical Investigation of Progressive Collapse through Demolition Scenarios and Computer Modeling. (Under the direction of Dr. Emmett Sumner.)

Within the past 40 years, abnormal loadings resulting from natural hazards, design flaws, construction errors, and man-made threats have induced progressive collapse in structures all over the world. As progressive collapse behavior has become more prominent, it has made the necessity for design and analysis tools evident. In effort to provide one of these tools, Applied Science International, Inc. introduced its Extreme Loading® for Structures (ELS®) software, capable of progressive collapse simulation.

This research evaluates the effectiveness of Extreme Loading® for Structures as an emerging, nonlinear dynamic analysis software package in modeling progressive collapse scenarios. The ELS® software utilizes the Applied Element Method (AEM) of numerical analysis, separating it from other available software packages. The software and analysis methodology's accuracy are investigated through simulation of two structural implosions. Comparing the predicted response to the documented response, each scenario is evaluated by analyzing the material models, failure criteria, local structural behavior, and global collapse behavior.

The two case studies, Crabtree Sheraton Hotel in Raleigh, North Carolina and Stubbs Tower in Savannah, Georgia, each include an experimental and analytical investigation. The experimental investigations include gathering existing structural information, coordinating with the demolition contractor to simulate the implosion sequence, as well as observing and obtaining documentation from the actual event. The analytical investigation utilizes the Extreme Loading® for Structures software to construct a model for each structure, simulate

the implosion sequence, and analyze the predicted behavior. To understand the effects of individual modeling parameters on the model's response, a parametric study was completed. Creation of an evaluation matrix allowed for systematic assessment of the parametric study, as well as the individual model's behavior. For the case studies, a completed evaluation matrix for each iteration can be found in the appendix, providing a rough quantification of the accuracy.

Observations from this research show that the software is capable of successfully modeling progressive collapse scenarios. The software allowed for realistic construction of the models and was effective on various levels in predicting the local and global collapse behaviors. Inaccuracies were discovered in each model and were investigated through subsequent iterations of the analysis. A solution was found for some of the inaccurate aspects, while recommendations for future research are proposed to address the others.

Allowing for the quick and effective assessment of structures, the Extreme Loading® for Structures software has the potential to become a valuable tool in design and analysis of structures for progressive collapse mitigation. Through continuous validation and verification, modeling techniques and parameters can be established, providing engineers with confidence when venturing into this relatively new realm. Eventually, the advancement in knowledge and computing integrated into this software could provide invaluable benefit to society, in the form of economic cost and life-safety.

Experimental and Analytical Investigation of Progressive Collapse through Demolition Scenarios and Computer Modeling

by
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Dedication

This thesis is dedicated to my father to express appreciation for the principles and values he instilled in me at an early age. Without his desire to see me succeed, it would have been difficult for me to develop the motivation I have today. His assistance and support throughout this research was beyond what could have ever been asked of him. Witnessing the knowledge and character that he possesses has provided me with something to strive for in the years to come.

Biography

Joshua Griffin was born and raised in Julian, NC to Rusty and Marie Griffin. His father, working in the demolition industry, exposed him to several high-scale projects at an early age. He developed an interest in structures well before he could comprehend the physical behavior; much less imagine where this past would take him. In 2002, he enrolled in North Carolina State University where he studied Civil Engineering with a concentration in structures. After graduating as Valedictorian in 2006 with his Bachelor's of Science degree, he continued his education by enrolling in graduate school at North Carolina State University to obtain a Master's of Science in Civil Engineering. Upon completion of his graduate studies in the spring of 2008, he plans to continue learning through application of his knowledge in the design industry, ultimately working towards a Professional Engineer's license.

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Appreciation is also extended to Applied Science International, LLC for providing the Extreme Loading® for Structures software along with extensive technical assistance. Further appreciation is given to D.H. Griffin Wrecking Co., Inc. for their cooperation in providing site access and the structural information relevant to the implosions, as well as documentation for the implosions. In addition, Demolition Dynamics Company is recognized for providing the blast sequence in both case studies, but prior to the implosion in the second study allowing for the software to be used in a predictive manner. Appreciation is also expressed to Protec Documentation Services, Inc. for their cooperation and assistance in documenting the implosions. Lastly, gratitude is expressed to the Housing Authority of Savannah and the City of Savannah for their assistance and hospitality.

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EXPERIMENTAL AND ANALYTICAL EVALUATION OF PROGRESSIVE COLLAPSE THROUGH DEMOLITION SCENARIOS AND COMPUTER MODELING

1. Introduction

1.1. Background

If you wish to control the future, study the past.

-Confucius

For the past 40 years, the structural engineering community has been attempting to comprehend and quantify the phenomena of progressive collapse in order to adapt design codes and incorporate within them a sufficient level of progressive collapse resistance.

Progressive collapse can be defined as the propagation of failure through a structure initiated by local damage. Progressive collapse concludes in two possible outcomes:

1. the structure stabilizes, resulting in partial collapse or
2. global collapse.

The interest in progressive collapse has come in waves, with each wave triggered by a catastrophic failure. The landmark event was the Ronan Point Apartment Tower collapse in England in 1968. Although only four people died, the collapse prompted the interest and concern of engineers all over the world (Levy and Salvadori, 2002). The following decade produced many of the authoritative papers in the realm of progressive collapse.

A second wave of interest succeeded the Oklahoma City bombing in 1995, in which a terrorist detonated a bomb outside of the Alfred P. Murrah Federal Building. This was the

first progressive collapse event in recent history of the United States that was not caused by human error or natural disaster. Because of this failure, several investigative papers on the damage and progression of collapse were written. Many of the papers also included design recommendations for consideration in future design codes.

The last wave, which is still ongoing, was a result of the collapse of the World Trade Centers and the partial collapse of the Pentagon on September 11, 2001. Since 2001, interest in progressive collapse has been at its highest level ever. A major factor in the climax of interest can be attributed to the advancements made in building systems and computational capabilities since the landmark event in 1968.

With knowledge constantly expanding in the field of structures, designers and engineers are creating quicker, more cost-effective construction solutions. These innovative building systems, combined with a growing population and rapidly increasing land costs, contribute to the increase of multi-use structures that accommodate more people than ever before. This is evidenced by World Trade Center Towers 1 and 2, which each had a footprint of 208 feet by 208 feet or approximately one acre. Each acre housed on top of it, 110 stories in which nearly 50,000 people worked and another 200,000 people visited or passed through on a daily basis (Johnson and Ross). Upon collapse of the towers, 2,152 civilians, not including emergency assistant personnel, were killed instantly (United States National Commission on Terrorist Attacks, 2004). In contrast, the Ronan Point Apartment Tower, which served as the inaugural cause for the progressive collapse movement, was only designed to house approximately 310 people (Pearson and Delatte, 2005). The magnitude of the death toll from the World Trade Centers illustrates the immeasurable liability that accompanies structures. The liability has always been present, but with today's structures,

the liability level is unprecedented. Therefore, progressive collapse prevention is an aspect of design and analysis that must be addressed.

Breakthroughs in computer technology have also enhanced the interest in progressive collapse. The ability to perform dynamic, nonlinear analyses with extraordinarily refined meshes, in a reasonable amount of time, has provided government agencies, private firms, and academic institutions with tools that they can use for design, analysis, and research purposes. A major segment of current research is attempting to validate computer software to ensure that it is predicting the actual behavior. This is being accomplished by projects that model a potential collapse scenario and then conduct full-scale testing for comparison (Karns, Houghton, Hall, Kim, and Lee, 2006). By analyzing actual test data, reverse analysis can be used to determine some of the dynamic and nonlinear parameters or coefficients that are incorporated in the computer code.

In addition, extensive research is being performed in attempt to modify the design codes. Subsequent to each major event, the design codes endeavored to address progressive collapse. However, the guidance within the codes has always been qualitative, leaving open the possibility for different interpretations among engineering professionals. The ultimate goal is to create prescriptive or quantitative provisions for the design codes that directly address how to design against progressive collapse.

1.2. Objectives

The primary objective of this research is to evaluate the effectiveness of an emerging nonlinear, dynamic analysis software program in modeling progressive collapse scenarios. The software utilizes a new method of numerical analysis, known as the Applied Element

Method (AEM). The research is performed to investigate the accuracy of the analysis method contained within the Extreme Loading® for Structures (ELS®) software in modeling progressive collapse of building structures through comparison with the response of each structure during implosion. Each model is evaluated by extensively analyzing the material models, failure criteria, local behavior, and global behavior. The ultimate goal is to establish modeling techniques and verify modeling parameters by iteratively modeling the structures before and after the implosions and comparing the analytical results to the experimental documentation. The establishment of modeling techniques and validation of the ELS® software and its new method of analysis will allow engineers to utilize it for a wide-range of design, analytical, and investigative applications.

1.3. Scope

To accomplish the objectives of this research, two case studies were completed. The first case study analyzed the Crabtree Sheraton Hotel in Raleigh, NC, which was imploded on May 7, 2006. The second case study was performed on Stubbs Tower in Savannah, Georgia, which was imploded on December 15, 2007. Each case study was accomplished through an experimental and analytical investigation. The experimental investigations consisted of gathering construction information on the structures, working with the demolition contractor to simulate the implosion sequences, as well as observing and attaining documentation from the implosions. The analytical investigations utilized ELS® software to construct models, simulating the implosion of each structure. A parametric study was completed to study the effects of individual parameters on the response of the model. An evaluation matrix was then created to allow for systematic assessment of each model and the

subsequent iterations. Lastly, recommendations and conclusions were made based on the final results obtained from each study.

1.4. Organization of Thesis

Presented within this thesis, a thorough review of progressive collapse literature and highlights of current research are contained in Chapter 2. Analytical background describing the fundamentals of the Applied Element Method, as well as the advantages it offers, can be found in Chapter 3. Chapters 4 and 5 present the experimental and analytical investigations that were performed for each case study. Finally, within Chapter 6, a summary of the research combined with recommendations and conclusions is presented. The last two chapters contain the references and appendices.

2. Literature Review

2.1. Introduction

This chapter provides a review of the current state of knowledge in the area of progressive collapse. Various types of abnormal loads that can influence structures are summarized along with the structural behavior that they induce. A review of current design code provisions is also encompassed, which details different design approaches for mitigating progressive collapse. General structural characteristics that are recommended when trying to prevent progressive collapse are included as well.

2.2. Definition of Progressive Collapse

The American Society of Civil Engineers defines progressive collapse in the commentary for ASCE 7-05, *Minimum Design Loads for Buildings and Others Structures* as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it." (American Society of Civil Engineers, 2005). A similar definition is offered by the General Services Administration in their *2005 Facilities Standards (P100)*, in which they describe progressive collapse as damage to a structure that is disproportionate to the initiating event (GSA 2005).

It is vital to state at this point that a structural engineer's objective is not to completely prevent progressive collapse. The absolute prevention of progressive collapse would require such economical resources that it is not feasible (Faschan, Garlock, and Sesil, 2003). Instead, the structural design goal is to inhibit disproportionate progressive collapse, which is what the codes are referring to when they use terms such as, "a disproportionately

large part" or "additional collapse". Progressive collapse is disproportionate when the global damage is not proportional to the event that triggered the collapse (Nair, 2006).

With regards to past failures, the Ronan Point Apartment Tower collapse is the best example of disproportionate progressive collapse. The triggering event was a gas explosion that did not take the life of the person in the room (Nair, 2003). Yet, the entire corner of the 22-story tower collapsed. Because the explosion only caused local damage to the exterior walls, but the failure propagated through the full height of the structure, the collapse can unquestionably be labeled as disproportionate.

The Murrah Federal Building bombing is difficult to categorize. Even though there was severe damage done to the structure, the triggering event, a truck loaded with approximately 4000-lbs of explosives, was also massive (Marchand and Alfawakhiri, 2005). Considering the design criteria for the structure when it was constructed, with this type of loading, extensive damage is to be expected. Therefore, it is hard to describe its collapse as disproportionate.

Utilizing the same definition, the collapse of the World Trade Center Towers was by no means disproportionate. With the impact of the aircraft and the resulting fire acting as the triggering event, the towers performed adequately in bridging the damaged area and temporarily arresting progressive collapse (Hamburger and Whittaker, 2003). The eventual collapse of each tower was eminent given the initial conditions produced by the aircraft.

2.3. Abnormal Loads

Structures can be subject to abnormal loading as a result of vehicular impacts, explosions, and even human error. Human error, in the form of faulty design or premature

removal of temporary supports can cause abnormal loads on a structure. Without considering human error, the rest of the loads can be characterized as either impact loads or impulse loads (Marchand and Alfawakhiri, 2005).

Impact loads consist of vehicle or aircraft collision on a structure. These loads are complex to quantify because the stiffness and crushing strength of the impacting object must be taken into account (Marchand and Alfawakhiri, 2005). Therefore, impact loads are not further addressed.

Impulse loads embody gas explosions and bomb detonations. A distinguishing characteristic within the category of impulse loads is the rate of the chemical reaction. The rate of reaction classifies the chemicals as either low explosives or high explosives (Longinow, 2004). Low explosives, such as propellants and natural gas, exert their energy through a rapid burning process known as deflagration (Marchand and Alfawakhiri, 2005). High explosives, such as TNT and Ammonium Nitrate, release energy through detonation (Longinow, 2004). The next section discusses in further detail, the behavior of explosive or blast loads.

2.3.1. Blast Loading Characteristics

An explosion is a rapid release of stored potential energy characterized by a bright flash and an audible blast (Marchand and Alfawakhiri, 2005). Explosions can originate from pressure vessels, the detonation or deflagration of dust particles, the release of detonable gas in a restricted space, improvised explosive devices (IED's, such as ANFO), or a wide variety of commercial and military explosives (Marchand and Alfawakhiri, 2005).

The location of an explosion when it occurs determines the behavior of the shock wave it produces. An air blast will create a spherical shockwave propagating outward from the point of detonation. An explosion on or relatively near the ground, like a car bomb, is known as a surface blast. In a surface blast, the initial wave is reinforced by the ground surface to produce a reflected wave. This creates a hemispherical shockwave that is more destructive than an equivalent air blast (Longinow, 2004).

The principal characteristics that define a blast wave are the peak pressure (amplitude), duration, shape, and impulse. Impulse is the integral of pressure over time (Marchand and Alfawakhiri, 2005). All blast waves produce an initial, instantaneous rise to the peak pressure followed by an exponential decay that leads into a negative pressure phase behind the shockwave. A simplified blast diagram from the Department of Defense's *TM5-1300: Structures to Resist the Effects of Accidental Explosions* can be seen below (1990).

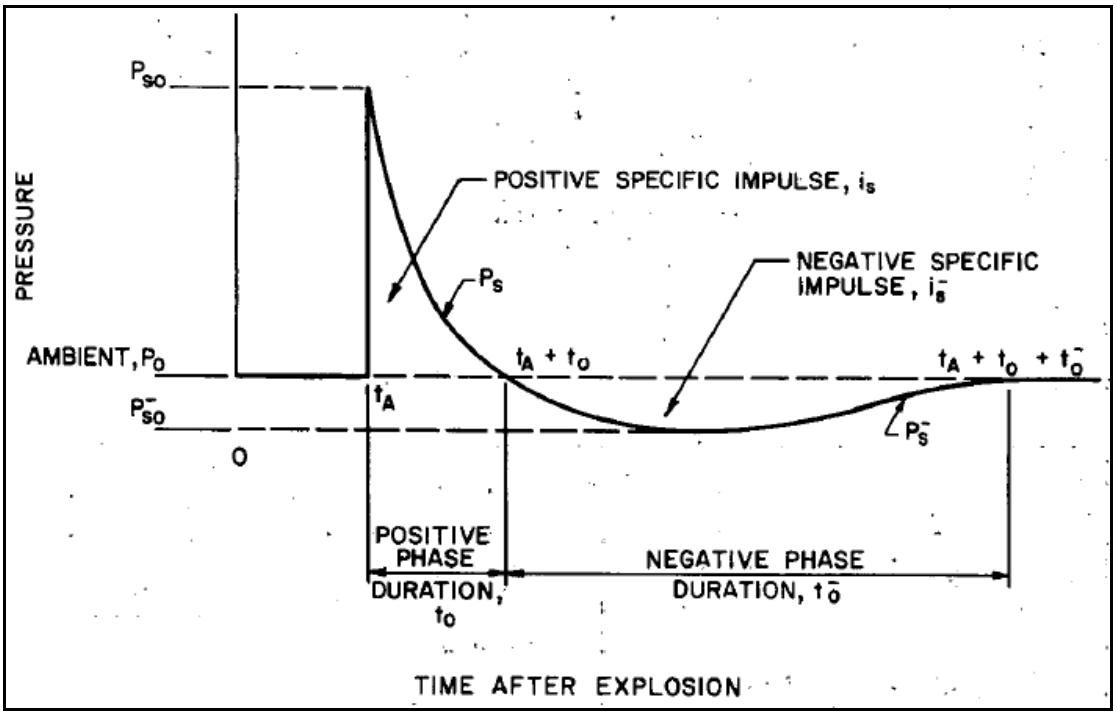


Figure 1. Free-Field Pressure vs. Time Blast Diagram (DoD, 1990)

Another parameter influencing the loads produced from an explosion is the location of the blast relative to the interior of a structure. If an explosive is detonated within a structure, quasi-static gas pressure loads can arise from the confinement of the products of the explosion (Longinow, 2004). This pressure can have a longer duration than the shock pressure and is dependent on the amount of confinement in the structure. If a structure is well-vented, the pressure will subside quicker (Longinow, 2004).

2.4. Structural Response to Abnormal Loading

The duration over which the blast impulse acts on a structure is usually a fraction of the structures natural period. Therefore, short duration, abnormal loads do not engage the structures global lateral force-resisting system (Marchand and Alfawakhiri, 2005). Instead, the damage that emanates from a blast is usually local. This local damage however, is what can initiate the progression of collapse.

2.4.1. Column Response

Past research has shown what type of behavior typically controls different structural members when subject to blast loads. For concrete columns, shear behavior controls the response due to the large amount of longitudinal steel that acts as flexural reinforcement. The vertical stirrup spacing is too large to prevent shear failure in the column (Oswald, 2005). In steel columns, behavior depends on the boundary conditions at the base of the column. If a base plate is used to anchor the column above the slab, connection failure will occur through the shear plane of the bolts. If the column is continuous into the slab, not providing a shear plane, then flexural response will control the failure (Karns et al., 2006).

2.4.2. Beam Response

The response of beams is rather complicated because of the direction of loading when subject to a surface blast. Because most explosions occur as surface blasts, beams are typically loaded laterally or from underneath. This produces twisting in the beam due to the fact that the top flange is connected to the slab. The effect of the column's response that the beam is framing into also magnifies the complexities. As a result of this behavior, gravity loads will have an adverse effect on the beam and its connections (Marchand and Alfawakhiri, 2005).

2.4.3. Slab Response

Historically, concrete slabs have behaved poorly in blast loading events because they are not designed for uplift pressure. In the Murrah Federal Building collapse, 112 of the 180 floor and roof panels failed either partially or completely (FEMA, 1996). When floor slabs fail, columns lose their lateral bracing. Loss of lateral bracing means columns have unbraced lengths much greater than their design values which often leads to instability and buckling (Princehorn and Laefer, 2005). This epitomizes how local damage, to a floor slab, can lead to global and disproportionate collapse.

2.4.4. Connection Response

A structural system is only as effective as its connections, similar to the weakest link in a chain. Even after individual members fail, proper connections can tie a structure together and provide stability. However, the design and detailing of connections to resist blast loads is extremely complex. The short duration of blast loading produces strain rate effects, which alters the connections physical properties. The strength is affected by a

dynamic increase factor. The dynamic increase factor scales the yield strength and to a lesser degree, the ultimate strength, of the material to account for the rapid loading rate. Along with increased strength, the material loses a portion of its ductility, which is vital for inelastic deformations (Krauthammer, 2003). Attempting to accurately account for all of these second-order effects is what makes modeling of connections so complex.

Connections can also be subject to combined moment and tension due to excessive deformation of the supported beams as described in the next section. This is yet another obstacle in the design of a connection capable of suppressing disproportionate progressive collapse.

2.4.5. Development of Catenary Forces

Failure of a column can create a two span condition in a structure. If the connection has sufficient ductility and rotational capacity, excessive deformation will occur in the double span. The beams or slabs will begin to transfer the vertical load through tension instead of by flexure (Marchand and Alfawakhiri, 2005). These components will act as cables between columns, developing significant tensile forces that the connection must be able to withstand. Many conventional connections and even newer, post-Northridge connections have been found to be inadequate to develop catenary forces because they are limited by either their tensile capacity when combined with moment or their rotational capacity (Krauthammer, 2003).

2.5. Structural Analysis for Abnormal Loads

2.5.1. Structural Analysis Methods

When analyzing a structure, four different methods can be implemented to investigate the structures behavior. The methods vary extensively on the amount of time and resources required for their usage. They are listed below in order of increasing complexity.

Linear Static

The linear static method of analysis is the most simplified form of analysis. Neither geometric nor material nonlinearity is considered. This method of analysis is only capable of analyzing very simple structures because the dynamic and nonlinear effects must be estimated by the user. The structures response is evaluated by demand to capacity ratios (Marjanishvili and Agnew, 2006).

The advantage this method offers is that it can be completed quickly, providing simple, fundamental results. However, with linear static analysis it is nearly impossible to accurately predict the dynamic effects in progressive collapse or blast scenarios.

Nonlinear Static

The nonlinear static method allows for nonlinear material properties to be defined. The most common nonlinear material model is an elastic-perfectly plastic curve. This analysis method is a step above the linear static model because of its ability to consider changing material properties. Nevertheless, this method still neglects the dynamic effects, so it provides little additional benefit in trying to understand a structures behavior.

Linear Dynamic

Dynamic analysis, whether linear or nonlinear, compared to static analysis is much more complex. Dynamic analysis inherently accounts for dynamic amplification factors, inertia and damping forces (Marjanishvili and Agnew, 2006). Thus, estimation of these parameters by the user is no longer required.

The limitation with linear dynamic analysis is its inability to account for geometric and material nonlinearity. Dynamic analysis demands significantly greater time compared to static analysis, but provides a substantial increase in the level of accuracy.

Nonlinear Dynamic

Nonlinear dynamic analysis is the most detailed and intricate analysis possible. It covers geometric nonlinearities, including second order effects such as P-delta. It allows material models to be specified which can define properties such as yielding, strain hardening, and strain rate effects. The nonlinear capabilities combined with the dynamic capabilities described in the linear dynamic analysis section, make this the ideal analysis method for modeling structures subject to abnormal loads (Marjanishvili and Agnew, 2006).

The only disadvantage to this method is that it requires the greatest amount of time to create the model and the longest amount of time to execute the model. Therefore, the type of model created to design or analyze a structure can become an economical issue because most models require at least a few iterations.

2.5.2. Progressive Collapse Analysis Software

There are several producers of structural analysis software, but not all of them allow for reasonable consideration of progressive collapse. Some structural analysis software, like

RISA-3D for example, can perform nonlinear dynamic analyses, but it does not have applications directly targeted at progressive collapse simulation thus making it difficult to be used in that manner. However, other nonlinear dynamic software programs either have direct applications intended for use in predicting progressive collapse or they have been adapted and modified so that they can be used in simulating collapse scenarios.

The capabilities of each software program vary, depending on what material models they include and what dynamic effects they consider. Below is a summary of the most common software packages currently being used for research or design in the fields of blast simulation and progressive collapse prediction.

FLEX

FLEX is a family of codes, first created by Weidlinger Associates Inc. in the 1980's, which derives its name from its ability to perform Fast Large EXplicit time-domain dynamic simulations (Weidlinger Associates Inc.). The software can be used to model a structure's response to blast and impact loads.

The FLEX software has a well-established history for modeling of structures subject to abnormal loads. In "Analytical Tools for Progressive Collapse Analysis", which was presented during a national workshop hosted by the National Institute of Building Standards in 2002, the FLEX software is compared side by side to other software programs on the market at that time (Smilowitz, 2002). The author begins by listing the software's capabilities that he deems vital for accurately modeling progressive collapse. The author states the software must be able to represent the sudden application of abnormal loading, the dynamic behavior of materials under very high strain rates, the inelastic post-damage

behavior of the materials, and the geometric nonlinearity resulting from large deformations (Smilowitz, 2002). All of which are included within the FLEX code.

FLEX software has been used to model various reinforced concrete and steel frames to determine critical levels of certain components when subject to column removal (Hansen, Wong, Lawver, Oneto, Tennant, and Ettouney, 2005). The goal of the research was to create a database by repeatedly analyzing typical structures while varying one component at a time. For example, the bay length was varied to see at what distance, progressive collapse would initiate after the removal of a column. The FLEX software allowed for the consideration of nonlinear material models for the concrete and reinforcing steel, including rate effects and softening. Element erosion based on strain was implemented to remove extremely distorted or damaged elements from the model (Hansen et al., 2005).

LS-DYNA

LS-DYNA is a general purpose transient dynamic finite element program capable of simulating extremely complex problems (Livermore Software Technology Corporation). In 1996, LS-DYNA's creator, Livermore Software Technology Corporation, partnered with ANSYS, Inc. to create a software package tailored specifically for structural analysis.

Researchers have used LS-DYNA in the United States and the United Kingdom. The University of Sheffield utilized LS-DYNA to investigate the magnitude of tying forces that were generated in a multi-story steel frame building upon removal of a column (Liu, Davison, and Tyas, 2005). By modeling a braced frame, typical of construction in the United Kingdom, LS-DYNA allowed them to remove columns and record the peak tying forces and the tying forces once equilibrium occurred. This research proved that tying force

requirements implemented with the UK design code after the Ronan Point collapse were inadequate to arrest progressive collapse for this type of structure (Liu, Davison, and Tyas, 2005).

Similar, more recent research in the United States, sponsored by the General Services Administration and the Defense Threat Reduction Agency, aims to evaluate the behavior of steel frame connections subject to blast and determine their post-blast integrity for the purpose of mitigating progressive collapse (Karns et al., 2006). The objective of the research is to quantify the capacity of typical steel frame connections, earthquake connections, and a new SidePlate® connection when an explosive charge is detonated in close proximity. Instead of modeling the entire structure, the only members included in the model were ones included in the field testing. The model consisted of one exterior column, two edge beams going off each side of the column, and an interior beam going off the back of the column. In order to capture all of the localized nonlinear behavior within the connection, some of the finite element models exceeded 2.5 million elements.

The initial test results showed that the finite element predictive analysis was accurate. The deformed shape was exactly as predicted and the magnitude of displacement was very close to the finite element results (Karns et al., 2006).

Additional research within the United States has attempted to model the behavior of simply supported reinforced concrete beams subject to blast loading inside of a shock tube (Yi, Agrawal, Ettouney, and Alampalli, 2007). The basis for this research was to investigate the effect of load generation, time step, and mesh size on the finite element simulation results obtained using LS-DYNA. The reinforced concrete beams were modeled using 3632

elements, approximately 1 inch in length, in attempts to capture the four steps in the failure process. The failure process when subject to blast loading was as follows (Yi et al., 2007):

1. Spalling of concrete on the back of loading face.
2. Cracking of tensile concrete in the region of maximum moment.
3. Severance of longitudinal tensile rebar.
4. Crushing of concrete in compression zone.

Failure criteria within the model was defined using values for reinforced concrete subject to high strain rates. These values, as listed below, have been extensively researched:

Table 1. Ultimate Strains Used in Reinforced Concrete FE simulation (Yi et al., 2007)

Material	Failure Strain
Steel Rebar	0.23
Core Concrete	0.005
Cover Concrete	0.002

Results of the research showed that load generation must be performed by a method that is not affected by eroding elements. In some analyses, when elements fail and they are removed from the model, the blast or pressure loading can be lost. Therefore, a new method of blast load generation is proposed that is not affected by eroding elements (Yi et al., 2007).

Investigation of the time step proved that smaller time steps attenuate the pressure faster. As a result, the time step used was the largest constant time step, which satisfied the critical time step condition. The critical time step corresponds to the time it takes the blast wave to travel across the shortest distance between any two nodes of one element. This ensures numerical stability by guaranteeing that the wave will not propagate over more than one element during a time step (Yi et al., 2007).

With respect to the finite element mesh, previous research by Krauthammer and Otani (1997) has shown that one element cannot accurately account for the effects of concrete and reinforcing steel together. In the current research, varying the mesh size caused less than 5% change in the reaction forces of the simply supported beam. However, with the same range of mesh sizes, the deflection varied over 400% (Yi et al., 2007). It was determined that deflection increases with increases in mesh size.

Overall, LS-DYNA was able to accurately predict the failure modes and magnitudes of various structural components subject to abnormal loading.

SAP2000, ETABS, and PERFORM 3D

Computers and Structures, Inc., CSI, was developed out of the University of California at Berkley in 1975 and since then they have produced several programs specifically tailored to civil engineering (Computers and Structures, Inc.). CSI's most renowned software is SAP2000, which is currently the industry standard for earthquake and structural analysis in the United States. SAP2000 allows for nonlinear dynamic analysis of any civil engineering structure. With the ETABS package in SAP2000, the analysis of multi-story building structures can be simplified.

CSI recently released PERFORM 3D which is a nonlinear software tool focused on using performance based design for structures. PERFORM 3D is compatible with SAP2000 and ETABS allowing models to be directly imported. PERFORM 3D considers dynamic loadings and completes nonlinear analysis using user-defined hysteretic loops. Vast amounts of structural information can be input into PERFORM 3D making it extremely useful in the design stages of a structure.

Recently, SAP2000 has been utilized to predict the behavior of a 10-story reinforced concrete building after an exterior column on the first floor was removed through an explosion (Sasani, Bazan, and Sagiroglu, 2007). The research aimed to identify the structural behavior generated to mitigate collapse as a result of the column removal. Therefore, the local or direct effects of the air blasts were not studied. For the analytical modeling, SAP2000 was used to create a model of the structure and investigate the redistribution of forces after the column was instantaneously removed.

Comparing the analytical results to the experimental data recorded through strain gages and linear potentiometers, the model was shown to be very accurate in predicting the final displacement of the joint above the removed column. The significant differences in the analytical results came from trying the estimate the amount of damping in the building at the time of the research. The experimental results appear to show that the structure had a damping ratio greater than 30 percent. However, it proved to be inconsequential because as the vibration dissipated, the residual displacement was approximately the same for the experimental and analytical data.

In conclusion, the researchers determined the main method of load redistribution was vierendeel or frame action, characterized by relative vertical displacement between beam ends and double curvature deformations of beams and columns. Vierendeel action transferred approximately 70 percent of the load to the adjacent column connected through the frame, while 12 percent was transferred to columns on each side of the frame through the one-way slabs (Sasani, Bazan, and Sagiroglu, 2007). The last 6 percent was transferred to columns located diagonally across the bays.

As this research shows, SAP2000, ETABS, and PERFORM 3D are all excellent tools when analyzing a structure that remains stable during loading. However, in a progressive collapse event, none of these programs allow for members to separate or simulate the collapse sequence. These programs can determine whether or not collapse will initiate, but once collapse initiates it requires several iterations within the model to determine which members will successively fail and how far collapse will progress.

Extreme Loading® for Structures

Extreme Loading® for Structures, ELS®, is a newer analysis program founded on completely different fundamental principles than the previous three. ELS® is based on the Applied Element Method whereas the previous programs are built on the Finite Element Method of analysis (Applied Science International, 2006). Similar to the other software, nonlinear dynamic analysis can be performed with ELS® and material models can be defined along with failure criteria. Further technical background is presented later.

ELS® has been used to simulate the behavior of past progressive collapse events such as the Murrah Federal Building. The software was utilized to perform iterations of the analysis, calculating the extent of progressive collapse for various locations of the bomb. ELS® illustrated how consequential stand-off distance is to a structure during a blast event (Applied Science International, 2006).

At the University of Miami, Dr. Antonio Nanni is using ELS® on two security-related projects. One project is a part of the Repair of Buildings and Bridges with Composites research, in which the effect of blast loading on concrete slabs strengthened with composite materials is being investigated. The other project involves the controlled

demolition and simulation of the Tenza Bridge, south of Naples, Italy. ELS® is being used to model the bridge prior to and during the explosive demolition (Homeland Defense, 2007).

ELS® has also been used commercially to simulate explosive demolition scenarios on high-priority structures. In June 2007, the software was used to model the implosion of the Charlotte Coliseum in Charlotte, NC (Homeland Defense, 2007). By coordinating with the explosive contractor, an accurate model was created prior to the demolition that verified the sequence of delays they specified. However, modifications were made to the blast plan by the explosives subcontractor shortly before the implosion differentiating it from the ELS® model.

2.5.3. Analysis Software Commentary

Given the four analysis software programs previously mentioned, there is not a definite answer for which one is superior. Some of the software has been around for a long time and developed a substantial resume in the research community. Most of them have had their accuracy quantified and meticulously scrutinized. However, through peer evaluations, all of them have been shown to be lacking to some degree, whether it is the load generation technique, the material models, the failure criteria, the erosion of elements, or even the fundamental principles. Therefore, it cannot be conclusively stated which one should be used for progressive collapse analysis and design. The one undeniable fact is that there is still work and research to be done in refining and validating all of the software.

2.6. Design for Progressive Collapse Mitigation

In the design of structures for abnormal loading, there are two distinct processes. The first process is blast mitigation or blast-resistant design, in which the effect of an explosion is

considered on individual structural members (Marchand and Alfawakhiri, 2005). The intent of blast mitigation is to eliminate debris and attempt to contain the damage to a localized area for human and asset safety. Blast mitigation investigates the behavior of non-structural components, such as glazing or cladding, as well (Princehorn and Laefer, 2005).

On a more comprehensive scale, design to resist progressive collapse attempts to provide sufficient redundancy to the structural system to prevent local damage from propagating. Design for progressive collapse mitigation either considers an abnormal loading on a member or assumes the structure has already sustained local damage. The design goal is then to identify alternate paths that can redistribute the load or different mechanisms that can be used to "bridge" the structure (Marchand and Alfawakhiri, 2005).

2.6.1. Design Methods

The codes and standards within the United States address progressive collapse in various ways, but generally rooted within them is either the direct design approach or the indirect design approach (Mohamed, 2006).

The direct design approach explicitly considers abnormal loading on a structure and considers its resistance to progressive collapse. Two methods arise from the direct design approach, the alternate path method and the specific local resistance method.

The alternate path method considers the immaculate removal of a load-carrying member. The structure is then analyzed to ensure that deflection or stress limits are not reached and that progressive collapse does not occur (Marchand and Alfawakhiri, 2005). Different codes specify acceptability limits for a structures reaction due to the removal of a member. In using the alternate path method, an analysis must be done on each unique

member. At a minimum this usually consist of a mid-side column on each face, a corner column, and an interior column (Marchand and Alfawakhiri, 2005).

As opposed to immediately considering a member failure, the specific local resistance method requires that critical members be designed to resist a specific load or threat (Mohamed, 2006). A loading is either specified or a structure specific risk assessment analysis can be done to determine possible threats. Then individual members are designed to provide sufficient strength to resist the loading (Faschan et al., 2003).

Within the indirect design approach, the removal of members or specific loads is not considered. Progressive collapse resistance is addressed through providing minimum levels of strength, continuity and ductility (Marchand and Alfawakhiri, 2005). Instead of considering individual threats, the goal of the indirect approach is to create a redundant structure that will perform under any conditions. Within some codes that prescribe the indirect design approach, such as the Department of Defense's *Unified Facilities Criteria 4-023-03: Design of Buildings to Resist Progressive Collapse*, particular detail is given to the way a structure is "tied" together (Department of Defense, 2005).

The tying approach is a tool that the indirect design method uses that presumes that the reserve axial capacity in members and connections is able to span over lost members (Marchand and Alfawakhiri, 2005). In spanning over "doubled" spans, this approach assumes that catenary forces will develop that are adequate to support the reactions from the lost member. To ensure that the structure will be able to support these forces, horizontal, vertical, and peripheral tying is required.

The basic design approaches are summarized in the following figure.

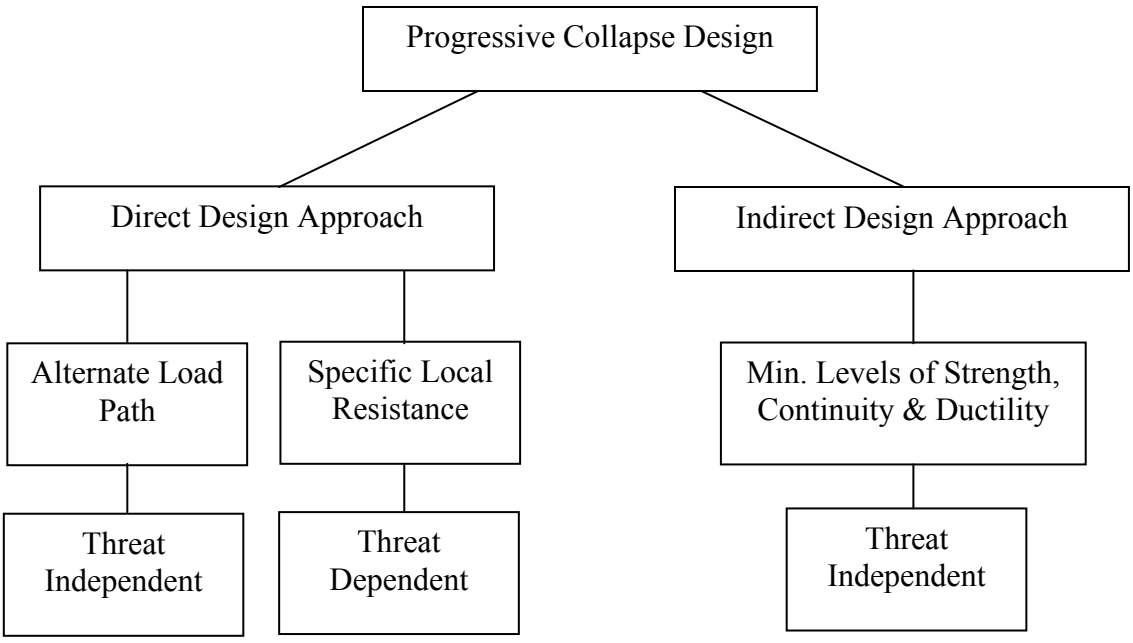


Figure 2. Progressive Collapse Design Approach Diagram

2.6.2. Design Codes and Standards

Different design codes and standards address progressive collapse to various degrees. With progressive collapse being a relatively new phenomenon from the design aspect, the extent to which it is discussed within the codes and standards is constantly evolving.

Most interesting to note is that the *2006 International Building Code (IBC)*, produced by the International Code Council does not mention progressive collapse at all (International Code Council, 2006). However, *ASCE 7-05*, which is the standard referenced throughout *IBC* on other topics, does discuss progressive collapse and the three design methods mentioned in the previous section (ASCE, 2005). The other major standards published by the General Services Administration and the Department of Defense also incorporate guidelines and criteria for progressive collapse design.

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

In Section C1.4 of *ASCE 7-05*, "General Structural Integrity", progressive collapse is defined and the Ronan Point and Murrah Federal Building collapses are presented as examples (2005). A list of factors contributing to the current risk of damage propagation in structures is presented. Some of the factors include non-load-bearing interior walls, lack of continuity as a result of quick erection processes, roof trusses with insufficient strength to maintain lateral stability upon the removal of an adjacent member, and the elimination of excessive safety factors in design codes over the past decades (ASCE, 2005).

Within Section C1.4, the alternate path, specific local resistance, and indirect design methods are described. In Section C2.5, "Load Combinations for Extraordinary Events", further detail is given on design using the direct design approach, either the alternate path method or specific local resistance method (ASCE, 2005). It is suggested that events with a probability of occurrence greater than 10^{-6} or 10^{-4} be identified and designed for, to ensure that the main load-bearing systems remain intact.

If the alternate path design method is used, the following load combination is given to evaluate the capacity of the remaining structure after the initial component is removed (ASCE, 2005):

$$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W \qquad \textbf{Equation 2-1}$$

In analyzing a structure using the specific local resistance method, the capacity of the structure or an individual component to withstand an extraordinary event should be checked using these equations:

$$1.2D + A_k + (0.5L \text{ or } 0.2S) \qquad \textbf{Equation 2-2}$$

$$(0.9 \text{ or } 1.2)D + A_k + 0.2W \qquad \textbf{Equation 2-3}$$

where A_k represents the load from the extraordinary event(ASCE, 2005).

General Services Administration, *2005 Facilities Standards (P100)*

ASCE 7-05 governs the construction and maintenance of civilian structures as referenced by *IBC* (ICC, 2006). Structures owned or occupied by the federal government must comply with the design standards presented by the General Services Administration (GSA) (General Services Administration, 2005). Therefore, the *2005 Facilities Standards* accounts for 3.4 billion square feet of space in 491,465 buildings (Smith, 2005). Due to the fact that federal buildings operate at a higher threat level when compared to civilian structures designed by *ASCE 7-05*, the standards set forth by the GSA are much more elaborate.

Within the standards, it is distinguished that, "design and analysis requirements for progressive collapse is not part of a blast analysis" (GSA, 2005). Blast loads are considered in the structural design; however they are independent from the progressive collapse analysis. For the progressive collapse analysis, the standard suggests using the alternate path method (Mohamed, 2006). For details on performing the progressive collapse analysis, the standard references another document published by the GSA, *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* (GSA, 2003).

The guidelines published in *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* specify performing a project-specific risk assessment of the structure. From this risk assessment, the potential

abnormal loads are determined and used within the progressive collapse and blast analyses (Dusenberry, 2002).

In utilizing the alternate path method for progressive collapse analysis it is suggested that nonlinear dynamic analysis be used if possible. However, if that is not appropriate due to time or economic constraints, a linear-elastic static model can be created using the following load case:

$$2.0(D + 0.25L) \qquad \textbf{Equation 2-4}$$

where the 2.0 accounts for the dynamic effects (Mohamed, 2006). The stresses resulting from the loading scenario are not to exceed unfactored ultimate capacities.

For a successful design, "mitigating progressive collapse would preclude the additional loss of 'primary structural members (or elements)' beyond this localized damage zone" (GSA, 2005). The *P100* standard classifies building components based on their role in the structural system. There are four categories, listed in their order of significance to structural integrity (GSA, 2005):

1. Primary Structural Elements –essential to building performance, including columns, girders, roof beams, and main lateral force resistance system.
2. Secondary Structural Elements – all other load bearing members, such as floor beams and slabs.
3. Primary Non-Structural Elements – elements that are essential for life safety, including ceilings and heavy suspended mechanical units.
4. Secondary Non-Structural Elements – elements not covered in primary non-structural elements, such as partitions and furniture.

These categories clarify the definition of a successful design by specifying which "primary structural members" are of concern in mitigating the progression of collapse outside of the initial damage zone.

After addressing progressive collapse, the *Facilities Standards (P100)* proceeds to discuss blast design and analysis. The information collected in the project-specific risk assessment report is used to quantify the abnormal loads. The specific local resistance method is used to design the individual components within the structure, such as exterior walls, to resist the abnormal loads.

Section 8.7, "Structural Engineering", provides "Good Engineering Practice Guidelines" for mitigating blast effects on structures (GSA, 2005). Some of the "rules of thumb" are listed below:

1. To design and analyze structures for blast loads, which are highly nonlinear both spatially and temporally, it is essential that proper dynamic analysis methods be used. Static analysis methods are generally unachievable or uneconomical.
2. The designer should recognize that structural components could be loaded in a direction they were not designed for due to engulfment by blast, the negative pressure phase, the upward loading of members, and the dynamic rebound.
3. Lap slices should develop the reinforcement's full capacity and be staggered.
4. Special shear reinforcement, including ties and stirrups, is generally required to allow large post-elastic behavior.

5. Maximum column spacing of approximately 30 feet.
6. Maximum floor-to-floor height of approximately 16 feet.
7. Designer should use fully grouted and reinforced CMU construction when CMU is used.

Although these are blast mitigation guidelines, many of them also aid in enhancing a structures capacity to prevent progressive collapse.

Department of Defense, 2005 United Facilities Criteria (UFC) 4-023-03

The Department of Defense's (DoD) guidelines published in *UFC 4-023-03: Design of Buildings to Resist Progressive Collapse*, apply to all military departments, DoD agencies, and DoD field activities (DoD, 2005). The design requirements presented in the *UFC* ensure that two structural response modes are developed within a structure. The first level of progressive collapse design employs "tie forces", which develop from catenary action in the structure. The second level utilizes the alternate path method, in which the structural mode is "flexural", bridging across a removed element (DoD, 2005).

New and existing construction is assigned a Level of Protection (LOP) corresponding to its usage (DoD, 2005). This LOP determines the extent to which progressive collapse is designed for. For Low LOP structures, only the indirect design approach is employed, by specifying minimum levels of tie forces. In the case that the required tie force cannot be developed in a particular member, then the alternate path method is applied to ensure that the structure can bridge over the inadequate member. For Medium and High LOP's, the alternate path method is always applied to verify that the structure contains sufficient flexural resistance in addition to the catenary resistance provided by tying (DoD, 2005).

The figure below shows the location and direction of ties that are necessary to "hold" a structure together when subject to localized damage:

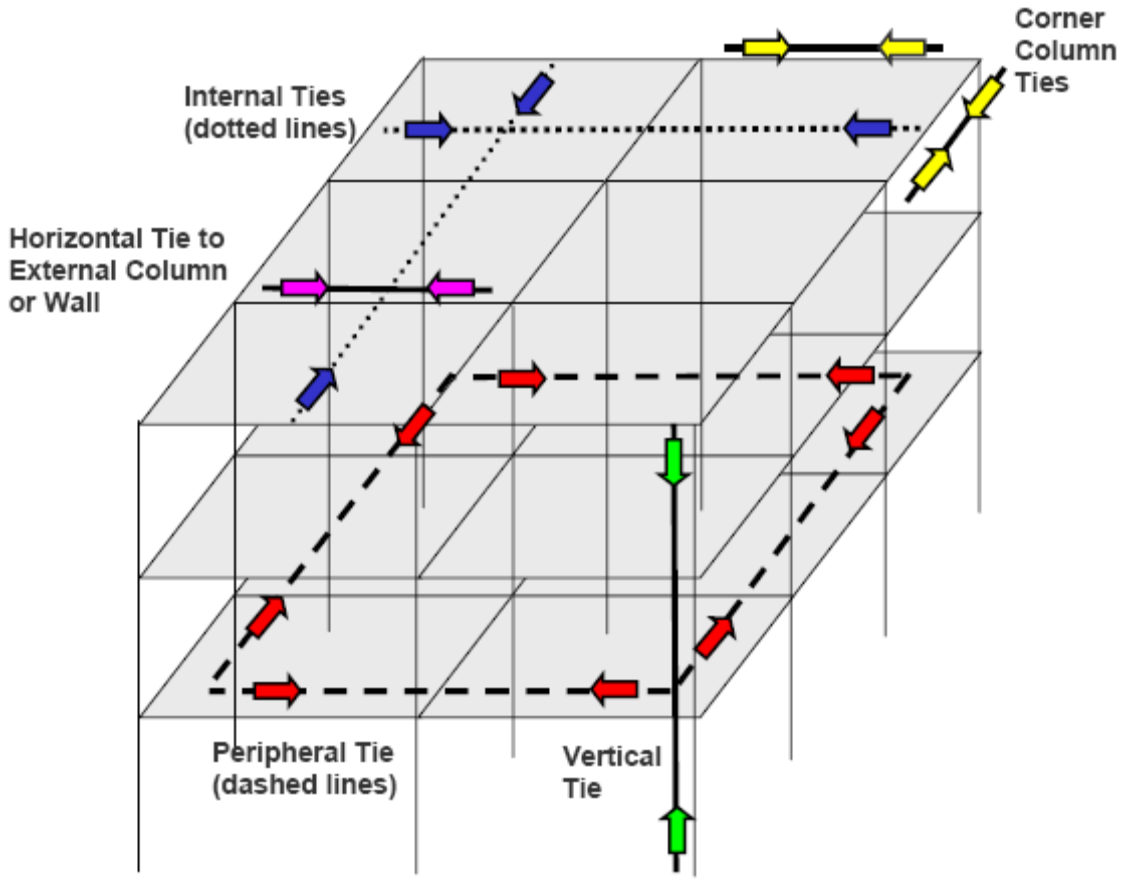


Figure 3. Diagram of Tie Forces in a Frame Structure (DoD, 2005)

For the alternate path method design, the following load combination is given for nonlinear dynamic analysis (DoD, 2005):

$$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W \qquad \textbf{Equation 2-5}$$

This is the same equation presented in *ASCE 7-05* (ASCE, 2005). If an element fails during analysis, the analysis is stopped and the element is removed. The load from the removed element is doubled and applied to the section directly below it. Doubling of the

load simulates the impact forces that would be created by collapse of the failed element (DoD, 2005).

If a nonlinear or linear static analysis is performed, the load combination used to account for the dynamic effects should be (DoD, 2005):

$$2.0\ [(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W \qquad \textbf{Equation 2-6}$$

The design guide also requires that columns and walls be designed for unsupported lengths equal to two-story heights. This accounts for the possibility of lost lateral support during progressive collapse. For instance, if negative uplift pressures cause floor slabs to fail, exterior columns could be left unbraced over multiple story heights. This was a major contributor in the Murrah Federal Building collapse (FEMA, 1996).

2.6.3. Commentary on Design Methods

Considering the three design methods presented in some or all of the standards, the alternate path method and the indirect design approach are both threat independent. Neither one considers the actual loading event. Ignoring the type or level of loading is reasonable for the indirect design approach because it specifies minimum levels for components regardless. However, for the alternate path method it is questionable. The alternate path method designates the removal of one column without any effect on the surrounding members. This process has sparked controversy due to the fact that if a column was removed by a shockwave, adjacent members would certainly be impacted, decreasing their load carrying capacity (Krauthammer, 2003). In addition, the alternate path method considers all members equal. This is unrealistic if there is a large, built-up section surrounded by smaller rolled

shapes. Any loading capable of removing the built-up section would undoubtedly cause failure in the smaller columns as well.

Even with these controversial characteristics, the alternate path method is still the preferred method. Designers are inclined to use the alternate path method due to its relative simplicity. Many codes allow the analysis to be done statically, specifying coefficients to account for dynamics and impact forces (Mohamed, 2006). It is not nearly as complex when compared to the specific local resistance method, which must be performed using dynamic, nonlinear analysis in order to capture the true inertial effects and peak forces and moments (Hansen et al., 2005).

2.6.4. Basic Design Characteristics

Through research and investigational studies, some codes are designating general characteristics that should be incorporated in a structure where possible. In ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures*, this list of specific suggestions is provided for enhancing structural integrity (ASCE, 2005):

1. Good plan layout, paying attention to spans and interior walls.
2. Provide an integrated system of ties among the principal elements of the structural system.
3. Return on walls.
4. Changing directions of floor slab so that in case of collapse, slab can span other direction.
5. Load-bearing interior partitions.
6. Catenary action of floor slab.

7. Beam action of walls, allowing them to act as the web of a beam with the slabs above and below acting as flanges.
8. Redundant structural systems, secondary load paths.
9. Ductile detailing.
10. Provide additional reinforcement to resist blast and load reversal when blast loads are considered in design.
11. Consider the use of compartmentalized construction in combination with special moment resisting frames for new buildings.

In general, it has been determined that a structures progressive collapse resistance can be elevated by including redundancy, interconnectivity, and ductility.

2.6.5. Behavior of Structural Systems Subject to Progressive Collapse

If a structure containing the above-mentioned characteristics is subject to abnormal loading and members are removed, it will have components capable of redistributing the load. The load redistribution will usually occur through three individual mechanisms (Khandelwal and El-Tawil, 2005):

1. Catenary action of slab and beams allowing gravity system to span adjacent elements.
2. Vierendeel action from the moment frame above the damaged structure.
3. Gravity load support provided by nonstructural elements such as partitions and infills.

Generally, these mechanisms are sufficient to tie a structure together and bridge over the damaged areas long enough to prevent extensive global damage and allow for the safe evacuation of the occupants.

2.7. Need For Research

Given the vast information illustrating the complexities involved when analyzing structural systems subject to abnormal loads and evaluating their potential for progressive collapse or disproportionate progressive collapse, it is evident why further research is necessary. Research is greatly needed to further understand the global behavior that occurs in different structural systems so that the comprehensive analysis and design programs can be refined and validated. Further understanding of the behavior and verification of the software programs will provide reliability for engineers, thus allowing design codes to be more prescriptive and thorough with their guidelines. Ultimately, through the current experimental and analytical research in the field of progressive collapse, the ambiguity of designing for prevention of disproportionate progressive collapse should be lifted, allowing for collapse mitigation to be designed through a somewhat simplified and accepted methodology.

3. Extreme Loading® for Structures Analytical Background

3.1. Applied Element Method

The ELS® software is founded on the Applied Element Method (AEM) of numerical analysis. The AEM was established at the University of Tokyo by Hatem Tagel-Din during his doctoral studies during the late 1990's (Tagel-Din, 1998). Motivation for the methodology arose from catastrophic earthquakes in the region (most recent, Kobe 1995) which brought about the realization that engineered structures could collapse. Acknowledging that engineered structures could collapse created an interest in analytical modeling for such events. However, of the numerical analysis methods available at the time, none were adequate to model the full-range of loading that is seen during an earthquake or potential collapse event. Thus, the AEM was created, designed to overcome the limitations of current methods and allow for the complete collapse analysis of structures.

3.2. What Separates the Applied Element Method

Numerical methods for structural analysis, other than the AEM, can be classified into two categories. In the first category, models are based on continuum material equations. The Finite Element Method (FEM) is a typical example in this category. Continuum models ignore the heterogeneous characteristics of objects and continually sub-divide the object into small, yet finite elements with properties being close to that of the bulk material. For small displacement analysis, continuum models are extremely accurate. Complications arise when the behavior becomes highly nonlinear due to either the material or geometry. For example,

it is very difficult for continuum models to simulate the behavior of a structure as it evolves from a continuum state to a perfectly discrete state, as it does during collapse.

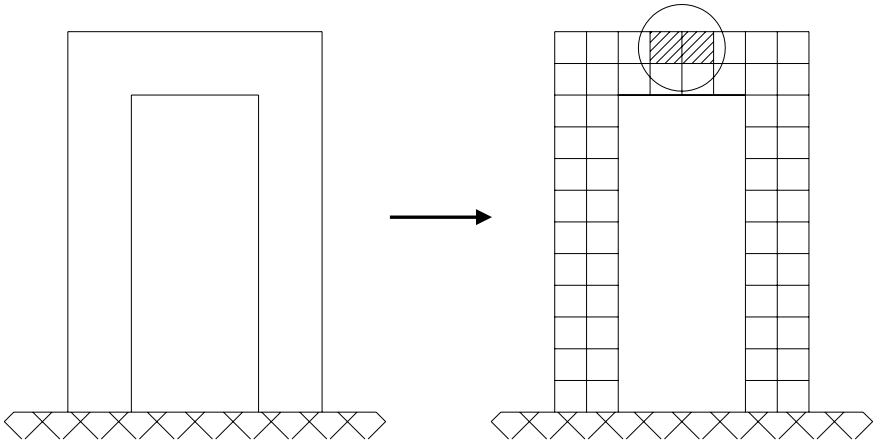
The second category of models is based on discrete element techniques. This includes methods like the Rigid Body and Spring Model and the Extended Distinct Element Method (Meguro, 2001). Discrete models simulate large groups of individual elements. The main advantage of discrete models is that they use relatively simple techniques to simulate the cracking process (Kikuchi, Kawai, and Suzuki, 1992). However, the crack propagation is highly dependent on the element shape, size and arrangement (Ueda and Kambayashi, 1993). In addition, the accuracy of discrete element methods in small deformation analysis is less than that of continuum models, or FEM models.

Bridging the gap between continuum models and discrete element models is the purpose of the AEM. The AEM provides techniques that allow a structure to be analyzed within one model from the unloaded stage, through small displacement loading, through large displacement loading, and throughout collapse. Therefore, with the AEM, crack initiation, crack propagation, separation of structural elements, rigid body motion, and the collapse process, including impact, can be followed.

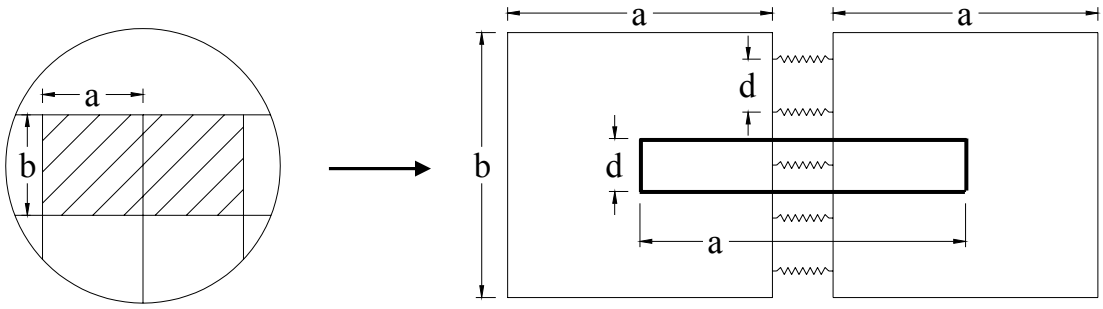
3.3. Applied Element Method Fundamentals and Advantages

With the AEM, a structure is modeled by virtually dividing it into an assembly of small elements. Adjacent elements are connected through series of normal and shear springs located at contact points that are distributed over the surface of each element. At each contact point, there is one normal spring and two shear springs in orthogonal directions. The figure below illustrates the division of a structure into elements and shows an example of the

conjoining springs. The area of influence on each element for a set of springs is highlighted in bold.



a.) Division of structure into small elements



b.) Spring distribution and area of influence for each pair of springs

Figure 4. Model Generation with AEM (Meguro and Tagel-Din, 2000)

Given the illustration, the spring stiffness equations are:

$$K_{normal} = \frac{Edt}{a} \quad \text{and} \quad K_{shear} = \frac{Gdt}{a} \qquad \textbf{Equation 3-1}$$

where E and G are Young's Modulus and shear modulus, respectively, d is the distance between springs, t is the thickness (into the page), and a is the length of the representative area. This is simply the equation for axial stiffness.

Each element has three degrees of freedom located at its centroid representing the rigid body motion of the element. Each set of springs on an elements surface can be geometrically related to the three degrees of freedom at the centroid, thus creating a stiffness matrix for that set of springs. The element stiffness matrix is then created by summing up the stiffness matrices of each individual set of springs. The model can then be analyzed by utilizing the following equation:

$$[F] = [K_G][\Delta] \qquad \textbf{Equation 3-2}$$

where [F] is the applied load vector, [Δ] is the displacement vector, and [K_G] is the global stiffness matrix.

For large displacement analysis, modifications must be made to the force side of the governing equation to account for incompatibility between spring strains and stresses and the geometrical changes in the structure. Formulation of this equation is presented by Meguro and Tagel-Din in "Applied Element Method Used for Large Displacement Structural Analysis" (2002).

Utilizing these equations and techniques to model structures is what separates the AEM from other methodologies. The AEM has been proven to be as accurate as the FEM in modeling linear structures in the small deformation range, but through implementation of its principles, the AEM is able to surpass the capabilities of the FEM in large displacement and collapse events (Meguro and Tagel-Din, 2000). The AEM offers four distinct advantages in structural modeling: automatic crack generation, element separation, element re-contact, and element impact.

Each of the advantages within the AEM arises from the use of springs to connect adjacent elements. For each set of normal and shear springs, stress and the corresponding

strain is calculated throughout the loading. Referencing default or user-defined material properties, once the criteria is reached, springs are cut. This can occur anywhere within the model, therefore no pre-conceived notion of where the cracks will initiate is necessary. Crack propagation follows the same principles.

If all of the springs connecting an element are severed, the element is allowed to separate from the structure. In a dynamic analysis, the element has an assigned mass and generates inertial forces. As the element falls, if it contacts another element, the contact is automatically detected and contact springs are generated between the colliding elements. These contact springs model the inertial forces transferred between the elements, as well as forces due to bearing and friction.

The ability to cover this vast range of structural behavior in a single model is what distinguishes the AEM from other analysis methods. The following sections go into further detail of the AEM and discuss some of the modeling techniques used in the ELS® software.

3.4. Material Models and Parameters

Several materials, including reinforced concrete, steel, masonry, mortar, glass, and aluminum can be considered in ELS®. Reinforced concrete and steel will be the only material models discussed herein because they were the only ones utilized within the research. Details of the other material models can be found in the ELS® Technical Manual (Applied Science International, 2006).

Before the material model for reinforced concrete is presented, it is necessary to step back and discuss how reinforcing bars are represented in elements. In a reinforced concrete structure, the exact reinforcement scheme can be input into the elements. For three-

dimensional problems this can be extremely difficult to generate, thus ELS® allows for AutoCAD files containing the reinforcement scheme to be imported into the drawing files. Once the AutoCAD file is imported into ELS®, the lines can be selected and converted to the appropriate size reinforcing bar. Placing unique reinforcement bars on different drawing layers in AutoCAD will allow for them to be toggled on and off within ELS®, further expediting the process of creating the reinforcement.

Once the reinforcement bars are created in ELS®, reinforcement springs will be generated on the elements. Reinforcement springs are located on the surface of the element at the exact location that the reinforcement bar crosses into the next element. At this location, a normal spring and two shear springs are created, representing the axial and shear behavior of the bar. The normal spring always takes the direction of the reinforcement bar irrespective of the element face direction. These springs are created in addition to the other springs on the elements surface that were previously described, which are known as matrix springs. The matrix springs represent the element material, in this case the concrete, while the reinforcement springs represent the reinforcement bar material.

In reinforced concrete sections, because the concrete and reinforcing steel is each represented by a unique spring, a matrix spring and a reinforcing spring, the material models can be broken down individually, into a concrete model and a steel model. For the modeling of concrete in compression, the Maekawa compression model was adopted (Okamura and Maekawa, 1991). This model utilizes the initial Young's modulus, a fracture parameter, and the compressive plastic strain to define the envelope for compressive stress and strain. This allows for unloading and reloading to conveniently be described as seen in the following figure.

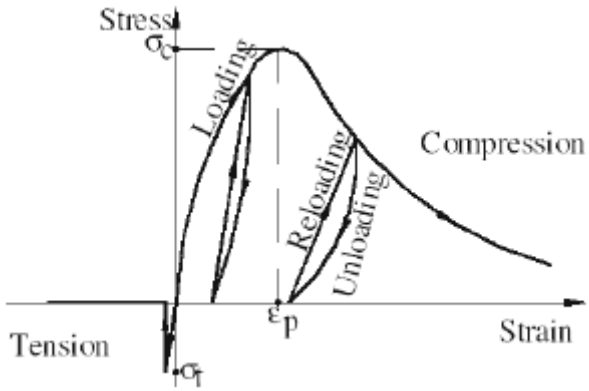


Figure 5. Model for Concrete Under Axial Stress (ASI, 2006)

After peak stresses, spring stiffness is assigned a minimum value to avoid negative stiffness. This results in a difference between calculated stress and stress corresponding to the spring strain. These residual stresses are redistributed by applying an equivalent force in the reverse direction during the next loading step.

For concrete springs in tension, spring stiffness follows the initial stiffness until the cracking point is reached. After cracking, the stiffness of springs subject to tension is set to zero. The residual stresses are then redistributed in the same manner as for compression.

The relationship between shear stress and strain for concrete springs is assumed linear until cracking. Following cracking, the shear stress drops to a level dependent on a parameter known as the friction coefficient. A model for this behavior can be seen below.

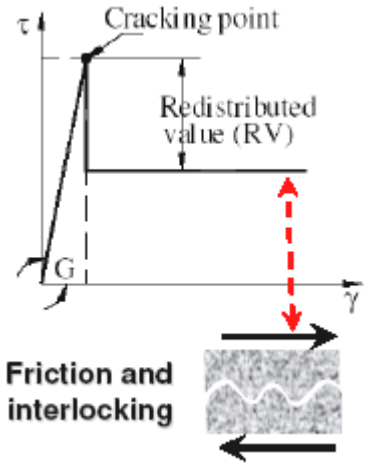


Figure 6. Model for Concrete Under Shear Stress (ASI, 2006)

To model the reinforcement bars, the reinforcing springs follow the model presented by Ristic, Yamada, and Iemura, which is illustrated below (1986). The tangent stiffness of the reinforcement spring is calculated based on the strain of the reinforcement spring, the loading status (either loading or unloading), and the history of the spring which controls the Bauschinger's effect. The advantage to this model is that it can easily consider the effects of partial unloading and the Bauschinger's effect without substantially complicating the analysis.

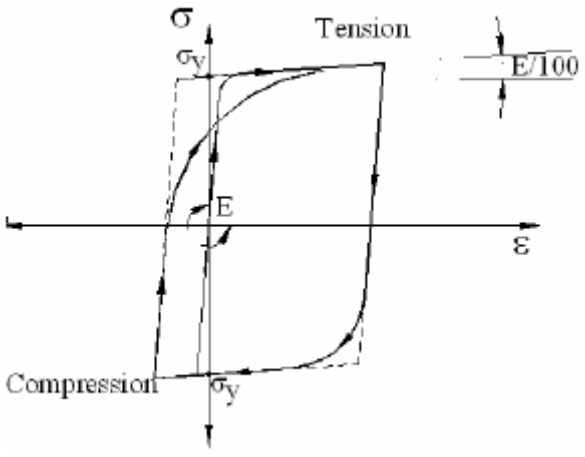


Figure 7. Model for Reinforcement Under Axial Stress (ASI, 2006)

For reinforcement springs, their failure criterion is unique because they are contained on an element in addition to the ordinary matrix springs or the concrete springs. There are two possible cases in which the reinforcement springs can be cut; 1.) the reinforcement bar reaches its tensile failure criteria, or 2.) the matrix (concrete) springs reach their separation strain limit, in which all springs on the element face are removed. The separation strain limit is a user-specified parameter in ELS®.

For structural steel elements, the behavior follows the same model as for reinforcement bars, except for the failure criterion. The failure criterion for structural steel elements is based on the principal tensile strain and the user-specified strain limits.

Beyond the stress-strain models, ELS® has three parameters for each material that define its behavior during separation and contact. The normal contact stiffness factor and the shear contact stiffness factor define the load-displacement relationship of the normal and shear springs during contact. The contact spring unloading stiffness factor is a number indicating the amount of energy dissipation during contact. The unloading stiffness is defined as the original stiffness, *K*, times the contact spring unloading stiffness factor, *n*. This load-displacement relationship for contacting elements can be seen below.

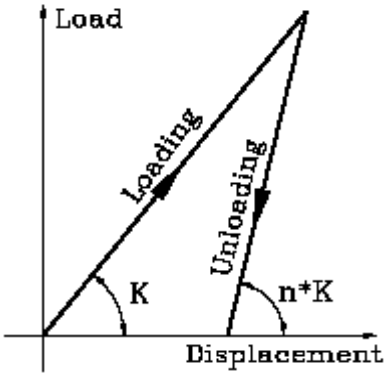


Figure 8. Load-Displacement Relation of a Contact Spring (ASI, 2006)

3.5. Mesh Generation

Each structural member, regardless of material, is divided into smaller elements creating a mesh. Mesh is used to describe the discretization of a structural member. A member can have a finer mesh or a coarser mesh depending on how many elements are used to represent it.

In ELS®, after inputting a member into a model, the mesh can be refined very easily, using the three principal directions to define it. The user specifies the number of elements in each direction and ELS® divides the distance evenly, placing the specified number of elements in that direction. The principal direction defines the number of elements along the length, while the other two directions define the mesh across the cross section for the member. The total number of elements used to represent a member is the product of the quantities in each direction. For example, a column discretized with a 10 x 3 x 3 mesh contains 90 elements. A cross section of the column would show 9 elements, a 3 x 3 mesh.

A significant advantage to ELS® over other FEM programs is in the way adjacent members mesh together. In most other programs, special considerations must be made when changing mesh size or modeling different materials adjacent to each other. Within ELS®, no transition elements are required and the interface between different materials can be considered through user-specified parameters. This greatly facilitates the modeling of large, complex structures.

3.6. Intersect Command

When constructing a model in ELS®, because of the way members are input, two or more members often occupy the same space. This can be due to various things like end

extensions or inputting members based off centerline dimensions. However, before the structure is analyzed, only one element can occupy a certain space. Therefore, the intersect command is used to specify which member should be given priority. The other members are truncated where they intersect the member that was given priority. If the truncated portions of the selected members contained reinforcement, the reinforcement will remain as part of the member with priority. A primary example of this is in a reinforced concrete structure where a column, girder, and slab all intersect at a joint. Priority can be specified for any of the members causing the other two to be truncated, but leaving the reinforcement continuous into the joint.

3.7. Loading Scenarios

Static and dynamic loading scenarios can be considered in ELS®. Static loadings can be lumped mass, load controlled, or displacement controlled. For dynamic loadings, ELS® can consider load and displacement controlled loading, earthquake loads, inertia loads, pressure loads, blast loads, moving loads, and element removal loads.

For this research, each analysis contained at least two loading stages, one static stage in which the self-weight of the structure was applied, and one dynamic stage in which the element removal or progressive collapse occurs. Additional static or dynamic stages were utilized when necessary to realistically represent the construction. For a static loading stage the only parameter that has to be defined is the number of increments over which the self-weight is applied. For a dynamic loading stage the duration, time step, and step divisions must be specified. The duration is the total length of the dynamic analysis. The time step is the increments over which the duration will be analyzed. Selection of the time step is crucial

to capturing all of the structural behavior and impact forces during a progressive collapse event. The time step is further divided by the number of step divisions specified by the user. Detail regarding how small the time step should be can be found in the ELS® Technical Manual (Applied Science International, 2006).

3.8. Element Removal

The element removal option available in ELS® can be utilized by the user to extract elements from the analysis. This is generally utilized in scenarios where the user is knowledgeable of which elements are going to fail or be removed, such as building implosions or progressive collapse design. The element removal option allows the user to create groups of elements that are immaculately removed from the analysis at specified times during the dynamic analysis. Immaculate removal of the elements means that everything contained within the element is removed from the model. Thus, for reinforced concrete elements, the reinforcement is extracted along with the concrete. The advantage to this method of modeling is the reduction in analysis time compared to a blast loading scenario. Specifying to the model which elements to extract shortens the computation time required for the program because it is not required to calculate the blast pressure, calculate the load, determine the stress and strain on the element, and then remove the element from the model. It must be emphasized again though, the element removal option is only useful when the failure locations are known.

3.9. Available Output

In analyzing a dynamic model with ELS®, an extensive amount of output data is generated. ELS® can calculate and illustrate element velocity, element acceleration, element

contacts, and member cracking. In addition, ELS® can display diagrams showing element forces and internal forces. Stresses and strains are calculated both with and without averaging in the X-Y-Z direction and in the principal direction. The user can turn off some of the output if it is not desired to minimize the output file size.

4. Experimental Investigation

The experimental investigation for each of the cases contains a description of the structure highlighting and illustrating the fundamental characteristics. Details are also given regarding what types of information were obtained and how they were acquired.

4.1. Case Study I – Crabtree Sheraton Hotel

4.1.1. Background

The Crabtree Sheraton Hotel was constructed in Raleigh, NC during the 1970's as the anchor hotel for Crabtree Valley Mall that was built a few years earlier. The hotel was located in the western corner of the Creedmoor Road and Glenwood Avenue intersection. After changing hotel affiliations several times, the property finally went into foreclosure in 2003. That is when the Soleil Group, Inc. of Cary, NC purchased the property. In 2005, Soleil unveiled plans for a 42-story, mixed-use building, to be known as Westin Raleigh Hotel & Residences at Soleil Center that would take the place of the Crabtree Sheraton. The demolition of the Crabtree Sheraton Hotel in 2006 made way for construction to begin on the new Westin Raleigh tower, which is set to debut in 2009 (Soleil Group, Inc.).

The Crabtree Sheraton Hotel was imploded on May 7, 2006. The demolition contractor in charge of the project was D.H. Griffin Wrecking Company, Inc. of Raleigh, NC. The explosive subcontractor hired by D.H. Griffin Wrecking Company, Inc. was Demolition Dynamics Company of Franklin, Tennessee. Prior to the implosion on May 7, the demolition company removed all structures on the site other than the 128-foot tall tower through means of conventional demolition.

4.1.2. Structural Description

The 128-foot tower section of the hotel, which is the only portion significant to this project, employed cast-in-place, two-way flat-plate (slab and column) concrete construction. The tower consisted of a ground or lobby level followed by 10 floors of hotel rooms. Above the 10th floor was a penthouse that covered approximately half of the roof area. An elevator and stair tower sat on top of the penthouse giving the structure its elevation of 128 feet. This can be seen in the following illustration, which shows the condition of the building just prior to the implosion, after all of the conventional demolition was performed.

Figure 9. Elevation of Crabtree Sheraton Hotel

The plan of the structure was roughly 58 feet by 117 feet, with three bays in the north-south direction and nine bays in the east-west direction. There were 40 column

locations. Some of the columns on the ground floor were non-rectangular sections. Above the ground floor, all columns, except for one, were rectangular, directional columns. Beams were utilized in a few locations for edge support. The beams were mainly on the east and west exterior walls as well as around the stair and elevator towers. Without beams across the entire structure, the building used the slabs two-way flat-plate behavior to transfer the loads to the columns. The following illustration presents the framing plan for the 1st-10th floor.

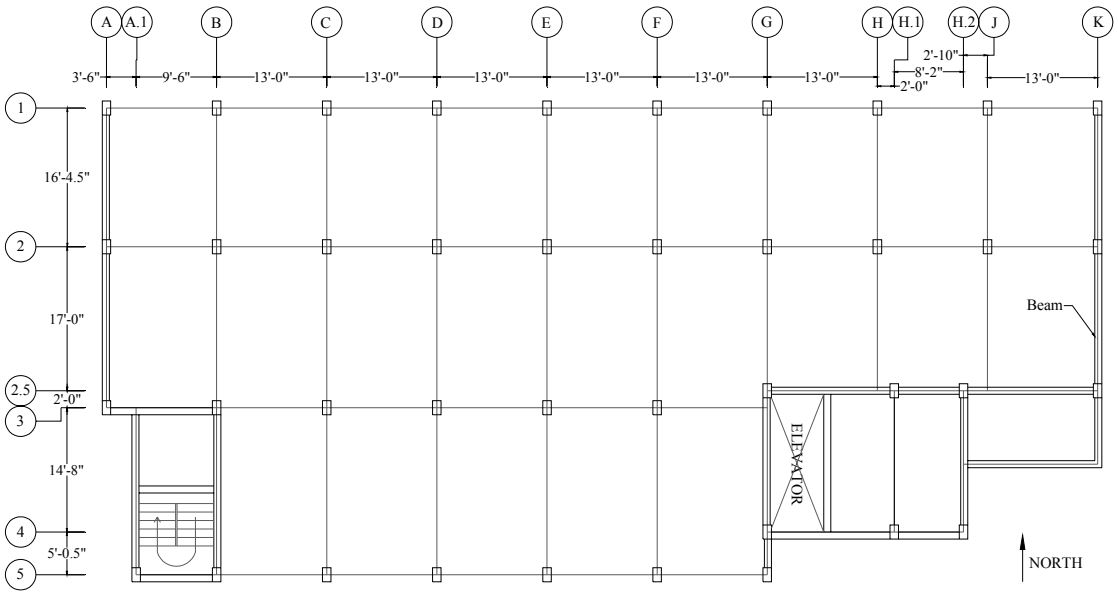


Figure 10. Crabtree Sheraton Hotel 1st – 10th Floor Framing Plan

4.1.3. Structural Documentation

This research was undertaken approximately one year after the Crabtree Sheraton Hotel implosion. Therefore, no on-site documentation was possible. All of the information had to be retroactively collected from the companies involved.

The initial contact was with D.H. Griffin Wrecking Company, Inc. who provided the structural drawings that they had preserved. Within the packet of information that they presented was a typical floor framing plan for floors 1 through 10 and column details for the

entire structure. They also included an elevation drawing of the hotel. The only information left to be desired was the beam reinforcement details and the slab reinforcement dimensions.

In an effort to obtain a full set of construction plans, the Soleil Group, Inc., who is the current owners of the property, was contacted. However, since their original plans were to completely start over with new construction on the site, they discarded the old plans as soon as they received them. Several additional attempts with previous owners, the original architect, and the construction firm, provided the same results; no plans.

Therefore, in order to estimate the details and amount of reinforcing used in the structure, older construction manuals were acquired. The 1971 version of *ACI 318, Building Code Requirements for Structural Concrete* and the Concrete Reinforcing Steel Institute's *Design Handbook*, which is based upon *ACI 318-71*, was referenced because the hotel was constructed in the mid-1970's. The *Design Handbook* provided typical longitudinal reinforcement details and stirrup spacing for beams of different spans. It also illustrated the typical overlap and splice dimensions for the slab reinforcement.

4.1.4. Implosion Documentation

For reference on how the structure was imploded, Demolition Dynamics Company was consulted. They provided information and illustrations describing how the dynamite was loaded, the drilling details, and the explosive time delay pattern. Upon request, Demolition Dynamics Company provided documentation video from the company contracted to record the implosion, Protec Documentation Services, Inc. The documentation video contained several views of the implosion from locations around the perimeter of the site.

In addition, various personal photographs and videos were utilized during the analytical evaluation of the model.

4.2. Case Study II – Stubbs Tower

4.2.1. Background

Construction of Stubbs Tower, located in Savannah, Georgia, was completed in 1970. The building was named in honor of J. Marcus Stubbs who was the postmaster of Savannah for 20 years. Upon his retirement from the postal service in the 1960's, he became involved in Savannah City Council. One of the main causes he championed was bringing about construction of new housing for the elderly. As a result, for the 34 years that this structure was in service housing the elderly, it was known as Stubbs Tower (Wittish).

In 2004, the building flooded causing the occupants to be permanently displaced. Stubbs Tower set vacant until funding was finally obtained for its demolition in 2007. The structure, as seen in the following picture, was imploded on December 15, 2007.

Figure 11. Elevation of Stubbs Tower

The demolition contractor overseeing the project was D.H. Griffin Wrecking Company, Inc. of Atlanta, Georgia. The explosive subcontractor hired by D.H. Griffin Wrecking Company, Inc. was once again Demolition Dynamics Company of Franklin, Tennessee. Prior to the implosion on December 15, the majority of the nonstructural elements, including all asbestos, was removed from the tower.

4.2.2. Structural Description

The footprint of Stubbs Tower was approximately 61 feet by 185 feet with the longitudinal direction oriented north-south as can be seen in the aerial picture presented.

Figure 12. Aerial View of Stubbs Tower (Google Earth)

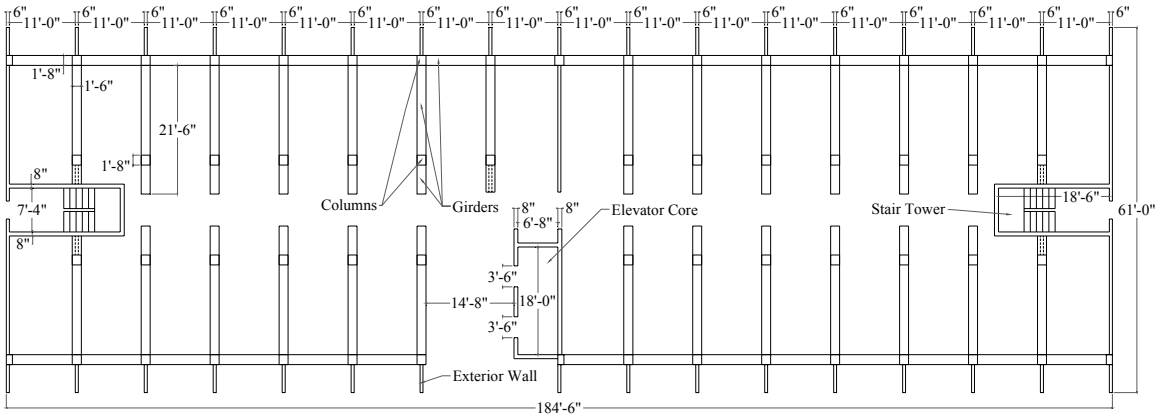
Stubbs Tower stood approximately 148 feet tall and contained 15 floors plus a penthouse. The structure was constructed of reinforced concrete using two types of load-bearing systems. On the first floor, to allow for open floor space, columns and girders were

used. On the subsequent floors, because it was appropriate for apartments, load-bearing walls were used to transfer the vertical loads. A comparison between these two systems can be seen in the following figure.

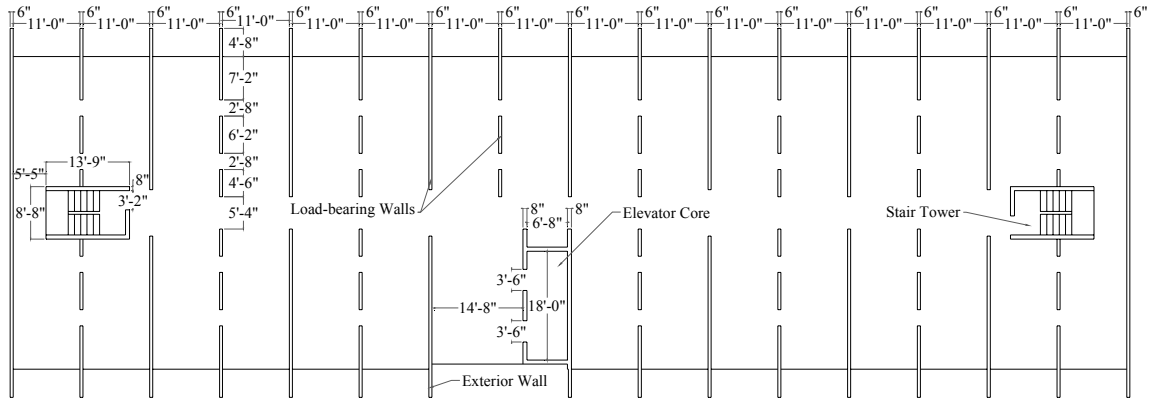
a.) Column-girder frames **b.) Load-bearing walls**

Figure 13. Structural Load-bearing Systems

The only other significant structural elements in the building were the stair towers located on each end and the elevator core located in the center of the building on the eastern side. Both stair towers and the elevator tower were cast-in-place with substantial amounts of reinforcement. The majority of the dimensions with regard to bay spacing and floor layout can be seen in the framing plans presented for the first floor and the typical subsequent floors.



a.) Stubbs Tower 1st Floor Plan



b.) Stubbs Tower 2nd – 15th Floor Plan

Figure 14. Stubbs Tower Floor Plans

The penthouse contained the elevator motors and other mechanical equipment. Steel beams were utilized in the roof of the penthouse to help span the long distance. An illustration showing the full height of the structure taken from the northeast corner can be seen in the following figure.

Figure 15. Northeast Elevation of Stubbs Tower

4.2.3. Structural Documentation

The main contact for the Stubbs Tower project was D.H. Griffin Wrecking Company, Inc. Upon initial contact regarding the project in September 2007, they were able to provide a thorough set of structural drawings. The drawings contained all of the dimensions and details necessary to model the structure.

Modeling Stubbs Tower prior to the implosion allowed for field documentation of the structure. The site was visited three times prior to the implosion to verify details found in the drawings and document the preparatory demolition work done on the structure.

September 10, 2007

The first site visit was used to visually inspect the structure and verify some of the significant dimensions on the structural drawings. During this time the majority of the work

being done was asbestos abatement and the removal of nonstructural elements. The photographs below illustrate the condition of the structure at that time.

a.) Western elevation of Stubbs Tower

b.) Column-girder frames

c.) Upper floor elevator core

Figure 16. Stubbs Tower Existing Conditions on September 10, 2007

December 3, 2007

The second site visit, approximately two weeks before the implosion allowed for all of the preparatory work to be documented. To ensure collapse during the implosion, portions

of the load bearing members were being removed, cables were being installed, and sawcuts were being made. Each of these modifications to the structure is further addressed in the analytical section.

At this time, the demolition contractor was also drilling holes at all of the locations specified by the explosive subcontractor. These holes are packed with dynamite and detonated to displace the concrete from the load-bearing members and create drop. Photographs similar to those presented in the previous figure can be seen below illustrating the level of work completed on the interior of the building through December 3, 2007.

a.) Column-girder frames with holes drilled

b.) Elevator core with material removed

Figure 17. Stubbs Tower Existing Conditions on December 3, 2007

December 11-15, 2007

The final site visit during the week leading up to the implosion allowed for final documentation of the structure. The explosive subcontractor was loading the previously drilled holes with dynamite, sealing the holes, and wiring the explosives together. At this point, the majority of the preparatory work was finished and the demolition contractor began

to wrap portions of the structure with chain link fence and a heavy fabric. The chain link fence and fabric is designed to catch any debris blown outward by the explosives.

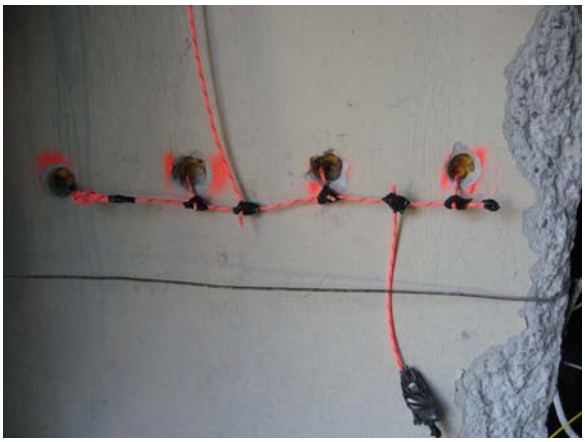
Below are several pictures illustrating the condition of the building and the type of work being done during the week leading up to the implosion. In the following figure, the two pictures depict the work completed on the same structural elements as in the previous figures.

a.) Column-girder frame loaded with dynamite

b.) Elevator core loaded with dynamite

Figure 18. Stubbs Tower Existing Conditions from December 11-14, 2007

The next figure presents two pictures that show further detail of the steps involved in preparing the structure for the implosion. The third picture illustrates how the structure as a whole was covered as a final line of defense to prevent debris from flying out.



a.) Holes loaded with dynamite, not yet sealed

b.) Column wrapped to catch local debris

c.) Eastern elevation showing fabric cover

Figure 19. Further Illustrations of Conditions from December 11-14, 2007

4.2.4. Implosion Documentation

In October 2007, Demolition Dynamics Company was initially contacted in reference to Stubbs Tower. At that time, they provided the preliminary blast plan, which included the drilling locations and explosive time delay pattern. Regular communication with Demolition Dynamics Company was used to learn of any modifications or adjustments made to the preliminary blast plan.

To document the implosion, D.H. Griffin Wrecking Company, Inc. hired Protec Documentation Services, Inc. The third-party documentation company was hired to record the implosion, as well as measure the air overpressure and seismic impact on surrounding structures. The audio and visual recording of the implosion is the only data relevant to this project.

To record the implosion, Protec Documentation Services, Inc. used three Sony HDR-FX1 Handycams. Each camcorder provided high-definition video of the implosion from a different angle at a rate of 30 frames per second. Copies of the audio and visual recordings were acquired through the demolition contractor to use, along with personal documentation, for comparison with the analytical model.

4.2.5. Post-Implosion Experimental Investigation

To aid in evaluation of the initial model, an experimental investigation was performed on a sample of the floor slab attained in the week prior to the implosion. A two foot by two foot cutout from a typical floor slab on the north end of the fifth floor was transported back to North Carolina State University where the individual components were tested.

Cores were taken from the slab specimen to determine the material properties of the concrete. Following ASTM C 42, the average compressive strength and density of the concrete was calculated. Samples of the wire mesh reinforcement were also extracted from the slab and tested under ASTM A 370. From the results, the average yield strength and average ultimate strength were determined.

5. Analytical Modeling and Evaluation

The analytical portion of this research includes developing representative models simulating the implosion of each structure by utilizing the information obtained during the experimental investigation. An evaluation is performed on each model, reviewing the behavior predicted by the model. To facilitate the systematic assessment of each model, a matrix was created.

5.1. Development of Evaluation Matrix

The evaluation matrix was created to roughly quantify the differences between the physical characteristics and behavior of the model and the documentation records. The ultimate goal behind the evaluation matrix is to allow for comparison of different models of the same structure using ELS® or an equivalent software program. Models are evaluated on both input and output through four categories: *Construction*, *Preprocessing*, *Postprocessing*, and *Visual Inspection*.

Models are critiqued based on the initial knowledge or information that is obtained for input and the demolition scenario. This provides a feeling for how well the model represented the construction and existing conditions of the structure. It also accounts for how precisely the implosion plan was simulated. Following the analysis, the behavior of the model is evaluated against the documentation records. *Postprocessing* looks closely at the behavior and failure of individual members, while *Visual Inspection* looks at the gross structure, examining the regional behavior and debris field. Completing the evaluation matrix for a simulation can provide a relative sense of accuracy and aid in identifying the contradictory aspects of the model.

The evaluation matrix varies slightly for each structure because of unique structural components. However, varying the components that are investigated does not affect the methodology and process used to evaluate each model. A completed evaluation matrix for each model that is analyzed in the following sections can be found in the appendices.

5.2. Case Study I – Crabtree Sheraton Hotel

5.2.1. Model Construction

Before the Crabtree Sheraton Hotel model was constructed in ELS®, an AutoCAD file containing a plan view of the base dimensions was imported. This grid representing the column rows allowed for precise placement of the structural members and could be toggled on and off as necessary.

Material Properties

The Crabtree Sheraton Hotel was purely a reinforced concrete structure. There were no other structural, load-bearing materials. The only other material in the structure was concrete masonry blocks used for infill walls on portions of the exterior.

The following table presents the material properties utilized in the model.

Table 2. Crabtree Sheraton Hotel Material Properties

Concrete	Compressive Strength	4,500 psi
	Tensile Strength	10% f_c' or 450 psi
	Young's Modulus, E	3,800,000 psi
	Shear Modulus, G	1,400,000 psi
Longitudinal Reinforcing Steel	Yield Strength	60,000 psi
	Ultimate Strength	84,000 psi
	Young's Modulus, E	29,000,000 psi
Transverse Reinforcing Steel	Yield Strength	40,000 psi
	Ultimate Strength	56,000 psi
	Young's Modulus, E	29,000,000 psi

Structural Components

In the Crabtree Sheraton Hotel there were 10 different column styles; four of them non-rectangular. Only one non-rectangular column occurred above the ground floor, but because of its dimensions and for modeling simplicity, it was assumed rectangular.

To create a non-rectangular column within ELS®, two columns sections are created. One column serves as the main part of the column and the other serves as the addition, which creates the non-uniform geometry. Standard reinforcement can be used in the main part of the column, but custom reinforcement must be used to define the addition because the reinforcement is not symmetrical within that section. The two individual columns are then drawn into the model and placed on top of each other to achieve the appropriate dimensions. The intersect command is used to mesh the two individual columns together. This entire process is shown in the following figure.

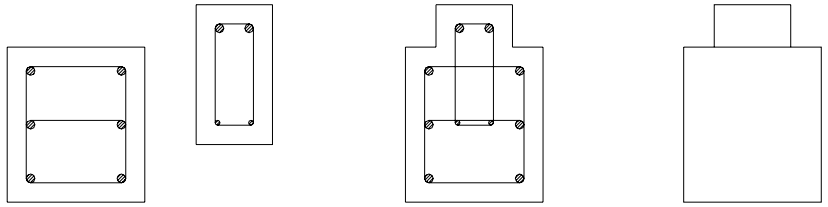


Figure 20. Generation of Non-Uniform Column

Upon insertion of the columns, the beams or girders were placed in the model. Because a complete set of original construction drawings were never obtained, both beam and slab depths had to be estimated from photographs and reference documents. All beams with slabs on top were estimated at 24 inches deep, including the slab thickness that was assumed seven inches. The beam in the stair tower without a slab on top was assumed to be 16 inches deep. Beams with a slab on top were not specified to contain any top steel. Reinforcement within the slab was used as the top, compression steel for these beams.

After inserting the beams into the model, slabs were overlaid on each floor. Slab reinforcement was detailed according to the structural drawings provided. The same process used initially to import the plan grid was used to import the slab reinforcement. In AutoCAD, the slab reinforcement was drawn and each unique group was placed on a different drawing layer. For instance, there was a layer for #4 bars on the bottom of the slab running in the north-south direction. Toggling each unique layer on independently, the lines representing the rebar could be selected and converted to the appropriate size reinforcement. By specifying the correct size of the reinforcement and setting the appropriate elevation, the extensive and intricate slab reinforcement was accurately created.

The intersect command was utilized to specify whether the columns, beams, or slabs should be given priority in the model. Priority was given to the slab, creating a continuous slab across the entire plan of the structure, truncating the columns wherever they intersected.

With the frame of the structure modeled through the full height, infill walls were placed in the locations where they remained after the conventional demolition. An illustration of the completed structure can be seen in the following figure.

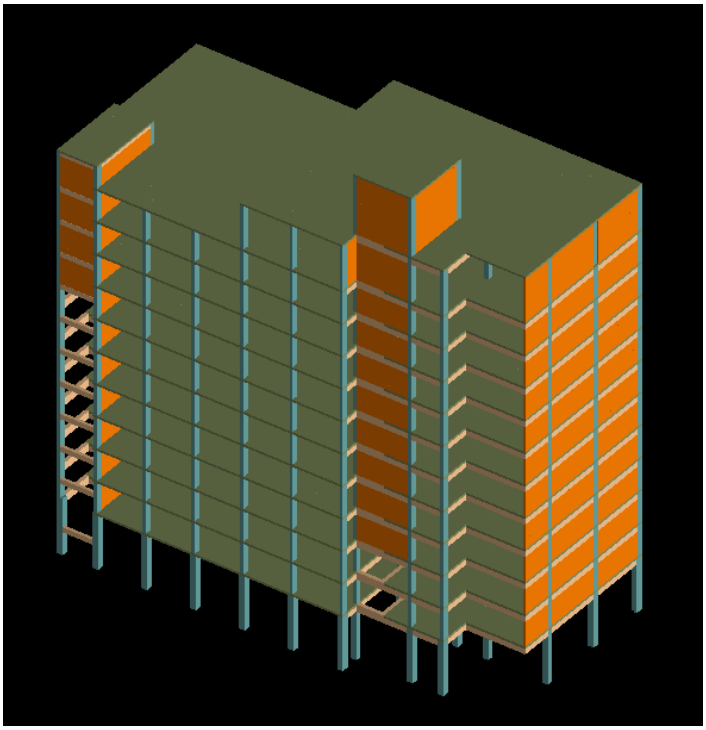


Figure 21. Crabtree Sheraton Hotel ELS® Model

5.2.2. Loading Scenario

The loading scenario consisted of two stages, Stage 1 and Stage 2. Stage 1 being the default stage, in which the self-weight of the structure was applied over 10 increments. Stage 2 is a dynamic stage during which the element removal or implosion takes place. The duration of Stage 2 was 10 seconds, allowing enough time for element removal and complete

collapse. The time step within Stage 2 was 0.005 seconds with 10 increments, creating a total of 20,000 steps.

Element Removal

To simulate demolition of the columns caused by the dynamite during the implosion, the immaculate element removal option within ELS® was utilized. Groups of elements were created by separating them based on their time of removal. Utilizing the information presented by the explosives subcontractor, each group was specified to be removed at the exact time it was during the implosion, thus creating a scenario that precisely matched the actual detonation and delay sequence.

The shortening pattern used by the explosives subcontractor was also modeled, which is where the lower portion of a column near the end of the delay sequence is blasted earlier than the remainder of the column to create a hinge. As the implosion sequence progresses and the structure gains velocity, prematurely blasting the bottom of the columns allows the structure to rigidly pivot on those columns before blasting the upper portion. Each column included in the shortening pattern was accounted for at the appropriate time delay steps in the removal groups.

The implosion sequence for the Crabtree Sheraton Hotel progressed diagonally through the structure from the southwest corner to the northeast corner. The primary locations of element removal were on the lobby, 1st, and 5th floors with the blasts separated by 500 millisecond delays. A total of 13 removal groups were created in ELS®.

5.2.3. Model Details

The total mesh for the structure consisted of 16,392 elements. For columns and beams, the mesh looked like slices of the cross-section. For built-up columns, there was more than one element when looking in cross-section as previously illustrated. Slabs consisted of a mesh that was approximately three feet by three feet. The slab thickness defined the depth or thickness of the slab elements. For the concrete block walls, each element represents one block. The blocks were modeled at twice their actual length and height, making them four times coarser than in reality. An illustration of the entire structure's mesh can be seen here.

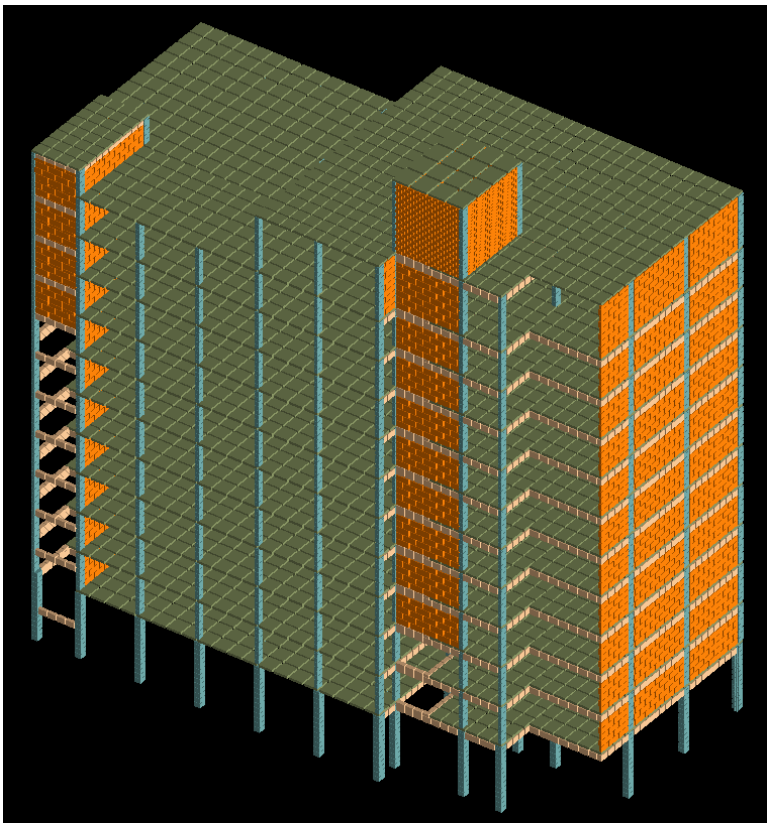


Figure 22. Illustration of Mesh for Crabtree Sheraton Hotel Model

The analysis would have benefited from refining the mesh, however the version of ELS® used for the analysis was only capable of handling 25,000 elements. Uniformly refining all of the columns or all of the slabs across the structure would have exceeded the allowable element count. Preference was not given to certain areas of the structure with regard to mesh refinement because the entire structure is undergoing collapse. Varying the mesh across the structure between common structural components could have instilled different behavior during the analysis.

The average aspect ratio in the model was 4.3, with an average element volume of 3.6 cubic feet. The analysis was performed on a standard desktop computer with an Intel® Pentium 4, 2.40 GHz processor and 1.5 gigabytes of RAM. It took roughly 66 hours to complete the analysis. The output file created, which included all of the default output, was 32.5 gigabytes.

5.2.4. Initial Model Inaccuracies

Evaluation of the initial model revealed one major inaccuracy during collapse of the structure. The model predicted the column-slab continuity to be much less than that witnessed in the documentation video. In the middle of the collapse, as the failure progressed across the structure, the slabs and columns separated prematurely in the model. As a result, the ductility of the structure was reduced and the full catenary forces were not allowed to develop. The difference between the model behavior and actual behavior is evident in the following illustration.

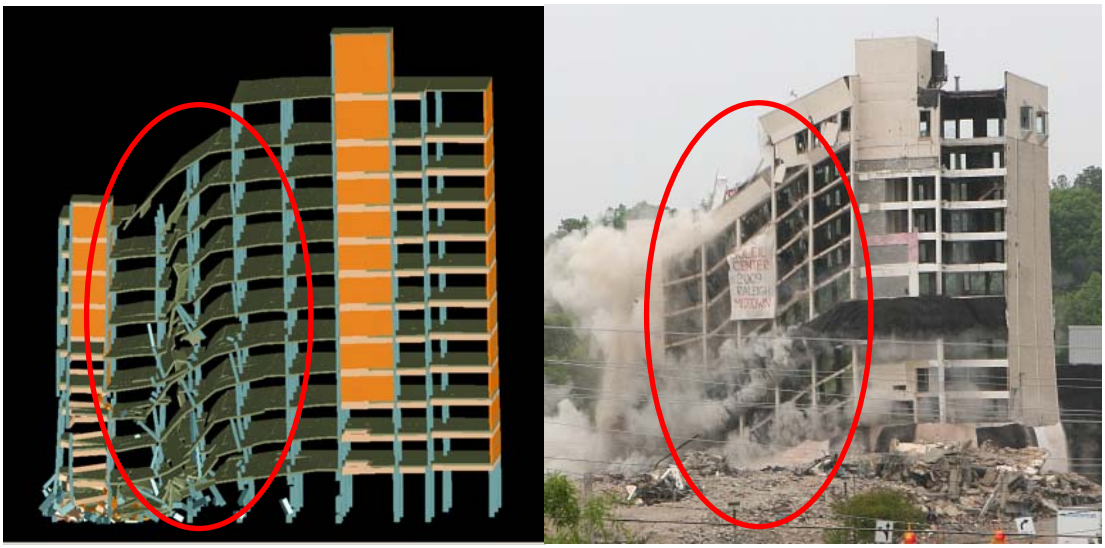


Figure 23. Initial Crabtree Sheraton Hotel Model Inaccuracy

Meticulously reviewing the progression of collapse in the model and scrutinizing the evolution of stress contours, three items were identified to be potentially contributing to the inaccuracy. The mesh discretization across the structure, the priority chosen for structural components using the intersect command, and the failure criteria for reinforced concrete were classified as possible hindrances.

5.2.5. Parametric Study

To investigate and isolate the effect of each modeling parameter on the structure's collapse behavior a parametric study was performed. The goal was to determine the ideal combination of parameters that most realistically simulated the collapse behavior for a reinforced concrete structure like the Crabtree Sheraton Hotel.

To limit the analysis time, a smaller structure was used for this study. A three-story, column and slab building using the same two-way flat plate behavior was utilized. The dimensions, reinforcement, element removal, and time steps were all similar to that in the Crabtree Sheraton Hotel model.

To investigate the effect of mesh discretization, the slab mesh was varied within the parametric study. Evidence from the initial trial proved that because the slab elements were much larger than the column elements that they connected to, excessive moment was transferred to the connection. The relative size difference between the column and slab elements can be seen in the 3-D illustration below.

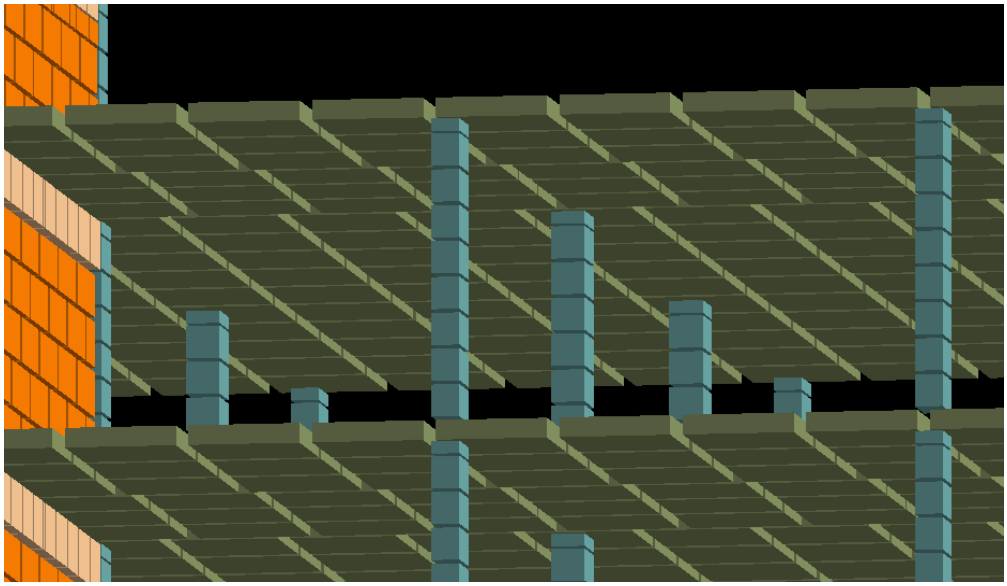


Figure 24. Initial Crabtree Sheraton Model Mesh Discretization

This connection potentially contributed to the premature failure at the column-slab interface, causing the lack of continuity.

The intersect command was investigated by specifying different structural component priority within the parametric models. Designating slab priority, as done in the initial model, creates a continuous slab across the entire floor and truncates the columns between each floor as seen in the previous figure. This creates an element interface in the plane of the top and bottom of the slab. If the failure criteria is reached with this element prioritization, the columns are allowed to "fall out" of the structure because they are not continuous or incorporated into the slab.

To investigate the failure criteria, the ELS® Technical Manual was referenced to identify what factors defined the behavior. The manual states that for reinforced concrete, the reinforcement springs representing the rebar are cut when either; 1.) the normal stress on the reinforcement reaches its ultimate tensile stress, or 2.) the matrix (concrete) springs reach their separation strain limit (Applied Science International, 2006). By defining the steel reinforcement failure criteria as a function of the concrete strain, it reduces the ductility capacity of the structure. This criterion does not allow the reinforcement to "tie" the structure together and develop the catenary forces present during collapse scenarios.

In effort to prevent reaching the failure criteria early, the shear modulus was varied to document the effect of transferring different amounts of shear force to the column-slab interface. Essentially, varying the shear modulus is an indirect way for the user to control the failure criteria.

Three other parameters were varied to see if they would effect the overall stability of the model during collapse. These parameters were the normal contact stiffness, shear contact stiffness, and the contact spring unloading stiffness factor or rebound stiffness factor. These parameters were treated as a group and toggled between their default values and newly recommended values from the ASI technical support team.

Thus, the file developed for the parametric study contained four structures that were identical with regards to dimensions and reinforcement as previously described. The structures, labeled A-D from left to right as seen below, were each unique however in terms of element priority and slab mesh discretization.

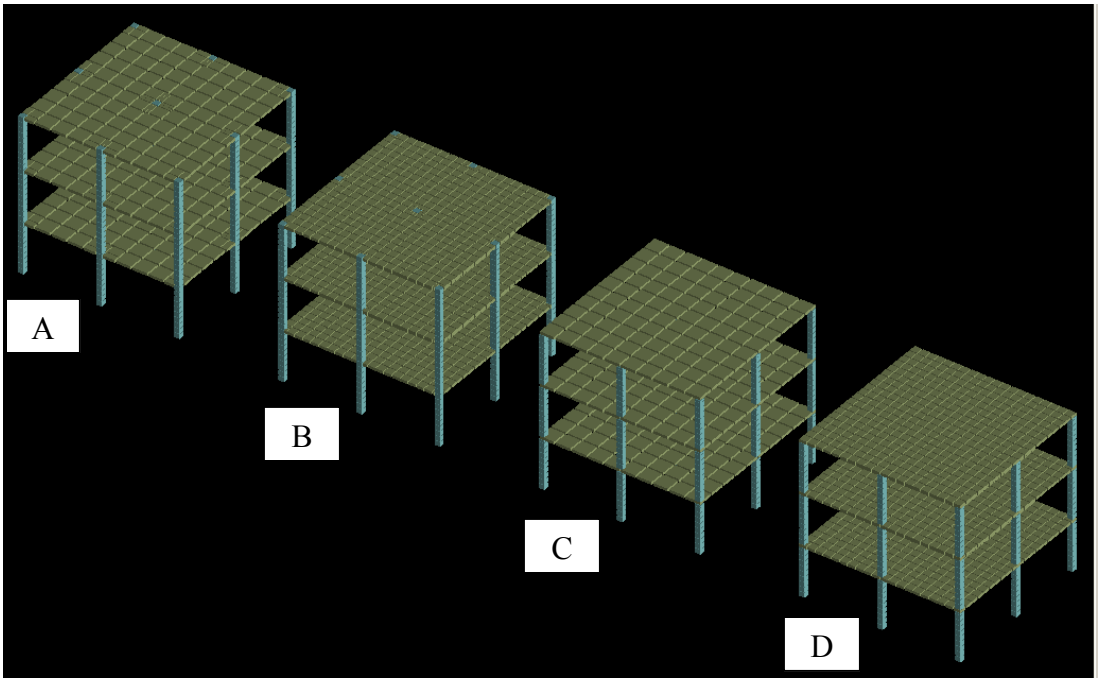


Figure 25. Parametric Study Models

Of the four structures, model A has column priority with a large slab mesh, model B has column priority with a small slab mesh, model C has slab priority with a large slab mesh, and model D has slab priority with a small slab mesh. The only other parameters investigated were the material properties previously described.

The parametric matrix consisted of six individual runs of the file containing four models, for a total of 24 unique scenarios. The complete matrix defining the parameters for each run and model is presented in the following table.

Table 3. Parametric Study Matrix

		Element Priority		Slab Mesh Size		Shear Modulus, G [1]			Other Parameters [2]	
Run	Model	Column	Slab	Large	Small	0.3E	0.37E	0.4E	Default	Revised
1	A	●		●		●			●	
	B	●			●	●			●	
	C		●	●		●			●	
	D		●		●	●			●	
2	A	●		●			●		●	
	B	●			●		●		●	
	C		●	●			●		●	
	D		●		●		●		●	
3	A	●		●				●	●	
	B	●			●			●	●	
	C		●	●				●	●	
	D		●		●			●	●	
4	A	●		●		●				●
	B	●			●	●				●
	C		●	●		●				●
	D		●		●	●				●
5	A	●		●			●			●
	B	●			●		●			●
	C		●	●			●			●
	D		●		●		●			●
6	A	●		●				●		●
	B	●			●			●		●
	C		●	●				●		●
	D		●		●			●		●

[1] The Shear Modulus, G is given as a percentage or fraction of Young's Modulus, E.

[2] Other Parameters include:

	Default	Revised
Normal Contact Stiffness	0.01	0.001
Shear Contact Stiffness	0.001	0.0001
Rebound Factor	10.0	3.0

Evaluating each model using the applicable criteria in the evaluation matrix helped decipher which combination of modeling techniques and parameters best simulated this type of collapse. Of the 24 individual models analyzed, model B of run 4 exhibited the most realistic behavior. That particular model utilized column priority, small slab mesh, smallest shear modulus, and revised material parameters.

Considering the *Postprocessing*, model B of run 4 had its tradeoffs with other models on the local level. Model B experienced punching shear of the columns, while model's C and D experienced slab shear in the middle of the spans. The main advantage model B had was at the global level, as the punching shear of the columns did not interfere with the progression of collapse. From the *Visual Inspection* criteria, when the slabs in C and D separated, it caused an erratic progression of failure, destroying the flow of collapse. Whereas in model B, punching shear of the columns did not inhibit the overall progression of collapse across the structure.

5.2.6. Revisions for Final Analysis

The Crabtree Sheraton Hotel model was re-created incorporating the set of parameters found to be optimal. Although the smaller slab mesh exhibited more realistic behavior, the slab mesh could not be revised in the Crabtree Sheraton Hotel model due to the limitation on the number of elements. Prior to analyzing the revised model, through further consultation with the technical support team at ASI, the normal contact stiffness factor and shear contact stiffness factor were reduced beyond the values used in the parametric study. A small investigation into this modification proved that the effect from a behavioral standpoint was negligible. From a computing standpoint however, the further reduced values offer greater numerical stability during collapse scenarios.

The parameters and techniques used in the final analysis of the Crabtree Sheraton Hotel model can be seen in the table below.

Table 4. Revised Crabtree Sheraton Hotel Model Parameters

Element Priority	Column
Slab Mesh Size	Identical to original model
Shear Modulus, G	0.3E or 1,140 ksi
Normal Contact Stiffness Factor	0.0001
Shear Contact Stiffness Factor	0.00001
Rebound (Unloading Stiffness) Factor	3.0

5.2.7. Model Evaluations

Evaluation of the initial model paralleled with evaluation of the revised model highlights the improvements in behavior as well as the inconsistent aspects. Using the criteria contained in the evaluation matrix as a rubric for comparison, the input was approximately the same for both models. The final model contained the revised parameters, but that is not a quantifiable difference with regards to input accuracy because they are modeling techniques or parameters that do not exist in actuality. Only the shear modulus is a physically verifiable parameter, but all of the information concerning the structures material properties was unknown. Therefore, the initial model and final model were exactly equal in terms of the evaluation matrix under *Construction* and *Preprocessing*.

The variance between the models became apparent when examining the structures behavior during collapse. The final model was revised in attempt to prevent the lack of continuity seen in the initial model. The revisions were successful in improving the behavior of the flat-plate construction during collapse, however as a byproduct, a new inaccuracy was introduced. Some of the columns within the structure punch through the slab upon impact resulting in full-length freestanding columns. This behavior was evident, to a much smaller degree, in the parametric study. The arrow in the following figure indicates the punching

shear of the columns on the back left corner. The figure compares the initial model, on the left, and the final model, in the middle, to the actual behavior on the right.

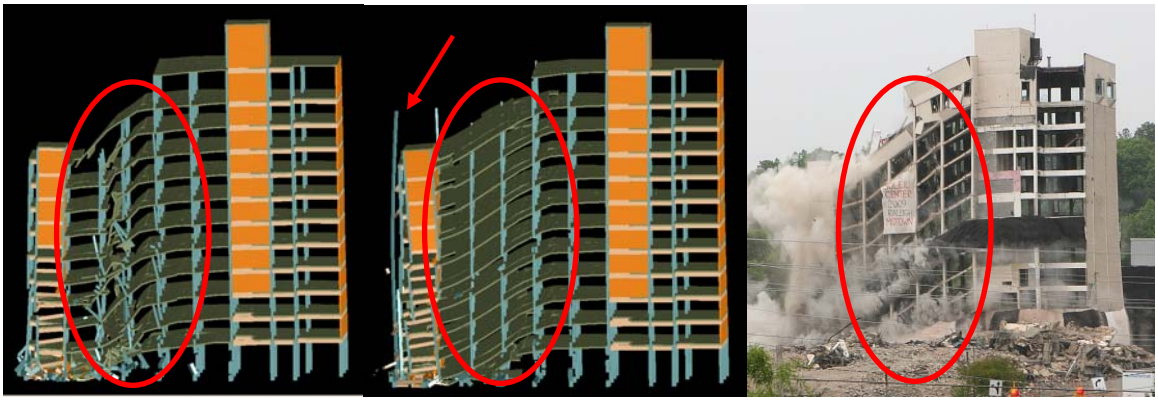
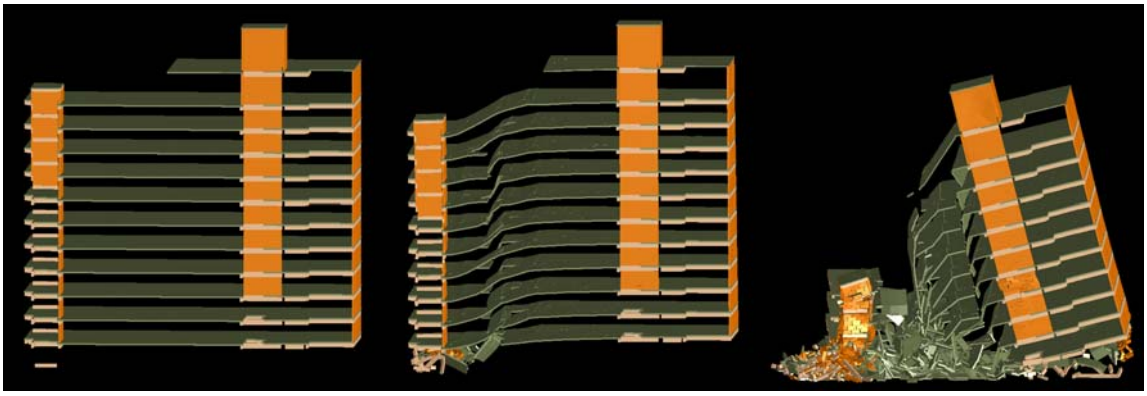
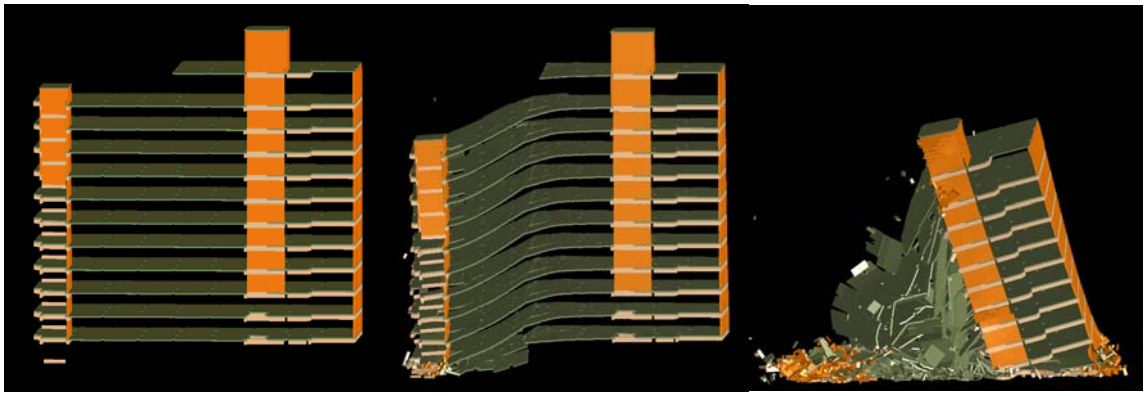


Figure 26. Comparison of Continuity in Crabtree Sheraton Hotel Model

Other than the obvious visual inaccuracy, the column behavior in the final model only has a minor adverse effect as it causes the masonry walls to fail early. Although the columns separate from the structure in the final model, it does not destroy the slab continuity, which allows for steady progression of collapse across the structure. Examining both models with the columns hidden, the slab behavior can easily be visualized. In the initial model, with slab priority, the continuity is lost before the catenary forces are fully transferred across the structure. In the final model, with column priority, the continuity is maintained, allowing the forces to act on the right side of the structure for a longer period of time. This is evident in the series of pictures below where in the last frame the tilt of the elevator tower and level of collapse is slightly further along for the final model.



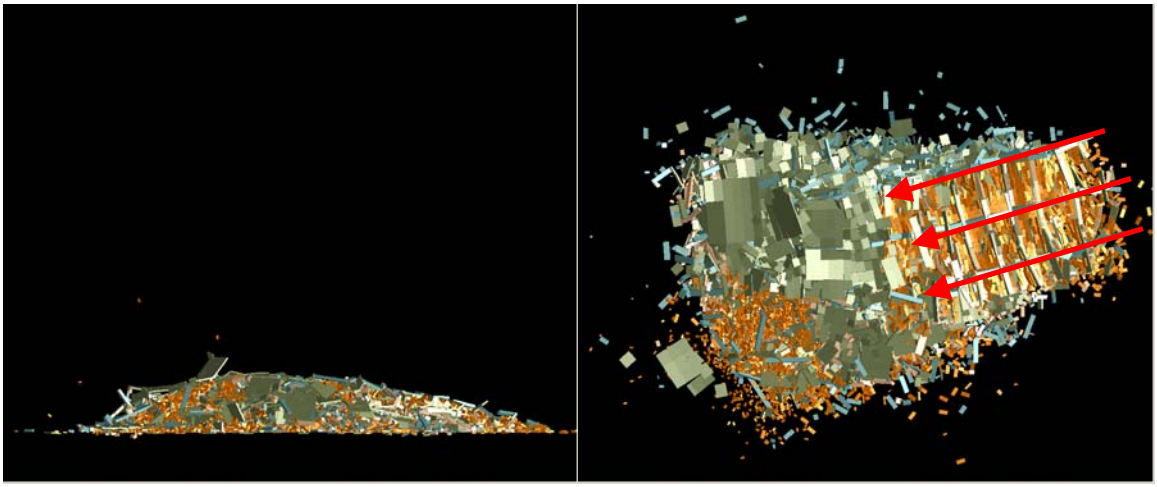
a.) Initial Model with Slab Priority



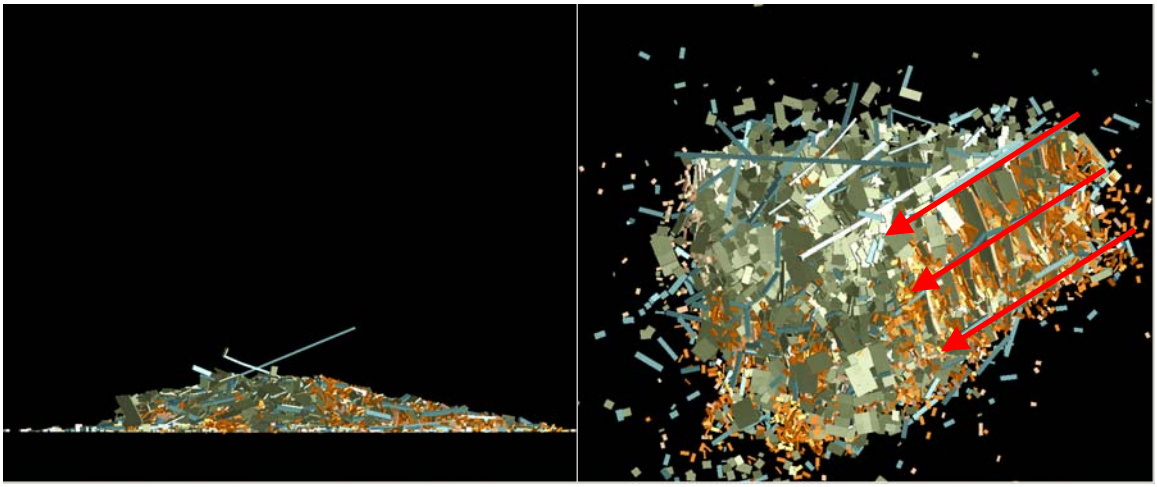
b.) Final Model with Column Priority

Figure 27. Crabtree Sheraton Hotel Models with Columns Hidden

In addition to altering the behavior during collapse, revising the model parameters had a slight impact on the final debris field. As a result of the slabs maintaining continuity longer, the debris field in the final model is angled forward more. The Crabtree Sheraton Hotel collapsed towards the southwest, in a direction perpendicular to the blast pattern. In the initial model, early separation of the structure caused the exterior wall on the right side to fall almost directly on top of the buildings footprint. The following illustrations emphasize this difference by highlighting where the exterior columns on the right side of the structure came to rest. An elevation of the debris field for each model is shown along with a plan view.



a.) Initial model debris field



b.) Final model debris field

Figure 28. Crabtree Sheraton Hotel Model Debris Fields

Complete evaluation matrices for the initial and final model of the Crabtree Sheraton Hotel can be found in the appendices.

5.2.8. Observations

For modeling flat-plate construction, column priority should be specified if the main goal is to predict the global collapse behavior. The column behavior itself may be

inaccurate, but a model with column behavior will best simulate the progression of collapse for the entire structure.

If the slab mesh can be defined so that the slab elements are the same size as the column elements then it does not matter what structural component is given priority. Having all elements approximately the same size is ideal, but as mentioned is not attainable for structures of this magnitude.

For reinforced concrete structures undergoing collapse, the revised material parameters presented in the following table should be used.

Table 5. Recommended Concrete Material Parameters

Shear Modulus, G	0.3E
Normal Contact Stiffness Factor	0.0001
Shear Contact Stiffness Factor	0.00001
Rebound (Unloading Stiffness) Factor	3.0

Modification of the shear modulus is not intended to be a permanent solution. ASI is continuously working to enhance the material models contained in ELS®. Once an adequate model is developed to more accurately simulate the post-cracking shear transfer of forces in reinforced concrete, the shear modulus can be defined using traditional formulas or experimental data.

The revised values for normal contact stiffness, shear contact stiffness, and unloading stiffness are intended for permanent implementation. ELS® over-estimates the contact and impact forces when a column drops after element removal and impacts the remaining lower portion of the column or the ground. The force is over-estimated because the contacting elements both have flat faces. Whereas in actuality, the remaining portions of the column adjacent to the blast are jagged and severely cracked. In addition, the rebar is still intact at

the location of the blast in an actual structure, further dampening the movement and contact between the remaining elements. Utilizing these revised material parameters accounts for these differences within the model and produces a more realistic collapse simulation, in addition to providing greater numerical stability.

5.3. Case Study II – Stubbs Tower

5.3.1. Model Construction

All of the structural dimensions, reinforcement details, and material strengths were attained off the as-built structural drawings dated February 1970. An AutoCAD file outlining the base dimensions in plan view of the structure was once again imported into ELS® to aid in construction of the model.

Material Properties

Stubbs Tower was constructed of reinforced concrete with steel beams in select locations. The material properties for all of the structural components were taken from the structural drawings. The following table presents the material properties utilized in this model.

Table 6. Stubbs Tower Material Properties

Concrete		Compressive Strength	4,000 psi
		f'c (Estimated) [1]	4,750 psi
		Tensile Strength	10% f_c' or 475 psi
		Young's Modululs, E	4,000,000 psi
		Shear Modulus, G	1,600,000 psi
Reinforcing Steel [2]	Column Longitudinal	Yield Strength	60,000 psi
		Ultimate Strength	80,000 psi
	Column Transverse	Yield Strength	33,000 psi
		Ultimate Strength	55,000 psi
	Slab Longitudinal	Yield Strength	50,000 psi
		Ultimate Strength	80,000 psi
	W.W.M.	Yield Strength	65,000 psi
		Ultimate Strength	85,000 psi
Structural Steel [2]		Yield Strength	36,000 psi
		Ultimate Strength	50,000 psi

[1] Estimated strength after 37 years.
[2] For all steel, E = 29,000,000 psi and G = 11,600,000 psi.

Structural Components

In Stubbs Tower there were four main structural components; columns, girders, slabs, and walls. On the first floor where columns were the load-bearing system, there were four column styles. Deep girders ran in the east-west direction spanning adjacent columns on each side of the hallway. Two smaller beams ran along the exterior on each side the full length of the structure in the north-south direction connecting all of the column girder frames.

Above the first floor, walls served as the gravity load-bearing system, running in the same direction as the column-deep girder frames. The wall style was modeled using 8-node elements so that the dimensions could easily be defined for each unique wall. The only other structural elements on the upper floors, besides the transverse walls, were the stair towers and

elevator core. The stair towers and elevator core were modeled using 8-node elements as well.

The 8-node elements were not specified to contain any reinforcement. Instead, all of the reinforcement for the vertical components was imported from AutoCAD and then converted. The illustration below shows the reinforcement for the stair towers and elevator core.

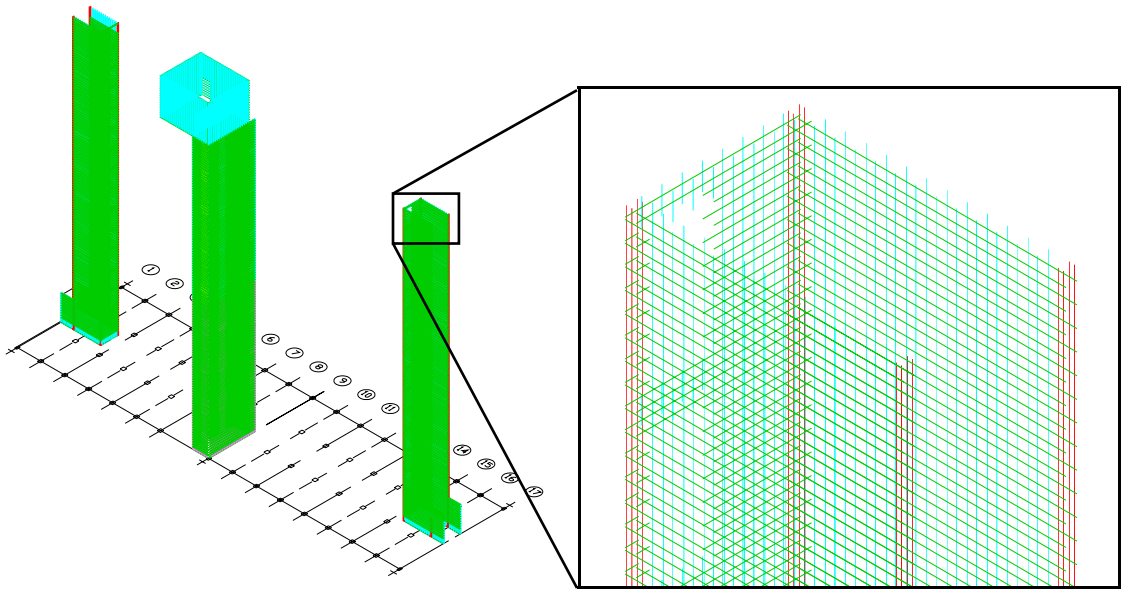


Figure 29. Stubbs Tower Stair Tower and Elevator Core Reinforcement

The last structural component was the slabs that were input into the model atop the column-deep girder frames on the first floor and above the transverse walls on the upper floors. Five slab styles were used to represent the typical floor on each level, the deeper slab in the bay of the elevator, the stair tower landings, the mechanical room floor, and the roof. The reinforcement for all the slabs except for the stair tower landings was imported.

Following insertion of the structural components, the elements were intersected giving priority in the order of slabs, girders, columns, and lastly walls. Column priority was not specified because they do not occur beyond the first floor.

Building Preparation

Upon completion of the model representing the as-built conditions, modifications were made to simulate the preparatory work performed by the demolition contractor. Other than removing the majority of the nonstructural elements in the building, there were three significant modifications made to the structure.

The first modification was sawcutting a select portion of the transverse walls to ensure that they would hinge during the collapse. The six-inch walls were cut approximately four inches deep on what would become the tension side during collapse. By cutting two-thirds of the wall away, the reinforcement mesh centered in the wall was severed. The following is an elevation of the structure with arrows detailing which walls were cut and from which side.

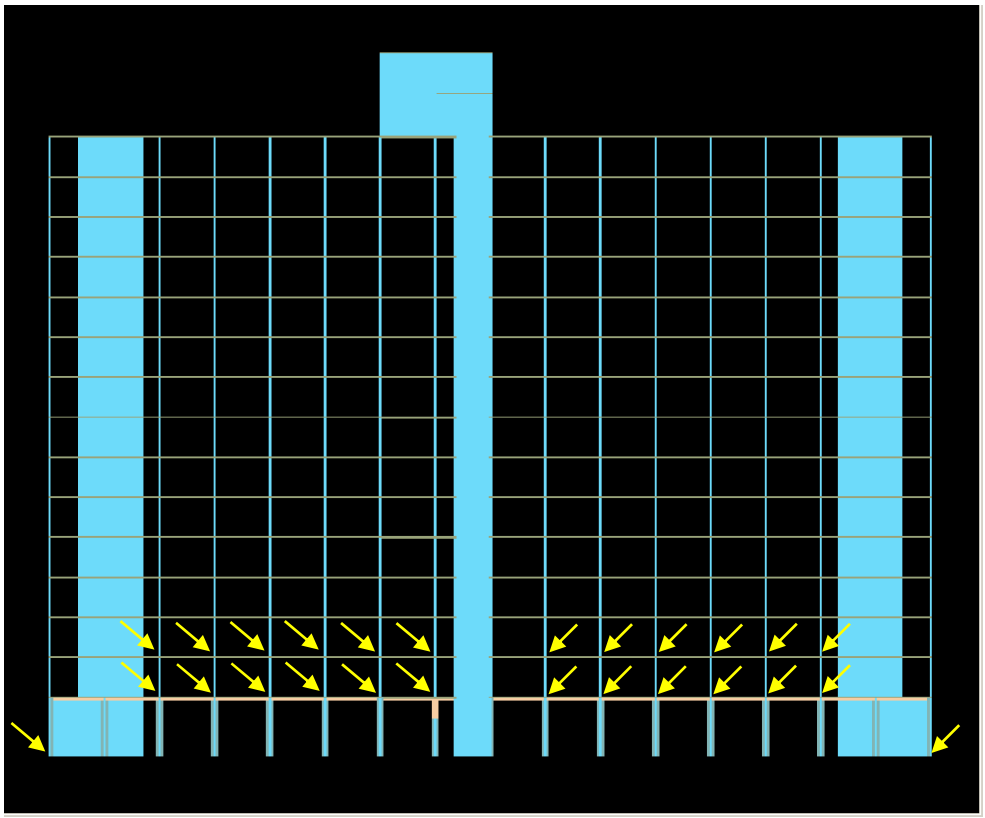


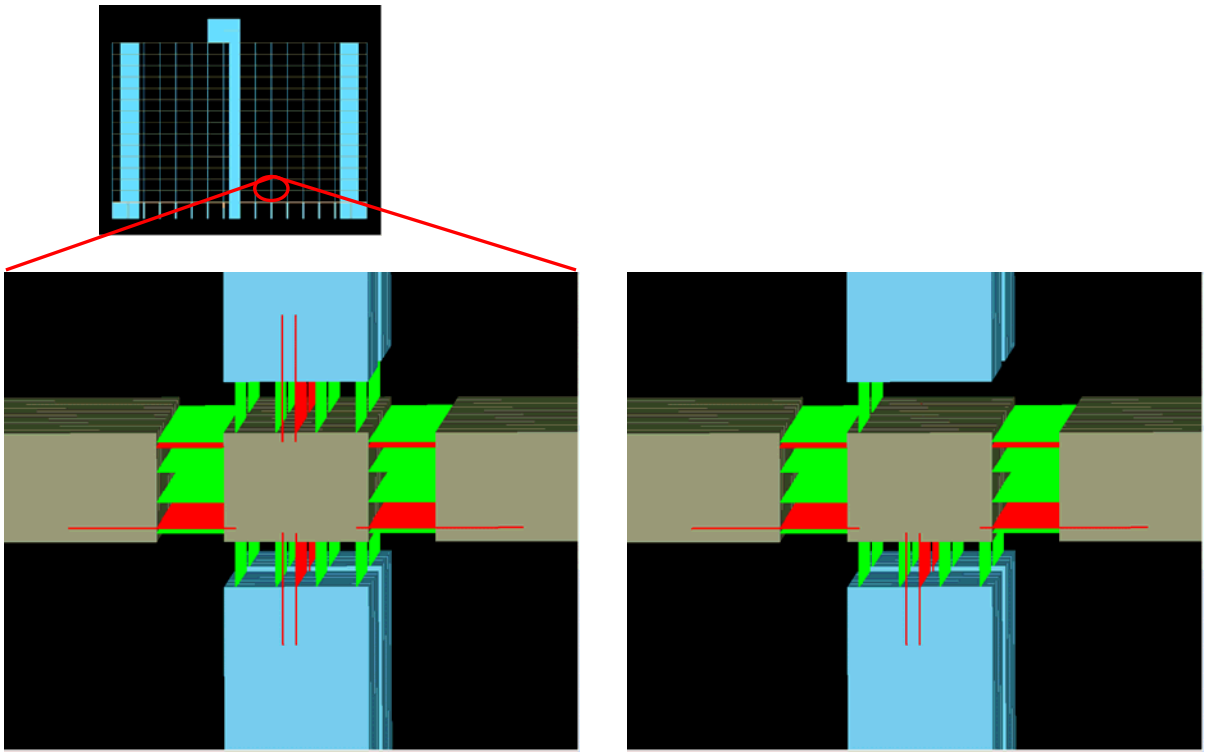
Figure 30. Location of Sawcuts

The sawcuts were made on approximately the same plane as the top of the floor slab. The following picture shows the sawcut being made on one of the interior walls.

Figure 31. Interior Wall Sawcut

To account for this condition in the model, the technical support team at ASI created an option allowing the user to specify planes in which there were no springs. By inputting

sawcut planes into a text file, ELS® was able to identify these locations and remove all of the matrix and reinforcement springs, thus making that location a contact surface. This method worked extremely well and can be seen in the following figure.



a.) Wall before sawcut **b.) Wall after sawcut**

Figure 32. Sawcut in ELS® Model

The second modification to the structure was removal of portions of the stair towers. Portions of the stair tower walls were removed on the first floor to create a hinge for the stair towers to rotate on during collapse. Removing the material prior to the blast is more economical than using explosives and it ensures that the hinge is in there in case the structure moves faster than predicted for the delay sequence.

The amount of removed material was calculated and the appropriate elements were deleted within the model. The following pictures depict the final condition of the stair towers prior to the implosion.

a.) Face of stair tower from hallway

b.) Side of stair tower

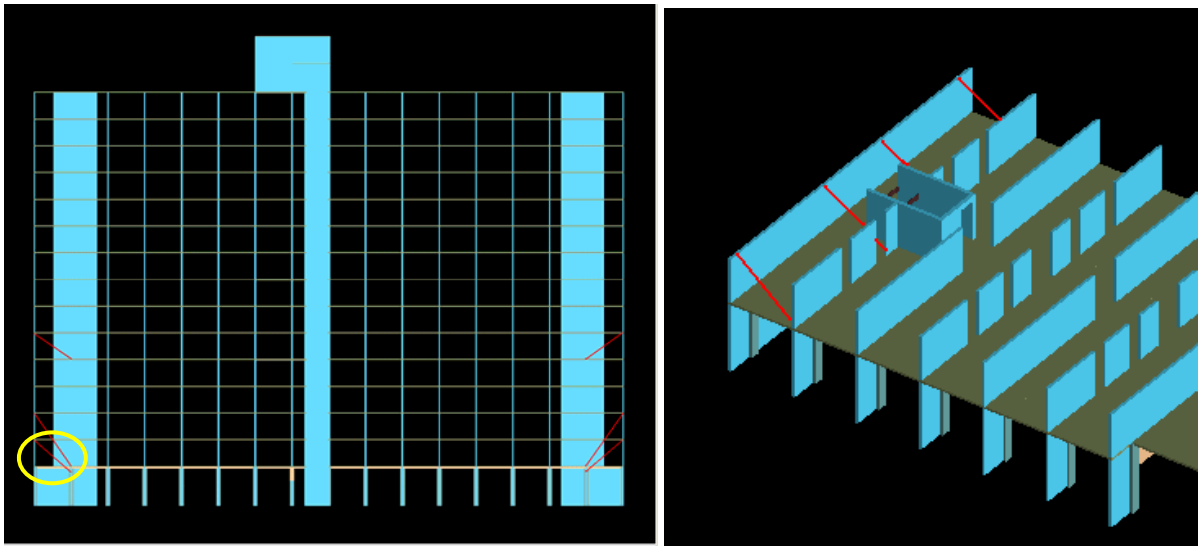
Figure 33. Preparatory Work on Stair Towers

The last alteration made to the existing structure was the installation of steel cables on both end walls. The cables were installed to insure that the exterior walls were pulled inward as the stair towers collapsed toward the middle of the building. A picture from inside the structure with the cables in place can be seen below along with a picture taken from the outside showing the cables coming through the outer wall.

a.) View of cables inside structure **b.) View of cables from outside**

Figure 34. Illustration of Installed Cables

Four, ¾-inch diameter cables were placed on the north and south end of the second, third, and sixth floors, for a total of 24 cables. An elevation of the structure below indicates the location of the cables. An isometric cross-section of the structure shows how the cables were placed on each floor in the model. The cross-section illustrates the cables installed on the south end of the second floor that are encircled in the first figure.



a.) Elevation of structure with cables **b.) Isometric cross-section showing cables**

Figure 35. Illustration of Cables in ELS®

To model the cables in ELS®, tension-only elements were used. The following table lists the material properties used to define the tension-only elements. The separation strain was approximated by calculating the expected elongation under normal conditions and multiplying it by a dynamic increase factor.

Table 7. Tension-only Element Material Properties

Tensile Capacity	35,000 lbs
Separation Strain	0.10
Modulus, E	29,000,000 psi

5.3.2. Loading Scenario

The loading scenario again consisted of two stages. During Stage 1, the self-weight of the structure was applied over 25 increments as opposed to the default of 10 because the sawcuts were being modeled during this period. Stage 2 was the dynamic stage containing the element removal. The duration of Stage 2 was 10 seconds with 0.005-second time steps and 10 divisions in each time step, creating a total of 20,000 steps.

Element Removal

The implosion sequence created by the explosives subcontractor, Demolition Dynamics Company, consisted of 11 timed delays occurring over four seconds. The sequence initiates in the middle of the structure in the elevator core. Blasts occur on four floors in the elevator core, removing material and creating vertical drop, which produces lateral pull through the slabs in the north-south direction. Once the lateral force has time to develop, the blast continues on the first floor by progressing outward from the elevator core. The graphic below, produced by the Savannah newspaper days before the implosion after witnessing the model prediction, illustrates the blast sequence.

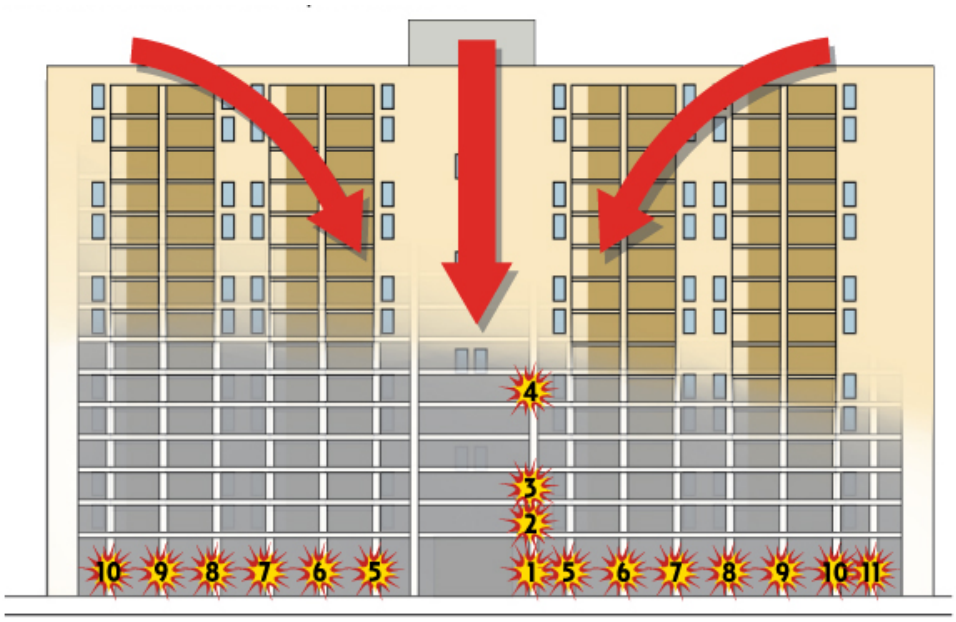


Figure 36. Stubbs Tower Implosion Sequence (Savannah Morning News, 2007)

A shortening sequence was used on the last two frames on each end of the structure. The shortening sequence was not directly input into the model. Instead, the timing of each shortened frame was averaged, creating a removal time that was after the blast at the base of the frame, but before the final blast at the top of the frame. An example of this can be seen below for the south end of the structure. For easier visualization, the only elements included in the graphic are the columns and girders on the first floor.

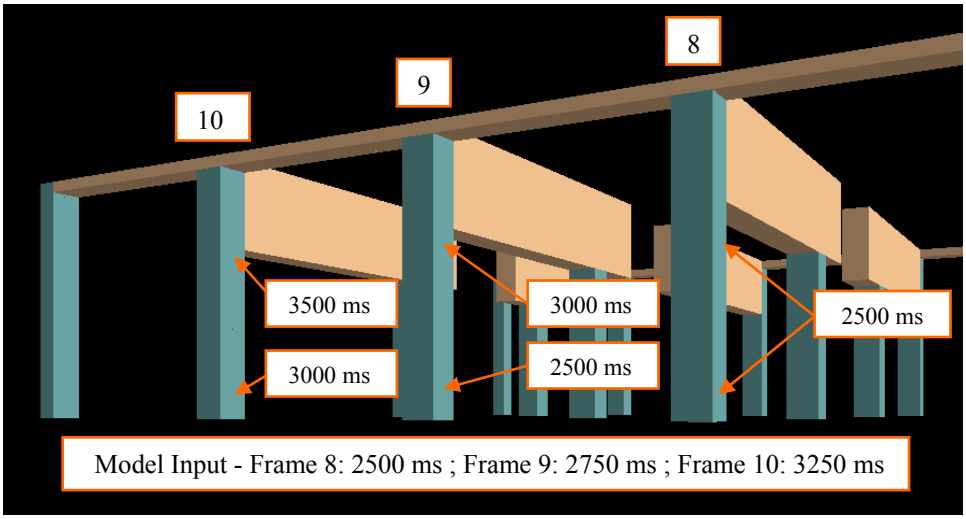


Figure 37. Model Approximation for Shortening Sequence

In using this methodology to approximate the removal times for the shortened frames, an additional removal group was necessary in the model. Therefore, the model required 12 removal groups to represent the implosion sequence.

In addition to approximating the shortening delays, consideration had to be given to the time delays input into ELS®. A preliminary analysis showed that the actual time delays could not be used directly because they were too slow for the model. Because ELS® removes elements immaculately, elements accelerate faster than in reality. In the preliminary analysis, the collapse out ran the element removal as can be seen in the illustration where the complete second and third floor has collapsed on top of the first floor columns which are not yet removed.

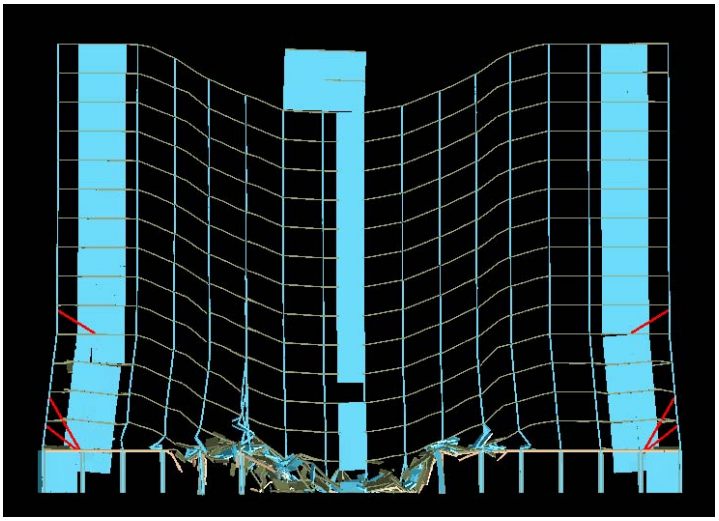


Figure 38. Preliminary Analysis Results Showing Need for Delay Timing Modification

The initial four blasts in Stubbs Tower removed 30 feet of material from the elevator core. In order to account for the models reaction to this amount of material removal, the timing of each delay was divided by a constant factor of 1.5. Speeding up the delay timing allows the structure to simulate the actual collapse behavior and continuous flow. The following table shows how the delay times from the previous figure would be modified.

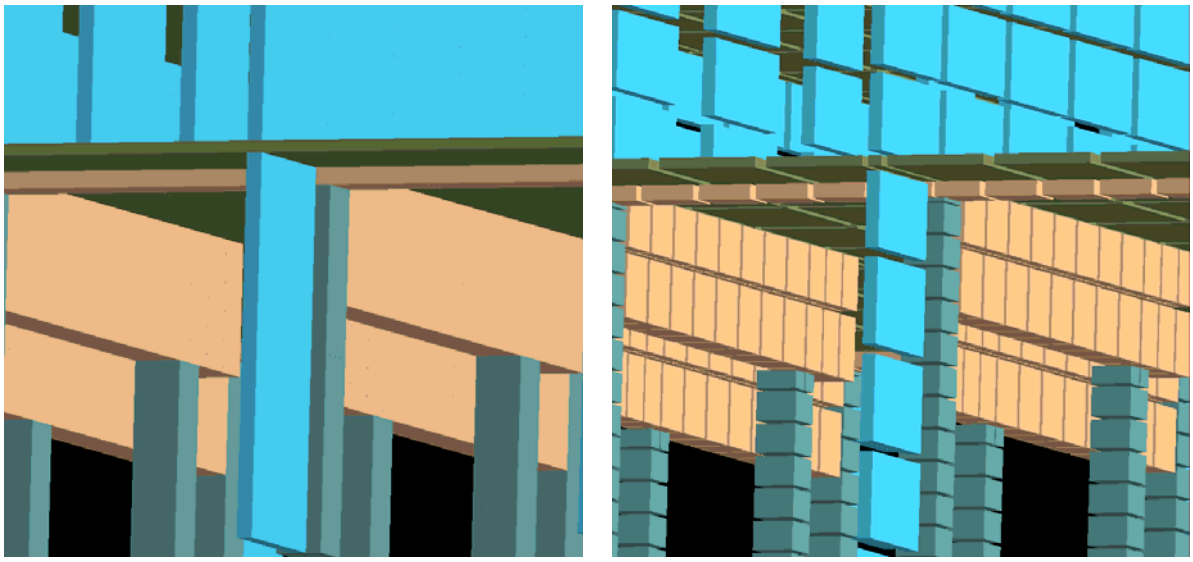
Table 8. Calculation of ELS® Removal Group Delay Timing

	Actual (with shortening approximation)	Model Input (modified by factor of 1.5)
Frame 8	2500 ms	1667 ms
Frame 9	2750 ms	1833 ms
Frame 10	3250 ms	2167 ms

5.3.3. Model Details

The total mesh for the Stubbs Tower model consisted of 21,463 elements. For columns and the two longitudinal girders on the first floor, the mesh was merely slices of the cross-section. For the deep girders that span east-west in the structure, the depth of the girder was cut in half to create two elements when looking at a cross-section. The walls were

modeled with elements that were approximately 3 feet by 3 feet, with the depth defined by the wall thickness. The following figure, which is looking upward at the first floor from ground level outside the building, aids in visualizing the mesh for these elements.



a.) First floor exterior column and girder frames **b.) Component mesh**

Figure 39. Stubbs Tower Model Structural Component Mesh

Special consideration was given to the slab mesh. To prevent the inaccuracy discovered in the Crabtree Sheraton Hotel model in response to the relative size of the slab mesh, in the Stubbs Tower model the slab was broken into regions directly over the walls and clear span regions. The region or portion of the slab that was directly over the wall had elements with the exact width of the wall. The depth was defined by the slab thickness. However, in the clear span, the slab elements were much coarser due to the limitation on the number of elements. This is evident in the following figure, which shows the mesh for typical bays on the upper floors.

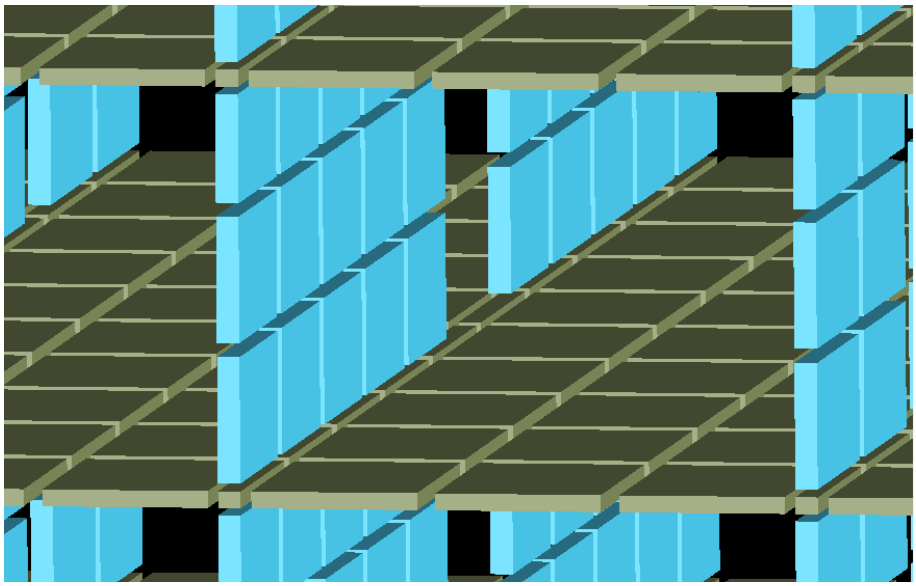


Figure 40. Stubbs Tower Model Slab Mesh

The mesh for the entire structure can be seen in the following figure.

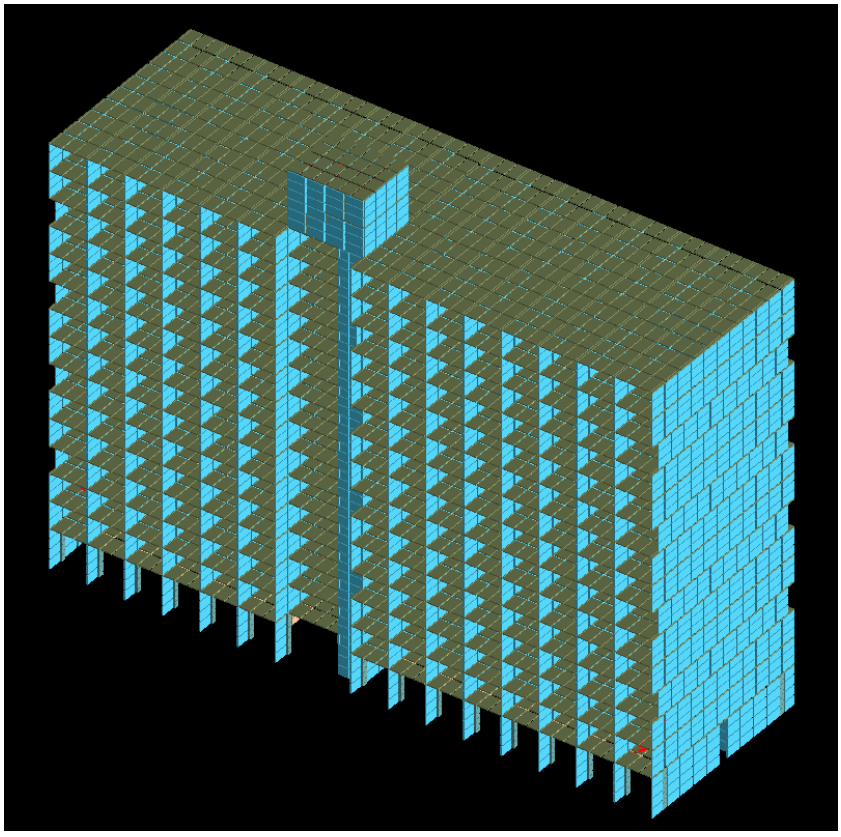


Figure 41. Stubbs Tower Model Element Mesh

The average aspect ratio for the elements in the model was 15.8 while the average volume was 5.81 cubic feet. The analysis was performed on a desktop computer running 64-bit Windows 2003, with a 2.4 GHz Core 2 Duo processor and 4 gigabytes of RAM. It took approximately 55 hours to complete. The resulting output file was 8.5 gigabytes and only contained element forces, velocities, accelerations, and contacts. Stress and strain computation was turned off to keep the file size relatively small.

5.3.4. Publication of Prediction

The initial Stubbs Tower model analysis was completed prior to the implosion. Upon approval from the demolition contractor, the explosives subcontractor, and ASI, videos of the simulation were released to the media. The videos depicted the expected collapse behavior and presented predictions for the debris field and surrounding impact. Upon receiving the videos, media outlets used them to communicate to the public what was to be expected.

In addition to releasing the videos, a joint presentation was given by North Carolina State University and D.H. Griffin Wrecking Company, Inc. to the residents at The Veranda. The Veranda was an adjacent property built to house the occupants that were displaced when Stubbs Tower flooded in 2004. Many of the residents at the Veranda had lived in Stubbs Tower for several years and had a personal interest in its demolition. The presentation served to answer their questions about the implosion procedure, what to expect, and how they would be affected.

All of this pre-implosion coverage is significant because it marks the first time that the predicted response to an implosion has been publicized with ELS® software. Publication of the model prior to the event certifies that the structural response was purely predicted

through ELS® without any modifications in hindsight. The publication does not guarantee that the prediction is entirely accurate, but it does guarantee that the structural response witnessed in the model is only calculated from the input information obtained from the collaborating companies.

5.3.5. Initial Model Inaccuracies

Evaluation of the initial model yielded two inaccuracies in the structural behavior during collapse. The model predicted a gradual, flowing collapse, similar to that which was sought after in the Crabtree Sheraton Hotel model. In actuality, the collapse did not flow steadily outwards due to different behavior or stiffness in some of the bays. Some bays contained nonstructural cold-formed walls on the exterior. These walls were not input into the model because they were not expected to have a significant impact on the behavior. However, during the collapse, the bays without the cold-formed walls behaved as predicted, while the bays with the walls were much stiffer, dropping rigidly and not distorting from their initial shape. This is evidenced in the following illustration where two bays on each side of the elevator core contain exterior walls. The difference can easily be visualized by looking at the lines formed by the slabs within the highlighted portion of the pictures.

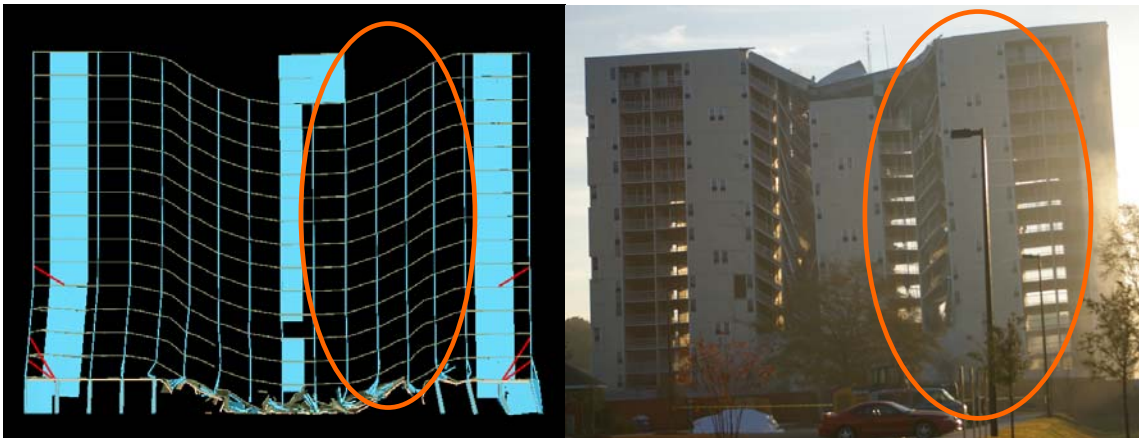


Figure 42. Initial Stubbs Tower Model Inaccuracy Due to Exterior Walls

The second minor inaccuracy in the model was the extent of damage done to the stair towers. The stair towers on each end of the structure are the last elements to collapse, rigidly falling onto the debris field created by the rest of the structure. The model predicts complete destruction of the stair towers, while in reality they remained predominantly intact. Below is a south elevation of the debris field illustrating the difference in behavior predicted by the model.

Figure 43. Initial Stubbs Tower Model Prediction of Debris Field

5.3.6. Revisions for Second Analysis

After the implosion, revisions to the initial Stubbs Tower model were justified through experimental investigation and analytical evaluation.

Post-Implosion Experimental Investigation

The experimental investigation performed after the implosion, which consisted of testing the concrete and welded wire mesh, yielded the following material properties.

Table 9. Post-Implosion Experimental Investigation Results

Concrete	Density	115 pcf
	Compressive Strength	2,540 psi
W.W.M.	Yield Strength	80 ksi
	Ultimate Strength	85 ksi

Material Revisions

Experimental testing discovered that the concrete specimen was semi-lightweight concrete. Further inquisition of the structural drawings showed that all of the components on the 2nd through 15th floor were constructed with semi-lightweight concrete. Thus, a new material was created in the revised model, altering the density to accurately represent the structure. Because the density was changed, Young's modulus was recalculated to account for the density using the equation presented in section 8.5.1 of Building Code Requirements for Structural Concrete and Commentary (ACI Committee 318, 2004). The shear modulus was appropriately revised relative to the Young's modulus.

The compressive strength of the concrete in the model was not changed to match the experimental results because it was a single, local sample greatly contradicting what was specified in the structural drawings. Had the building contained an extensive amount of

concrete that was less than 65% of the specified strength, it could not have passed inspection without some type of documentation.

For the welded wire mesh, the yield strength was adjusted to match the experimental testing. The ultimate strength remained the same because the experimental results exactly matched the initial value used in the model.

The following table lists the properties used to define the concrete and welded wire mesh in the revised Stubbs Tower model. Materials or components not listed did not change.

Table 10. Revised Stubbs Tower Model Material Properties

Concrete	Density	115 pcf
	Compressive Strength	4,750 psi
	Young's Modulus, E	2,800,000 psi
	Shear Modulus, G	840,000 psi
W.W.M.	Yield Strength	80,000 psi
	Ultimate Strength	85,000 psi

Structural Revisions

Two structural revisions were made to the model to more accurately simulate the existing conditions of the structure during the implosion. The first revision was addition of nonstructural exterior cold-formed stud wall panels that were not removed prior to the implosion. An illustration of the walls from inside the structure is shown in the following figure.

Figure 44. Exterior Cold-Formed Stud Wall

To model the cold-formed stud walls, the strength and stiffness was estimated for the wall as a system. Before estimating the system properties, the tensile strength and modulus of elasticity was first referenced for individual studs (The Steel Network, 2007 and Abdel-Rahman and Sivakumaran, 1997). The individual stud properties were then combined to estimate the system properties. To convert the actual system properties into model properties, the strength and stiffness had to be proportioned with respect to areas. The elements representing the wall were solid and did not contain the voids found between the studs. Therefore, the properties of the elements were scaled based on their cross-sectional area versus the cross-sectional area of the cold-formed studs in the wall. A total of 196 walls were added, creating an additional 2,352 elements in the model.

Adding elements to account for the presence of the walls did not completely satisfy the actual conditions. Estimating the system properties for the wall accurately models the behavior of the wall itself, but the interface between the wall and the surrounding structure

must be defined to represent the construction methodology. Because the walls are nonstructural infill, there is not a substantial force transmitting connection between them and the surrounding concrete. To model this weak interface, an interface material was defined with properties representing the strength and stiffness of the connecting elements, steel screws. The following table provides the material properties used to represent the cold-formed stud wall and the interface between it and the concrete.

Table 11. Material Properties for Cold-Formed Walls and Interface Materials

Cold-formed Wall	Wall Weight	5 psf
	Compressive Strength	1.0 ksi
	Tensile Strength	0.15 ksi
	Young's Modulus, E	60 ksi
	Shear Modulus, G	25 ksi
Interface Material	Compressive Strength	1.0 ksi
	Tensile Strength	0.10 ksi
	Young's Modulus, E	65 ksi
	Shear Modulus, G	27 ksi

The second structural revision to the initial model was the addition of concrete stair tower landings between each floor. The initial model contained the steel beams spanning between the walls, but not the concrete topping slab. Considering the extent of destruction to the stair towers in the model, the stair tower landings were added in attempts to provide more continuity and strength between the walls. It is likely that steel stairs themselves provided a substantial amount of strength and continuity within the tower; however, it is not realistic to attempt to include them in the model.

Preprocessing and Demolition Scenario Revisions

With addition of the exterior cold-formed walls, the structure was predicted to be more resistant to collapse. Therefore, to adjust for the structure's improved ability to inhibit

collapse, the delay timing modification factor was removed. Without the delay timing modification factor of 1.5, the removals were 50% slower than in the initial model. Belief was that the exterior walls would sufficiently resist collapse to prevent out running the delay timing.

Also, by inputting the exact delay timing utilized in the actual event, the revised model consists of the most precise information known about the implosion. Thus, this revision contains the most accurate input possible in all aspects.

5.3.7. Revisions for Final Analysis

Evaluating results from the first revision of the Stubbs Tower model established a bearing point for comparison because all levels of its input closely matched the actual conditions and exact loading for the structure. Some inaccuracies were identified within the revised model output and warranted further revision and analysis. Therefore, a second revision and final analysis was performed on the Stubbs Tower model.

In the second revision, the method in which the cold-formed stud walls were input into the model was changed. In the first revision, the cold-formed stud walls were created simultaneously with the structural members before the self-weight was applied. However, because they are flush in-filled walls, when the self-weight was applied to the structure, the cold-formed walls that were supposed to be nonstructural ended up taking load. Thus, for the final revision, a new loading stage was created. A second static loading stage was created in which the nonstructural walls were added to the model after all of the structural members were input. This allowed the structural members to settle due to their self-weight without loading the nonstructural walls.

The only other alteration was re-implementation of the delay timing modification factor. Some excessive discontinuity was witnessed in the first revision, thus a factor of 1.2 was used in the second revision.

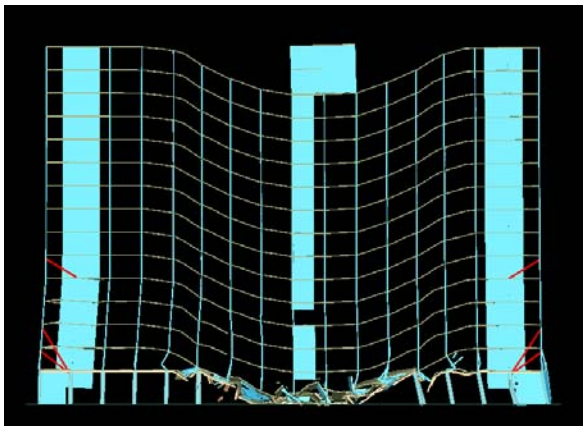
5.3.8. Model Evaluations

Parallel evaluation of all three models against the actual documentation emphasizes the differences in behavior between the models, as well as, the impact caused by some of the modeling decisions. Following the categories and criteria contained in the evaluation matrix, the relative accuracies of each model can be identified.

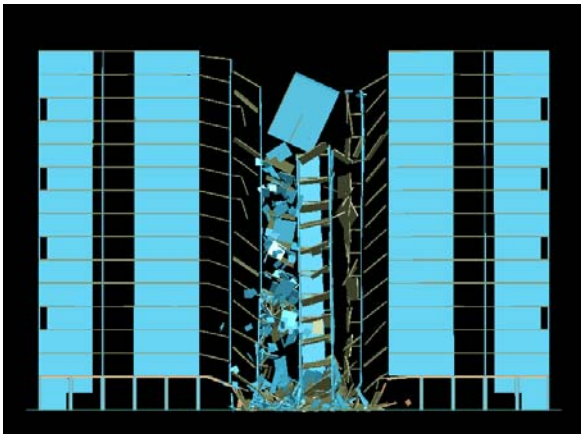
Construction of each model varied slightly. Improvements were made in each analysis through modifying the material properties and considering what components were included in the model. Other than inclusion of the nonstructural walls, there were no distinguishable differences. With regards to *Preprocessing*, or the demolition scenario, the models had recognizable and quantifiable differences. Through inclusion of the delay timing modification factor, the model was altered from an exact representation of the event into an equivalent representation considering the element removal method. The delay timing modification factor was the only difference between the models in the *Preprocessing* category, otherwise the removal groups and order of progression was identical. Although the delay timing modification factor can be identified at this stage of the evaluation, the influence of it cannot yet be visualized. The factor's effect on the collapse behavior becomes evident in the subsequent evaluation of member behavior and global collapse progression.

In evaluating the collapse behavior of the models side-by-side, the inconsistencies are transparent. The most notable difference resulted from the addition of the nonstructural cold-

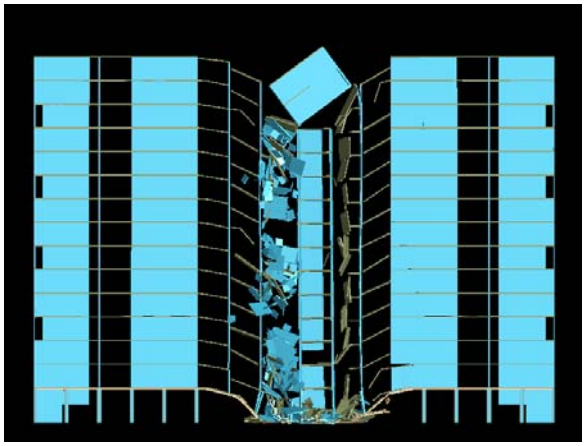
formed stud walls in the first revision. Addition of the nonstructural wall panels greatly stiffened the bays that contained them when subject to shear distortion and inhibited collapse of the structure. The resistance provided by the nonstructural walls worked collectively with the slower, actual delay timing sequence. The following pictures, taken from the western side just prior to the seventh removal group, illustrate the differences in behavior.



a.) Initial trial



b.) First revision



c.) Second revision

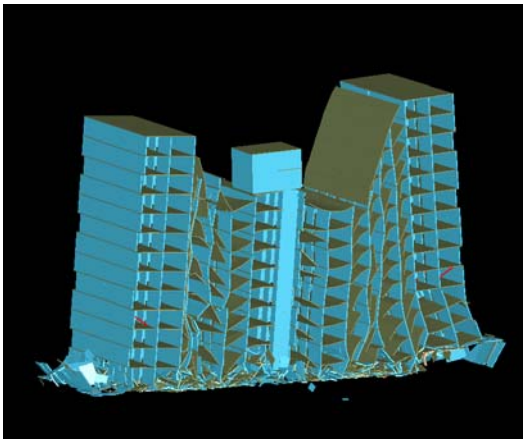
d.) Actual documentation

Figure 45. Effect of Nonstructural Wall Panels in Stubbs Tower Models

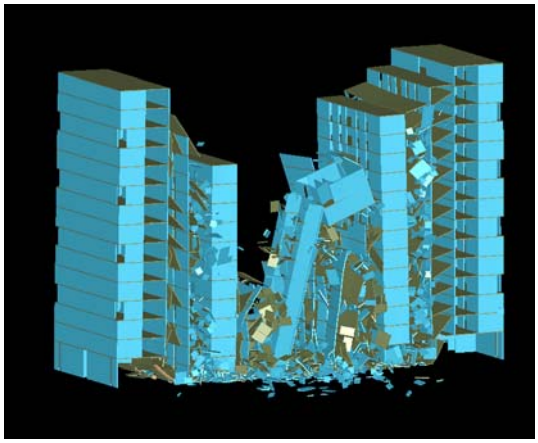
The pictures clearly show that the nonstructural walls play a significant role in allowing the structural system to resist the lateral forces transferred through the slabs. In the initial trial, at the instant shown, the catenary forces have pulled the exterior transverse walls

inward at the top. The lack of resistance has allowed for the slabs to remain continuous and sustain their tensile forces. However, in the models containing the exterior panels, the resistance to the lateral forces has caused the slabs to fail. In the second revision, where staged construction was used for the nonstructural walls, the lack of initial load allowed the panels to provide slightly more resistance than in the first revision. This resistance by the nonstructural walls and hinging of the slabs is seen in the picture documenting the actual implosion.

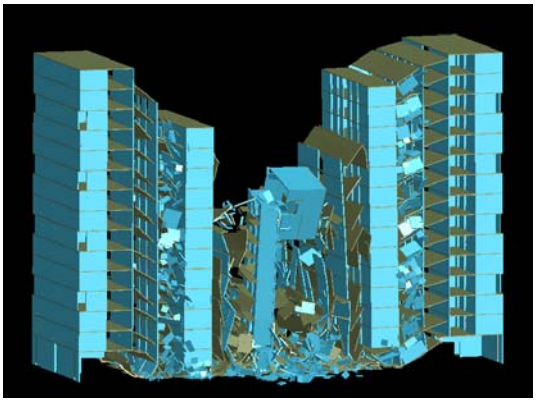
Although addition of the exterior panels had a positive effect on the local member behavior during collapse, they had an adverse effect from the global collapse progression aspect. The increased stiffness, which led to premature separation of the slabs and a decrease in the lateral pull, caused the stair towers to collapse in a different fashion than documented. In the actual event, the slabs fail at the transverse walls, but still transmit tensile forces through the reinforcement, causing the north stair tower to collapse first with the south tower directly following it. However, in the first and second revision the south tower collapses well before the north tower does. The following figure shows the regional and global collapse behavior of the various models and the actual event approximately halfway through collapse.



a.) Initial trial



b.) First revision



c.) Second revision

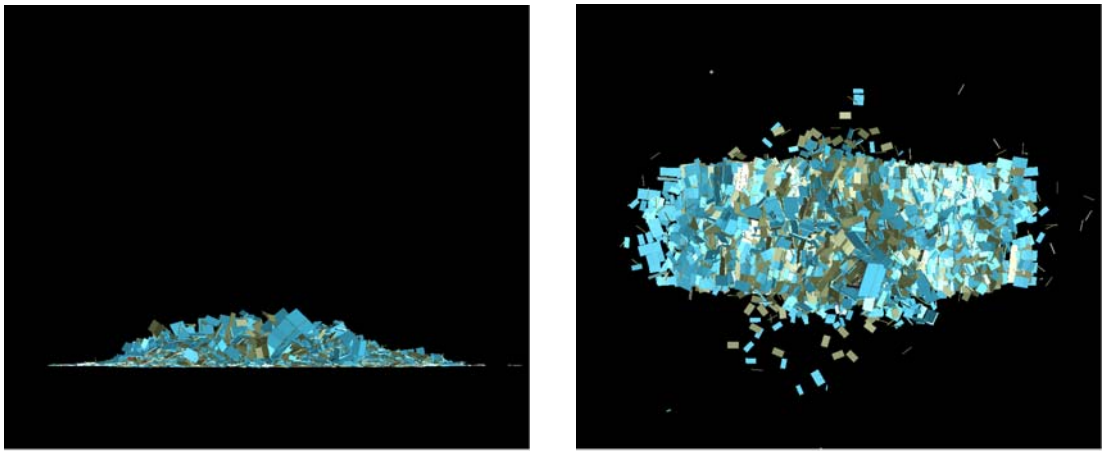
d.) Actual documentation

Figure 46. Comparison of Collapse Behavior in Stubbs Tower Models

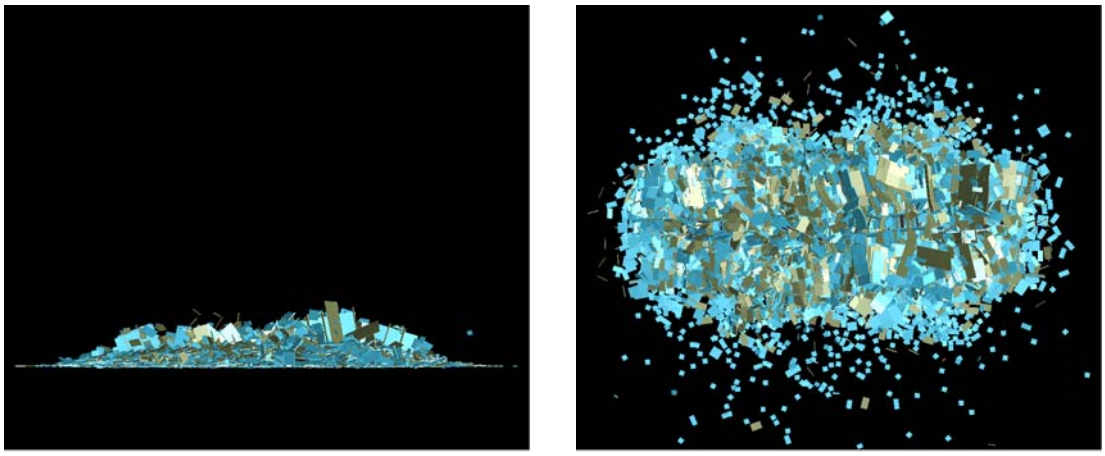
Logically, it makes sense for the south side to collapse first because there are fewer bays on that side of the elevator core. But, the structure does not fail in that sequence, most likely because having more bays allows for greater generation of catenary forces throughout the collapse. If this is true, the inaccuracy in the model could be a result of the failure criteria prematurely removing the matrix springs within the slabs, not allowing the catenary forces to fully develop.

Beyond the progression of collapse, in analyzing the debris field resulting from each model, the level of continuity once again played a significant role. In the initial model,

which sustained continuity for the longest period during the collapse, the debris field was the most consolidated. The revised models, partly due to the nonstructural wall elements failing and dropping outward and in part due to the loss of continuity resulting from influence of the nonstructural walls, had a noticeably more widespread debris field. The illustrations below show an eastern side elevation and plan view of the resulting debris field from the initial model and first revision. The second revision is not shown because it nearly replicates the debris field seen in the first revision.



a.) Initial trial debris field elevation and plan



b.) First revision debris field elevation and plan (second revision approximately the same)

Figure 47. Comparison of Debris Field in Stubbs Tower Models

As in the initial model, neither of the revised models was able to accurately predict the degree of damage done to the stair towers. The results from the revised models showed a slight increase in the amount of the stair towers that remained in tact. However, overall the models still predicted for the most part, complete failure and destruction of all major structural components.

Complete evaluations for the initial model, first revision, and second revision can be found in the appendices.

5.3.9. Observations

Iterations of the Stubbs Tower model directly illustrated the effect of nonstructural elements during collapse. The nonstructural elements, in this case exterior cold-formed steel walls, had a significant impact in the timing and progression of collapse. Although addition of the nonstructural elements caused some global inaccuracies, their presence is critical to understanding the system behavior.

Parallel to the incorporation of nonstructural elements in the model, is how they are input into the model. Staged construction of the model is necessary to correctly represent the conditions and stress present in the actual structure. Without staged construction, the nonstructural elements initially contain stress, thus increasing the structural stiffness, but leading to their early failure.

The demolition scenario, combined with the structural system, presented the need for a delay timing modification factor. Due to the amount of material removed in the elevator core of Stubbs Tower, the delay timing modification factor was necessary to prevent the structural collapse from outrunning the rest of the element removal on the first floor.

Development of the factor's value is directly related to the amount of material removed in a single load path and the structural system's stiffness. Thus, determination of the value can be dependent on the influence of nonstructural walls, as was the case between the initial model and the first revision.

Preparatory work performed by the demolition contractor was effectively modeled using techniques established by the ASI technical support team. Sawcutting can easily be modeled, providing an accurate representation of modifications made to structures. In addition, the use of link members to represent the steel cables worked well without substantially adding to the element count or analysis time.

6. Summary, Recommendations, and Conclusions

An investigation of the accuracy of ELS® software in modeling progressive collapse was completed through comparison with the responses of two structural implosions. The objective was accomplished through experimental and analytical investigations, which included studying the behavior and documentation of each implosion and comparing that to models generated using the ELS® software. To revise and improve the accuracy of the models, a parametric study was performed which examined the effect of individual modeling techniques and parameters. The iterations of each model were assessed using an evaluation matrix, created to allow for rough quantification of each model's level of accuracy. The resulting recommendations and conclusions are presented herein.

6.1. Modeling Recommendations and Conclusions

The following conclusions are directed towards modeling reinforced concrete structures and their application to other structures should be done on a case-by-case basis if found advantageous.

When modeling reinforced concrete, revisions to the default material parameters for concrete will improve the structural behavior, as well as the numerical stability. The following table presents the revised parameters that were proven beneficial through investigation in the parametric study and subsequent use in the Stubbs Tower model.

Table 12. Concrete Material Parameters

Shear Modulus, G	0.3E
Normal Contact Stiffness Factor	0.0001
Shear Contact Stiffness Factor	0.00001
Rebound (Unloading Stiffness) Factor	3.0

In modeling reinforced concrete, the mesh discretization can have a considerable impact. Greatly varying the size of adjacent elements can produce inaccuracies within the behavior. Defining all elements to be approximately the same size is ideal, but not always realistic. Consideration should be given at the beginning of a project as to how fine the mesh can be, accounting for the amount of material being modeled and the computing requirements.

At a minimum though, if the entire mesh cannot be refined, defining slab elements to be the same size as the structural elements above and below them will prevent premature failure and discontinuity in the middle of the bays during collapse. The necessity for this was witnessed in the first case study. However in the second case study, where this modeling technique was used, the models exhibited behavior in which the slabs hinged and failed at the joints, as evidenced in reality.

For some reason, if the mesh cannot be sufficiently refined, utilizing column or vertical element priority will produce the most realistic collapse simulation. Caution should be given to local effects resulting from utilizing column priority because it can instill behavior different from the same model with slab priority.

Beyond the modeling conclusions previously stated, which are applicable to all reinforced concrete structures, some considerations are structure dependent and not always applicable. The first of these considerations is whether or not immaculate element removal is a reasonable estimate for the actual event being modeled. Depending on the demolition scenario, if the instantaneous removal is substantial enough, the structure will "over-accelerate", necessitating the use of a delay timing modification factor. The first case study did not require a modification factor because the amount of material removed in each column

did not create a significant enough of a void for the structure to "over-accelerate". In the second case study though, where thirty foot of material was removed in the elevator core, a modification factor was necessary.

A delay timing modification factor is necessary when a significant portion of load-bearing material is removed from a single vertical load path. The value of the modification factor required is dependent upon the amount of material removed and the structural system. The greater the percentage of the load path removed, the greater the factor will need to be. Similarly, the greater the flexibility of the structural system, the greater the modification factor will need to be. A relatively stiff system will not allow as quick acceleration in response to the removed material and therefore will not require the same level of modification.

Also influencing the necessity for a delay timing modification factor is the inclusion of nonstructural elements in the model. In creating the model, nonstructural elements should be considered as to whether they will impact the behavior. If their impact is negligible then they should be ignored to keep the model as efficient as possible. To determine how consequential the nonstructural elements will be, the demolition scenario and direction of loading must be investigated. For example, in the second case study, had the demolition scenario progressed across the structure in the transverse direction, the panels would not have had an impact because they would have been loaded in tension and they would have failed at the interface, pulling away from the concrete. But, because the demolition scenario progressed longitudinally, they were loaded in shear, thus providing substantial strength.

If nonstructural elements are included, it is vital to get an accurate estimate of their properties in the direction of loading. For some nonstructural elements, as in the cold-formed

stud walls seen in this research, the strength and stiffness is dependent upon the direction or type of loading. Special attention can be given to documenting nonstructural elements on a site visit if their role in the model can be forecasted.

Again, it is important to state that these conclusions only stem from observations made on the two case studies presented within this research. Therefore, it is possible that these modeling techniques and parameters will not be the most advantageous for every reinforced concrete structure. Each structure should be thoroughly studied to determine exactly what is happening within the system during the abnormal loading. The most beneficial thing in modeling is performing iterations. By varying parameters, comparing the results, and using sound engineering judgment, the correct or most realistic behavior can be identified and analyzed.

6.2. ELS® Conclusions

This research has shown that given reliable input and an accurate demolition scenario, ELS® is capable of accurately simulating the collapse behavior for reinforced concrete structures. The results show that local, regional, and global collapse behaviors, as well as the debris fields, were modeled with substantial accuracy.

However, one main concern evolving from this research is the failure criteria for reinforced concrete. Because ELS® severs the reinforcement between elements once the separation strain for concrete is reached, it greatly hinders the material's ductility. This was evident in both case studies where premature removal of the reinforcement caused a loss of continuity in the structural system during some stage of the collapse.

For progressive collapse analysis, this issue must be addressed because continuity is what enables catenary forces to develop. These catenary forces, primarily in floor slabs, are a major contributor to the nonlinear strength in a reinforced concrete structure, possibly inhibiting collapse.

This inaccuracy did not have an extensive impact on the two case studies, mainly evidenced by slab failures in the initial Crabtree Sheraton Hotel model and stair tower destruction in all of the Stubbs Tower models. However, in the future if a structure is to be analyzed with only local failure to predict if progressive collapse will initiate, this behavior will have a vital impact in determining the level of collapse mitigation. Ignoring the effect of the failure criteria on local structural behavior will result in overly conservative design and inaccurate analysis. This issue must be resolved before consideration can be given to using ELS® to design reinforced concrete structures with the alternate load path method.

6.3. Future Research

Specifically, within ELS® the two topics that should be addressed in future research are the reinforced concrete failure criteria and the form of element removal for demolition scenarios. The reinforced concrete failure criteria will need to be investigated from a program definition standpoint to see if it can be improved to more accurately address the issues previously discussed.

The form of immaculate element removal for reinforced concrete structures could be investigated with the goal of eliminating usage of the delay timing modification factor. The removal technique could be analyzed and modified to better represent the existing conditions of a column removed during an implosion. Detonation of the dynamite only displaces the

concrete and buckles the reinforcement. Thus, the material above the blast is not completely free to accelerate downward without interference. The buckled cage and crushed pieces of entrapped concrete remain, dampening acceleration of the structure. If the dynamics of this process can be approximated, then the delay timing modification factor will not be necessary.

As far as usage of this removal technique in conjunction with the alternate load path method, it will be sufficient because only one member is removed, unlike a demolition scenario where successive members are removed from the same load path. Immaculate removal is the method actually prescribed by most guidelines because of the analysis time that it saves.

For demolition scenarios though, there are several possibilities for modeling a blasted column that could improve upon the immaculate removal technique. One option would be to input a link member spanning the distance of the removed material so that once the elements are removed, the link member remains. By estimating and defining the link member properties, the buckling of the reinforcement and damping due to failed material may be able to be modeled. Another option could be to refine the concrete mesh in the blast area and only remove the interior elements. This would create four small columns that would buckle or crush under the axial load from above.

A preliminary study was performed to estimate the feasibility of using these methods. To compare the methods, an ELS® model with six structurally identical columns was created, as can be seen below.

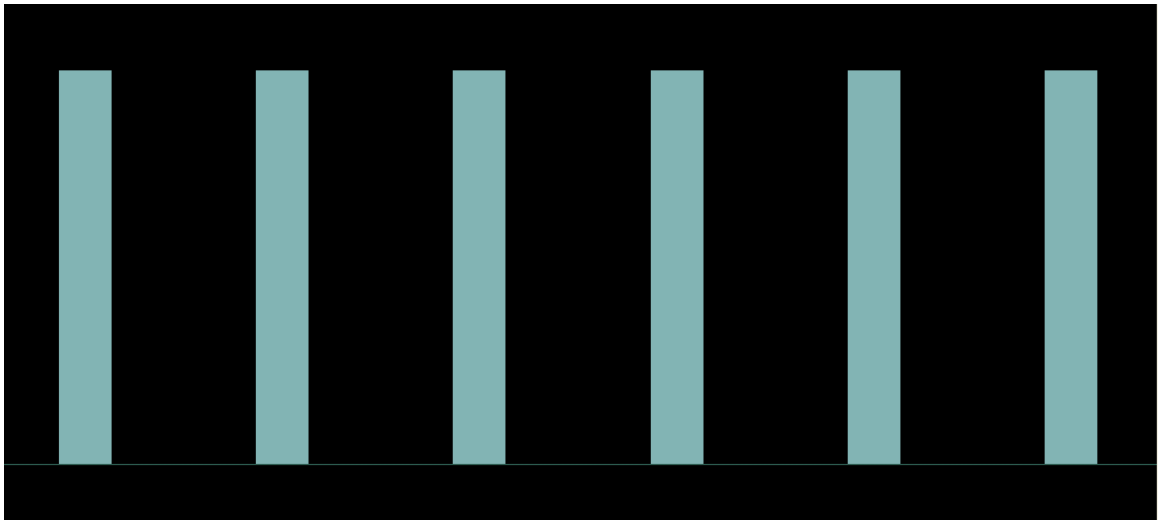


Figure 48. Preliminary Element Removal Research Model

Modifications were made to each one to create six unique columns with respect to the element removal scenario. Numbered from left to right, the first column was the control column, with no material removed. The second column utilized the same immaculate element removal technique used in both case studies. The third, fourth and fifth columns used the immaculate element removal technique as well, but contained link members that spanned the removed elements. The link members were grade 36 steel with areas of 1 in^2, 2 in^2, and 4 in^2, respectively. Lastly, the sixth column contained a refined mesh in the section of the column were the blast would occur. Therefore, instead of removing all of the elements, only the interior elements were removed, creating four exterior corner columns. An axial load of 144 kips was applied to the top element in each column. The illustration below shows the columns at the time step in which the element removal takes place.

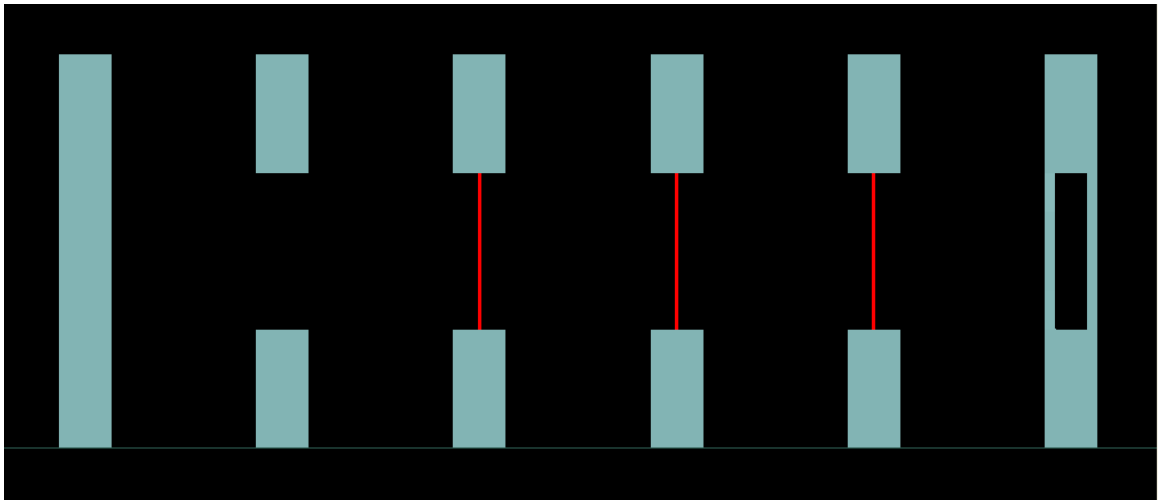


Figure 49. Columns Directly Following Element Removal

The purpose of this preliminary study was to see how different modeling techniques affected the element acceleration and dynamics of a structure. The following illustration shows the columns approximately four-tenths of a second later.

Figure 50. Acceleration of Columns due to Element Removal

The figure clearly indicates that the acceleration can be controlled through the use of different modeling techniques. Utilizing link members is the most practical and efficient method. By specifying the area and strength, the time in which it takes the link member to

fail can be controlled. In the illustration, the link members within the third and fourth columns have surpassed yield and ultimate, thus the software has removed them. However, variance in height between the top portions of the columns, relative to the second column, clearly shows how the area affected the acceleration upon removal.

The sixth column also performed well, but it required much more time to input and created many more elements, thus it is not recommended for an entire structure where this procedure would be used at several locations.

Completion of this preliminary study shows that it is possible to control the downward acceleration of a structure after immaculate element removal. For proper use though, some type of experimental research is necessary to estimate the residual strength and actual acceleration in a reinforced concrete column following a blast. Using well-protected accelerometers, the data required to calibrate this modeling technique should be obtainable. However, there will be substantial cost because it is likely that the accelerometers would not be salvageable.

Parallel to this possible future research focusing on immaculate element removal, ELS® would benefit from further calibration through modeling a structures response to selective demolition. This research would be similar to other academic research currently being done where prior to a full-scale implosion, a potential progressive collapse scenario is analyzed locally on the structure. This research, through both case studies, has proven that ELS® can simulate scenarios when collapse is imminent, but the question remains can it accurately determine if and when collapse will initiate and to what extent it will progress.

6.4. Potential Impact and Resulting Overview

Complete and thorough validation of the ELS® software through extensive research and case studies would create an invaluable tool for structural engineers. With the growing threats placed on today's structures, extreme or abnormal loadings will eventually become a standard part of design for moderate to high-risk structures. Because of the highly nonlinear behavior that occurs as a result of these abnormal loadings, it is not as straightforward to design for as traditional gravity or wind loads. Even earthquake loads are more straightforward than blast loads or impact loads.

The economic and life-safety liability that is inherited by the size and usage of today's structures make progressive collapse an even more substantial concern. With the succession of each progressive collapse event in the United States, the economic cost and death toll has risen. Forty years ago, prior to the Ronan Point collapse, progressive collapse mitigation could not be economically justified in structural design cost. But, now with the magnitude of structures that are being designed and the cost of construction, design to inhibit disproportionate progressive collapse failure is becoming a reasonable consideration. It will never be practical to consider designing to prevent any local damage because with man-made threats the loading can always be increased to overcome the design. The practicality emerges when engineers use sound judgment to determine what level of collapse is acceptable for a given loading. In the future, as it is to a certain extent now, this allowable extent of collapse will be quantified in design codes. That is when ELS® will have an opportunity to make an unquestionable impact in the structural engineering community. At that point, providing engineers with a tool that they can utilize for progressive collapse design, analysis, and investigation would be the ultimate accomplishment.

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8. Appendix

Contained within the appendices is a completed evaluation matrix for each analysis performed in this research. The evaluation for each model consists of the *Model Construction*, *Preprocessing*, *Postprocessing*, and *Visual Inspection* categories, each on an independent page. Order of the evaluations are as follows, the initial Crabtree Sheraton model, the revised Crabtree Sheraton model, the initial Stubbs Tower model, the first revision for Stubbs Tower, and the second or final revision for Stubbs Tower.

Initial Crabtree Sheraton Model

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
CONSTRUCTION	**Dimensions**								
	Height	128'	128'	Y					●
	Plan	58' x 117'	58' x 117'	Y					●
	System and Components	As existing conditions	Existing conditions	Y					●
	Structural Member Sizes								
	Columns	Exact	Provided by DHG Wrecking Co.	Y					●
	Beams	Estimated	Unknown	Y			●		
	Slabs	Estimated	Unknown	Y			●		
	Walls	Oversized to lower number of elements	8" CMU	Y				●	
	Reinforcement Details								
	Columns	Exact	Provided by DHG Wrecking Co.	Y					●
	Beams	Estimated using ACI & CRSI	Unknown	Y			●		
	Slabs	Approximate	Most aspects given in drawings	Y				●	
	Walls	No reinforcement	Unknown	Y				●	
	Material Properties								
	Concrete	Estimated	Unknown	Y			●		
	Structural Steel	Estimated	Unknown	Y			●		
	Reinforcement	Estimated	Unknown	Y			●		
	Tension-only Elements			N					
				Total	0	0	6	3	5

Weighted Average	73.2%

Initial Crabtree Sheraton Model (Cont.)

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
PREPROCESSING	**Removal Groups (Delays)**								
	Number of Steps	13	13	Y					●
	Delay Timing	0.48 sec	~ 0.5 sec	Y					●
	Order of Progression								
	Delay Flow Through Structure	Exact	Sequence provided by Demolition Dynamics	Y					●
	Element Removal								
	Blast			N					
	Complete Removal	Removal of concrete and reinforcement	Only concrete is removed upon detonation	Y				●	
				Total	0	0	0	1	3

Weighted Average	93.8%

Initial Crabtree Sheraton Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
POSTPROCESSING (STRUCTURAL MEMBER BEHAVIOR)	**Columns**								
	Failure?	Some shear and separate during collapse	Mostly continuous throughout collapse	Y			●		
	Location of Failure	At column ends		Y	●				
	Time of Failure	~ 3.5 sec		Y	●				
	Beams								
	Failure?	Separate from slab and shear	No failure seen before dust cloud	Y				●	
	Location of Failure	Near end of beam		Y				●	
	Time of Failure	~ 7 sec		Y				●	
	Slabs								
	Failure?	Widespread failure	Continuous, hinges at columns	Y			●		
	Location of Failure	Middle of bays and adjacent to columns		Y			●		
	Time of Failure	~ 3 sec		Y		●			
	Walls								
	Failure?	No discernable difference	No failure seen before dust cloud	Y					●
	Location of Failure			Y					●
	Time of Failure			Y					●
				Total	2	1	3	3	3

Weighted Average	58.3%

Initial Crabtree Sheraton Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
VISUAL INSPECTION	**Initial Movements**								
	First Movement	Drop in stair tower at ~ 2 sec	Drop in stair tower at ~ 2 sec	Y					●
	Regional Behavior								
	Stair Tower	Drops then pulls inward	Drops then pulls inward	Y					●
	Elevator Tower	Pulled over to the left	Pulled over to the left	Y					●
	Building Façade	Pulled to the left, buckles above 1st floor	Pulled to the left, buckles above the 1st floor	Y					●
	Bay/Floor Cells	Separate during collapse	Columns remain vertical, slabs hinge and pull downward	Y			●		
	Global Behavior								
	Overall Collapse	Flow slightly broken by floor slabs breaking up	Flows from front left to back right, pulling exterior wall over on top	Y				●	
	Debris Field								
	Footprint/Height	90' x 140' / 20', falls almost across the original footprint	Approximately the same, birms placed to hold debris in pile	Y					●
	Consolidation	Very good	Very good	Y					●
	Large Projectiles Off of Pile	None	None	Y					●
				Total	0	0	1	1	7

Weighted Average	91.7%

CONSTRUCTION	73.2%
PREPROCESSING	93.8%
POSTPROCESSING	58.3%
VISUAL INSPECTION	91.7%
Total	79.2%

Revised Crabtree Sheraton Model

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
CONSTRUCTION	**Dimensions**								
	Height	128'	128'	Y					●
	Plan	58' x 117'	58' x 117'	Y					●
	System and Components	As existing conditions	Existing conditions	Y					●
	Structural Member Sizes								
	Columns	Exact	Provided by DHG Wrecking Co.	Y					●
	Beams	Estimated	Unknown	Y			●		
	Slabs	Estimated	Unknown	Y			●		
	Walls	Oversized to lower number of elements	8" CMU	Y				●	
	Reinforcement Details								
	Columns	Exact	Provided by DHG Wrecking Co.	Y					●
	Beams	Estimated using ACI & CRSI	Unknown	Y			●		
	Slabs	Approximate	Most aspects given in drawings	Y				●	
	Walls	No reinforcement	Unknown	Y				●	
	Material Properties								
	Concrete	Estimated	Unknown	Y			●		
	Structural Steel	Estimated	Unknown	Y			●		
	Reinforcement	Estimated	Unknown	Y			●		
	Tension-only Elements			N					
				Total	0	0	6	3	5

Weighted Average	73.2%

Revised Crabtree Sheraton Model (Cont.)

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
PREPROCESSING	**Removal Groups (Delays)**								
	Number of Steps	13	13	Y					●
	Delay Timing	0.48 sec	~ 0.5 sec	Y					●
	Order of Progression								
	Delay Flow Through Structure	Exact	Sequence provided by Demolition Dynamics	Y					●
	Element Removal								
	Blast			N					
	Complete Removal	Removal of concrete and reinforcement	Only concrete is removed upon detonation	Y				●	
				Total	0	0	0	1	3

Weighted Average	93.8%

Revised Crabtree Sheraton Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
POSTPROCESSING (STRUCTURAL MEMBER BEHAVIOR)	**Columns**								
	Failure?	Some experience punching shear throughout full height	Mostly continuous throughout collapse	Y				●	
	Location of Failure	At each floor slab		Y		●			
	Time of Failure	Throughout collapse		Y	●				
	Beams								
	Failure?	Separate from slab	No failure seen before dust cloud	Y				●	
	Location of Failure	Along length of beam		Y			●		
	Time of Failure	~ 7 sec		Y				●	
	Slabs								
	Failure?	Across middle of structure	Continuous, hinges at columns	Y				●	
	Location of Failure	Middle of bays and adjacent to the columns		Y			●		
	Time of Failure	~ 4 sec		Y			●		
	Walls								
	Failure?	Stair tower wall fails before impact	No failure seen before dust cloud	Y				●	
	Location of Failure	Stair tower		Y				●	
	Time of Failure	~ 7 sec		Y				●	
				Total	1	1	3	7	0

Weighted Average	58.3%

Revised Crabtree Sheraton Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
VISUAL INSPECTION	**Initial Movements**								
	First Movement	Drop in stair tower at ~ 2 sec	Drop in stair tower at ~ 2 sec	Y					●
	Regional Behavior								
	Stair Tower	Drops then pulls inward	Drops then pulls inward	Y					●
	Elevator Tower	Pulled over to the left	Pulled over to the left	Y					●
	Building Façade	Pulled to the left, buckles above 1st floor	Pulled to the left, buckles above 1st floor	Y					●
	Bay/Floor Cells	Separate during collapse, but later than in initial model	Columns remain vertical, slabs hinge and pull downward	Y				●	
	Global Behavior								
	Overall Collapse	Slabs hold together better, punching shear of columns, but good flow	Flows from front left to back right, pulling exterior wall over on top	Y				●	
	Debris Field								
	Footprint/Height	105' x 145' / 20' - comes further out in transverse direction	Approximately the same, birms placed to hold debris in pile	Y					●
	Consolidation	Very good	Very good	Y					●
	Large Projectiles Off of Pile	None	None	Y					●
				Total	0	0	0	2	7

Weighted Average	94.4%

CONSTRUCTION	73.2%
PREPROCESSING	93.8%
POSTPROCESSING	58.3%
VISUAL INSPECTION	94.4%
Total	79.9%

Initial Stubbs Tower Model

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
CONSTRUCTION	**Dimensions & Structure**								
	Height	148 '	148 '	Y					●
	Plan	61' x 185'	61' x 185'	Y					●
	System and Components	No stair tower landings, no exterior non-structural walls	Existing conditions	Y				●	
	Structural Member Sizes								
	Columns	Exact	Given in construction drawings	Y					●
	Beams	Exact	Given in construction drawings	Y					●
	Slabs	Exact	Given in construction drawings	Y					●
	Walls	Exact	Given in construction drawings	Y					●
	Reinforcement Details								
	Columns	Exact	Given in construction drawings	Y					●
	Beams	Approximate	Partially given in construction drawings	Y				●	
	Slabs	Exact	Given in construction drawings	Y					●
	Walls	Exact	Given in construction drawings	Y					●
	Material Properties								
	Concrete	Estimated from strength specified in drawings	28-day strength specified in drawings	Y				●	
	Structural Steel	As specified	Specified in drawings	Y					●
	Reinforcement	As specified	Specified in drawings	Y					●
	Tension-only Elements	Approximate	3/4" steel cables used during implosion	Y				●	
				Total	0	0	0	3	11

Weighted Average	94.6%

Initial Stubbs Tower Model (Cont.)

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
PREPROCESSING	**Removal Groups (Delays)**								
	Number of Steps	12, no shortening	11	Y				●	
	Delay Timing	Modification factor of 1.5	Provided by Demolition Dynamics, verified at 0.50 sec	Y				●	
	Order of Progression								
	Delay Flow Through Structure	Exact	Sequence provided by Demolition Dynamics	Y					●
	Element Removal								
	Blast			N					
	Complete Removal	Removal of concrete and reinforcement	Only concrete is removed upon detonation	Y				●	
				Total	0	0	0	3	1

Weighted Average	81.3%

Initial Stubbs Tower Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
POSTPROCESSING (STRUCTURAL MEMBER BEHAVIOR)	**Interior Transverse Walls**								
	Failure?	Remain intact and vertical throughout collapse	Remain intact and vertical throughout collapse	Y					●
	Location of Failure			Y					●
	Time of Failure			Y					●
	Exterior Transverse Walls								
	Failure?	South wall fails	Remains intact till atleast 20° over	Y				●	
	Location of Failure	~ 13 feet up, at top of stair tower wall on first floor	-	Y		●			
	Time of Failure	1.6 sec (2.4 sec factored)	~ 4 sec	Y		●			
	Exterior Longitudinal Walls								
	Failure?	Not included in initial model	Cause shear rigidity and fail by buckling out of the building	Y	●				
	Location of Failure			N					
	Time of Failure			N					
	Slabs								
	Failure?	Yes	Yes	Y					●
	Location of Failure	Adjacent to transverse walls	Adjacent to transverse walls	Y					●
	Time of Failure	1.7 sec (2.6 sec factored)	~ 1.6 sec (in the two bays without panels north of elevator core)	Y			●		
				Total	1	2	1	1	5

Weighted Average	67.5%

Initial Stubbs Tower Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
VISUAL INSPECTION	**Initial Movements**								
	First Movement	30' drop in center of building, penthouse tilts to the north	30' drop in center of building, penthouse tilts to the north	Y					●
	Regional Behavior								
	Stair Towers / Outerwalls	Stair tower and two adjacent bays rigidly fall, north side first	Stair tower and two adjacent bays rigidly fall, north side first	Y					●
	Elevator Tower	Drops down, tilts slight to northeast	Drops down, almost exactly vertical	Y				●	
	Penthouse	Rigidly rolls off elevator tower to the south upon impact	Rigidly rolls off elevator tower to the south upon impact	Y					●
	Bay/Floor Cells	Continuous, hinge at all floors and bays	Hinge only at bays with no exterior panels stiff where panels are	Y			●		
	Global Behavior								
	Overall Collapse	Core drops, penthouse rolls off, north side collapses before south	Core drops, penthouse rolls off, north side collapses before south	Y				●	
	Debris Field								
	Footprint/Height	80' x 205' / 21', pile spreads out slightly on eastern side	Approximately same footprint, height ~ 25' excluding rigid stair towers	Y					●
	Consolidation	Very good, dense pile, stair towers broken up	Very good, dense pile consisting of layers of walls and slabs.	Y				●	
	Large Projectiles Off of Pile	None	None	Y					●
				Total	0	0	1	3	5

Weighted Average	86.1%

CONSTRUCTION	94.6%
PREPROCESSING	81.3%
POSTPROCESSING	67.5%
VISUAL INSPECTION	86.1%
Total	82.4%

First Revision Stubbs Tower Model

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
CONSTRUCTION	**Dimensions & Structure**								
	Height	148 '	148 '	Y					●
	Plan	61' x 185'	61' x 185'	Y					●
	System and Components	Matches existing conditions	Existing conditions	Y					●
	Structural Member Sizes								
	Columns	Exact	Given in construction drawings	Y					●
	Beams	Exact	Given in construction drawings	Y					●
	Slabs	Exact	Given in construction drawings	Y					●
	Walls	Exact	Given in construction drawings	Y					●
	Reinforcement Details								
	Columns	Exact	Given in construction drawings	Y					●
	Beams	Approximate	Partially given in construction drawings	Y				●	
	Slabs	Exact	Given in construction drawings	Y					●
	Walls	Exact	Given in construction drawings	Y					●
	Material Properties								
	Concrete	Approximated from specified strength and lab tests	28-day strength specified in drawings	Y				●	
	Structural Steel	As specified	Specified in drawings	Y					●
	Reinforcement	Verified through lab tests	Specified in drawings	Y					●
	Tension-only Elements	Approximate	3/4" steel cables used during implosion	Y				●	
				Total	0	0	0	3	12

Weighted Average	95.0%

First Revision Stubbs Tower Model (Cont.)

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
PREPROCESSING	**Removal Groups (Delays)**								
	Number of Steps	12, no shortening	11	Y				●	
	Delay Timing	Exact	Provided by Demolition Dynamics, verified at 0.50 sec	Y					●
	Order of Progression								
	Delay Flow Through Structure	Exact	Sequence provided by Demolition Dynamics	Y					●
	Element Removal								
	Blast			N					
	Complete Removal	Removal of concrete and reinforcement	Only concrete is removed upon detonation	Y				●	
				Total	0	0	0	2	2

Weighted Average	87.5%

First Revision Stubbs Tower Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
POSTPROCESSING (STRUCTURAL MEMBER BEHAVIOR)	**Interior Transverse Walls**								
	Failure?	Remain intact and vertical throughout collapse	Remain intact and vertical throughout collapse	Y					●
	Location of Failure			Y					●
	Time of Failure			Y					●
	Exterior Transverse Walls								
	Failure?	Remain intact longer than initial model, close to 20° before crumbling	Remains intact till atleast 20° over	Y				●	
	Location of Failure	~ 7 feet up, buckles outward	-	Y				●	
	Time of Failure	4.2 sec	~ 4 sec	Y					●
	Exterior Longitudinal Walls								
	Failure?	Buckle outward due to shear distortion of the bays	Cause shear rigidity and fail by buckling out of the building	Y					●
	Location of Failure	Walls in bay adjacent to elevator fail in shear	Walls in bay adjacent to elevator drops rigidly	Y				●	
	Time of Failure	Early in some bays, late failure or no failure in other bays	Continuous failure outward from center as structure collapses	Y				●	
	Slabs								
	Failure?	Yes	Yes	Y					●
	Location of Failure	Adjacent to transverse walls	Adjacent to transverse walls	Y				●	
	Time of Failure	1.7 sec (in the two bays without panels north of elevator core)	~ 1.6 sec (in the two bays without panels north of elevator core)	Y				●	
				Total	0	0	0	6	6

Weighted Average	87.5%

First Revision Stubbs Tower Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
VISUAL INSPECTION	**Initial Movements**								
	First Movement	30' drop in center of building, penthouse tilts to the north	30' drop in center of building, penthouse tilts to the north	Y					●
	Regional Behavior								
	Stair Towers / Outerwalls	Both rigidly fall, south side much before north	Stair tower and two adjacent bays rigidly fall, north side first	Y			●		
	Elevator Tower	Drops down, but falls to the north upon impact	Drops down, almost exactly vertical	Y			●		
	Penthouse	Rigidly rolls off elevator tower to the north, before impact	Rigidly rolls off elevator tower to the south upon impact	Y			●		
	Bay/Floor Cells	Hinges at bays with no panels, mostly stiff where panels are	Hinge only at bays with no exterior panels stiff where panels are	Y				●	
	Global Behavior								
	Overall Collapse	Panels greatly reduce the continuity and flow, disrupts regional behavior	Core drops, penthouse rolls off, north side collapses before south	Y			●		
	Debris Field								
	Footprint/Height	110' x 230' / 21', pile spreads out more due to addition of wall panels	Approximately same footprint, height ~ 25' excluding rigid stair towers	Y					●
	Consolidation	Very good, dense pile, stair towers broken up but recognizable	Very good, dense pile consisting of layers of walls and slabs	Y				●	
	Large Projectiles Off of Pile	None	None	Y					●
				Total	0	0	4	2	3

Weighted Average	72.2%

CONSTRUCTION	95.0%
PREPROCESSING	87.5%
POSTPROCESSING	87.5%
VISUAL INSPECTION	72.2%
Total	85.6%

Final Revision Stubbs Tower Model

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
CONSTRUCTION	**Dimensions & Structure**								
	Height	148 '	148 '	Y					●
	Plan	61' x 185'	61' x 185'	Y					●
	System and Components	Matches existing conditions, uses staged construction	Existing conditions	Y					●
	Structural Member Sizes								
	Columns	Exact	Given in construction drawings	Y					●
	Beams	Exact	Given in construction drawings	Y					●
	Slabs	Exact	Given in construction drawings	Y					●
	Walls	Exact	Given in construction drawings	Y					●
	Reinforcement Details								
	Columns	Exact	Given in construction drawings	Y					●
	Beams	Approximate	Partially given in construction drawings	Y				●	
	Slabs	Exact	Given in construction drawings	Y					●
	Walls	Exact	Given in construction drawings	Y					●
	Material Properties								
	Concrete	Approximated from specified strength and lab tests	28-day strength specified in drawings	Y				●	
	Structural Steel	As specified	Specified in drawings	Y					●
	Reinforcement	Verified through lab tests	Specified in drawings	Y					●
	Tension-only Elements	Approximate	3/4" steel cables used during implosion	Y				●	
				Total	0	0	0	3	12

Weighted Average	95.0%

Final Revision Stubbs Tower Model (Cont.)

Category	Criteria	Model Properties	Actual Properties	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
PREPROCESSING	**Removal Groups (Delays)**								
	Number of Steps	12, no shortening	11	Y				●	
	Delay Timing	Modification factor of 1.2	Provided by Demolition Dynamics, verified at 0.50 sec	Y				●	
	Order of Progression								
	Delay Flow Through Structure	Exact	Sequence provided by Demolition Dynamics	Y					●
	Element Removal								
	Blast			N					
	Complete Removal	Removal of concrete and reinforcement	Only concrete is removed upon detonation	Y				●	
				Total	0	0	0	3	1

Weighted Average	81.3%

Final Revision Stubbs Tower Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
POSTPROCESSING (STRUCTURAL MEMBER BEHAVIOR)	**Interior Transverse Walls**								
	Failure?	Remain intact and vertical throughout collapse	Remain intact and vertical throughout collapse	Y					●
	Location of Failure			Y					●
	Time of Failure			Y					●
	Exterior Transverse Walls								
	Failure?	Remain intact longer than initial model, close to 20^{o} before crumbling	Remains intact till atleast 20^{o} over	Y				●	
	Location of Failure	~ 13 feet up, buckles outward	-	Y				●	
	Time of Failure	4.0 sec (4.8 sec factored)	~ 4 sec	Y				●	
	Exterior Longitudinal Walls								
	Failure?	Buckle outward due to shear distortion of the bays	Cause shear rigidity and fail by buckling out of the building	Y					●
	Location of Failure	Walls in bay adjacent to elevator fail in shear	Walls in bay adjacent to elevator drops rigidly	Y				●	
	Time of Failure	Early in some bays, late failure or no failure in other bays	Continuous failure outward from center as structure collapses	Y				●	
	Slabs								
	Failure?	Yes	Yes	Y					●
	Location of Failure	Adjacent to transverse walls	Adjacent to transverse walls	Y				●	
	Time of Failure	1.4 sec (1.7 sec factored)	~ 1.6 sec (in the two bays without panels north of elevator core)	Y				●	
				Total	0	0	0	7	5

Weighted Average	85.4%

Final Revision Stubbs Tower Model (Cont.)

Category	Criteria	Model Behavior	Actual Behavior	Comparable (Yes or No)	Level of Accuracy				
					0%	25%	50%	75%	100%
VISUAL INSPECTION	**Initial Movements**								
	First Movement	30' drop in center of building, penthouse tilts to the north	30' drop in center of building, penthouse tilts to the north	Y					●
	Regional Behavior								
	Stair Towers / Outerwalls	Both rigidly fall, south side first, but not as early as in first revision	Stair tower and two adjacent bays rigidly fall, north side first	Y			●		
	Elevator Tower	Drops down, but falls to the east upon impact	Drops down, almost exactly vertical	Y			●		
	Penthouse	Rigidly rolls off elevator tower to the north, before impact	Rigidly rolls off elevator tower to the south upon impact	Y			●		
	Bay/Floor Cells	Hinges at bays with no panels, mostly stiff where panels are	Hinge only at bays with no exterior panels stiff where panels are	Y				●	
	Global Behavior								
	Overall Collapse	Panels reduce the continuity and flow, lateral pull not able to develop	Core drops, penthouse rolls off, north side collapses before south	Y			●		
	Debris Field								
	Footprint/Height	120' x 220' / 22', pile spreads out more on sides due to wall panels and	Approximately same footprint, height ~ 25' excluding rigid stair towers	Y					●
	Consolidation	Very good, dense pile, stair towers broken up but recognizable	Very good, dense pile consisting of layers of walls and slabs.	Y				●	
	Large Projectiles Off of Pile	None	None	Y					●
				Total	0	0	4	2	3

Weighted Average	72.2%

CONSTRUCTION	95.0%
PREPROCESSING	81.3%
POSTPROCESSING	85.4%
VISUAL INSPECTION	72.2%
Total	83.5%