Effects of Various Blowout Panel Configurations on the Structural Response of Los Alamos National Laboratory Building 16-340 to Internal Explosions
Edited by Keith G. Pohs of Group IM-1
Photocomposition by Deidre A. Plumlee of Group IM-1

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Photo on Cover: Blowout panel side of Building 340 with test instrumentation in place

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on the Structural Response of Los Alamos National
Laboratory Building 16-340 to Internal Explosions

Jason P. Wilke
This thesis is accepted on the behalf of the 
Faculty of the Institute by the following committee:

[Signatures and titles]

[Signatures and titles]

[Signatures and dates]

Date

I release this document to the New Mexico Institute of Mining and Technology

[Signature and dates]
Effects of Various Blowout Panel Configurations on the Structural Response of Los Alamos National Laboratory Building 16-340 to Internal Explosions

By

Jason P. Wilke

Thesis

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ABSTRACT

The risk of accidental detonation is present whenever any type of high explosives processing activity is performed. These activities are typically carried out indoors to protect processing equipment from the weather and to hide possibly secret processes from view. Often, highly strengthened reinforced concrete buildings are employed to house these activities. These buildings may incorporate several design features, including the use of lightweight frangible blowout panels, to help mitigate blast effects. These panels are used to construct walls that are durable enough to withstand the weather, but are of minimal weight to provide overpressure relief by quickly moving outwards and creating a vent area during an accidental explosion.

In this study the behavior of blowout panels under various blast loading conditions was examined. External loadings from explosions occurring in nearby rooms were of primary interest. Several reinforcement systems were designed to help blowout panels resist failure from external blast loads while still allowing them to function as vents when subjected to internal explosions. The reinforcements were studied using two analytical techniques, yield-line analysis and modal analysis, and the hydrocode AUTODYN.

A blowout panel reinforcement design was created that could prevent panels from being blown inward by external explosions. This design was found to increase the internal loading of the building by 20%, as compared with nonreinforced panels. Nonreinforced panels were found to increase the structural loads by 80% when compared to an open wall at the panel location.
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LIST OF SYMBOLS

\( a \) – speed of sound in air, length of a slab or panel

\( b \) – empirically determined decay constant for air shocks, width of a slab or panel

\( c \) – speed of sound in solids

\( h \) – plate or slab thickness

\( c_o \) – bulk sound speed

\( k \) – spring constant

\( m \) – mass

\( p \) – pressure

\( p_o \) – initial ambient air pressure

\( p_r \) – reflected shock overpressure

\( p_s \) – incident shock overpressure

\( q \) – artificial viscosity

\( s \) – a constant relating shock speed to

\( s_i \) – deviatoric stress

\( u_p \) – particle velocity

\( v \) – initial, porous-specific volume

\( v_s \) – solid, compacted volume

\( w_u \) – uniform pressure load on a slab

\( A \) – initial static yield strength

\( A_x, A_y, A_z \) – acceleration in the \( x, y, \) and \( z \) directions.

\( B \) – strain hardening constant

\( C \) – strain rate hardening constant
$C_L$ – a constant related to the Landshoff artificial viscosity formulation

$D$ – cylindrical charge diameter

$E$ – modulus of elasticity

$E_{mat}$ – internal material energy

$G$ – Lame constant, shear modulus

$H$ – altitude above sea level (feet)

$I$ – second moment of area

$I_1, I_2, I_3$ – strain invariants

$I_P$ – impulse

$J_L$ – second deviatoric stress invariant

$K$ – bulk modulus

$L$ – cylindrical charge length, length of a yield-line

$P_I$ – incident elastic stress wave pressure

$P_R$ – reflected elastic stress wave pressure

$P^*$ – standard sea level pressure

$P^+_S$ – peak absolute shock pressure

$R$ – radius from the charge (feet)

$T^+$ – positive phase duration of an air shock

$T_{H^M}$ – homologous temperature

$U_o$ – general stress loading

$U_S$ – shock velocity

$V$ – blowout panel velocity

$W$ – equivalent TNT charge weight (pounds)
\( Y \) – axial yield strength

\( Z \) – scaled distance factor

\( Z^\ast \) – altitude adjusted scaling factor

\( \alpha \) – instantaneous material porosity

\( \alpha_i \) – angle of incidence between a shock front and a surface

\( \gamma \) – mass per unit area

\( \varepsilon_p \) – plastic strain

\( \varepsilon_p^* \) – normalized effective mean strain rate

\( \theta \) – angle off the axis of a cylindrical charge, angle of a yield-line

\( \lambda \) – \( \ln(0.0893 \ Z) \), Lame constant

\( \lambda_{ij} \) – dimensionless frequency parameter of a rectangular plate

\( \nu \) – Poisson’s Ratio

\( \rho \) – density

\( \sigma \) – stress

\( \sigma_e \) – effective yield stress

\( \varphi \) – angle of internal friction in a material

\( \Delta \) – center deflection of a slab

\( \Delta_e \) – grid spacing
LIST OF ACRONYMS

ALE – Arbitrary Lagrangian/Euler
C-4 – Composition-4, RDX-based explosive
CEMED – Center for Energetic Materials and Energetic Devices
CJ – Chapman-Jouget detonation pressure
CTH – Chart Three-Dimensions
DDESB – Department of Defense Explosives Safety Board
DOE – U.S. Department of Energy
EMRTC – Energetic and Materials Research and Testing Center
ESA-MEE – Engineering Sciences and Applications Division–Materials and Explosives Engineering Group
ESA-WR – Engineering Sciences and Applications Division–Weapons Response Group
FEA – finite element analysis
HE – high explosive(s)
HEET – High Explosives Engineering Training Program
JCAT – Joint Characterization and Analytical Testing at the TA-16-340 Complex
JWL – Jones-Wilkins-Lee equation of state
LANL – Los Alamos National Laboratory
LLNL – Lawrence Livermore National Laboratory
PBX – plastic bonded explosive
SAIC – Science Applications International Corporation
SPH – smooth particle hydrobody
TA – technical area
TMD – theoretical maximum density
TNT – trinitrotoluene
1.0 INTRODUCTION

1.1 Background

The risk of accidental detonation is present whenever any type of processing activity is performed with high explosives (HE). The Materials and Explosive Engineering Group of the Engineering Sciences and Applications Division (ESA-MEE) at Los Alamos National Laboratory (the Laboratory or LANL) is responsible for formulating, pressing, and machining HE charges for use in various research activities. Buildings 260 and 340 were constructed in the 1950s at the Laboratory’s Technical Area (TA)16 to provide facilities in which explosive formulation and processing operations could be carried out with reduced risk. The buildings were used concurrently until 1995 (Trujillo, 2004). Both of these buildings include several features that were designed to mitigate the effects of any accidental detonation.

In 1959, a fatal accident occurred in Building 260 in which two workers were killed while performing a drilling operation on approximately 7 lb of HE. The resulting shock wave blew out a wall composed of lightweight blowout panels, exited the processing bay, wrapped around a dividing wall, and punched a section of the abutting bay’s blowout panels inward. This accident prompted two studies by C. A. Anderson (1969) in which eighth scale models were used to determine the blast overpressures within Building 260, the structure’s corresponding response, and the effectiveness of various blast mitigation features including a labyrinth floor plan and frangible blowout panels.
Recently, Building 340 was decommissioned, decontaminated, and scheduled for demolition. ESA-MEE took advantage of this opportunity by conducting a series of full-scale tests within the bays of Building 340. The purpose of these tests was to determine how the shape of the bays mitigated shock pressures within the bays and adjacent corridors. The performance of various blast mitigation features and the structural response of the building were also of interest. Furthermore, this test series was used to provide real data with which to validate numerical codes such as Sandia National Laboratories’ Chart Three-Dimensions (CTH) and the commercial package AUTODYN (Century Dynamics, 2003).

1.2 Purpose and Outline of Study

This study was conducted to determine the effects of various blowout panel reinforcement assemblies on the blast loading or impulse imparted to the structure of Building 340 if an internal explosion occurs inside the building’s processing bays. The resulting structural response of the building was also of interest. The goal was to find a
blowout panel reinforcement assembly that would withstand the force of an external shock wave from an explosion in an abutting bay pushing inward on the blowout panels, but would not greatly increase internal loading of the structure by blocking the vent opening if the accident occurred within the bay. The author believes that a reinforcement scheme can be developed that will provide the necessary reinforcement for the blowout panels while not increasing the blast load seen by the processing bay structure. Any proposed blowout panel reinforcement assembly must be easily installed on an existing structure and be fairly inexpensive.

Both closed-form solutions and AUTODYN computer models were used to calculate the blast loadings for various blowout panel reinforcement assemblies. These results were verified, as much as possible, against test data gathered from the Joint Characterization and Analytical Testing (JCAT) performed on Building 340 at TA-16 over the summer of 2004. Once the AUTODYN blast and structural response predictions for tested panel configurations were verified with JCAT data, calculations on untested panel assemblies were made to determine their effects on blast impulse and structural response. The results of this study could be applied to Building 260, in which future HE processing operations will be carried out.

The first section of this report, Section 1, gives descriptions of Buildings 260 and 340 as well as the existing blowout panel assembly. The two proposed blowout panel reinforcements are described and the JCAT test series is summarized. The AUTODYN finite element analysis (FEA) hydrocode software package is also described in Section 1.
The second portion of this report explains the procedures used in both the closed-form and computer modeling calculations. There is a review of various techniques classically used to determine blast overpressures, and the method with which the blowout panel velocities from various explosions were determined using the calculated blast pressures is outlined. The AUTODYN remap facility is outlined and the AUTODYN constituent models for the steel rebar and concrete, as well as the setup of the 3-D AUTODYN bay model, are described in Section 2.

Data collection methods used in the JCAT series are described in the third section of this thesis. This chapter includes a description of accelerometers, piezoelectric pressure gages, signal amplifiers, and data collection equipment and the various gage-mounting configurations.

The fourth section compares both the experimental and computational results. The results of pressure predictions for 2.2-lb, spherical trinitrotoluene (TNT) charge air blasts made using AUTODYN are compared against empirical data. The comparison of AUTODYN pressure predictions and empirical data to cylindrical charges of various sizes is extended. Air blast pressure verification calculations made using a 3-D AUTODYN bay model are compared against JCAT data and the effects of using different charge densities in the AUTODYN 3-D air blast models are briefly outlined. The measured and calculated blowout panel velocities are compared and data from these measurements and calculations are used to size appropriate blowout panel reinforcements, as described in Section 4.7. The results of solutions obtained using closed-form techniques given in a U.S. Department of Energy (DOE) manual are presented in Section 4.8. A comparison of both the measured and calculated
effectiveness of the blowout panels at increasing structural blast loading is presented. Ceiling damage from the large sidewall test and the large bay-centered test is described. AUTODYN structural response predictions are compared against JCAT results in Section 4.14 and, finally, AUTODYN predictions for the structural response of the building, had the blowout panels been present during testing, is also presented.

Section 5 is the last section of the report and gives a summary of the effects of various blowout panel configurations. Possible future work is outlined, including scale-testing configurations that could be used for further investigation of the blowout panel reinforcement and the associated effects in structural response. The report concludes with recommendations for improving the safety of Building 260 that are based on the findings of this study.

1.3 **Description of Building 340**

Building 340 is an explosive processing facility located at the Laboratory that was scheduled for demolition. The building was constructed in the 1950s to house various explosives processing tasks, including mixing, pressing, and machining. Building 340 consists of nine processing bays arrayed side by side. Each bay is similar to a self-storage rental unit except that the bays are much larger, and, in place of the rollup “garage” door there is a series of frangible “blowout panels.” The bays are typically grouped together in pairs by a single hallway that leads to a long corridor connecting all the bays. Two bays that are paired together are referred to as adjacent bays. The hallways and corridors are designed to create a labyrinth that will mitigate shock pressures in adjacent rooms or bays if an accident occurs in any one of the bays. The wall between any bay and its corresponding access hall is always referred to as the
adjacent wall. The full-length, 27-ft-long walls separating bay pairs are referred to as abutting walls.

**Figure 2.** Floor plan of a typical bay pair.

Three of the bay walls are constructed of heavily reinforced concrete that is typically 2 ft thick with 1.25-in.-diameter reinforcing bars, spaced 6 in. apart, running vertically and horizontally on both faces. The fourth wall is a frangible blowout panel that is designed to provide overpressure relief by moving outwards during an accidental detonation. The bays have interior widths of either 20 or 24 ft, interior lengths of approximately 27 ft, and ceiling heights of 20.5 ft. The ceiling consists of 18 in. of reinforced concrete topped with 2 in. of insulation and weather-proofing materials. The ceiling reinforcement consists of 1.25-in.-rebar, spaced 9 in. apart, running both directions on both faces and doubly reinforced near the edges. The free edge of the ceiling has a 42-in.-high by 18-in.-wide parapet that provides additional reinforcement. The back corners of the room have 2-ft, heavily reinforced concrete gussets. The
blowout panels stretch horizontally across almost the entire width of the room and vertically from the floor to within 6 in. of the ceiling, where they meet the bottom of the parapet, which protrudes approximately 6 in. below the ceiling lever.

![Figure 3. Blowout panel side of Building 340 with test instrumentation in place.](image)

All blowout panels are located on the north side of the building. The south side of the building has a number of small rooms such as offices, equipment rooms, and break rooms that are not analyzed in this thesis. Passage between the back rooms and the processing bay is via a 9-ft-high, 6-ft-wide hallway that leads to the center of the bay pair.
1.4 **Description of Building 260**

Building 260 is very similar to Building 340. The major difference between the two buildings is that the bays in Building 260 are 12 ft high, as opposed to 20 ft high in Building 340. The ceiling of Building 260 is 15 in. thick, while Building 340 has an 18-in.-thick ceiling. The bays of Building 260 are 20 ft long, 7 ft shorter than those in Building 340.

1.5 **Description of the Blowout Panels**

The blowout panels are designed to leave their initial position quickly, creating a vent through which the expanding explosive byproduct gases can escape the bay. While being as lightweight as possible, they must still be strong enough to withstand the types of bumps and abuse any wall can be expected to receive during daily operations, as well as wind and snow loadings.

The construction of a strong, lightweight shell was achieved by using 16-gauge aluminum sheet for the outer surfaces. Two sheets were stamped together to form a panel. The interior of the panel was filled with fiberglass insulation having a density of 2.5 lb/ft³. Each panel is 3 in. thick, 16 in. wide, and 6 to 20 ft long, depending on its location. Individual panels are fastened together at the ends to form a wall 3 in. thick. The entire wall of panels is secured to the building by number 10 sheet metal screws that are run into 3.5-in. by 1.5-in. by 0.25-in. aluminum angles that run the length of the bay opening, along the top and bottom. The aluminum angles are securely fastened into the building’s concrete frame by 5/16-in. bolts. To prevent the panels from hanging up on the angles, the aluminum angles are located on the room side of the blowout panels.
The areal density of the complete panel assembly is approximately 5 lb/ft$^2$. In some places, various electrical and plumbing fixtures are attached to the blowout panels, raising the local panel density.

![Figure 4. Elevation view of a typical blowout panel assembly and installation (from LANL drawing C-11139).](image)

**1.6 Description of the Proposed Blowout Panel Reinforcements**

The purpose of this study was to find a panel reinforcement assembly that would not greatly increase the blast loading of the bay structure. Any type of panel reinforcement must improve the protection of bay occupants by preventing the blowout panels from being pushed inward by a shock wave leaving an abutting bay in which an accident has occurred. While being strong enough to prevent inward panel movement,
the reinforcement must also have a low drag profile and not significantly increase the blast loading on the structure if an accident should occur within the bay. Additionally, any type of proposed reinforcement should be simple enough to be easily installed on an existing structure with minimal interruption of production work. Also, the reinforcement will obviously need to be on the inside of the blowout panels, so that the panels will have something to rest against when pushed inward, but at the same time will not be impeded if blown outward.

Several blowout panel arrangements were designed to meet these requirements and then evaluated with the hydrocode AUTODYN (refer to Section 1.8 for a description of AUTODYN). All reinforcement beams were sized using data obtained by incident pressure gages outside the abutting bay panels, 1 ft in front of the panels, on a 75-lb test. The first reinforcement scheme consisted of a vertical array of 2-in. by 6-in. by 3/8-in. rectangular, structural steel tubes. A 20-ft section of tubing, running from the floor to the ceiling, would be placed at every blowout panel seam. The rectangular tubes could be secured at their ends by either welding or bolting them to a 0.75-in. by 6-in. steel bar running the length of the bay opening that is fastened to the bay’s concrete floor and parapet with lag bolts threaded into concrete anchors.
Figure 5. Type 1 blowout panel reinforcement.

A second possible method for reinforcement of the panels would be the addition of large I-beams or steel tubes, running horizontally behind the blowout. Calculations show that the blowout panels can withstand the shock from a 25-lb charge detonating in an abutting bay while spanning a gap of 7 ft between the individual reinforcement beams.
Figure 6. Type 2 blowout panel reinforcement.

1.7 Summary of the JCAT Test Series

The JCAT series was conducted at Building 340 in late June and early July of 2004 and consisted of the detonation of a number of small, 25- to 75-g HE charges, followed by a series of larger HE charges. The small charges were used to calibrate instrumentation and gage the response of the building’s blowout panels to small blast loadings. Charges 25 lb or larger were used to determine the effects of designed mitigation features on shock pressures as well as the structural response of the building itself. Several HE charges, on the order of several hundred pounds, were detonated to determine the bay’s structural response and the extent of damage to the building.
These HE charges will always be referred to in this report as the large charges. All of the larger charges were detonated at heights of 3.3 and 6.6 ft above the floor at various locations within the building. A series of bay-centered tests consisting of HE charge sizes of 25, 75, and 100 lb and the larger charges were conducted. Additionally, a series of tests in which the charge was located 3.3 ft from the back wall was conducted. This test series included 5-, 50-, and 100-lb HE charges. A large HE charge was detonated 3.3 ft from the sidewall of an end bay, simulating a blast near the back wall. Conducting the test in an end bay allowed high-speed photography of the event from a distant location outside the building.

A sequence of 25- to 55-g tests was conducted first in an attempt to determine the charge size required to fail the blowout panels. Price had predicted that 0.17 lb would be required to fail the blowout panels (Price, 2004). Blowout panel failure did not occur during these tests. The 25- and 75-lb HE charges were the first of the large charges to be detonated. These tests were conducted in bays at opposite ends of the building, allowing both tests to be conducted in an undamaged bay with all blowout panels in place. The data from these two shots will hopefully enable ESA-MEE to determine the blast mitigation effectiveness of the blowout panels in accidents involving HE charge sizes typically used in daily operations.

1.8 **Description of AUTODYN Nonlinear Analysis Software**

AUTODYN is a commercial code developed by Century Dynamics for use in solving nonlinear dynamic problems involving the high strain rates associated with impact and explosive loading problems (Century Dynamics, 2003). A two-dimensional (2-D), three-dimensional (3-D), and an academic version are currently available. The
program uses various processors to solve blast and impact problems. These are selectable by the user and include Eulerian, Lagrangian, Arbitrary Lagrangian / Eulerian (ALE), and smooth particle hydrobody (SPH) processors. Different types of processors can be used concurrently to model different parts of the same problem. A large library containing properties for many common materials is included with the software. Additionally, the user is allowed to make modifications to the material properties and equations of state. The AUTODYN graphical interface allows the user to watch problems run in “real time” rather than having to wait for data outputs at the end of the problem—a feature that the author found extremely useful for determining exactly what was occurring as the problem ran and for troubleshooting modeling errors.
2.0 CALCULATION PROCEDURE

2.1 Determination of Blast Impulse

The total blast impulse imparted into the walls, ceiling, and blowout panels of a bay was calculated using several different methods, including the use of both analytical techniques and various software packages. Cylindrical charges, such as those tested in Building 340, create irregular blast fronts that make shock pressure and impulse calculations difficult. The irregular bay geometry and moving blowout panels further complicate the analysis. Obtaining an exact analytical solution is nearly impossible. However, the calculation may be greatly simplified by making a series of assumptions and using empirically generated data. DOE’s Pantex Fabrication Facility, located in Amarillo, Texas, publishes a manual that provides a procedure for estimating structural loads resulting from internal explosions (DOE, 1980). The manual, entitled “A Manual for the Prediction of Blast and Fragment Loadings on Structures,” will be referred to as the Pantex Manual throughout this report.

When an explosion occurs within an enclosed room, the impulse that loads the structure may be broken into two distinct parts: the shock front load and a quasi-static gas pressure load. The initial blast loading is a result of the primary shock front and secondary reflected shock fronts. Air shock loadings typically have very short time durations, but may have peak pressures of several thousand pounds per square inch or more. At later times, the structure is loaded primarily by a low-intensity, long-duration “gas impulse” that is created by expansion of the explosive byproduct gases within the structure.
The intensity of the primary, nonreflected shock impulse from a spherical charge can be easily determined using empirically generated tables and Hopkinson and Sachs scaling laws (Kingery and Bulmash 1984). The empirical values for shock overpressure and impulse are usually listed for TNT detonations at sea level. Other explosives are given a TNT equivalency factor that the actual HE charge weight must be multiplied by to obtain an equivalent TNT weight. The equivalent TNT weight may then be used with the tables to obtain scaled pressure, shock time of arrival, and blast impulse. All tests at Building 340 were conducted using plastic bonded explosive (PBX)-9501, which has a TNT equivalency of 1.129 (Lawrence Livermore National Laboratory, 1985).

TA-16 is approximately 7500 ft above sea level. At this altitude, the shock properties of air are significantly different than at sea level, the altitude at which most scaling curves and relationships are valid. Fortunately, blast parameters can be adjusted for altitude if the ambient pressure and the speed of sound are known. These values can be determined using the following equations (DOE, 1980):

\[ p = 14.6965 \left( \frac{288.15}{288.15 - 0.0019812H} \right)^{-5.2588} \quad \text{and} \quad (2.1) \]

\[ a = 65.77 \left(288.15 - 0.0019812H\right)^{0.50} \quad (2.2) \]

The first relationship gives pressure, \( p \), in pounds per square inch, while the second expression gives the speed of sound, \( a \), in feet per second. \( H \) is the altitude above sea level in feet. From these equations, the ambient air pressure at TA-16 is 11.12 psi, and the speed of sound is 1087 ft/s.

The scaled distance factor is given by

\[ Z = \frac{R}{W^{1/3}} \quad (2.3) \]
in which $R$ is the radius from the charge in feet, and $W$ is the equivalent TNT charge weight in pounds. The scaled distance can be adjusted to account for altitude (DOE, 1980):

$$Z^* = Z \left[ \frac{p}{p^*}\right]^{\frac{1}{3}}. \quad (2.4)$$

In Equation (2.4) ambient pressure is given by $p$ and standard sea level pressure is given by $p^*$. The altitude-adjusted scaling factor can be used in place of the standard scaled distance with any expression or chart that uses a scaled distance.

![Diagram of end detonated cylinder with coordinates](image)

**Figure 7.** Coordinates for determining the pressures from an end detonated cylinder.

Unfortunately, blast-overpressure and shock-intensity data is usually tabulated only for spherical charges in free air or hemispherical charges lying on the ground. All of the JCAT charges were cylinders. The Pantex Manual gives the following empirical curve fit for determining the blast pressures from an end-detonated cylindrical charge with an aspect ratio less than or equal to one (DOE, 1980):

$$x = \ln\left(\frac{L}{D}\right), \quad (2.5)$$
\[ y = (2.0467 - 0.1753x + (0.1285 + 0.0728x)\cos(\theta) + (0.0621 - 0.2503x)\cos(2\theta) \\
+ (-0.0029 + 0.0079x)\cos(3\theta) - 0.1534\cos(4\theta)) + ((-2.1616 + 0.0464x) \\
+ (-0.2079 - 0.2174x)\cos(\theta) + (-0.4178 + 0.3426x)\cos(2\theta) + (-0.1372 - 0.1171x)\cos(3\theta) \\
+ (-0.3484 - 0.3449x)\cos(4\theta))\lambda + (0.4366 + 0.0053x + (0.0138 + 0.0006x)\cos(\theta) + \\
(0.1178 - 0.2656x)\cos(2\theta) + (0.2556 + 0.2072x)\cos(3\theta) + (0.3123 - 0.2140x)\cos(4\theta)) \]

(2.6)

and

\[ P = 1.22[\exp(y)] , \]

(2.7)

in which \( L \) is the charge length, \( D \) is the charge diameter, \( P \) is the peak pressure in psi, and \( \theta \) is the angle off the charge axis. The detonator is located at \( \theta = 180 \) degrees. \( \lambda \) is given by the expression

\[ \lambda = \ln(0.0893Z) , \]

(2.8)

where \( Z \) is the scaled distance from the charge. The Pantex Manual gives plots of empirical data overlaid with this curve fit for various charge aspect ratios. The curve fit is fairly accurate for most scaled distances and gives peak pressures that are typically within 5%. However, the curve fit is only valid for peak side-on pressures of 2 to 100 psi (DOE, 1980). Many of the calculated and recorded pressures within the Building 340 bays were on the order of 1000 psi or more, well beyond the Pantex Manual’s cylindrical charge table range.

Blast-X (U.S. Army Engineer Research and Development Center, 2001) is a computer program that will determine pressures for a Composition-4 (C-4) or TNT cylinder having a length-to-diameter aspect ratio of 1.0, 2.0, 4.0, or 6.0 by using tabulated empirical values and scaling laws. All charges in the test series had a length-
to-diameter aspect ratio of 1.0 or less, prohibiting the direct use of Blast-X in the actual blast calculations.

Because both the Pantex Manual and Blast-X could not be applied to the actual charges used in Building 340, AUTODYN had to be used for all blast calculations. Any charge geometry and scaled distance can be modeled in an AUTODYN program, provided the user has a computer powerful enough to handle the required grid size. The results allow for verification of AUTODYN against both the Blast-X and Pantex Manual data.

A series of 2-D calculations were run on AUTODYN and the peak pressure predictions were compared against values given by both Blast-X and the Pantex Manual. Two-dimensional blast calculations are the first step in all AUTODYN structural response models and are therefore very important. All mass, momentum and energy data developed in a 2-D calculation was “remapped” into a 3-D grid by AUTODYN. (For more information see Section 2.3 – “Description of the AUTODYN Remap Facility.”) A significant advantage of using a 2-D grid is the allowance of a finer mesh size, cutting down on the amount of shock wave smoothing performed by the artificial viscosity terms of the mass, momentum, and energy relations used by AUTODYN to determine material states. (See Appendix B for a description of artificial viscosity.) The high-fidelity, 2-D grid gives higher peak pressures that are closer to the real values. (See Appendix B for information on the effects of grid size on peak predicted pressures and predicted blast impulse.) The 2-D calculations also run much faster than 3-D calculations.
The AUTODYN 2-D verification calculations were first run for a small, 6.593-lb, C-4 charge, i.e. large scaled distance. The results were then compared to the Pantex Manual curves. These calculations were run for length-to-diameter ratios of 1.0, 0.75 and 0.50—the aspect ratios of the various charges used in actual JCAT series. Once comparisons were made for the small charge, the 25-lb and larger cylindrical charge detonations could be modeled on AUTODYN 2-D with an understanding of the program’s prediction characteristics.

Although the exact blast pressures and impulse distributions of the cylindrical charges used in the JCAT series could not be determined by analytical methods, an estimate of the total impulse was possible. For initial computational purposes, the charges were assumed to have a spherical geometry. The initial shock impulse seen by the walls and ceiling can be easily determined using Kingerly’s empirical tables (Kingerly and Bulmash, 1984).

An idealized shock wave, moving through undisturbed air, is shown in Figure 8 (Baker, 1973). The initial rise time is essentially zero: the air pressure jumps from ambient to peak pressure conditions instantaneously. Following the initial rise, the pressure decays to a negative value in an exponential fashion. The shock pressure is usually divided at the decay line’s zero crossing into two regions: a positive pressure phase and a negative pressure phase. The total impulse delivered to an object struck by the shock is the integral of this decay curve.
Many researchers have assigned curve fits to the shock decay curve. Baker (1973) gives eight curve fits of differing complexity and accuracy. He recommends using the modified Friedlander equation, given by

\[ p(t) = p_0 + P^+_s \left( 1 - \frac{t}{T^+} \right) e^{-bt} \]

in which \( P^+_s \) is the peak absolute shock pressure, \( T^+ \) is the positive phase duration, and \( b \) is a decay constant to be determined from experimental data. Baker also gives three other, more complicated forms. The author recommends using these three curves because they are more accurate and are readily calculated with personal computers. The eight equations given by Baker fit the positive phase of the shock only. However, the impulse associated with the negative phase of the shock is typically ignored because it is generally much smaller than the positive impulse and doing so is generally considered conservative. Therefore, one may find the shock impulse by integrating any curve fit over the positive phase of the shock. Unfortunately, this integral will not account for the effects of reflection, which can vary widely. To obtain the actual
impulse imparted to the object’s surface, the free-air impulse must be multiplied by some type of reflection factor.

The Pantex Manual recommends using the modified Friedlander equation, Equation (2.9), to obtain the free-air shock impulse. The manual then gives a plot for determining reflection multipliers (Figure 9).

Figure 9. Reflected shock overpressures as a function of peak pressure and angle of incidence (from DOE, 1980).

The data given in Figure 8 and Figure 9 is useful for calculating the impulse that an exterior wall will be exposed to, but it is too cumbersome to use when determining an interior wall’s total blast load. However, the Pantex Manual has a procedure for determining the total initial and reflected shock impulse for each surface of an enclosed or partially vented bay that is based on the surface’s distance from the charge, the angle of incidence between the shock wave and the surface, and the room volume.

The calculation for the internal surface blast impulse is carried out by determining the volume of the room, equivalent TNT weight of the charge, vent area,
and radius from the charge of various surfaces. These factors are used with a series of nondimensionalized charts to find the peak shock pressures, shock impulse at the various surfaces, gas impulse, and impulse duration. The method depends on the user reading a series of log-log plots, each of which uses information read off the previous chart. Therefore, a large margin of error would be expected. Furthermore, all of the charts are approximations. Although this method is not precise, the author still used it to obtain data that was compared to that generated in the AUTODYN predictions.

When analytical techniques fail, FEA programs must be used to predict blast loadings. Two FEA programs were used by ESA-MEE to investigate shock behavior. Parker (2004) used Sandia National Laboratories’ hydrocode, CTH, to run Eulerian calculations. Price and the author (Price, 2004) used AUTODYN to investigate the shock and building interactions using both Eulerian and Lagrangian 2-D and 3-D computational grids.

2.2 Determination of Blowout Panel Velocities

The duration and intensity of the gas impulse is highly dependent on the room’s volume, aspect ratio, and vent area. The purpose of the blowout panels is to provide an instantaneous vent route for the expanding explosive byproduct gases if an accident occurs, while keeping the weather out during normal operations. In order to determine how quickly the byproduct gases start to vent from the bay, the blowout panel velocities must be determined. The blowout panel velocities are also required if the panels are to be accurately modeled in the 3-D bay models. Fortunately, the panel velocities can be calculated if the blast impulse is known.
In the case of the Building 340 blowout panels, the lightweight panels will leave their positions fairly quickly, so only the early time shock impulses need to be determined in order to find the panel velocities. By the time static gas loading within the bay becomes significant, the panels will have traveled a large distance and will have left a large opening through which the static gas pressure can vent, mitigating quasi-static loading of the bay. The blast impulses can be found using either the analytical methods given in the Pantex Manual or with hydrocodes. In addition to using the analytical methods, the author used a series of 3-D AUTODYN calculations to determine the shock timing and total impulse received at various points on the blowout panels. These values were then compared with the data that was available to the author at the time the calculation was made. The impulse from the test data was determined by numerical integration of pressure recordings using Microsoft Excel. In the bay-centered and sidewall shots, the back wall pressure gage data were used to determine the impulse seen by the blowout panels. The room is fairly symmetric, so at early times the shock wave can be assumed to propagate symmetrically if the charge is placed at the room’s horizontal centerline.

When a plane wave makes a transition from one material to another, a portion of its energy will be transmitted into the second material while the remaining fraction is reflected back into the first material. Every material has a characteristic impedance given by \( \rho c \), where \( \rho \) is the material’s density and \( c \) is the speed of sound in the material. If the incident pressure of the air shock is known, the pressure of the wave reflected off the aluminum surface, at these low pressures, can be approximated by the expression (Tedesco, 1999)
\[
\frac{P_R}{P_I} = \frac{(\rho c)_{Al} - (\rho c)_{Air}}{(\rho c)_{Al} + (\rho c)_{Air}}.
\]  

(2.10)

In the case of the blowout panel, the ratio of \(P_R\) to \(P_I\) is nearly one because \(\rho_{Air} \ll \rho_{Aluminum}\). Thus, the entire shock can be assumed to reflect off the panels.

In all blowout panel velocity calculations the panels were assumed to initially hold together as a single sheet and not rotate about their vertical axis for the first several milliseconds after the initial shock front struck them. Therefore, the effects of gap formation between individual panels could be ignored. The author believed this assumption was valid for several reasons. First, the panels receive the greatest portion of their kinetic energy in a fraction of a millisecond, when the initial, and typically strongest, shock front strikes them. In the time required for the first shock to strike and reflect off the blowout panels, they cannot move very far, and must therefore remain in the same orientation. Second, after traveling only a couple of feet the panels start to provide effective venting, and their influence on the bay’s structural response becomes unimportant. The panels may then be ignored. Any gaps that start to form at this point will have little effect upon the bay’s structural response.

After examining the remains of the blowout panels after the shot, the author noticed that many long sections of the panels had remained intact and straight. This indicated that many of the panels did not sustain much damage while they were accelerated by the blast, although some panels were severely bent. The blowout wall doors in particular, which were composed of three to four 6-ft-long, 16-in. blowout panels fastened together at the top and bottom of the doorframe, seemed to hold together fairly well.
Despite these observations, AUTODYN 2-D calculations have shown that if vertical stringers are positioned correctly in front of the panels, the panels will rotate and create gaps almost instantly. The gaps form quickly enough to allow a portion of the first shock front to vent. As discussed previously, this behavior was ignored in the closed-form calculations.

Once the blast impulse at various points on the blowout panels was known, the panel velocities could be easily determined using the relationship

\[
V = \frac{I_P}{m}
\]

in which \(V\) is the panel velocity, \(I_P\) is the impulse received by the panel per unit area, and \(m\) is the panel mass per unit area. Since the total impulse received at any point on the panel was known as a function of time, the blowout panel velocity at that point could be easily determined for any time.

In addition to the blowout panel velocity calculations made using Equation (2.11) and the pressure time history data collected from experimental tests, several attempts to estimate panel velocities were made using AUTODYN 3-D models. The first attempt at modeling the blowout panels was made using one of the 3-D blast calculation models. A Lagrangian blowout panel was coupled to an existing Eulerian air subgrid. The author has found that any Lagrangian grid must be coupled to the Eulerian grid that it is contained within at all nodes on both surfaces of the Lagrangian grid. If the nodes on only one side of the Lagrangian grid are coupled with the Eulerian grid, AUTODYN will input force due to ambient air pressure into the coupled side of the object only. This results in a force imbalance, and the object begins to accelerate as if propelled by a pressure of \(P_0\). Therefore, the Lagrangian subgrid node spacing must be
equal to that of the Eulerian grid to ensure that the nodes on both sides of the Lagrangian mesh are coupled. A rather large Eulerian air mesh spacing of 3.54 in. was required in order to run calculations on the entire room in a reasonable amount of time. An attempt was made to lower the density of the fill aluminum in the Lagrangian blowout panel grid in order to have the correct amount of mass within the grid while maintaining a large mesh size. Lowering the aluminum density, however, created several problems. AUTODYN bases the computational time step on the amount of time that the fastest particle or shockwave speed in the problem would require to traverse the diagonal of the smallest mesh element (Century Dynamics, 2003). The speed of sound is the slowest possible shock speed in any material, and is given by $\sqrt{\frac{E}{\rho}}$, in which $E$ is the material’s modulus of elasticity and $\rho$ is the density of the material. Decreasing the density of the aluminum in the blowout panels to 10% or 20% of the metal’s natural density increased the shock speed by a factor of three or more. The modulus of elasticity was then lowered to counter this effect and keep the time step large. This in turn created more problems within the constituent model for the material, causing AUTODYN to either not run or completely crash. The author tried using all the aluminum alloys given in the AUTODYN material library and could not find an alloy that worked well. Next, attempts were made to model the blowout panels with nylon and other plastics or synthetic materials of low density, with similar results. Eventually polycarbonate was found to work, but the time step was still too small to allow any calculations to be performed within a reasonable time period.

At this point, the fully coupled, 3-D blowout panel response calculation attempts were dropped in favor of other methods. A 2-D, axial-symmetric model could be built
using a mesh size equal to the thickness of the aluminum sheet required to simulate the
blowout panel mass. This allowed the Lagrangian blowout panel mesh to be filled with
aluminum having the proper density. This same strategy was also used to create 2-D
models having planar symmetry.

The 2-D, axial-symmetric models were run for the large, bay-centered charge
located 3.3 ft off the floor. The purpose of the models was to check the calculations
made using Equation (2.11). The panel mass, including the weight of the fiberglass
insulation, was simulated by a 0.355-in.-thick layer of 6061-T6 or 1100 aluminum that
was modeled using a Lagrangian subgrid that was fully coupled to the Eulerian air
subgrid. The charges were detonated from the top and burned downward; the initiation
train used for the actual JCAT charges. In one model, flow-out boundary conditions
were specified on the top and bottom of the 78.7-in.-wide grid, simulating a free-air
explosion for comparison with tabulated data. In another model, the lower flow-out
boundary condition was removed, simulating presence of the bay floor. Pressures and
panel velocities were recorded by gages at various points within the air and panel
subgrids. The air gages remained fixed in the Eulerian mesh while the Lagrangian
blowout panel gages moved with the panel grid.

Plan view computations with planar symmetry were also run on AUTODYN
2-D using a 0.827-in. grid to investigate the effect of vertical Type 1 stiffeners on panel
loading. These computations included an entire bay, and therefore required larger
computational cell sizes. The Lagrangian blowout panel grid was filled with
polycarbonate because the material’s low density allowed for the use of larger
computational mesh spacing. One disadvantage of using this material for the panel
simulation is that the effective stiffness of the polycarbonate model panels is about one hundredth of the actual panel stiffness. The polycarbonate was given no failure strength, in effect making it infinitely strong. In JCAT testing all blowout panel walls came apart at the seams, and the aluminum was not typically observed to tear, only bend. Therefore, sand was used to fill a “gap” in the Lagrangian blowout panel grid every 16 in. to simulate the panel seams. The density of the sand was decreased to match that of the panels and the erosion strain was set to 1.2. With an erosion factor this low, AUTODYN will automatically remove the sand and associated grid nodes if much deformation takes place within the cell. This way, if the panels started to move in relation to each other, the sand gaps would be removed, and high-pressure air on the inside of the panels could vent to the outside.

Uranium was used in the plan view models to simulate the gusseted corners of the rooms and the Type 1 reinforcements. All calculations were based on the assumption that the reinforcement beams were securely fastened to the bay structure at both the top and bottom ends and would not move. The reinforcement stiffeners could therefore be treated as reflecting surfaces, as were the back corner gussets. Uranium worked well for this because it could be filled directly into the air grid. Boundary conditions were not required because the dense uranium fill blocks in the grid, simulating the reinforcements and concrete corner gussets, did not move during the computation period.
2.3 Description of the AUTODYN Remap Facility

The Jones-Wilkins-Lee (JWL) equation of state is normally used by AUTODYN to model detonations, although a Lee-Tarver burn model is also available. Once the pressures in the problem drop below 200 atm, AUTODYN version 5.0 automatically switches to the Ideal Gas Law (Century Dynamics, 2003). To accurately model the detonation of an HE charge using the JWL equation of state, at least 10 grid cells should be contained within the explosive. Because of the large difference of scale between the explosive charge and the bay, it would be impossible to model the entire problem in three dimensions with a small enough grid using the computing resources available to the author. Fortunately, AUTODYN includes a remap package that allows problems run in 1- or 2-D grids to be mapped into other 2- or 3-D computational grids. The advantage of this technique is that the detonation phase of the problem can be run very quickly and accurately using a fine grid size. At later computational times, in which the entire bay is part of the problem, a larger grid spacing may be used.
For spherical charges, the initial detonation can be modeled using the AUTODYN wedge facility, which is essentially a one-dimensional grid that diverges into two dimensions to model spherical geometries. The initial detonation can also be modeled using a 2-D, axial-symmetric grid that allows any charge shape with an axis of symmetry to be accurately modeled. The results obtained from either of these two grid types may then be mapped into a 3-D model by wrapping the results around one or more axes of symmetry. The grid size of the 3-D model can be much larger than the remap grid size, providing a method for dealing with large differences in scale between different parts of the same problem. AUTODYN will map the mass, momentum, and energy data from each 2-D remap file cell into the appropriate 3-D model cell. This data is then used as the initial conditions for the new problem.

2.4 Description of the AUTODYN Constituent Model for Steel Rebar

The shock equation of state was used to relate particle velocity \( u_p \) in the rebar and the shock velocity \( U_S \) (Century Dynamics, 2003). In its simplest form, this relationship has the form (Cooper, 1996)

\[
U_S = c_0 + s u_p,
\]

in which \( c_0 \) is the material’s bulk sound speed and \( s \) is an empirically determined constant that accounts for the increase in shock speed with increasing particle velocity. Particle velocities in the blast problems modeled in this study were relatively slow, so the shock velocity was nearly \( c_0 \).

The Johnson-Cook equation of state was used to determine the yield point of the rebar steel. This equation of state was developed using data obtained from Taylor cylinder impact tests and is most applicable to medium strain rate conditions such as
those encountered in ballistic impact problems. However, the Johnson-Cook relation is based on yielding of materials under static loading conditions so it works well at low strain rates. The form of the Johnson-Cook equation of state implemented in AUTODYN is (Century Dynamics, 2003)

$$\sigma_{\text{yield}} = (A + B\varepsilon_p^n)(1 + C \log \varepsilon_p^*)(1 - T_H^m).$$

The initial static yield strength of the material is given by the constant $A$. Work hardening from material strain is accounted for by the $B\varepsilon_p^n$ term, in which $B$ and $n$ are empirically determined constants. Dynamic strength increases are accounted for by the second bracketed term where $C$ is another empirically determined constant and $\varepsilon_p^*$ is the normalized effective mean strain rate. The last term accounts for softening from thermal effects. $T_H^m$ is the homologous temperature.

The rule of thumb when making closed-form blast loading calculations is to increase the effective rebar strength by approximately 20% to account for strain hardening effects. Obviously, some strain hardening occurs in these types of problems. The effects of the homologous temperature term are probably small.

### 2.5 Description of the AUTODYN P-Alpha Concrete Model

AUTODYN uses a variation of Herrmann’s P-Alpha equation of state to describe the behavior of concrete. This constitutive model was developed as an attempt to describe the behavior of both the porous and solid phases of a porous material as it is compressed into a solid state (Herrmann, 1969). Early investigators had treated these regions separately by developing different equations for the elastic, plastic, elastic-plastic, and unloading phases of the material loading.
Herrmann gave an equation of state having the form

\[ p = f\left(\frac{v}{\alpha}, E_{\text{mat}}\right) \]  \hspace{1cm} (2.14)

in which \( E_{\text{mat}} \) is the material’s internal energy, \( v \) is the initial, porous, specific volume of the material, and \( \alpha \) is the instantaneous material porosity, \( v/v_s \). The material’s solid, compacted density is given by \( v_s \). To complete the description of the material, an equation of state for \( \alpha \) is also needed. For material compression occurring within the elastic range, empirical data can be used to determine an appropriate form of the expression (Herrmann, 1969)

\[ \alpha = g\left(p, E_{\text{mat}}\right). \]  \hspace{1cm} (2.15)

Herrmann found that a third order polynomial allowed for accurate curve fitting of \( \alpha \) in the plastic region.

Carroll and Holt (Carroll and Holt, 1971) modified Equation (2.14) by assuming that the material was homogenous and that the porosity affected pore collapse pressures. They suggested the following form for the P-alpha equation of state:

\[ p = f\left(\frac{v}{\alpha}, E_{\text{mat}}\right)\left(\frac{1}{\alpha}\right). \]  \hspace{1cm} (2.16)

AUTODYN uses this form of the P-alpha equation of state (Century Dynamics, 2003).
In his equation of state formulation, Herrmann neglected shear strength and the strain rate dependence of the pore collapse. Carroll and Holt (1971) sought to include rate dependence in the P-alpha model and in the process discovered that under static pressure conditions the matrix undergoes uniform dilation—meaning that deviatoric stress has the greatest effect on porosity. Because the compression rate of the concrete in the author’s models is fairly low, rate dependence effects on $\alpha$ will be small and can be ignored. Overall compression of the concrete matrix will have the greatest effect on the concrete’s behavior.

The initial compaction pressure for the concrete was set to 3425 psi for all AUTODYN calculations in this study. The full solid compaction pressure of the concrete was set to 882,000 psi, while the compressive failure strength of the concrete was only 6000 psi. The relatively low strain rates that the bay concrete was exposed to meant that only the very lowest pressure regions of the plastic hugoniot were encountered. However, large amounts of compression can occur at even these relatively

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**Figure 11.** Porosity-volume Hugoniot for the P-alpha equation of state (from Century Dynamics, 2003).
low pressures because a cusp in the P-V hugoniot occurs at the elastic limit.

AUTODYN’s Drucker-Prager strength model and failure criteria were used to
determine when the concrete would break. For more information on the Drucker-Prager
models, see Appendix C.

2.6 Setup of AUTODYN Eulerian/Lagrangian 3-D Structural Models

A fully coupled, 3-D blast and structural response model is required to
accurately determine the building’s response to the complex loading history created by
an internal explosion. A very large computational grid is required to precisely capture
both the shock behavior within the building and the reaction of the structure. If a purely
Eulerian calculation is made, the required number of cells becomes much larger because
the mesh must be very fine near the walls in order to accurately predict wall deflections.
Modeling the structural elements with moving Lagrangian meshes alleviates this
problem and cuts down on the required number of cells, but two incompatible mesh
types must then be coupled to each other.

The structural models in this study were run on AUTODYN 3-D, which
simultaneously calculates both shock propagation and structural response. Both
AUTODYN 2-D and 3-D allow the user to fully couple the Lagrangian structural
meshes with Eulerian air meshes. The Lagrangian boundary mesh nodes are tied to
spatially corresponding nodes in the Eulerian mesh at the start of the problem. When a
pressure disturbance traveling through the Eulerian mesh reaches the boundary of a
Lagrangian mesh, the boundary is treated as a reflecting surface and the disturbance
begins to reflect off the structural element (Century Dynamics, 2003). The force
imparted on each Lagrangian node by the reflecting disturbance is transmitted, as a
boundary condition, to the Lagrangian mesh. AUTODYN 2-D and 3-D offer several options for dealing with moving Lagrangian elements. The tie-in locations of the Lagrangian nodes may be kept track of and moved spatially within the Eulerian grid as the problem progresses, or they may remain tied to their original Eulerian node locations. The second method requires less computational resources to perform, but negates the major advantage of using this type of coupling: the ability to account for changing boundary conditions within the Eulerian grid created by the moving structural elements.

The major disadvantage of using fully coupled Lagrangian and Eulerian grids is that both the blast and corresponding structural response portions of the problem must be simultaneously computed. Running the two problems in parallel requires more memory and processor time than a back-to-back serial run would require. The author initially ran the problems on a laptop computer having 256 MB of RAM and a 2.4-GHz Pentium IV processor. One half of the bay and ceiling could be modeled using 4-in. air-grid cells. The corresponding run time was two to three days. In July of 2004 Price and the author received a workstation from the Laboratory with 2 GB of RAM and a 3.2-GHz Intel Xenon processor. A fully coupled problem, including the entire room volume, the ceiling, one sidewall, and having a mesh size of 3.543 in., could be run in five to six days on this workstation.

The model used for running the majority of the simulations was designed to allow several types of structural behaviors to be studied including the ceiling response, the response of the sidewall to the larger charges, the effectiveness of the parapet, and the coupled response of the ceiling and sidewall acting together. The model included the
entire ceiling with parapet, and one sidewall. All structural elements were modeled using Lagrangian grids that were enclosed almost entirely within, and fully coupled to, the Eulerian blast grid. The basic cell or mesh element size, a 3.543-in. cube, was selected because it was the smallest cell that could be used while allowing the computation to be completed in a reasonable amount of time. The dimensions and shape of the structural cells could be, and almost always were, varied. The only stipulation for the Lagrangian cells was that the outer node positions of the sidewall and ceiling match up with the positions of nodes in the Eulerian blast grid. Therefore, the coordinates of all Lagrangian nodes to be coupled had to be evenly divisible by 3.543 in.

The weakest element in the bay structure was the ceiling, which was only 18 in. thick, as opposed to the 24-in.-thick walls. It also contained less rebar than the walls. This part of the bay was expected to fail before the other bay components and therefore received the most attention. The 18-in.-thick ceiling slab was modeled using five separate subgrids that were layered on top of one another. The top and bottom layers, representing the 2 in. of concrete covering the top and bottom rebar layers, had 3.543-by 3.543- by 1.77-in. cells that were filled with 6000 psi p-alpha concrete to simulate the bay concrete. The bay concrete, which was more than fifty years old, was believed to have a yield strength of approximately 6000 psi in compression. The center portion of the ceiling was constructed of 5 layers of 3.543- by 3.543- by 2.362-in. cells, filled with 6000-psi p-alpha concrete. These layers were tied together with two rebar layers, composed of a single vertical layer of cells, having dimensions of 3.543- by 3.543- by 1.181-in. The rebar layers were filled with p-alpha concrete. Every ninth row, in both directions, was then filled with 59-ksi structural steel to simulate the rebar. The
building’s drawings (Drawing Numbers 11144 and 11148) call for number 10, or 1.25-in., rebar, spaced on 9-in. centers, each way, each face. The estimated yield strength of the steel reinforcing bar was 59 ksi. The rebar fill described above gives the same volumetric steel-to-concrete fill ratio that the drawings call for. Containing the rebar within the same grid as the concrete simulates an infinite coefficient of friction between the rebar and concrete.

The sidewall was modeled in a manner almost identical to that of the ceiling. All subgrids had the same element sizes as their corresponding grids in the ceiling. Because the sidewall is 6 in. thicker than the ceiling, 2 additional layers of cells in the center layer of concrete were required to achieve the correct sidewall thickness. Another difference between the ceiling and the sidewall was the sidewall contained number 10 rebar, spaced on 6-in. centers, requiring a denser steel fill pattern in the rebar layers.

The parapet was modeled using six layers of individual subgrids: two below the ceiling and four above. The construction of the parapet was similar to that of the ceiling and wall except for the rebar layer, which was run in only one direction.

**Figure 12.** Material locations in the 3-D model.
A series of “corner” grids were used to tie the ceiling and sidewall subgrids together into a cohesive structure. The corner grids were layered upon one another, and were sized to ensure that the nodes in all grids were connected together. Rebar was run from the ceiling, through the corner grids, and into the sidewall grid.  

The material layout is given in Figure 12. The bay opening is on the edge of the cube closest to the observer. The parapet runs above the opening for the length of the building and can be seen in Figure 12. The right hand side of this cube represents the other bay wall, which is modeled by a reflecting condition on the side of the grid. The back wall, the side of the cube farthest from the observer, was also modeled as a reflecting surface. The corner gussets at the back of the wall were modeled by filling the proper grid cells with concrete, in a diagonal pattern. This can be seen in the back, left corner of the room.  

The drawings (Drawing Numbers 11144 and 11148) call for a doubling of the rebar at the edges of all walls and the ceiling to prevent shear failure and handle the higher bending moments expected at these locations. Additional rebar was included in the structural model at the appropriate places to simulate the extra reinforcement. Examination of the rebar that was exposed after the large sidewall shot showed that only the extra rebar layers extended into the foundation at the bottom of the sidewall. This construction detail was added to the AUTODYN model by fixing the $x$, $y$, and $z$ coordinates of only the extra layer of rebar which extended into the foundation.
All structural subgrids were joined at their nodes to create a monolithic structure. The nodes along the back edges of the sidewall and ceiling, and on the ceiling side where the adjacent wall would be, were constrained in the $x$, $y$, and $z$ directions to simulate the constraints of the back and adjacent walls. At the bottom of the wall, the short rebar coming out of the foundation was given a boundary condition, while the longer rebar that runs through the entire wall was allowed free movement. This boundary condition simulated the actual building construction, in which half the rebar running through the wall ended at the foundation.

**Figure 13.** Layout of reinforcing bars in the 3-D model.
Figure 14. Rebar cage of 3-D model with all boundary conditions highlighted in orange.

To help minimize the computational grid size, the floor, back wall, and adjacent wall were not included in the structural model. The floor was not included because predicting damage to the floor was of little or no interest to this study. The back wall has a smaller area than the sidewalls and is fully constrained on all four edges. Thus, this bay element is expected to withstand much higher blast loadings before failing. The adjacent wall is six feet shorter than the abutting wall, so it does not have as much drag area and can be expected to withstand higher shock loads. It does however have a floor on the backside, halfway up, that couples it to the other adjacent wall of the bay pair.
The interaction of the two adjacent walls and the connecting floor might be of interest but was not studied by the author.

The Eulerian air subgrid was large enough, having a length of 27 ft (the same as the bay) and a height and width of approximately 23 ft, to contain the sidewall and ceiling. The air subgrid was composed of approximately 550,000 3.543- by 3.543- by 3.543-in. cells. The bottom node positions of the air grid corresponded to the floor level, and the adjacent wall side was even with the inner surface of the adjacent wall. A flow-out boundary condition was located along the adjacent wall edge of the air subgrid, near the blowout panels, in the location of the bay entrance from the corridor. At the blowout panel location a flow-out boundary condition was turned on at precise times in the calculation to simulate the removal of the blowout panels by the blast. (See the next section of this document for information on the blowout panel modeling.) The air subgrid was filled with air having an initial pressure of 11.172 psi and a temperature of approximately 70º F, simulating standard operating conditions within the bays. Remap files from various charge sizes and geometries were then mapped into grid at desired locations.

Vertical and horizontal fixed target arrays were set, in the air subgrid, along the back and abutting walls. A target array was also run in both the \( y \) and \( z \) directions, along the centerlines of the room, just below the ceiling. Another fixed target array was run in the lengthwise direction of the room, just below the ceiling. In addition to the fixed air subgrid targets, moving targets were also placed in the Lagrangian ceiling and wall grids at strategic locations. The main purpose of the moving targets was to record
ceiling and wall deflections, as well as the stress, strain, damage, plastic strain, and possibly other measurements of the reinforcing steel’s condition, if needed.

![AUTODYN 3D model](image)

**Figure 15.** Target locations for the 3-D bay model.

### 2.7 AUTODYN Modeling of Various Blowout Panel Configurations

A major goal of this study was to determine what modifications could be made to the blowout panels of buildings similar to Building 340 to decrease the chances of panels being blown inward by a shock emitted from an abutting bay, while not greatly increasing the structural loading of the particular bay that the accident occurred within. A series of AUTODYN models were run to help determine the effects of various blowout panel assemblies on the blast impulse imparted to the bay structure.

Simulating the case of a bay with no blowout panels present at any time was very easy. A flow-out boundary condition was imposed at the blowout panel location in the bay opening for all problem time. If a blowout panel was modeled in the problem, the velocity had to be predetermined using methods described in previous sections of
this document. Once the positions of the panel at various times had been calculated, the panels could be modeled by turning the flow-out condition on at the appropriate time to replicate the removal of the blowout panel. In the cases of the larger HE charges that were of interest in the structural response calculations, the panels would typically completely reflect the first shock and then move away by the time the second shock reached the panel location. This was modeled by allowing the first shock to hit a reflecting surface at the panel location, and then turning on a flow-out boundary condition after the first shock had been reflected.

The proposed blowout panel reinforcements would be located on the inside of the panels, and not attached in anyway. The reinforcing is assumed to be strong enough to not move in any type of blast condition.

The Type 1 blowout panel reinforcement, consisting of 2- by 6- by 3/8-in. steel columns arrayed vertically at every blowout panel seam, was modeled by removing vertical strips from the flow-out boundary condition. The reinforcement column sizing was made using calculations that were based upon incident pressures recorded outside the bay, 1 ft in front of the blowout panels, during the 75-lb test. The steel columns are 2 in. wide while the model had a node spacing of 3.543 in. To keep the proper ratio of reinforcement tubing to air, the vertical strips in the subgrid had to be cleared out of every eighth cell. This gave a spacing of 28.35 in.
Figure 16. Simulation of the blowout panel reinforcements using boundary conditions. The orange areas are flow-out boundary conditions.

The Type 1 blowout panel reinforcement model was first run for the large bay-centered charge. A reflecting condition was imposed at the blowout panel location for the first 4.5 ms, long enough for the first shock to strike the panel and reflect off. The modified flow-out boundary condition, with vertical strips of cleared material, was then imposed upon the blowout panel location for the rest of the computation. These results were then compared against those obtained from the existing and Type 2 blowout panels.
3.0 COLLECTION OF TEST DATA

3.1 Recording of Pressure Gage Data

All JCAT pressure and acceleration data was collected by Science Applications International Corporation (SAIC). Incident pressures were recorded using PCB Piezoelectric type 101A, 102A, 102M, and 106B pressure gages mounted in torpedo tubes. Reflected pressures were also recorded using these gages, which were screwed into custom-made nylon mounts that threaded into a shock-absorbing mount that was held in a countersunk hole with epoxy. All exposed surfaces of the reflected pressure gage mount assemblies were flush with the surrounding wall, column, or ceiling surface. For the 25-lb and larger shots, the reflected pressure gages were coated with a thin layer of vacuum grease and metal foil to provide protection from thermal radiation.

Figure 17. Reflected pressure gage mounting assembly.
The data from all gages traveled through approximately 600 ft of coaxial cable before reaching a PCB model 584 signal conditioner. A 32-channel National Instruments PXI-1000 digitizer collected incoming data for 250 ms at a sampling rate of 200,000 counts/s. A Tektronix TDS 210 oscilloscope was used to view test signals during the setup and checkout of the gages prior to each shot. After each shot, test data was printed on-site and saved to flash drives. SAIC transferred the data over to the Laboratory in the form of both binary and text files. The author imported text file information into Microsoft Excel to perform calculations of impulse, timing, etc.

3.2 Recording of Ceiling and Wall Acceleration Data

Displacement data was collected using PCB model 305A02, 305A03, and 352N50 accelerometers. Although other types of gages such as linearly variable displacement transducers would have been more appropriate, the piezoelectric gages were compatible with the existing electronics being used on the test series to capture pressure gage data.

One problem associated with using accelerometers is unwanted high-frequency vibrations and shocks. Once the blast wave strikes any part of the concrete bay, high-speed shock and sound waves will race through the concrete ahead of the air shock moving through the bay air. These waves will reflect off concrete/air and concrete/rebar interfaces, creating a large amount of high-frequency noise that will obscure the long period, low-acceleration structural vibrations that are desired. However, the high-frequency vibrations have a much smaller amplitude than the low-frequency structural vibrations. The high-frequency vibrations can be easily filtered out using electronic, software, and mechanical filtering without affecting the desired measurements.
Personnel involved with the test had obtained decent displacement data on a similar project in the past by isolating accelerometers using a soft rubber mount, similar to the types that are used for vibration isolation mounts in machines.

The first type of accelerometer mount assembly is shown below in Figure 18. The spring constant of this type of mount was found to be far too stiff, resulting in very noisy data.

![Figure 18. Type 1 accelerometer mounting assembly.](image)

After the first accelerometer mounting configuration was found to work poorly, a soft rubber machine mount from McMaster-Carr (PN 4403K13) was used. The mount was 0.75 in. high and 1.00 in. in diameter. The rubber hardness was shore A 30. The accelerometer was screwed into the top of the soft rubber mount. Both the mount and accelerometer were then screwed onto the top of a second Shore A 70 rubber block. This type of mounting configuration was found to act as a reasonable mechanical filter.
Figure 19. Type 2 accelerometer mount assembly.

The raw data collected from the Type 2 accelerometer configuration was clean enough that no electronic wave smoothing was required. The data was then imported directly into Microsoft Excel.

3.3 **Video and High-Speed Photography**

Parker, of Engineering Sciences and Applications–Weapons Response (ESA-WR), set up several 30 frame/s JVC color video cameras on each shot. Depending on the shot, these cameras were oriented to capture the fireball leaving the bay, and blowout panel response if applicable, from head-on and side-on views. On the smaller shots, these cameras were pointed directly at the charge, so that the fireball expansion within the bay could be seen. Mirrors were used to allow the camera to be positioned behind protective cover. The cameras were standard home video cameras that were started several minutes before the test. The camera pictures were then recovered after the test. The images they captured were randomly timed with respect to the detonation time.
High-speed photography of all test shots was also performed. The frame rate varied between 300 and 3000 images/s. A Hycam II camera was used to record all high-speed film sequences. Typically two 2000 or 3000 frame/s high-speed film cameras were used on each shot. On all larger shots, a high-speed camera was always set up outside the building to capture the fireball and blowout panel behavior. On the 25- and 75-lb tests a high-speed camera was set directly on top of the building, looking down at the bay openings. The data from this camera was used to determine the blowout panel velocities. In addition to the roof camera movies, shots of fireballs and blowout panels were taken inside the building on several of the tests.
4.0 COMPARISON OF TEST DATA AND NUMERICAL PREDICTIONS

4.1 Verification of AUTODYN 2-D Calculations for a 2.2-lb TNT Sphere

The cylindrical shape of the test charges made the axial symmetric 2-D remap facility a natural choice for modeling the detonation phase of the problem. To verify the JWL equation of state and Eulerian grid processor used in the AUTODYN calculations, the blast pressures created by a 2.197-lb sphere of center-initiated TNT were calculated using AUTODYN version 4.3. The pressure predictions for points in the surrounding air were checked against experimental values published in Kingerly and Bulmash (1984) and against values predicted by the Department of Defense Explosive Safety Board (DDESB) Blast Effects Calculator.

The spherical charge problem was run using several different grid sizes. A grid size of 0.197 in. was found to work well during the detonation phase of the remap, but a large void would occur about 1100 computational cycles into the problem, at which time the shock wave had expanded to a diameter of approximately 9.8 ft. The grid size was then switched to 0.264 in. and the calculation was rerun. The 0.264-in. grid was found to work better. However, a void would still form. Eventually a grid size of 0.394 in. was determined to be small enough to capture the details of the shock propagation while remaining numerically stable. Figure 20 shows the results of an AUTODYN calculation using a 0.197-in. grid size. The computational grid was 3.3 ft high and 13.1-ft long. In Figure 20, the grid has been mirrored about its lower edge, the axis of cylindrical symmetry. In this particular calculation, the upper axis was given a reflecting boundary condition. Other calculations were run with a flow-out boundary
condition along the top edge. The boundary condition used did not affect the initial
shock pressures recorded by various targets, unless the target was far enough from the
charge to allow a mach stem to form and grow large enough to overtake the shock front
at the target’s particular height above the floor.

Upon receiving AUTODYN version 5.0, another series of 2-D calculations was
run. The newer version of AUTODYN did not have any problems with void formation
when a 0.197-in. Eulerian grid was used.

Figure 20. Formation of large void area in a 2-D axial symmetric calculation.

A series of targets were set up to record the pressure histories at distances
ranging from 7.9 in. to 13.12 ft, as shown in Figure 21. The targets were located both on
and off of the axis of symmetry to see if AUTODYN had any problems dealing with
this region.
As expected, AUTODYN predicted peak blast pressures that were slightly lower than those given by the experimentally derived data, as shown in Figure 22. This is most likely due to the small amount of shock smoothing performed by AUTODYN using artificial viscosity to ensure numerical stability. (See Appendix B for information on artificial viscosity.) Note that all the peak pressure values converge at distances of over 2.5 m. A similar trend was discovered in the time of arrival data. Time of arrival predictions between AUTODYN, DDESB, and Kingerly and Bulmash close to the charge are presented in Figure 22.
Figure 22. AUTODYN verification calculation comparing peak incident pressures for a 2.2-lb charge and a 0.39 in. grid at 0.76 bar air pressure.

Figure 23. AUTODYN verification calculation comparing shock times of arrival for a 2.2-lb spherical TNT charge and a 0.39 in. grid at 0.76 bar air pressure.

4.2 Comparison of AUTODYN Cylindrical Charge Calculations to Data

Once AUTODYN 2-D was found to obtain fairly accurate results for spherical charge geometries, calculations were performed to check the accuracy of AUTODYN 2-D predictions for the shock behavior associated with cylindrical charge geometries.
These calculations were performed for two reasons. First, 2-D, axially symmetric shock profiles were required for remapping into the 3-D Building 340 models. Second, these calculations were used to check AUTODYN pressure predictions against those of Blast-X and the tabular values given in the Pantex Manual.

**Figure 24.** Target locations for the AUTODYN cylindrical charge models.

**Figure 25.** Blast-X target array for investigation of cylindrical charge pressures.
A Blast-X calculation was run for a 3-kg cylinder of C-4 to provide an additional pressure data source. The tabulated data for an Energetic and Materials Research and Testing Center (EMRTC) cylinder was used by Blast-X to determine pressures. The blue squares in Figure 25 represent the 24 target locations used in the calculation. The cylindrical charge location is indicated by the red square. All targets were located at a radius of 12.96 ft from the charge. The entire assembly was placed in a 50- by 50- by 50-ft room to ensure that no reflected shock pressures were recorded by the targets. The calculation was run to 20 ms. The room was filled with air having a temperature of 70º F and a pressure of 11.172 psi. The results from the Blast-X calculation are plotted in Figure 26 against predictions made by AUTODYN and values given in the Pantex Manual. The Blast-X code gave a maximum pressure of 93 psi at a theta of zero degrees.

![Figure 26. Comparison of AUTODYN, Blast-X, and Pantex Manual curve predictions for a 6.6 lb C-4 cylinder, L/D = 1.](image-url)
The AUTODYN data plotted in Figure 26 should match the red plot of the empirical data for a $Z$ of 8.31. AUTODYN is close between $60^\circ$ and $180^\circ$, but misses the high-pressure region opposite the detonator location. Calculations performed for spherical charges show that AUTODYN puts the correct amount of energy into the problem. It should because each cell in the grid that is filled with HE is programmed to input a given amount of energy into the problem when the HE detonates. This energy corresponds to the heat of detonation for the particular explosive being modeled and can be determined by experiment. Extensive tables of heat of detonation values for many common explosives are available from different sources. Century Dynamics (2003) uses the values given in the Lawrence Livermore National Laboratory (LLNL) Explosives Handbook. Once the heat of detonation is known, programming the computational algorithm to release this energy is simple and exact. Energy is conserved in the hydrodynamic calculations, so AUTODYN will eventually load the structure with the correct shock impulse.

The region between $0^\circ$ and $30^\circ$ was pointed at the floor in all computations in this report. The structural loading process involved the repeated collision of reflected shock waves with the bay’s walls and ceiling and a quasi-static loading period. During this process the shock energy released by the charge will be mixed up, so small discrepancies in the initial energy distribution will not affect the final structural response. This was especially true in the models run in this study because the region of greatest initial pressure inconsistencies was always pointed at the floor.
4.3 Comparison of AUTODYN Blast Predictions and JCAT Pressure Data

The final and most important check on the accuracy of the AUTODYN pressure calculations was to compare them against actual data collected from the JCAT series. At the time of writing, data from the 25-lb, 75-lb, and 100-lb bay-centered shots was available for comparison. The pressures recorded from a bay-centered 75-lb HE charge are plotted against the predictions of AUTODYN 2-D and 3-D in Figure 27. The target point was in the center of the back wall, 3.3 ft off the floor. The charge height was also 3.3 ft off the floor. A comparison of AUTODYN pressure predictions and the test data is presented in Figure 27.

AUTODYN 2-D predicts the timing and intensity of the initial shock, but misses the late time data because only axial or planar geometry can be adequately used. As expected, the large cell size required by AUTODYN 3-D causes the peak pressure predictions to be low (see Appendix B). The timing is also retarded because the lower pressure shock waves travel slower than the corresponding real shocks. However, AUTODYN 3-D does well at predicting the arrival of reflected shocks, but tends to predict large static pressures after 12 ms.

The plots in Figure 27 show excellent agreement between AUTODYN 3-D and the test data, but this was not always the case. Comparison of the blast impulse as a function of time for the same gage in Figure 28 shows that the AUTODYN 3-D timing always lags 1 to 2 ms behind, but it does give accurate total impulse values. The large jump in the AUTODYN impulse data after 16 ms is due to the static pressure prediction inaccuracies mentioned above. Even though AUTODYN 3-D does not always make
accurate pressure predictions, it does match the 75-lb shot impulse data fairly closely. Total blast impulse is important because it has the greatest effect on structural response of all shock parameters. Therefore, based on AUTODYN 3-D’s ability to accurately predict blast impulse, it is believed that the program may be used with confidence when calculating structural blast loads.

As mentioned earlier in the section on cylindrical charge calculation, Section 4.2, AUTODYN makes low predictions for the initial shock pressures directly above the charge. During testing, the measured shock pressures directly above the charge were much higher than those predicted. However, in the case of larger charge sizes, the gas impulse contributes much more to the total blast impulse than the shock impulse does. The gas impulse loading is fairly uniform on all interior surfaces of the bay, so the large spike in pressure directly above the charge can be neglected.

![Figure 27. Comparison of AUTODYN back wall reflected pressure predictions and test data for a 75-lb bay-centered shot (RP3).]
4.4 Effect of Charge Density on Structural Loading AUTODYN Calculations

The high performance HE charges that ESA-MEE typically presses and machines have a density at or near the explosive’s theoretical maximum density (TMD). Accordingly, calculations performed using AUTODYN 3-D were made with PBX-9501 cylinders having a density of 0.064 lb/in.$^3$, the explosive’s TMD. All shots on the JCAT test series, except for the largest tests, were conducted with cylinders pressed nearly to TMD. Pressing the bulky cylinders of the largest test charges would have been problematic and possibly unsafe for ESA-MEE. To avoid having to press and handle big cylinders, the 2 large charges were fabricated by filling cardboard tubes, of the type used as forms for concrete columns, with PBX-9501 prills, that were then hand tamped to a density of approximately 0.036 lb/in.$^3$. A 25-lb pressed cylinder of PBX-9501, having a density of 0.064 lb/in.$^3$, was located in the center of the charge to ensure full detonation of the lower-density cylinder. The top of the smaller cylinder stuck out approximately 1 in. above the surrounding tamped prills. The smaller cylinder was detonated at the center of the top surface.

Figure 28. Comparison of AUTODYN back wall impulse predictions and test data for a 75-lb bay-centered shot (RP3).
The change in density will change the Chapman-Jouget (CJ) steady-state
detonation pressure and detonation velocity within the charge itself (Fickett and Davis,
1979). However, the peak pressures and blast impulses seen by the bay walls and
ceiling should not change because the total amount of energy expended by the charge
remains relatively constant. It was not known that the lower-density charge
configuration would be used in the JCAT series until late June of 2004, at which point
many simulation models for the larger HE charges had already been run on
AUTODYN. Furthermore, it was felt that the consistency provided by a series of
calculations based on TMD charges was more desirable than an exact match of test
charges because there would be one less parameter to model, assuming that the
modeling results still matched the test data. Also, ESA-MEE typically works with high-
density charges and the AUTODYN material library contains JWL equation-of-state
parameters for PBX-9501 at TMD only.

To ensure that the use of a TMD cylinder in the AUTODYN model was going to
produce accurate results, a large bay-centered shot in a narrow bay was run using a
TMD and a 0.036 lb/in.\(^3\) cylinder. The lower-density, large charge model was built and
run in AUTODYN 2-D by Price (2004), who recalculated the new JWL parameters
needed to model the PBX-9501 detonation at the lower density. Price’s results were
then mapped into the author’s 3-D model and run out to a time of 20 ms. The final
results of the 3-D model are compared in Figures 29, 30, and 31.
Figure 29. Comparison of reflected ceiling pressures between two large bay-centered charges having differing densities of 0.036 and 0.064 lb/in.\(^3\), respectively.

Figure 30. Comparison of reflected abutting wall pressures between two large bay-centered charges having differing densities of 0.036 and 0.064 lb/in.\(^3\), respectively.
Figure 31. Comparison of ceiling impulses between two large bay-centered charges having differing densities of 0.036 and 0.064 lb/in.³, respectively.

The plots clearly show that AUTODYN predicts a higher peak pressure for the high-density charge, and a nearly equal impulse for the two charges. There is a slight discrepancy between ceiling impulses created by the two charges. Examination of impulses recorded by targets at other locations in the model revealed that the higher-density charge impulse was sometimes slightly lower than the model 116 impulse, indicating that the amount of energy scattered throughout the room by both charges is likely equal in magnitude, but not in distribution. The important point is that all gages examined showed a difference in recorded impulse of 10% or less between the low-density and high-density charges. This slight discrepancy may be safely ignored when calculating structural response.

4.5 Blowout Panel Velocity Calculations and Measurements

The primary factor affecting blowout panel venting performance is the initial panel velocity. A high initial panel velocity is desirable because a large vent area is quickly created as the panels leave their initial position.
As detailed in Section 2.2 “Determination of Blowout Panel Velocities,” all velocity calculations were performed with the assumption that no gaps formed between individual panels. As expected, the panels came off the building as a sheet, with little space between individual panels. In Figure 32 the blowout panels are shown moving away from a bay in which a 5-lb charge has just been detonated. The photograph was taken 40 ms after the detonation. The panels are clearly moving away from the building in a single sheet. The two large smoke clouds are detonation products that have escaped the bay by passing through two vents, located at the top and bottom of the panels.

![Image of blowout panels moving away from the building](image)

**Figure 32.** Blowout panel location 80 ms after a bay-centered, 5-lb shot.

Initially, a small free-air AUTODYN model of the Type 1 blowout panel reinforcement configuration was created that showed that the vertical reinforcements tend to create unequal load distributions on the blowout panels if the shock wave strikes the panels at an oblique angle. This type of behavior matches intuition. The reinforcement beam closest to the charge will shield some areas of the panel while the
other side of the panel will initially see high pressures because the far stiffener will reflect a portion of the shock wave back onto the panel as depicted in Figure 33.

The author felt that this type of unequal panel loading might be potentially useful because an initial rotation could be imparted on each individual blowout panel. The panels are not held together well at their seams, so a series of rotating panels might create gaps that would allow a reflected wave, and possibly even the initial shock wave, to vent at an earlier time. The rotating panels would also create a broken surface, which would act as a poor shock reflector, helping to dissipate energy.

A large model, as described in Section 2.2, was set up to investigate the behavior of the blowout panels with the Type 1 vertical stiffeners in front of them.

![Figure 33. Type 1 reinforcements creating low- and high-pressure regions on the blowout panels.](image)

The uneven loading of the blowout panels by the initial shock is clearly evident in the right-hand plot of Figure 34. For comparison, the existing blowout panels, with no reinforcements, are shown on the left side of the figure. The panels without the reinforcement receive a much more even pressure and impulse distribution, causing
them to leave their position as a smooth sheet. Figure 35 shows both blowout panel configurations 5.2 ms after detonation. The existing blowout panel configuration, on the left side of the figure, is still holding together as one piece. However, the Type 1 reinforcement modifications have imparted a nonuniform shock loading distribution into the panels, causing them to break apart and form gaps between individual panels. Unfortunately, the gaps do not form quite fast enough to vent the first shock. Additionally, in the case of the large bay-centered shot, the wave off the back wall does not reach the panels until 8.1 ms after detonation, at which point the panels are 5 ft or more from their original position.

![Pressure plots at 1.3 ms for a bay-centered shot with existing panels and Type 1 reinforcements.](image)

**Figure 34.** Pressure plots at 1.3 ms for a bay-centered shot with existing panels and Type 1 reinforcements.
Figure 35. Air pressures at 5.2 ms during a bay-centered shot with existing and Type 1 blowout panel locations.

The 2-D model was run again, this time for a charge located 3.3 ft from the abutting and back walls. Most processing operations are likely to take place near a wall or corner of the room, so this model is a better representation of real operating conditions than the bay-centered model. Additionally, having the charge near the back wall decreases the time of arrival between the initial shock wave and the shock reflected off the back wall at the blowout panels. This would allow the second shock to actually bounce off the panels, so any differences resulting from the rotating panels could be observed.

The results of the corner shot calculations are shown in Figure 36. The pressures are plotted 4.0 ms after detonation. Note that the predicted pressures are higher than the expected pressures in the actual room because the planar model geometry does not
allow the shock wave to expand in three dimensions. The bay-centered plots are more accurate because the explosion could be allowed to expand further in the wedge grid before being remapped into the planar grid. However, because all conditions were identical between the two corner charge models, up until the time at which the shock wave hits the blowout panels, the models are accurate enough for a comparison.

At 4.0 ms after detonation, the gas from the second shock completely bounces off the existing panel, shown on the left, but partially vents through the Type 1 modified panels. Unfortunately, the reinforcements cause a smaller portion of the shock wave to be transmitted to the panels, causing them to initially move off slower, and therefore reflect a greater portion of the shock wave energy back into the room. A comparison of the impulses recorded by a series of targets stretching across the room, 5 ft in front of the panels, was made. The average incident pressure impulse recorded by the target array with the existing panels was 2410 psi*ms while the targets in front of the Type 1 panel arrangement recorded an average impulse of 2650 psi*ms. The loss of pressure from early shock venting, due to the rotating panels, was apparently not great enough to overcome the additional overpressure from the shock reflection off the reinforcement beam surfaces.
Figure 36. Pressures at 4.0 ms after a large corner shot.

Figure 37. Pressures recorded on a large corner test by a target array 5 ft inside the blowout panels with no reinforcements.

The 2-D, large-charge blowout panel velocity calculation was rerun in axial symmetry to obtain an accurate prediction of the panel velocities. This type of
symmetry is much more accurate than planar symmetry because AUTODYN accounts for dispersion in the third dimension as the shock moves away from the axis of symmetry. The AUTODYN pressure predictions are plotted at 1.00- and 2.15-ms in Figure 38. At 1.00 ms the shock front is just about to strike the blowout panels where they meet the floor. The bottom edge of the plot represents the floor location, the right edge of the plot is the axis of symmetry, and a flow-out boundary condition, indicated by the beige lines, was imposed upon the upper and left sides of the grid. At 2.15 ms the shock has almost completely reflected off the blowout panels, which are just about to leave the computational grid.

![Figure 38. Side-on view of interaction of the large charge shock and the blowout panels.](image-url)
A series of targets were embedded in the blowout panel to capture the panel’s velocities, which are plotted below in Figure 39. The targets were spaced 7.87 in. apart. The first areas of the panels to move are those closest to the floor, which also obtain the highest velocities. Other parts of the panels follow the bottom section. The average predicted velocity for the panels is about 900 ft/s.

![Figure 39. Blowout panel velocities from a large bay-centered charge 3.3 ft off the floor, as predicted by AUTODYN 2-D.](image)

The velocities imparted to the blowout panels by the large charge were determined from the 3-D models by recording pressure data along a horizontal target array on the rear wall, 6.6 ft above the floor. Both the bay-centered and sidewall shots were located at the lengthwise midpoint of the room. The panel velocities were calculated from the impulse data and then integrated to determine the panel position as a function of time. Panel positions for a large sidewall shot are plotted in Figure 40. The panels on the charge side of the bay leave their positions first, and obtain the highest
velocities. The charge side of the bay will start to have a significant vent area at 3.00 ms, when a gap of just over 2 ft will be created between the panel and the wall. Venting will not occur on the far side of the panels until 7 or 8 ms after the shot.

The panel positions calculated using the impulses obtained from the 3-D AUTODYN model are in fair agreement with the panel positions predicted by AUTODYN 2-D. This increases confidence in the accuracy of the AUTODYN 3-D impulse predictions and the method of velocity computation used by the author.

![Graph showing blowout panel positions](image)

**Figure 40.** 3-D model of narrow bay blowout panel positions of a large sidewall shot as a function of time (calculated using targets 6.6 ft above the floor).

### 4.6 Measured Pressures at the Abutting Bay Panels for the 75-lb Test

From a Building 260 surety standpoint, one of the most important shots of the test series was the 75-lb bay-centered shot with undamaged blowout panels. Several cameras were set up to capture exterior images of the blowout panels leaving their positions. A high-speed camera was positioned on the parapet of Bay 103, directly above the panels, looking downward, to record the panel behavior at 2000 frames/s.
An additional camera was set up in Bay 104, which abuts the test bay. This camera filmed the abutting bay blowout panels from inside the bay at 300 frames/s.

The locations of some of the pressure gages used for this 75-lb shot are shown in Figure 41. Reflected pressure gages are labeled RP and are shown in green. Incident pressure gages are labeled IP and are in red. The IP gages have an arrow corresponding to the direction that the gage faced. The camera in the interior of the abutting bay was located at the end of the bay entrance hall and was aligned parallel to the blowout panels, looking towards the charge. A photometric grid was marked out on the abutting wall to measure the blowout panel displacement. The blowout panel doors are shown in the open position in the figure for clarity, but were closed during the test. The charge was located in the center of the bay, 3.3 ft off the floor. All reflected and incident pressure gages were also 3.3 ft above the floor level. A reflected pressure gage, located in the ceiling directly above the charge, is not shown in Figure 41.

**Figure 41.** Instrumentation map for the 75-lb bay-centered test.
The pressures recorded by the three gages located directly in front of the abutting bay blowout panels are plotted in Figure 42. The shock wave clearly loses intensity as it moves down the loading dock. IP-8, the gage nearest the explosion, records a long duration negative pressure phase after 30 ms that the other gages do not capture. This negative pressure region is of concern because it will deplete the air from the abutting bay, possibly injuring any occupants. However, the only way to prevent this from happening is to restrict the outward movement of the abutting panels, which would obviously hamper their effectiveness if an explosion were to occur in the bay.

Fortunately, the pressures recorded inside the abutting bay, as plotted in Figure 43 by the red trace, do not appear to drop more than a few psi below ambient. This means that the personnel within the bay will not be exposed to a high-vacuum environment, even if the fireball from an accident involving a 75-lb charge is depleting the abutting bay of air.

**Figure 42.** Incident pressures in front of the abutting bay panels during a 75-lb bay-centered shot.
Figure 43. Incident pressures inside and outside the abutting bay blowout panels during a 75-lb bay-centered shot.

The expected blowout panel motion can be determined using the pressure versus time data given in Figure 42. The blowout panel velocities were calculated with the assumption that the panels experience the same reflected pressures that were recorded as incident pressures by IP-8, which was 1 ft in front of the panels. The calculated panel positions were compared against those recorded by a 300 frame/s camera. The calculated positions are plotted in Figure 44. Three of the frames from this camera are shown in Figure 45. The leading edge of the blowout panel, highlighted by the blue line in Figure 45, could be seen clearly when the consecutive frames were scanned through quickly. A 12-in. photometric grid was taped onto the abutting wall. This grid has been highlighted with black lines. The results, plotted in red in Figure 44, show that the blowout panel calculations underestimated the initial panel motion. The positive incident pressures were doubled to account for shock reflection and the calculation was rerun. The results from this second calculation are plotted in light blue in Figure 44. The results of a third calculation, in which a reflection factor of 1.7 was used, are given by the green trace. This trace matches the actual maximum panel deflection closely, but does predict a high panel velocity.
Figure 44. Calculated and measured abutting bay inner blowout panel positions as functions of time during a 75-lb bay-centered test.

The plots in Figure 9 indicate that a 10-psi shock wave striking a surface at an angle of 60° from perpendicular will create reflected pressures that are almost twice as high as the initial peak pressures. The multiplier of 1.7 gives results even closer to the measured panel motion. It can be seen from Figure 45 that only the panels nearest the bay containing the charge move. This same abutting bay panel behavior was observed on Building 260 after the accident in 1959 and may be a result of shock waves refracting around the abutting wall and striking the nearest panels at a high angle. As the shock moves down the loading ramp it straightens out and moves perpendicular to the blowout panels. The panels near the center of the bay do not see as high a pressure load, so using a multiplier of 1.7 will give a conservative loading.
4.7 **Blowout Panel Reinforcement Sizing Calculations**

The information acquired from the 75-lb, bay-centered test by the incident pressure gages and blowout panel photography was used to determine the reinforcement beam sizes required to prevent the panels from being displaced inward. The use of vertical stringers to induce panel rotation had already been deemed ineffective and eliminated. Horizontal reinforcement beams were found to create lower static pressures within the bay because they had a smaller combined surface area when compared with the vertical reinforcements. In addition, the horizontal reinforcements would be easier to install than the vertical reinforcements. Because of these characteristics, horizontal beams were chosen for further study.

The blowout panel reinforcement beams were sized using the pressure data collected by IP-8, the gage 1 ft in front of the abutting bay blowout panels on the side of the bay closest to the explosion. There were two other gages in front of the abutting bay panels, but further from the bay in which the explosion took place. The pressures recorded by the gage nearest the explosion were used because they were the highest and would give the most conservative beam size. Based on the motion of the panels
captured by the high-speed camera, the incident pressure trace was multiplied by a “reflection factor” or multiplier of 1.7.

The loading was approximated by a triangular pulse having an amplitude of 10 psi and a time duration of 14 ms. Each horizontal panel reinforcement beam was required to support a strip of blowout panels 6 ft wide. After checking that the blowout panels could support the expected blast loading over the 6-ft span between reinforcement beams, the reinforcement beams were sized.

The beams are supposed to withstand the overpressures from a 25-lb explosion in an abutting bay. However, the gages stationed in front of the abutting bay panels on the 25-lb test did not record any data. Therefore, the data from the 75-lb shot was scaled down to a 25-lb equivalent using cube root scaling laws. The result was a peak overpressure of 7 psi.

Using this data and modal analysis techniques, a 4- by 12- by 0.50-in. rectangular structural steel tube with 4- by 0.75- bars welded to both 4-in.-wide surfaces was found to be sufficiently strong to withstand the external shock loading on the panels. The beam was oriented with its largest section modulus resisting the horizontal motion of the panels. This beam would have a second moment of area in the strongest direction of 449 in.\(^4\) and a weight of 74.5 lb/ft. In an accident, the beam might suffer some permanent deflection, but would not buckle, thus saving any occupants in the abutting bay. The beam should be composed of structural steel having a yield strength of 38 ksi or greater. The largest wall thickness commonly available in this size of rectangular tubing is 0.50 in. The thick-walled tube was selected because it provided the largest bending resistance in the smallest cross-sectional area. Welding the 4- by
0.75-in. bars to the sides of the beam would further increase the strength while keeping the profile small. Adequate connections to the existing bay walls must be included on both ends of all reinforcement beams.

The factor of safety with respect to material yielding for the selected reinforcement beam is fairly low at 1.18. However, the shock wave profile used to size these beams was collected by the gage nearest the explosion, but the pressure will decrease by the time the wave reaches the center of the span where it will induce the greatest bending moment in the reinforcement beams.

Alternatively, a W16 by 58 I-beam could be used. This beam has a second moment of area of 748 in.$^4$ The higher moment is required because this beam, which is lighter, will have a higher natural frequency than the other beam, so the time duration of the shock loading will be closer to the beam’s natural period. Although the I-beam has a higher drag profile than the 12- by 4-in. rectangular tube, it would be lighter and does not require any fabrication.

A 5- by 3- by 0.375-in. rectangular steel tube will be adequate for the sill and top beams, which prevent the ends of the panels from moving inward. The bending moment resistance of these beams does not need to be very high but they must be securely fastened to the concrete at regular intervals. The sill and top beam should be oriented with the 5-in.-wide face against the panels to provide as much protection as possible against the blowout panel ends rising up over the sill beams.

4.8 Results of Calculations Made Using the Pantex Manual Procedure

Preliminary calculations of the HE charge size required to fail the ceiling were made using the techniques given in DOE’s Pantex Manual. The purpose of these
calculations was to provide a baseline with which to compare the AUTODYN 3-D calculations.

The first step in determining the charge size required to yield the ceiling was to calculate the maximum static pressure load that the ceiling could support using yield-line analysis. The failure pressures for both uniform pressure and nonuniform, “center-biased” pressure were calculated. Yield-line analysis is typically used to design structures that must withstand roof, snow, or other fairly uniform loadings. However, the JCAT test results and AUTODYN models have shown that a bay-centered cylindrical charge preferentially loads the center of the ceiling. Therefore, failure pressures for this center-biased loading were also calculated. The center-biased failure pressures are given in terms of the average pressure subjected to the ceiling.

In yield-line analysis, failure criteria are based on the amount of work required to yield the slab. The work imparted on a particular section of slab by a load, defined as the product of force and distance, is dependent on the location of the centroid of the load because the amount of vertical displacement the load pushes the slab through depends on this location. The slab is assumed to act as a hinged mechanism which folds downward at the slab edges; therefore, the distance between the edge of the slab and the load’s centroid is very important. The centroid of a uniform load acting on a triangular plate will always be one-third the height from the base, but the exact location of the centroid of a nonuniformly loaded plate must be calculated. The following formula was used (Bradley and Smith, 1995):

\[
(x, y) = \left( \frac{M_{xz}}{l}, \frac{M_{yz}}{l} \right),
\]

(4.1)
in which $l$ is the total load, and $M_{xz}$ and $M_{yz}$ are the moments about the $xz$- and $yz$-planes, given by the expressions

$$M_{xz} = \iint_R yP(x, y, z) dVol$$

and

$$M_{yz} = \iint_R xP(x, y, z) dVol.$$  \hspace{1cm} (4.2)

The total load, $l$, is determined by integrating the volume under the entire pressure loading curve. Blast-X (Blast-X, 2001) was used to obtain a profile for the load distribution because Blast-X was found to provide impulse distributions that were closest to the test data.

A series of targets was set up in a line on the ceiling in the Blast-X model and the impulses from a large bay-centered charge, integrated to 20 ms, were recorded by each target and noted. The impulse values versus target locations were plotted with Microsoft Excel and a curve fit was created. The curve fit was very poor because of the sharp peak in the center of the data. In an attempt to improve the fit, the targets from only one half of the room were plotted and a second curve fit was created that gave much better results. The equation of this curve was mapped into a 2-D array in Microsoft Excel in such a way as to simulate rotating the equation around a vertical axis located at the center of the room. The resulting surface of revolution is plotted in Figure 46. The area under this curve was divided into sections representing the different regions of slab deformation after yielding had occurred. The centroid of the load on each section was then found by numerically integrating under the curve. This calculation showed that the centroid of a center-biased load is approximately in the center of a triangular-shaped piece of slab. As a result, a given average pressure from a
center-biased load does more work than an equivalent uniform load pressure because the centroid is further from the edge of the wall, where the slab section undergoes more vertical movement.

![Graph showing total impulse versus total blast duration with different load pressures and time intervals.

Figure 46. Ceiling blast loading for a large bay-centered charge generated using Blast-X data. Curve fit is rotated about charge location.

Once the centroids for the nonuniform loading had been determined, yield-line analysis of the ceiling was carried out. The ceiling was modeled as having both three and four fixed edges. The results are listed in Table 1.

Table 1. Static Ceiling Failure Loads for the Uniform and Center-biased Loads

<table>
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<tr>
<th>Clamped on 3 edges</th>
<th>Uniform Static Failure Load (psi)</th>
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<tr>
<td>Uniform Load in 20 ft Bay</td>
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<tr>
<td>Uniform Load in 24 ft Bay</td>
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<td>Center Biased Load in 20 ft Bay</td>
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<td>Center Biased Load in 24 ft Bay</td>
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<table>
<thead>
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<th>Clamped on 4 edges</th>
<th>Average Static Failure Load (psi)</th>
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</thead>
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<td>Center Biased Load in 24 ft Bay</td>
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The ceiling of the narrow bay can support 25% to 30% higher loads than the wider bay’s ceiling. The center-biased load that a bay-centered charge is expected to impart to the ceiling lowers the failure load by about 30%. Note that the pressures listed for the center-biased loads correspond to the average loading, not the peak loading in the center of the ceiling.

In Table 1, the slab supported on three edges can support 25% of the load that the fully supported slab can support. Having a fourth edge supported increases the load capacity of the ceiling by a factor of 4 or more. The actual bay ceiling has a parapet on the fourth edge that provides additional support but not as much as a fixed wall. Therefore, the bay ceiling will behave somewhere between these two extremes. The load capacities of the ceiling fixed on four edges and subjected to a center-biased load were used to determine if a given load would yield the ceiling.

Once the static failure load for the ceiling was determined, the equivalent static pressures for different charge sizes, locations, and panel configurations were evaluated. If the equivalent pressure for the charge was greater than the failure load, the ceiling slab would yield. The blast loading was divided into two distinct regimes: a transient shock loading and a quasi-static gas loading. The impulse from both of these loadings was determined and then summed to give a total predicted blast impulse.

The shock and gas impulses calculated using the Pantex Manual are listed in Table 1. The initial shock pressure seen by the ceiling is independent of the blowout panel type, but the strength of the first reflected shock will be highly dependent upon whether or not there are blowout panels present. The reflected shock and all subsequent shocks, plus the quasi-static gas loading, are accounted for in the gas impulse column.
The total ceiling impulse is the sum of the initial ceiling shock impulse and the gas impulse. The predicted time required for the static gas loading to vent out of the building is also listed.
Table 2. Equivalent Static Loading for Various Shots in the Bays of Building 340

a) 20-Ft-Wide Bay with Open Wall

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<th>Charge Size (lb)</th>
<th>Gas Impulse (psi*msec)</th>
<th>Ceiling Shock Impulse (psi*msec)</th>
<th>Total Ceiling Impulse (psi*msec)</th>
<th>Time Duration (msec)</th>
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b) 20-Ft-Wide Bay with Existing Panels

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c) 20-Ft-Wide Bay with Type 1 Panels

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d) 20-Ft-Wide Bay with a Solid Wall

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e) 24-Ft-Wide Bay with Open Wall

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f) 24-Ft-Wide Bay with Existing Panels

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<table>
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An equivalent static load was calculated for each charge and blowout panel configuration using modal analysis. The load profile was assumed to be triangular in shape and to have a time duration equal to the time duration listed in Table 1, if the duration was less than the natural period of the ceiling. The load profile used to approximate the shock and impulse loading is shown in Figure 47. The natural periods of the ceiling and the walls of the bay were determined by finding the modulus of elasticity for the reinforced concrete slabs and applying the following formula from Blevins (1995) for the natural frequencies of plates with three and four clamped edges:
In this formula $E$ is the modulus of elasticity, $h$ is the plate or slab thickness, $a$ is the length of the plate, $b$ is the width, $\nu^2$ is the material’s Poisson Ratio, and $\gamma$ is the mass per unit area of the plate. The dimensionless frequency parameter of rectangular plates, $\lambda_{ij}$, is listed by Blevins for all 21 possible clamping conditions and for varying length-to-width ratios. If the static load time duration was longer than the ceiling’s natural period, the natural period of the ceiling was used as the load duration in the modal analysis. The peak pressure of the assumed triangular loading profile was found by setting the area under the triangle equal to the total ceiling impulse and solving for the peak amplitude. The equivalent static load was then found by multiplying the peak pressure by the maximum gamma factor. The maximum gamma factor is the ratio of the maximum deflection of a spring mass system subjected to a dynamic forcing function and the deflection of the spring mass system when subjected to a constant force equal in magnitude to the peak amplitude of the forcing function. If the load duration was very long, as in the case of the solid wall calculations, the peak pressure of the loading triangle would be very low. This is the reason the load duration was limited to the natural period of the ceiling.
Figure 47. Typical triangular loading profile used in modal analysis structural response calculations (from DOE, 1980).

4.9 Effects of the Blowout Panels on Measured Pressures

A test having the same charge size test was not conducted with and without blowout panels on the JCAT series. Therefore, a measure of blowout panel effectiveness could only be made by comparing 75- and 100-lb test data.
Figure 48. Comparison of back wall reflected pressures recorded from test of bay-centered charges, with and without panels.

The back wall reflected pressures recorded from a 75-lb, bay-centered test with blowout panels and a 100-lb, bay-centered test without blowout panels are plotted in Figure 48. As expected, the 100-lb shot generates the highest initial pressures. However, the reflected shock off the blowout panels can only be seen in the blue, 75-lb test trace. At late times, several more reflected shocks hit the gage’s position. The impulses recorded by the same two gages are plotted in Figure 49. Again, the 100-lb HE charge generates a higher initial impulse, but the 75-lb test eventually catches up after 13 ms as reflected shocks are integrated into the curve. The Pantex Manual predicts an impulse at this point of approximately 675 psi*ms for the 100-lb shot, and 775 psi*ms for the 75-lb shot. These values compare well with the experimental data.

Both processing bays were thoroughly examined after the two shots. In both cases, the 2-ft-thick walls showed no signs of damage. A cantilever ceiling support beam extending from the adjacent wall out to the blowout panel was cracked in both cases, although the 75-lb test created more and larger cracks. No signs of ceiling
yielding could be found in either case, but the 75-lb charge created shear cracks that were visible along all edges of the ceiling, approximately 1 to 2 in. inward from the walls. Small cracks were visible in the same places after the 100-lb test, but they were not as substantial as those from the 75-lb test.

Figure 49. Comparison of back wall reflected impulses recorded from tests of bay-centered charges, with and without panels.

4.10 Calculated Effects of the Blowout Panels on Blast Impulse

Using AUTODYN, a large HE charge was modeled inside a narrow bay with no blowout panels, with the existing panels, and with the Type 1 and Type 2 reinforced panels. Pressure recordings from various points within the bay were plotted against each other to determine the effects of the panels on structural loading. Because the ceiling is the weakest portion of the bay structure, the ceiling pressures were of primary interest. These pressures are plotted in Figure 50. The same four traces were then numerically integrated to give the impulse results given in Figure 51. The pressures were identical in all four cases until 4.5 ms. At this time, the reflected shock from the panels and back wall reached the point on the ceiling directly above the charge. The presence of the blowout panels greatly increases the reflected shock pressure and impulse. After this
time, a bay with no blowout panels will quickly vent, while a bay with panels will retain a much higher portion of the initial explosion energy, in the form of quasi-static pressure.

The second shock reflection from the walls reaches the target position at 13 ms. If the panels were not initially present, this shock is almost negligible and barely registers as a green trace. If the panels were present, this second shock is fairly significant. At 14.5 ms the second reflected shock from the reinforcements reaches the gage position. The Type 1 vertical reinforcement beam arrangement gives a higher reflected shock pressure than the Type 2 reinforcement which has a smaller reinforcement surface area. Both the Type 1 and Type 2 reinforcements contain a higher static pressure than the existing panels after 15 ms.

**Figure 50.** Comparison of AUTODYN reflected ceiling pressure predictions for various blowout panel configurations for a large bay-centered shot.

Even a cursory glance at the impulses plotted in Figure 50 reveals that the additional shock reflection provided by a panel of any type greatly amplifies the impulse that the ceiling receives between 5 and 15 ms. While the ceiling receives a total
impulse of 2000 psi*ms at 30 ms without the blowout panels present, the existing panels will raise the impulse to over 3600 psi*ms. The impulse has almost been doubled by the presence of any type of reflecting surface at the bay opening. The Type 1 and Type 2 reinforcement beams raise the total impulse a little higher, to 4300 and 4400 psi*ms respectively, as compared with the existing panels. The performance difference between the Type 1 and Type 2 panels is very slight, suggesting that the panel reinforcement design should be optimized to provide the greatest reinforcement strength and ease of installation, rather than minimal surface area.

![Image](image_url)

**Figure 51.** Comparison of AUTODYN ceiling impulse predictions for various blowout panel configurations during a large bay-centered shot.

The fireball from a bay-centered large HE charge is shown in Figure 52. The blowout panels were not present during this test, and an 80-ft-diameter fireball expanded out of the bay. The white blur seen on the trees to the left of the fireball is a fire retardant foam which has been blown off the trees by the shock wave. Although much of this energy would still have escaped the bay, the blowout panels would have
reflected a large portion of it back into the bay. The parapet, which is approximately 23 ft above the loading dock, can be seen extending away from the camera position and towards the explosion. The loading dock is between 30 and 36 in. above the parking lot. The fireball later expanded another 20 ft and fully engulfed the first row of trees. If this energy were confined to the bay the wall and ceiling loads would be expected to greatly increase.

Figure 52. The external fireball of a large bay-centered explosion with no blowout panels in place (photo from JCAT test data).

4.11 Ceiling Deflection and Damage from the Large Sidewall Shot

The vertical wall displacement contours of one of the test bays were mapped before and after a large sidewall test using a transit and laser level. All vertical measurements were taken with respect to a fixed reference point located above the back
wall of an adjacent bay. Figure 53 gives permanent deflections, as measured after the
test, for various points on the roof.

The center of the bay ceiling deflected the greatest amount. However, the area
directly above the HE charge and along the sidewall also showed significant upward
movement. The area along the sidewall displayed the largest amount of shear damage.
The roof appears to have yielded in the expected trapezoidal pattern. Cracks of various
sizes were observed in the walls and ceiling. The larger cracks are shown in Figure 54
using thicker lines. Hatched areas indicate areas where the top layer of concrete had
spalled away from the roof.
Figure 53. Permanent ceiling yield created by a large sidewall test (the numbers indicate vertical permanent displacement in in.).
Figure 54. Yield-line crack patterns in the roof of the bay after a large sidewall test.

4.12 Ceiling Deflection and Damage from the Large Bay-Centered Shot

The ceiling was mapped before and after this test using a transit and the same techniques that were used with the large sidewall shot. As expected, the ceiling deflections were highest in the areas directly above the HE charge. The ceiling deflected in a fairly symmetric pattern that was not greatly influenced by the location of the bay.
entrance. The deflections along the parapet were low, indicating that the parapet acts almost as a fixed boundary condition for the ceiling. The ceiling should be expected to support loads that are closer to those of the fully constrained slab, given in Table 1. In this case, the yield pattern did not match any of the expected patterns.

**Figure 55.** Ceiling deflections created by a large, bay-centered test (the numbers indicate vertical permanent deflection in in.).
4.13 **Comparison of AUTODYN Structural Predictions with JCAT Data**

The final benchmark of the 3-D AUTODYN bay model was a comparison of structural damage predictions and data. Large bay-centered and sidewall tests were modeled using the full 3-D bay model, which was run to very late times when nearly all structural motion had ended. These were the same models used to make the pressure predictions cited in other sections of this thesis. After the models had been run to
completion, the deflections recorded by targets located along the longitudinal centerline of the model were compared with those of the actual test.

The ceiling deflections from a large bay-centered test are plotted in Figure 57. AUTODYN’s prediction is very close to the actual test data near the center of the room, but shows larger deflections at the parapet and back wall than actually occurred. AUTODYN predicted too much shear failure near the back-wall, a result that may have been caused by errors in the concrete constituent model or by rebar details that were not modeled exactly. The large parapet deflection predicted by AUTODYN is probably due to premature failure of the parapet in the simulation. Vertical stirrups, which prevent the lower and upper reinforcing bar layers from separating when 45º cracks form because of the shear load, strengthened the real parapet. These stirrups were formed from 0.25-in.-diameter wire and were too small to accurately include in the model. As a result, the parapet in the AUTODYN model separated vertically and failed at an earlier than expected time.

![Deflection vs Distance](image)

**Figure 57.** Comparison of predicted and measured ceiling centerline deflections for a large bay-centered test without panels.
The deflection predictions and results for a large sidewall HE charge test are plotted in Figure 58. The effects of the model’s low parapet strength are even more apparent. Had the simulated parapet acted more like a fixed slab support, more of the slab’s energy would have been transferred to the center of the ceiling causing greater deflection there. The predicted centerline deflections would have compared better with the measured deflections. Instead, AUTODYN predicted deflections more in-line with the expected deflections of a slab with an unsupported fourth edge.

Figure 58. Comparison of predicted and measured ceiling centerline deflections for a large sidewall test without panels.

Figure 59 shows the damage incurred by the bay as a result of the large sidewall test and the plastic strain predicted by AUTODYN 3-D for the same problem. The viewing angle is nearly identical in both pictures. AUTODYN predicted well the large amount of shearing failure that occurred along the bottom of the sidewall and near the charge. The damage and vertical shear failure of the parapet near the building corner were also predicted accurately. The model inaccurately predicts the extent and location...
of damage along the right-hand side of the ceiling slab. Most of the error at this location can be attributed to the use of a fixed boundary condition that is a poor simulation of the actual boundary conditions seen by the slab, especially along the cantilever beam. The cantilever beam would probably twist if significant ceiling deflections occurred, lowering nearby bending moments in the ceiling. Furthermore, the real parapet acted more as a fixed edge than the model parapet, lowering deflections and the associated cracking. The test photo shows cracks near the cantilever beam, but not along the parapet.

Figure 59. Comparison of actual and predicted bay damage from a large sidewall test.

4.14 AUTODYN Predictions for Structural Behavior with Panels Present

The goal of this study was to determine what effects blowout panel modifications would have on the structural response of Building 340 to internal blast loadings. After the AUTODYN predictions had been verified using the test results, the bay model could be used to model untested accident scenarios with confidence. The JCAT series was performed using a series of HE charge sizes that increased uniformly to a certain size and then jumped up to the larger charge sizes. In between these sizes
there was a range of untested sizes. AUTODYN was used to determine the structural response of the processing bays to these charge sizes. One HE charge size in particular that fell within this group of untested charge sizes was modeled and is always referred to in this document as the “medium-sized” charge.

In addition to a gap in HE charge size increments, there were other areas where incomplete data was gathered from the JCAT series. Because of safety concerns, no large HE charge tests were performed with the blowout panels in place. AUTODYN was used to estimate the increase in structural blast loading of the structure, as described in Section 4.10, and the resulting additional bay damage that the building would have incurred if the blowout panels had been in place during these tests.

The JCAT tests results and AUTODYN predictions for the bay damage resulting from a large, bay-centered charge with no blowout panels present are given in Sections 4.12 and 4.13. The measured and predicted maximum permanent deflection was approximately 3.25 in. The AUTODYN calculation was rerun with the blowout panels present, modeled by a reflecting boundary condition for the first 4.5 ms after the charge detonation. The blast impulse, which increased by 180% when the blowout panels were present, drove the ceiling’s behavior far into the plastic regime.

The results of a calculation performed in which the blowout panels are present on a large bay-centered shot are shown on the right-hand side of Figure 60. The results from the same model with no blowout panels present are plotted on the left side of the figure for comparison. The maximum deflections of the ceiling were over 3 ft. The parapet concrete underwent large amounts of plastic strain, much of which was due to model inaccuracies arising because the vertical stringers that tied the top and bottom
layers of rebar together could not be included in the model. This caused the parapet to fail prematurely and act more like a free edge than a fixed boundary.

**Figure 60.** Damage from a large, bay-centered test without (left) and with (right) blowout panels.

The posttest yield stress associated with the large, bay-centered shot with the panels present is shown at the bottom of Figure 61. The yield stress from the model having no blowout panels present is plotted in the upper portion of the figure. The yield stress in the ceiling has been increased from work hardening that the rebar steel underwent as the ceiling deflected. The top layers of rebar in the parapet are just starting to fail. The rebar at the bottom of the sidewall slab has been work hardened, but has not failed.
Figure 61. Comparison of rebar final yield strength from a large, bay-centered test without (top) and with (bottom) panels.
The results of a model simulating a large sidewall test with the blowout panels present are plotted on the right side of Figure 62. The results from the model having no blowout panels are plotted for comparison on the left. Again the parapet deflected much more than expected. The sidewall underwent 12 in. of permanent deflection in the areas closest to the charge location.

Figure 62. Structural displacements resulting from a large, sidewall test, without (left) and with (right) panels.

Yield stresses from the large sidewall shot with the panels present indicated that the majority of rebar tying the sidewall into the foundation had failed. The test performed with no blowout panels present, as well as the equivalent model, showed no rebar failure in these areas.
5.0 CONCLUSIONS

5.1 Effects of Various Panel Configurations on Structural Response

If the blowout panels in Building 340 had been reinforced with either Type 1 or Type 2 reinforcements, as compared with the existing panels only having been in place, the loading on the bay’s ceiling due to a larger HE charge exploding would increase by approximately 20%. This prediction is based on a series of AUTODYN 3-D models run in succession in which the only parameter that was changed was the reflecting boundary conditions used to simulate the different blowout panel configurations. Calculations made using this model show very little difference in the blast impulse loading of the structure between the Type 1 and Type 2 reinforcements. The calculations for the Type 2 reinforcements were performed using boundary conditions that modeled the 12- by 4-in. rectangular tubes with 4- by 0.75-in. bars welded to either 4-in. face. The W16 by 58 I-beam would be expected to have a slightly higher drag coefficient than the rectangular tube.

There was a large difference between the response of a bay with an open wall and a bay having the existing panels. Adding the existing panels raised the blast impulse at the center of the ceiling by 80% as compared to a bay with no panels. The ceiling, being thinner and having less rebar than the bay walls, is of interest because it is typically the first slab in the structure to yield. Most of the tests on the JCAT series were conducted with no blowout panels present because the panels had been removed early in the test series by small charges. Summing the 80% increase in blast impulse caused by the addition of the blowout panels and the 20% increase due to the panel
reinforcements produces approximately twice as much expected blast impulse with these features. Therefore, the results from a given test conducted with no blowout panels during the JCAT series can be used to gage the response of the building if the reinforced panels had been in place and the charge size was roughly half the test charge size.

5.2 Future Work

There are several possible extensions of this work. Using the existing bay model, the effects of positioning the HE charge in different locations can be investigated. This report focused on the structural response of the bay-centered charges. Most machining operations would likely be placed in a corner or along the side of a room, where the effects on the ceiling will be lessened. Also, only cylindrical charges with their axis of symmetry oriented vertically were modeled. If lathes or other machine tools are used in which the charge orientation varies, different charge orientations could also be modeled.

A complete AUTODYN model of a Building 260 bay could be created. Anderson’s (1969) scale model work could be used to validate the results of this model, which would provide data applicable to the Laboratory’s current operations.

During the JCAT testing, a wing configuration that extended outward from the abutting wall was tested. The purpose of the wing was to create a barrier that would lower the shock pressures that reached the outside of the abutting bay panels. This wing was found to be very effective. Price (2004) ran a 2-D calculation that showed similar behavior to the test results. A 3-D AUTODYN model could be run to investigate the behavior of this barrier.
5.3 **Building 260 Modification Recommendations**

Several changes can be made to Building 260 to increase the safety of current operations. As much work as possible should be done remotely through the use of numerically controlled computer machine tools and robots. When possible, work should be scheduled in such a way as to prevent operations from simultaneously occurring in abutting bays.

If machine operators must be present in a bay while HE processing is carried out in an abutting bay, the blowout panels should be reinforced. Although Building 260 is shorter than Building 340, it is nearly identical in many other regards. A bay-centered, 75-lb HE charge can be expected to generate pressures in front of the abutting bay panels that are slightly higher than those measured on a 75-lb, bay-centered test with blowout panels. The smaller volume of the bays in Building 260 would probably raise the outside pressures. Assuming that the pressures from a Building 260 accident would be fairly close to the 75-lb test pressures, they can be used as a guideline for determining the size of the structural tubing needed to reinforce the Building 260 panels.

A 12- by 4- by 0.50-in. structural steel tube with 4- by 0.75-in. steel bars welded to both 4-in. faces is recommended for reinforcing the blowout panels. This beam must be composed of structural steel having a yield strength of 38 ksi or more and a minimum ultimate strength of 55 ksi. A W16 by 58 I-beam may also be used. The I-beam is 400 lb lighter and no time would be required to weld the composite structural tube together. However, the I-beam would have a higher drag coefficient and would probably raise the blast loading on the building by several percent over that of the
composite tube beam. This geometry was too complicated to simulate in the 3-D model using boundary conditions only. The beam should be run horizontally, just above the door, as close as possible to the horizontal centerline of the bay opening. Adequate end support must be provided for the beam to prevent it from moving into the bay.

A 3- by 5- by 0.25-in. sill and top beam must be installed to prevent the panel ends from kicking inward. These beams should be turned so that the 5-in. sides are oriented vertically to provide the largest possible ledge for the panels to rest against. The top and sill plates must be securely fastened to the floor and ceiling at close intervals to provide sufficient lateral support.
APPENDIX A
Yield-line Analysis Calculation Procedure

Yield-line analysis is based on the observation that concrete slabs tend to act as hinged mechanisms when failing. Typically, a yield-line pattern is assumed along which the plate or slab will fail. Some common yield-line patterns for a slab supported on four sides are shown in Figure 63. Using a method based on either virtual work or static equilibrium, the exact location of the yield lines can be analytically determined. The slab will fail in the manner that requires the least amount of energy, and several possible yield-line patterns must usually be investigated. The virtual-work method was used for all calculations in this report.

Figure 63. Possible yield patterns for a rectangular slab.

There are four fundamental assumptions that are required when using yield-line analysis to predict slab failure. First, bending and twisting moments are evenly distributed along all yield lines, and the moments are equal to the maximum moment capacity of the plate. Second, the slab forms separate segments upon plastically failing. Third, the steel rebar at the yield lines is fully yielded at failure. Finally, the amount of plastic deformation occurring at the yield lines is much greater than the elastic.
deformation in the slab. This means that individual segments of the slab are assumed to act rigidly during failure.

The first step in performing a yield-line analysis is to determine the maximum allowable bending moments of the slab in both the $x$ and $y$ directions for both positive and negative moments. Normally the standard bending moment sign convention in which a positive moment creates negative deflection at the center of the slab is used.

![Figure 64. Bending moments acting on a triangular-shaped section of slab.](image)

Once the maximum moments are determined, the combined bending moment acting on an arbitrary yield line can be found using the following expressions (Wang, 1979):

\[
M_{ub} (L) = M_{ux} (L \sin \theta) \sin \theta + M_{uy} (L \cos \theta) \cos \theta \\
M_{ub} = M_{ux} \sin^2 \theta + M_{uy} \cos^2 \theta \\
M_{ub} = \frac{M_{ux} + M_{uy}}{2} - \frac{M_{ux} - M_{uy}}{2} \cos 2\theta
\]  

(A.1)

Similarly, the twisting moment can also be determined as follows:
\[ M_{ut}(L) = M_{ux}(L \sin \theta) \cos \theta - M_{uy}(L \cos \theta) \sin \theta \]
\[ M_{ut} = (M_{ux} - M_{uy}) \sin \theta \cos \theta \]
\[ M_{ut} = \frac{(M_{ux} - M_{uy})}{2} \sin 2\theta \]  \hspace{1cm} (A.2)

**Figure 65.** Combined twisting moment on a triangular section of slab.

If the above slab undergoes a rotation in the \( \theta_x \) and \( \theta_y \) directions, the absolute value of the work done by \( M_{ut}(L) \) and \( M_{ut}(L) \) must equal the value of the positive work done by \( M_{ux}(L_y) \) and \( M_{uy}(L_x) \).

Starting with the first of three possible yield patterns for a rectangular slab given above, and assuming a center deflection of \( \Delta \), the centroids of the triangles A, B, C, and D will deflect \( \Delta/3 \). Assuming the entire surface of the rectangular panel is loaded uniformly, the work required to deflect the panel is

\[ W = w_c ab \left( \frac{\Delta}{3} \right) \] \hspace{1cm} (A.3)
The uniform load or pressure is given in Equation (A.3) by \( w_u \), while \( a \) and \( b \) are the panel dimensions. This work term can be equated to the work done by moments acting on the boundaries of the four triangular panels:

\[
\begin{align*}
w_u ab \left( \frac{\Delta}{3} \right) &= 2 \left( M_{any} + M_{upy} \right) \left( a \right) \left( \frac{2\Delta}{b} \right) + 2 \left( M_{uxx} + M_{upx} \right) \left( b \right) \left( \frac{2\Delta}{a} \right). \\
\end{align*}
\]  

(A.4)

This expression can be solved for the uniform load, \( w_u \):

\[
\begin{align*}
w_u &= 12 \left[ \frac{M_{uxx} + M_{upx}}{a^2} + \frac{M_{any} + M_{upy}}{b^2} \right]. \\
\end{align*}
\]  

(A.5)

Unless the slab is a square, with equal ultimate moment capacities in both the \( x \) and \( y \) directions, the actual yield-line pattern is unknown, and the other two possible patterns given in Figure 63 must be investigated. These yield patterns are slightly more complicated but the analysis is not difficult. The first step is to divide the two trapezoidal-shaped pieces of the broken slabs into rectangles and triangles as depicted in Figure 66.

**Figure 66.** Sectioning of a slab (from Wang, 1979).

The total work required to deflect the yield-line intersections of the plate by \( \Delta \) can be easily determined using the following equation:

\[
\begin{align*}
W = W_{A_1} + W_{A_2} + W_{A_3} + W_B + W_C + W_D. \\
\end{align*}
\]  

(A.6)
By symmetry, \( W_{A1} + W_{A2} + W_{A3} \) equals \( W_C \) and \( W_B \) equals \( W_D \). These relations then give the following: (Wang, 1979)

\[
W = 2w_u \left( a - 2x \right) \left( \frac{\Delta}{2} \right) + 4 \left( \frac{1}{2} w_u x \frac{b}{2} \right) \left( \frac{\Delta}{3} \right).
\]

\[
W = \frac{w_u \Delta}{6} (3ab - 2bx).
\]

(A.7)

The above relation can again be set equal to the work done by the moments acting on all the slab segment boundaries:

\[
\frac{w_u \Delta}{6} (3ab - 2bx) = 2 \left( M_{uxx} + M_{upy} \right) \left( a \left( \frac{2\Delta}{b} \right) + 2 \left( M_{uxy} + M_{upy} \right) \left( b \left( \frac{\Delta}{x} \right) \right). \quad (A.8)
\]

As in the case of the previous yield pattern, this is solved for \( w_u \), and is expressed as:

\[
w_u = \frac{12 \left[ b^2 \left( M_{uxx} + M_{upy} \right) + 2ax \left( M_{uxy} + M_{upy} \right) \right]}{b^2 \left( 3ax - 2x^3 \right)}.
\]

(A.9)

All variables in this function are known, except for \( x \). Because the slab is assumed to break in a way that minimizes the required failure energy, the above function can be minimized using the differential calculus by setting the derivative with respect to \( x \) equal to zero:

\[
4a \left( M_{uxy} + M_{upy} \right) x^2 + 4b^2 \left( M_{uxx} + M_{upy} \right) x - (3ab^2 \left( M_{uxx} + M_{upy} \right)) = 0. \quad (A.10)
\]

This quadratic relationship can be solved for \( x \), which can then be plugged into the equation for \( w_u \).

The analysis used to determine the uniform pressure required to break the slab into the third pattern is similar to the second analysis. The variables \( a \) and \( b \) are
switched, as are the subscripts $x$ and $y$. In this case the positions of the yield-line intersections, and the ultimate load are then given by the following:

\[
4b \left( M_{ux} + M_{uxx} \right) y^2 + 4a^2 \left( M_{uxy} + M_{uxy} \right) y - \left( 3ba^2 \left( M_{uxy} + M_{uxy} \right) \right) = 0 \quad \text{and (A.11)}
\]

\[
w_u = 12 \left[ \frac{a^2 \left( M_{uxy} + M_{uxy} \right) + 2by \left( M_{uxx} + M_{uxx} \right)}{a^2 \left( 3by - 2y^2 \right)} \right]. \quad \text{(A.12)}
\]

In addition to the three possible yield-line patterns outlined above, there are three other yield-line patterns that may have even lower ultimate loads associated with them. These patterns are referred to as corner yield patterns and are plotted below in Figure 67.

\[\text{Figure 67. Possible corner yield patterns (from Wang, 1979).}\]

In some cases, the corner yield patterns will allow the slab to break at slightly lower ultimate loads. Because of the complexity of the analysis associated with these patterns, designers often calculate loads using simpler patterns and then use a 10% safety factor to account for the possibility of the slab yielding along a corner yield pattern. In this report, the author ignored corner effects completely.

Although the bay ceilings and back walls are effectively constrained on four sides, the sidewalls have a free edge. Obviously, these types of panels will have a different set of yield-line patterns. Several common yield patterns of a slab supported on three sides are shown in Figure 68.
Following an analysis similar to that of the fully constrained slabs, the ultimate uniform load can be found for plates that are supported on only three edges. For the pattern shown on the left side of Figure 68, the uniform yield load is

\[ w_u = \frac{6\left(b^2M_{wx} - x^2M_{wy}\right)}{2b^2x^2}, \]  

(A.13)

where the value of \(by\) is determined using the quadratic expression

\[ 3a\left(\frac{M_{wy}}{M_{xx}}\right)x^2 + 4b^2x - 3ab^2 = 0. \]  

(A.14)

The uniform load required to create the pattern on the right in Figure 68 is given by

\[ w_u = \frac{24bM_{wx}}{a^2(3b - 2y)}. \]  

(A.15)

The quadratic equation used to find \(y\) in the above expression is

\[ 4by^2 + 2a^2\left(\frac{M_{wy}}{M_{xx}}\right)y - 3a^2b\left(\frac{M_{wy}}{M_{xx}}\right) = 0. \]  

(A.16)
APPENDIX B –  
Conservation of Impulse When Large Computational Cells are Used

The primary factor that determines structural response is the total blast impulse delivered to various structural elements. The complete shock loading history for the rectangular bays of Buildings 260 and 340 can only accurately be determined using a fully 3-D finite element analysis (FEA) computation. Because of the large size of the machining bays, a rather large computational cell size is required, which has a characteristic cell length that is much larger than the actual shock front width.

Real shock fronts have a rise time of essentially zero. The infinite slope on the leading edge of the shock wave creates numerical difficulties when an FEA scheme is used to model the shock. Prediction of shock wave behavior was one of the first problems to be tackled using early digital computers. Von Neumann and Richtmyer (von Neumann and Richtmeyer, 1949) developed a mathematical scheme in which an “artificial viscosity” term was added to the equations of state used to describe a shock wave. The artificial viscosity term proposed by von Neumann and Richtmeyer had the form

\[ q = c_0^2 \rho (\Delta x)^2 \left( \frac{\partial x}{\partial x} \right)^2, \]  

(B.1)
in which the material density is given by \( \rho \), the direction of shock travel is given by \( x \), \( \Delta x \) is the grid spacing, and \( c_0 \) is a constant. The equations of motion, energy, and continuity in a Lagrangian system, with artificial viscosity take, then take the form

\[ \rho_0 s \left( \frac{dU}{dw} \right) = \left( \frac{d}{dw} \right)(p + q), \]  

(B.2)
\[
\left( \frac{dE_0}{dw} \right) + (p + q) \left( \frac{dV}{dw} \right) = 0 , \text{ and}
\]

\[
\rho_0 s \left( \frac{dV}{dw} \right) = - \frac{dU}{dw} ,
\]

(B.4)

in which \( E_0 \) is the fluid’s internal energy, \( \rho_0 \) is the initial density, \( p \) is the static pressure, specific volume is defined by \( V \), and \( w = - st \). The shock speed relative node points is denoted by \( s \).

The purpose of the artificial viscosity term, \( q \), is to eliminate any discontinuities in the solutions to Equations (B.2), (B.3), and (B.4). The last term in Equation (B.1) smears the shock front out over several computational cells. The exact number of cells is determined by the constant \( c_0 \), which is typically around 2.0. Choosing a smaller value for \( c_0 \) will create a steeper shock front but will cause the waveform to oscillate behind the initial shock front. Obviously, one wants the closest approximation of reality possible without creating overshoot behind the shock front. Because the shock front is smeared over several grid elements the calculated peak pressures will be much lower than the actual peak pressures. This disparity will become greater if the grid spacing is much larger than the shock front width, as is typically the case.

Fortunately, structural response is determined primarily by the total blast impulse, not the peak shock pressure. Von Neumann and Richtmyer showed that their form for the equations of motion, energy, and continuity can be algebraically manipulated and solved to give (von Neumann and Richtmeyer, 1949)

\[
\rho_0 s V + U + C_1 = 0 ,
\]

(B.5)

\[
\left( \rho_0 s \right)^2 V + p + q + C_2 = 0 , \text{ and}
\]

(B.6)
\[ E_0 + (p + q)V + \frac{(\rho_0 s)^2}{2} V^2 + C_s = 0 . \]  
\[ \text{(B.7)} \]

Noting that \( q \) is negligible away from regions near the shock, the three constants in the solution equations can be determined using the boundary conditions. Upon solving these equations, mass momentum and energy can be shown to be conserved, even though the artificial viscosity term \( q \) is present.

A linear artificial viscosity term was introduced by R. Landshoff in 1955 of the form (Wilkins, 1980)

\[ q = c_l \rho \Delta x a \left| \frac{\partial \dot{x}}{\partial x} \right| \]  
\[ \text{(B.8)} \]

if \( \left| \frac{\partial \dot{x}}{\partial x} \right| < 0 \), otherwise \( q = 0 \). In this expression, \( a \) is the local sound speed. While Equation (B.8) reduces oscillations in the waveform behind the shock front, it gives a greater overshoot at the shock front than Equation (B.1) does. Also, the Landshoff equation spreads the shock front over more grid cells than does the von Neumann and Richtmeyer term. By combining both the quadratic von Neumann and Richtmeyer and linear Landshoff artificial viscosity terms, the best attributes of the two terms can be obtained as in the following:

\[ q = c_0^2 \rho (\Delta x)^2 \left( \frac{\partial \dot{x}}{\partial x} \right)^2 + c_l \rho \Delta x a \left| \frac{\partial \dot{x}}{\partial x} \right| . \]  
\[ \text{(B.9)} \]

The Landshoff term is also dependent on the change in velocity with respect to position, although the dependence is linear and not quadratic. Therefore, it will not affect conditions away from the shock layers and thus also conserves mass, momentum, and energy.
Wilkins showed that Equation (B.9) can be extended to shocks traveling through 3-D grids, although he uses a rather difficult strain rate term (Wilkins, 1980),

\[
\frac{ds}{dt} = \frac{1}{A_x^2 + A_y^2 + A_z^2},
\]

where \(A_x, A_y,\) and \(A_z\) are accelerations in the \(x, y,\) and \(z\) directions, respectively.

AUTODYN uses a very similar artificial viscosity term (Century Dynamics, 2003),

\[
q = \rho \left( c_0 d \left( \frac{\dot{V}}{V} \right) \right)^2 - \rho c_L a \left( \frac{\dot{V}}{V} \right)
\]

if \(\left| \frac{\partial \dot{V}}{\partial V} \right| < 0\), which is the change in volume, otherwise \(q = 0\). Thus, this expression only smooths the shock front rise and allows the back of the front to naturally decay as a Taylor refraction, partially avoiding the annoyance of decay wave oscillations. The terms \(c_0\) and \(c_L\) are again constants and are chosen in such a way as to smooth the wave front as much as possible. The local sound speed is \(a\) and \(d\) is a characteristic length determined by dividing the cell volume by the cell’s longest diagonal. Because Equation (B.10) is based on the Landshoff artificial viscosity term, the mass, momentum, and energy, and therefore impulse, are shown to be conserved.
In all models run by the author, both the steel reinforcing bar and the concrete were assumed to act as isotropic materials. Additionally, both materials were assumed for numerical computation purposes to be homogenous. Although concrete is an aggregate type material, composed of sand, gravel, cement, and void areas, it will act as a homogenous material over the macroscopic regions defined by the rather large grid elements used to compose the walls and ceiling in these models.

The 3-D state of stress for these types of materials can be described at any point by the six stress components (Boresi, 2003)

\[ \sigma_{xx}, \sigma_{yy}, \sigma_{zz}, \sigma_{xy}, \sigma_{yz}, \text{and} \sigma_{zx}. \]  

By rotating the coordinate axis, one can always eliminate the shear stresses so that the only terms in the stress matrix which do not become zero are (Century Dynamics, 2003)

\[ \sigma_{xx}, \sigma_{yy}, \text{and} \sigma_{zz}. \]  

Figure 69. Stress components acting on a differential material element (from Boresis, 2003).
These principle stresses are denoted in the AUTODYN literature by $\sigma_1$, $\sigma_2$, and $\sigma_3$ and fill in the diagonal of the general stress tensor. If the material is behaving in a completely elastic manner, the three primary elastic strains may be solved for using Hooke’s law:

$$\varepsilon \sigma = \lambda \left( \frac{\text{Vol}}{\text{Vol}} \right) + 2G\dot{\varepsilon}, \quad \text{(C.3)}$$

where $\lambda$ and $G$ are the Lame constants and $i$ is summed over from 1 to 3. Volume is denoted by $\text{Vol}$. This formula determines the elemental strain in relation to the element’s strain on the previous calculation cycle, not the original strain. The hydrostatic pressure experienced by an element is determined by taking the mean value of the three principle stresses:

$$p = -\frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3). \quad \text{(C.4)}$$

Hydrostatic pressure changes the volume but not the shape of an element (Abernathy, 2003). However, hydrostatic pressure can be used to find the element’s deviatoric stresses, which do change the shape. Deviatoric stress can be determined using the relationship (Century Dynamics, 2003)

$$s_i = \sigma_i + p. \quad \text{(C.5)}$$

Differentiating the equation with respect to time gives

$$\dot{s}_i = \dot{\sigma}_i + \dot{p}. \quad \text{(C.6)}$$

Due to the linear stress-strain relationship given by Hooke’s law, the same relationships hold for material strain. Here deviatoric strain is denoted by $\theta$:
The principle strains can be related to the total change in the element’s volume between computational cycles by the equation of continuity, from which the following is derived:

\[ \varepsilon = \frac{1}{3} (\varepsilon_1 + \varepsilon_2 + \varepsilon_3) \]
\[ \theta_1 = \varepsilon_1 + \varepsilon \quad \text{(C.7, C.8, C.9)} \]
\[ \theta_2 = \varepsilon_2 + \varepsilon \]

For this relationship to hold true,

\[ \theta_1 + \theta_2 + \theta_3 = 0 \quad \text{(C.11)} \]

and

\[ \dot{\varepsilon} = \frac{\dot{Vol}}{Vol}. \quad \text{(C.12)} \]

Using the relationships derived above, Hooke’s law may be rewritten in terms of the three principle deviatoric stresses (Century Dynamics, 2003):

\[ \dot{s}_i = 2G \left( \varepsilon_i - \frac{\dot{Vol}}{3Vol} \right). \quad \text{(C.13)} \]

Under certain conditions, the deviatoric stresses may then be used to determine when the element will fail, as described below in the principle stress failure criteria section.

For an isotropic material, a bulk modulus, denoted by K, may be defined as

\[ K = \left( \lambda + \frac{2G}{3} \right). \quad \text{(C.14)} \]
If the user selects the hydrodynamic tensile failure model AUTODYN, the user simply compares the hydrostatic pressure calculated from Equation (C.4) with a constant hydrodynamic tensile limit set by the user. This type of model works well for ductile materials but has many drawbacks. Although this is a computationally inexpensive model to implement, it works very poorly with porous materials such as concrete. The model does not predict failure of the material due to high compressions and will allow elements subjected to shear stresses only to reach stresses three times greater than the actual failure limit. This is because of the use of Equation (C.4) to predict failure. Moreover, if the failure limit is not carefully set, the hydrodynamic tensile failure model will give incorrect predictions because it “reheals” materials by setting the internal energy to zero if the material’s pressure drops below zero. The author found that this model gave very unrealistic results when it was used to predict concrete failure.

The AUTODYN principle stress failure model works by defining the principle directions within in an element so that the shear stress lying on the principle plane is zero. Maximum shear stress is determined from the shear stresses lying on planes oriented 45° from the principle directions. The user inputs a maximum tensile stress and a maximum shear stress.

This failure model is based on the Von Mises yield criterion and is mathematically described by the following (Boresi, 2003):

\[ 2Y^2 = (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2, \quad (C.15) \]

where \( Y \) represents the axial tensile yield strength, which is given by the user. Some minor arrangements may be performed using linear algebra to give the expression
\[ 2Y^2 = (s_1 - s_2)^2 + (s_2 - s_3)^2 + (s_3 - s_1)^2. \] \ (C.16)

Setting the trace of the deviatoric stress tensor to zero, (Century Dynamics, 2003)

\[ s_1 + s_2 + s_3 = 0, \] \ (C.17)

yields

\[ \frac{2Y^2}{3} = s_1^2 + s_2^2 + s_3^2. \] \ (C.18)

Therefore, the deviatoric stresses alone can be used to find the onset of plastic deformation. However, this is only true for some materials, such as ductile metals like the steel used for the rebar in the AUTODYN models. Experimental data shows that the yield point in some metals is independent of hydrostatic pressure, but in many materials hydrostatic pressure will increase the yield strength. This is especially true with concrete, where the strength of individual aggregate particles is much higher than the surrounding cement media strength. Concrete tends to fail along cracks propagating through the cement that binds the aggregate together. Hydrostatic pressure prevents these cracks from forming, therefore increasing the shear load that the concrete can carry. For these types of materials the above equation is invalid and another failure model must be used.

The Von Mises yield criterion is often referred to as the distortional energy density criterion because it is actually based on the theory that yielding begins when the distortional strain energy at a point equals the uniaxial tension yield distortional strain-energy density. From Hooke’s law:

\[ work = \frac{1}{2} k (\text{disp})^2, \] \ (C.19)
where \( k \) is the material’s spring constant. For any elastic material, strain energy is dependent on principle strains only, and if a material is linearly elastic the strain energy at any point is

\[
U_o = \frac{1}{2} C_{ij} \varepsilon_i \varepsilon_j \quad i, j = 1, 2, 3.
\]  

(C.20)

One can set the principle axes to any arbitrary direction. If the principle axes are set in the direction of principle strain, then it can be shown that three invariants exist (Boresi, 2003):

\[
I_1 = \varepsilon_{xx} + \varepsilon_{yy} + \varepsilon_{zz}, \\
I_2 = \varepsilon_{xx} \varepsilon_{yy} + \varepsilon_{xx} \varepsilon_{zz} + \varepsilon_{yy} \varepsilon_{zz} - \varepsilon_{xy}^2 - \varepsilon_{xz}^2 - \varepsilon_{yz}^2, \text{ and} \\
I_3 = \begin{pmatrix}
\varepsilon_{xx} & \varepsilon_{xy} & \varepsilon_{xz} \\
\varepsilon_{yx} & \varepsilon_{yy} & \varepsilon_{yz} \\
\varepsilon_{zx} & \varepsilon_{zy} & \varepsilon_{zz}
\end{pmatrix}.
\]

(C.21, C.22, C.23)

Using these invariants, and some algebraic manipulation, one can derive the expression for the strain energy density of an isotropic material subjected to a general stress loading:

\[
U_o = \frac{1}{2E} \left[ \sigma_{xx}^2 + \sigma_{yy}^2 + \sigma_{zz}^2 - 2\nu \left( \sigma_{xx} \sigma_{yy} + \sigma_{xx} \sigma_{zz} + \sigma_{yy} \sigma_{zz} \right) + 2(1-\nu) \left( \sigma_{xy}^2 + \sigma_{xz}^2 + \sigma_{yz}^2 \right) \right] > 0.
\]  

(C.24)

If the coordinate axes are oriented so that they align with the principle stress axis, the above equation becomes

\[
U_o = \frac{1}{2E} \left[ \sigma_{1}^2 + \sigma_{2}^2 + \sigma_{3}^2 - 2\nu \left( \sigma_1 \sigma_2 + \sigma_1 \sigma_3 + \sigma_2 \sigma_3 \right) \right] > 0.
\]  

(C.25)

In more familiar notation:

\[
U_o = \frac{1}{2E} \left[ \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\nu \left( \sigma_1 \sigma_2 + \sigma_1 \sigma_3 + \sigma_2 \sigma_3 \right) \right] > 0.
\]  

(C.26)

Using the definition for the shear modulus,
the bulk modulus may be written as

\[ K = \frac{E}{3(1-2\nu)} \]  

(C.28)

The strain energy expression may then be rewritten as (Boresi, 2003)

\[ U_o = \frac{(\sigma_1 + \sigma_2 + \sigma_3)^2}{18K} + \frac{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}{12G}. \]  

(C.29)

The first term on the right in the above equation is due to the hydrostatic pressure. The second term on the right is the strain energy due to distortion of the material. The second deviatoric stress invariant is

\[ J_2 = -\frac{1}{6} \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right]. \]  

(C.30)

The second term on the right in Equation (C.29) can be written in terms of the second stress invariant or the first and second strain invariants:

\[ U_D = \frac{|J_2|}{2G} = I_2 - \frac{I_1^2}{3}. \]  

(C.31)

Under uniaxial stress, the \( \sigma_2 \) and \( \sigma_3 \) terms are zero, and the second stress invariant becomes

\[ |J_2| = \frac{\sigma_1^2}{3} = \frac{Y^2}{3}. \]  

(C.32)

The von Mises yield criterion can now be written as

\[ f = \frac{1}{6} \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right] - \frac{Y^2}{3} \quad \text{and} \quad f = \sigma_e^2 - Y^2. \]  

(C.33, C.34)
where $\sigma_e$ is the effective stress. The second of the two relations above may be solved to find the effective yield stress (Boresi, 2003):

$$\sigma_e = \sqrt{\frac{1}{2} (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}. \quad (C.35)$$

In Cartesian coordinates the effective stress becomes

$$\sigma_e = \sqrt{\frac{1}{2} \left[ (\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{yy} - \sigma_{zz})^2 + (\sigma_{zz} - \sigma_{xx})^2 \right] + 3 \left( \sigma_{xx}^2 + \sigma_{yy}^2 + \sigma_{zz}^2 \right)}. \quad (C.36)$$

The above equation is essentially identical to equation (C.35), which was derived earlier in a different manner. Once again the deviatoric stresses alone define the yield stress.

AUTODYN implements its own concrete failure model, referred to as the RHT failure model. The author has not found any information about this model in the documentation supplied with the program by Century Dynamics. However, this failure model should be fairly similar to either the Mohr-Coulomb or Drucker-Prager failure criteria.

To extend the von Mises strain energy failure criterion to cohesive materials, the Mohr-Coulomb yield function was developed. The major assumption behind the Mohr-Coulomb model is that the effective yield stress increases as hydrostatic pressure increases. Effective yield stress is also assumed to be lowered if the hydrostatic pressure is negative. Furthermore, the yield surface forms a cone, rather than a cylinder. In the Mohr-Coulomb model, the equation for this surface is given by the following (Century Dynamics, 2003):

$$f = \sigma_1 - \sigma_3 + (\sigma_1 + \sigma_3 \sin \phi - 2c \cos \phi, \quad (C.37)$$
where $\sigma_1 > \sigma_2 > \sigma_3$. The cohesion of the material is $c$ and $\phi$ is the material’s angle of internal friction. This surface is plotted in Figure 70. This function will allow a cohesive material subjected to an infinite hydrostatic pressure to support an infinite shear load. This is obviously unrealistic, so AUTODYN allows the user to specify whether or not a hemispherical cap is added to the conical failure surface cone at some point. The default setting for this feature is on. All models of Building 340 that have been run by the author have used the cap.
Figure 70. Cylindrical and conical yield failure surfaces.
Yield-line Analysis of the Building 340 Ceiling
January 3rd, 2004

Determine the flexural capacity of the ceiling as calculated using "Design of Reinforced Concrete Structures:"

**Known:**
The ceiling slab is 18 in. thick.
Number 10 rebar is spaced on 9-in. centers, each way, each face.
The rebar yield strength is 59 ksi.
The concrete failure compression pressure, \( f_c \), is 6 ksi.
Assume 2 in. of cover on the rebar. (This appears to be how deep EMERTC could drill before hitting rebar.)

**Solution:**
\[
\begin{align*}
\text{As} &:= (12/9) \times (1.25); \\
A_{\text{prime}_s} &:= (12/9) \times (1.25); \\
A_{\text{prime}_s} &:= 1.666666667 \\
A_{\text{prime}_s} &:= 1.666666667 \\
\text{As1} \times f_y &:= 0.85 \times f_c \times a \times b \\
a &:= \text{As} \times f_y / (0.85 \times f_c \times b);
\end{align*}
\]
The volumetric steel ratio is
\[
\rho := \frac{A_s}{b \cdot d}; \\
\rho' := \frac{A'_{s}}{b \cdot d}; \\
\rho : = 0.00868055556 \\
\rho' := 0.00868055556.
\]

To yield the compression steel
\[
(rho-rho_prime) > 0.85 \times \beta_1 \times \left(\frac{f'c}{f_y}\right) \times \left(\frac{d'}{d}\right) \times \left(\frac{87}{87-f'y}\right)
\]
\[
\text{lefths} := rho-rho_prime; \\
\text{righths} := 0.85 \times \beta_1 \times \left(\frac{f'c}{f_y}\right) \times \left(\frac{d'}{d}\right) \times \left(\frac{87}{87-f'y}\right);
\]
Because the left-hand side is not greater than the right-hand side, therefore the steel yields in tension.
\[
\text{lefths} := 0. \\
\text{righths} := 0.0001595645058 \beta_1
\]

The yield strength of the rebar increases because of the high strain rate.
The multiplier \(\phi\) accounts for this
\[
\phi := 1.1:
\]

The flexural capacity of the steel is then (lbf\cdot in)
\[
\mu := \phi \times A_s \times f_y \times (d-a/2); \\
\mu := 1817565270 \; 10^7.
\]

The volumetric steel ratio is
\[
\rho := \frac{A_s}{b \cdot d}; \\
\rho' := \frac{A'_{s}}{b \cdot d}; \\
\rho : = 0.00868055556 \\
\rho' := 0.00868055556.
\]

To yield the compression steel
\[
(rho-rho_prime) > 0.85 \times \beta_1 \times \left(\frac{f'c}{f_y}\right) \times \left(\frac{d'}{d}\right) \times \left(\frac{87}{87-f'y}\right)
\]
\[
\text{lefths} := rho-rho_prime; \\
\text{righths} := 0.85 \times \beta_1 \times \left(\frac{f'c}{f_y}\right) \times \left(\frac{d'}{d}\right) \times \left(\frac{87}{87-f'y}\right);
\]
Because the left-hand side is not greater than the right-hand side, therefore the steel yields in tension. This matches intuition.
\[
\text{lefths} := 0. \\
\text{righths} := 0.0001595645058 \beta_1
\]

The yield strength of the rebar increases because of the high strain rate.
The multiplier \(\phi\) accounts for this
\[
\phi := 1.21:
\]
The flexural capacity of the steel is then (lbf*in)

\[ M_{ult} := \phi \cdot A_s \cdot f_y \cdot (d-a/2); \]

\[ M_{ult} := 1.999321797 \times 10^7. \]

**Yield pressure of Building 340 with uniform load in 20-ft-wide bays with 3 fixed edges:**

The length of the panel is 27 ft (inches):

\[ \text{restart}; \]

\[ a := 27 \times 12; \]

The width of the ceiling panel is 20 ft (inches):

\[ b := 20 \times 12; \]

The ceiling thickness is ????? (inches):

\[ h := 18; \]

The drawings call for 1.25-in. rebar on 9-in. spacing, each way, each face.

Calculating the volumetric percentage of steel by looking at a 1- by 1-ft block (inches^3):

\[ \text{Block}_\text{vol} := 12 \times 12 \times 18; \]

The steel volume is

\[ \text{Steel}_\text{vol} := 1.25 \times 12 \times 2 \times 4; . \]

The volumetric percentage of steel is 1%, or 0.05% in each direction.

\[ \rho := \text{Steel}_\text{vol} / \text{Block}_\text{vol} \times 100; \]

The area of steel on the top or bottom of the cross-section is (inches^2/ft):

\[ A_s := 1.25 \times 12 / 9; \]

\[ \text{Block}_\text{vol} := 2592 \]

\[ \text{Steel}_\text{vol} := 120.00 \]

\[ \rho := 4.629629630 \]

\[ A_s := 1.666666667. \]

The ultimate bending moments in the x and y directions are assumed to be equal. They are (in*lbf):

As calculated by hand using "Design of Reinforced Concrete Structures." (Wang and Salmon)

\[ M_{uy} := 2.00E6 / 12; \]

\[ M_{ux} := 2.00E6 / 12; \]

Assuming that the ceiling yields along three lines, in a Y-pattern, the work done by a uniform load on the entire panel is (in*lbf):

\[ W_{wu} := \left( \frac{1}{2} \right) \times wu \times b \times x \times \Delta / 3 + 2 \times wu \times (a-x) b / 2 \times Delta / 2 + 2 \times \left( \frac{1}{2} \right) \times wu \times x \times (b/2) \times (Delta / 3); \]

\[ W_{wu} := 80 \times wu \times x \times \Delta + 120 \times wu \times (324 - x) \times \Delta \]

The work done by the yield-line moments is (in*lbf)

\[ W_{edg} := 2 \times M_{uy} \times a \times ((2 \times Delta) / b) + M_{ux} \times b \times (Delta / x); \]
\[ \text{Wedg} := 900000.0000 \Delta + \frac{4000000001 \times 10^{8} \Delta}{x} \]

Setting the load and yield-line work equal to each other:

\[ \text{eq1} := \text{Wwu} = \text{Wedg}; \]

\[ \text{eq1} := 80 \, \text{wu} \, x \Delta + 120 \, \text{wu} \, (324 - x) \Delta = 900000.0000 \Delta + \frac{4000000001 \times 10^{8} \Delta}{x} \]

Solving equation 1 for the uniform load:

\[ \text{load} := \text{solve (eq1, wu)}; \]

\[ \text{load} := -0.000250000000 - \frac{90000000 \times 10^{8} \Delta + 4000000001 \times 10^{10}}{x (x - 972.)} \]

Find \( x \), the point at which the three yield-lines meet, by taking the derivative with respect to \( x \) and setting it equal to zero:

\[ \text{eq2} := \text{diff (load, x)} = 0; \]

\[ \text{eq2} := -22500.00000 \frac{1}{x (x - 972.)} + \frac{0.000250000000 \times (0.90000000 \times 10^{8} \Delta + 4000000001 \times 10^{10})}{x^{2} (x - 972.)} + \frac{0.000250000000 \times (0.90000000 \times 10^{8} \Delta + 4000000001 \times 10^{10})}{x (x - 972.)^{2}} = 0 \]

Solving the quadratic equation for \( x \) (in):

\[ \text{X} := \text{solve (eq2, x)}; \]

\[ \text{X} := 168.1003947, -256.9892836 \]

The static failure load is (psi)

\[ \text{static} := \text{eval (load, x=168.1)}; \]

\[ \text{static} := 35.38853088 \]

The failure load is dynamic (psi):

\[ \text{dynamic} := 1/0.86 \times (\text{eval (load, x=160.34)}); \]

\[ \text{dynamic} := 41.16849666 \]

The average shear load at the ceiling edges due to this load is (psi)

\[ \text{tau} := \text{a*b*51.3/(2*a*h+b*h)}; \]

\[ \tau := 249.5675676 \]

**Yield pressure of Building 340 with uniform load in 24-ft-wide bays with 3 fixed edges:**

The length of the panel is 27 ft (inches)

\[ \text{restart}; \]

\[ \text{a} := 27*12; \]

The width of the ceiling panel is 24 ft (inches)

\[ \text{b} := 24*12; \]

The ceiling thickness is (inches):
> \( h:=18 \):
The drawings call for 1.25-in. rebar on 6-in. spacing, each way, each face. Calculating the volumetric percentage of steel by looking at a 1- by 1- ft block (inches^3):

> \( \text{Block} \_\text{vol}:= 12*12*18; \)
The steel volume is

> \( \text{Steel} \_\text{vol}:= 1.25*12*2*4; \).
The volumetric percentage of steel is 1%, or 0.05% in each direction.

> \( \rho:=\text{Steel} \_\text{vol}/\text{Block} \_\text{vol}*100; \)
The area of steel on the top or bottom of the cross-section is (inches^2/ft)

> \( \text{As}:= 1.25*2; \)

\[
\begin{align*}
\text{Block} \_\text{vol} & := 2592 \\
\text{Steel} \_\text{vol} & := 120.00 \\
\rho & := 4.629629630 \\
\text{As} & := 2.50
\end{align*}
\]

The ultimate bending moments in the \( x \) and \( y \) directions are assumed to be equal, they are (in*lbf):

As calculated by hand using "Design of Reinforced Concrete Structures."

> \( \text{Muy}:=2.00E6/12; \)
> \( \text{Mux}:=2.00E6/12; \)

Assuming that the ceiling yields along three lines, in a Y-pattern, the work done by a uniform load on the entire panel is (in*lbf)

> \( \text{Wwu}:= (1/2)*\text{wu}*b*x*\Delta /3+2*\text{wu}*(a-x)*b/2*\Delta /2+2*(1/2)*\text{wu}*x*(b/2)*(\Delta /3); \)

\[
\text{Wwu} := 96 \text{wu} x \Delta + 144 \text{wu} (324-x) \Delta
\]

The work done by the yield-line moments is (in*lbf)

> \( \text{Wedg}:= 2*\text{Muy}*a*((2*\Delta )/b)+\text{Mux}*b*(\Delta /x); \)

\[
\text{Wedg} := 750000.000 \Delta + \frac{480000001 \times 10^8 \Delta}{x}
\]

Setting the load and yield-line work equal to each other:

> \( \text{eql}:=\text{Wwu}=\text{Wedg}; \)

\[
\text{eql} := 96 \text{wu} x \Delta + 144 \text{wu} (324-x) \Delta = 750000.000 \Delta + \frac{480000001 \times 10^8 \Delta}{x}
\]

Solving equation 1 for the uniform load:

> \( \text{load}:=\text{solve(eql,}\text{wu}); \)

\[
\text{load} := -0.0002083333333\times \frac{750000000 \times 10^8 x+ 4800000001 \times 10^{10}}{x(x-972.)}
\]

Find \( x \), the point at which the three yield-lines meet, by taking the derivative with respect to \( x \) and setting it equal to zero:

> \( \text{eq2}:= \text{diff(load,}\text{x})=0; \)
\[ eq2 := -15625.00000 \frac{1}{x(x-972.)} + \frac{0.002083333333 (0.7500000 \times 10^8 x + 0.480000001 \times 10^{10})}{x^2(x-972.)} + \frac{0.002083333333 (0.7500000 \times 10^8 x + 0.480000001 \times 10^{10})}{x(x-972.)^2} = 0 \]

Solving the quadratic equation for \( x \) (inches):

\[
> X := \text{solve}(eq2, x);
\]

\[
X := 193.4956310, -321.4956312
\]

The failure load is static (psi)

\[
> \text{static} := \text{eval} (\text{load}, x=193.5);.
\]

\[
\text{static} := 26.70900277
\]

The failure load is dynamic (psi):

\[
> \text{dynamic} := 1/0.86*(\text{eval} (\text{load}, x=160.34));
\]

\[
\text{dynamic} := 31.31931592
\]

The average shear load at the ceiling edges due to this load is (psi)

\[
> \tau := a*b*51.3/(2*a*h+b*h);
\]

\[
\tau := 284.1230769
\]

**Yield pressure of Building 340 with center-biased load in 20-ft-wide bays with 3 fixed edges:**

A vertically oriented cylindrical charge will load the ceiling directly above it with very high pressures.

Assume that the centroid of the load on a triangular panel may be shifted from 1/3 of the panel's length out from the wall to 1/2 the panels length.

Assume that a rectangular panel will be loaded 2/3 of the way out from the wall.

\[
> \text{restart}:
\]

\[
> \text{The length of the ceiling panel is 27 ft (inches)}:\
> a := 27*12:
\]

The width of the ceiling panel is 20 ft (inches):

\[
> b := 20*12:
\]

The ceiling thickness is (inches)

\[
> h := 18:
\]

The ultimate bending moments in the \( x \) and \( y \) directions are assumed to be equal, they are (in*lbf):

As calculated by hand using "Design of Reinforced Concrete Structures."

\[
> \text{Muy} := 2.00E6/12:\n\]

\[
> \text{Mux} := 2.00E6/12:\n\]

Assuming that the ceiling yields along three lines, in a Y-pattern, the work done by a center-biased load on the entire ceiling panel is (in*lbf)

\[
> \text{Wwu} := (1/2)*wu*b*x*Delta/2+2*wu*(a-x)*b/2*Delta*(2/3)+2*(1/2)*wu*x*(b/2)*(Delta/2);
\]
\[ W_{wu} := 120 wu x \Delta + 160 wu (324 - x) \Delta. \]

The work done by the yield-line moments is (in*lbf)
\[
Wedg := 2 \times Muy \times a \times ((2 \times \Delta) / b) + Mux \times b \times (\Delta / x) ;
\]
\[
Wedg := 900000.0000 \Delta + \frac{4000000001 \times 10^8 \Delta}{x}.
\]

Setting the load and yield-line work equal to each other:
\[
> eq1 := Wwu = Wedg;
\]
\[
eq 1 := 120 wu x \Delta + 160 wu (324 - x) \Delta = 900000.0000 \Delta + \frac{4000000001 \times 10^8 \Delta}{x}.
\]

Solving equation 1 for the uniform load:
\[
> load := solve(eq1, wu);
\]
\[
load := -0.0002500000000 - \frac{90000000 \times 10^8 x + 40000000001 \times 10^{10}}{x \times (x - 1296.)}
\]

Find \( x \), the point at which the three yield-lines meet, by taking the derivative with respect to \( x \) and setting it equal to zero:
\[
> eq2 := diff(load, x) = 0;
\]
\[
eq 2 := -22500.0000 \frac{1}{x \times (x - 1296.)}
\]
\[
+ \frac{0.0002500000000 \times (0.90000000 \times 10^8 x + 0.40000000001 \times 10^{10})}{x \times (x - 1296.)^2}
\]
\[
+ \frac{0.0002500000000 \times (0.90000000 \times 10^8 x + 0.40000000001 \times 10^{10})}{x \times (x - 1296.)^2} = 0
\]

Solving the quadratic equation for \( x \) (inches):
\[
> X := solve(eq2, x);
\]
\[
X := 199.6360928, -288.5249817
\]

The average static failure load is (psi)
\[
> static := eval(load, x=199.6);
\]
\[
static := 25.09122588
\]

The failure load is dynamic (psi):
\[
> dynamic := 1/0.86*(eval(load, x=160.34));
\]
\[
dynamic := 29.42326224
\]

**Yield pressure of Building 340 with center-biased Load in 24-ft-wide bays with 3 fixed edges:**

\[
> restart:
\]

The length of the panel is 27 ft (inches):
\[
> a := 27*12:
\]

The width of the ceiling panel is 24 ft (inches):
\[
> b := 24*12:
\]

The ceiling thickness is (inches)
The ultimate bending moments in the x and y directions are assumed to be equal. They are (in*lbf):

\[ M_{uy} := \frac{2.00 \times 10^6}{12} \]

\[ M_{ux} := \frac{2.00 \times 10^6}{12} \]

Assuming that the ceiling yields along three lines, in a Y-pattern, the work done by a center-biased load on the entire ceiling panel is (in*lbf)

\[ W_{wu} := \frac{1}{2} w_u b x \Delta + 192 w_u \left( \frac{324 - x}{\Delta} \right) \Delta \]

The work done by the yield-line moments is (in*lbf)

\[ W_{wedg} := 2 M_{uy} a \left( \frac{2\Delta}{b} \right) + M_{ux} b \left( \frac{\Delta}{x} \right) \]

Setting the load and yield-line work equal to each other:

\[ eq1 := W_{wu} = W_{wedg} \]

Solving equation 1 for the uniform load:

\[ load := \text{solve}(eq1, w_u) \]

Find x, the point at which the three yield-lines meet by taking the derivative with respect to x and setting it equal to zero:

\[ eq2 := \text{diff}(load, x) = 0 \]

Solving the quadratic equation for x (inches):

\[ X := \text{solve}(eq2, x) \]

The average static failure load is (psi)

\[ static := \text{eval}(load, x=231); \quad static := 18.73615430 \]

The failure load is dynamic (psi):

\[ dynamic := \frac{1}{0.86} \times \text{eval}(load, x=160.34); \]
**Yield pressure of Building 340 with uniform load in 20-ft-wide bays with 4 fixed edges:**

> **restart:**

The length of the panel is 27 ft (inches):

> \( a := 27 \times 12 \)

The width of the ceiling panel is 24 ft (inches):

> \( b := 20 \times 12 \)

The ceiling thickness is (inches)

> \( h := 18 \)

The ultimate bending moments in the \( x \) and \( y \) directions are assumed to be equal, they are (in*lbf):

As calculated by hand using "Design of Reinforced Concrete Structures."

> \( \text{Mup} := \frac{2.00 \times 10^6}{12} \)

> \( \text{Mup} \times := \frac{2.00 \times 10^6}{12} \)

> \( \text{Mun} := 2 \times \text{Mup} \times \)

> \( \text{Mun} := 2 \times \text{Mup} \)

Assume that the ceiling yields in an "X" pattern:

> \( \text{wu} := 12 \times \left( \frac{(\text{Mun}+\text{Mup})}{a^2} + \frac{(\text{Mun}+\text{Mup})}{b^2} \right) \)

\( \text{wu} := 161.325881 \)

Assume that the ceiling yields in a modified "X" pattern:

> \( \text{wu} := 12 \times \frac{(b^2 \times (\text{Mun}+\text{Mup})+2 \times a \times x \times (\text{Mun}+\text{Mup}))}{b^2 \times (3 \times a \times x - 2 \times x^2)} \)

\( \text{wu} := \frac{1}{4800} \times \frac{0.2880000001 \times 10^{11} + 0.3240000000 \times 10^9 \times x}{972 \times 2 \times x^2} \)

Find \( x \), the point at which the three yield-lines meet, by taking the derivative with respect to \( x \) and setting it equal to zero:

> \( \text{eq2} := \text{diff(wu,x)} = 0 \)

\( \text{eq2} := \frac{1}{972 \times 2 \times x^2} - \frac{1}{4800} \times \frac{0.2880000001 \times 10^{11} + 0.3240000000 \times 10^9 \times x}{(972 \times 2 \times x^2)^2} (972 - 4 \times x) = 0 \)

Solve quadratic equation for \( x \):

> \( x := \text{solve(eq2,x)} \)

\( x := 137.1669329, -314.9447108 \)

Plugging the value of \( x \) back into the expression for the \( \text{wu} \):

> \( \text{wu} := 24 \times a \times (\text{Mun}+\text{Mup}) / (2 \times b^2 \times x + 3 \times b^2 \times (a-2 \times x)) \)

> \( \text{wu} := 6 \times (\text{Mun}+\text{Mup}) / x^2 \)

\( \text{wu} := 159.4491704 \)
Yield pressure of Building 340 with uniform load in 24-ft wide bays with 4 fixed edges:

\[ wu_3 := 159.4492965 \]

The length of the panel is 27 ft (inches):
\[ a := 27 \times 12. \]
The width of the ceiling panel is 24 ft (inches):
\[ b := 24 \times 12. \]
The ceiling thickness is (inches):
\[ h := 18. \]
The ultimate bending moments in the \( x \) and \( y \) directions are assumed to be equal. They are (in*ft):
\[ M_{uy} := \frac{2.00 \times 10^6}{12}; \]
\[ M_{ux} := \frac{2.00 \times 10^6}{12}; \]
\[ M_{unx} := 2 \times M_{ux}; \]
\[ M_{uny} := 2 \times M_{uy}; \]
Assume that the ceiling yields in an "X" pattern:
\[ wu_1 := 12 \times \left( \frac{M_{uny} + M_{ux}}{a^2} + \frac{M_{uny} + M_{uy}}{b^2} \right) ; \]
\[ wu_1 := 129.4938843 \]
Assume that the ceiling yields in a modified "X" pattern:
\[ wu := 12 \times \left( \frac{b^2 \times (M_{ux} + M_{uy}) + 2 \times a \times x \times (M_{uny} + M_{uy})}{b^2 \times (3 \times a \times x - 2 \times x^2)} \right) ; \]
\[ wu := \frac{1}{6912} \times 4.147200001 \times 10^{11} + 0.3240000000 \times 10^9 \times x \\
\times 972. x - 2 \times x^2 \times 972. \]
\[ wu := 16912 \left( \frac{0.4147200001 \times 10^{11} + 0.3240000000 \times 10^9 \times x}{(972. - 4 \times x)^2} \right) = 0 \]
Solve quadratic equation for \( x \):
\[ x := \text{solve}(eq2, x) ; \]
\[ x := 152.3426475, -408.3426476 \]
Plugging the value of \( x \) back into the expression for the \( wu \):
\[ x := 152.3426 ; \]
\[ wu_2 := 24 \times a \times (M_{ux} + M_{uny}) / (2 \times b^2 \times x + 3 \times b^2 \times (a - 2 \times x)) ; \]
\[ wu_2 := 129.2641307 \]
Yield pressure of Building 340 with center-biased load in 20-ft-wide bays with 4 fixed edges:

```
restart:
The length of the panel is 27 ft (inches):
a:=27*12:
The width of the ceiling panel is 24 ft (inches):
b:=20*12:
The ceiling thickness is (inches)
h:=18:
The ultimate bending moments in the \( x \) and \( y \) directions are assumed to be equal, they are (in*lbf):
As calculated by hand using "Design of Reinforced Concrete Structures."
Mupx:=2.00E6/12:
Mupy:=2.00E6/12:
Munx:=2*Mupx:
Muny:=2*Mupy:
W1:=wu*a*b*(Delta/2);
W1 := 38880 \( wu \Delta \)
```

```
W2:=2*(Muny+Mupy)*(a)*(2*Delta/b)+2*(Munx+Mupx)*(b)*(2*Delta/a);
W2 := .4181481482 \( 10^7 \Delta \)
```

```
eq1:=W1=W2:
solve (eq1,wu);
107.5483920
```

Yield pressure Building 340 with center-biased load in 24-ft-wide bays with 4 fixed edges:

```
restart:
The length of the panel is 27 ft (inches):
a:=27*12:
The width of the ceiling panel is 24 ft (inches):
b:=24*12:
The ceiling thickness is (inches)
h:=18:
The ultimate bending moments in the \( x \) and \( y \) directions are assumed to be equal, they are (in*lbf):
As calculated by hand using "Design of Reinforced Concrete Structures."
Mupx:=2.00E6/12:
Mupy:=2.00E6/12:
Munx:=2*Mupx:
Muny:=2*Mupy:
```
\[ W_1 = w_u a b \left( \frac{\Delta}{2} \right) \]
\[ W_1 = 46656 \, w_u \Delta \]

\[ W_2 = 2 (M_{u_n} + M_{u_p}) (a) \left( \frac{2 \Delta}{b} \right) + 2 (M_{u_n} + M_{u_p}) (b) \left( \frac{2 \Delta}{a} \right) \]
\[ W_2 = 0.402777778 \times 10^7 \, \Delta \]

\[ eq1 := W_1 = W_2 \]
\[ solve (eq1, w_u) \]
\[ 86.32925622 \]
REFERENCES


Parker, R., “Numerical Predictions of Overpressures within TA-16, Bldg. 340 HE Machining Bay Subject to detonation of 75lbs. PBX-950,” presentation at Los Alamos National Laboratory (December 2, 2003).


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