

# AD AO 58653

# TARGET VULNERABILITY AND HARDNESS UNCERTAINTY ANALYSIS

TRW Defense and Space Systems Group One Space Park Redondo Beach, California 90278

31 January 1977

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DNASBIE 4531 F, AD-E300 315 SECURITY CLASSIFICATION OF THIS PAGE (When Date READ INSTRUCTIONS BEFORE COMPLETING FORM REPORT DOCUMENTATION PAGE REPORT NUMBER 2. GOVT ACCESSION NO. 3. RECIPIENT'S CATALOG NUMBER DNA 4534F Final Repart for Period TITLE (and Subtitle) 30 Mar 76-31 Jan 77 TARGET VULNERABILITY AND HARDNESS UNCERTAINTY ANALYSIS 29426-6001-RU-00 CONTRACT OR GRANT NUMBER(S) AUTHORIS Juen S. Chiu DNA\_001-76-C-0229 John J. |Farrell Norman Lipners Augusto'L. Soux PROGRAM ELEMENT. PROJECT, TASK AREA & WORK UNIT NUMBERS ERFORMING ORGANIZATION NAME AND ADDRESS TRW Defense and Space Systems Group NWED Subtask One Space Park Y99QAXSC062-06 Redondo Beach, California 90278 11. CONTROLLING OFFICE NAME AND ADDRESS REPORT DATE 31 January 1977 Director NUMBER OF PAGES Defense Nuclear Agency Washington, D.C. 20305 96 15. SECURITY CL · (of this report) MONITORING AGENCY NAME & ADDRESS(II dillerent from Controlling Offic:, UNCLASSIFIED 154. DECLASSIFICATION/DOWNGRADING SCHEDULE 16. DISTRIBUTION STATEMENT (of this Report) Approved for public release; distribution uni imited.  $\mathbb{D}_1$ 17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report) ショ9川26-600エーポン-00 18. SUPPLEMENTARY NOTES This work sponsored by the Defense Nuclear Agency under RDT&E RMSS Code B344076464 Y99QAXSC06206 H2590D. 19. KEY WORDS (Continue on reverse side if necessary and identify by block number) Nuclear Weapon Effects Target Hardness Uncertainty Industrial Buildings Shallow Buried Bunkers Structural Response ABSTRACT (Continue on reverse side if necessary and identify by block number) Large uncertainties on target vulnerability and hardness have influenced U.S. strategic targeting and R&D planning. The objective of this study was to develop quantitative methods for analyzing uncertainty in target vulnerability and hardness predictions. Methods for target hardness predictions and the analysis of the associated uncertainty given good intelligence, were evaluated for application to two targets, namely, 1) above-ground industrial building; and 2) shallow-buried command and control bunker. -Stl bunc DD FORM 1473 ECITION OF 1 NOV 6 UNCLASSIFIED CLASSIFICATION OF THIS PAGE (When Data Entered) 401 631

UNCLASSIFIED SECURITY CLASSIFICATION OF THIS PAGE(When Data Entered) 20. ABSTRACT (Continued) conti The procedure for evaluating target hardness uncertainty was started with the formulation of mathematical models of the weapon effects and the target response and failure. Structural responses models for both targets were developed based on classical structural dynamics considerations. However, for the buried bunker, the structural model is more uncertain because of the inadequate data base. Uncertainties on model parameters were characterized and propagated through the analysis using a Taylor-series expansion method, to evaluate the best estimate hardness and the hardness uncertainty as a function of weapon yield. The results were evaluated and compared with the Physical Vulnerability System. UNCLASSIFIED

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#### PRIFACE

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| mils                   | millimeters                                | 0.0254       |
| inches                 | centimeters                                | 2.54         |
| feet                   | meters                                     | 0.3048       |
| miles                  | kilometers                                 | 1.6093       |
| square inches          | square centimeters                         | 6.4516       |
| square feet            | square meters                              | 0.0929       |
| square miles           | square meters                              | 2,589,998.0  |
| cubic inches           | cubic centimeters                          | 16.38706     |
| cubic feet             | cubic meters                               | 0.0283       |
| cubic yards            | cubic meters                               | 0.764555     |
| gallons (U.S.)         | liters                                     | 3.785        |
| gallons (Imperial)     | liters                                     | 4.542        |
| ounces                 | grams                                      | 28.349       |
| pounds                 | kilograms                                  | 0.454        |
| tons (short)           | kilograms                                  | 907.185      |
| tons (long)            | kilograms                                  | 1,016.047    |
| pounds per foot        | newtons per meter                          | 14.59390     |
| pounds per square inch | newtons per square<br>centimeter           | 0.6894757    |
| pounds per cubic inch  | kilograms per cubic<br>centimeter          | 27,679.90    |
| pounds per square foot | newtons per square<br>meter                | 47.88026     |
| pounds per cubic foot  | kilograms per cubic<br>meter               | 16.0185      |
| inches per second      | centimeters per second                     | 2.54         |
| inch-pounds            | meter-newtons                              | 0.1129848    |
| inch-kips              | meter-kilonewtons                          | 0.0001129848 |
| Fahrenheit degrees     | Celsius degrees or<br>Kelvins <sup>a</sup> |              |
| kilotons               | terajoules                                 | 4.183        |

aTo obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use C = (5/9)(F-32). To obtain Kelvin (K) readings, use K = (5/9)(F-32) + 273.15.

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# TABLE OF CONTENTS

1

Casa yang

. . . . .

| 1.0  | INTR | DUCTION   |
|--|------|---|
| 2.0  | STUD | APPROACH  |
|  | 2.1  | Statistical Considerations  |
|  | 2.2  | Evaluation of Uncertainties   |
| 3.0  | INDU | TRIAL TARGET  |
|  | 3.1  | Target Description  |
|  | 3.2  | Structural Response Analysis  |
|  |      | 3.2.1       Side-on Loading       . |
|  | 3.3  | Structural Response Uncertainties   |
|  |      | 3.3.1Uncertainty in Loads.203.3.2Uncertainty in Intelligence Data203.3.3Uncertainty in Target Model and Response213.3.4Uncertainty Propagation21  |
|  | 3.4  | Comparison with PVN System  |
|  | 3.5  | Attacker's Requirements   |
| 4.0  | BURI | D TARGET  |
|  | 4.1  | Target Description  |
|  | 4.2  | Structural Response Analysis  |
|  |      | 4.2.1Elastic Response Phase304.2.2Small Deformation Plastic Response Phase354.2.3Large Deformation Plastic Response Phase374.2.4Structural Capability38   |
|  | 4.3  | Structural Response Uncertainties   |
|  | 4.4  | Comparison With PVN System  |
|  | 4.5  | Attacker's Requirements   |
| 5.0  | CONC | USIONS  |
| 6.0  | RECO | IMENDATIONS   |
| 7.0  | REFE | ENCES   |
| APPENDIX A - FREE FIELD EQUATIONS          |      |   |
| APPENDIX B - ELASTO-PLASTIC ANALYSIS MODEL |      |   |
| APPENDIX C - SIDE-ON LOADING ANALYSIS      |      |   |
| APPENDIX D - END-ON LOADING ANALYSIS       |      |   |

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# LIST OF FIGURES

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| Figure No. | Title Pa  | age | No. |
|------------|---|-----|-----|
| 3-1        | Industrial Building Model • • • • • • • • • • • • • • • • • • •                   | 14  |     |
| 3-2        | Variation of Range with Weapon Yield at Failure<br>(Side On Loading Case)         | 18  |     |
| 3-3        | Variation of Range with Weapon Yield at Failure<br>(End-On Loading Case)          | 19  |     |
| 3-4        | Target Hardness   | 22  |     |
| 3-5        | Attacker's CEP and Yield Requirements for $P_{K} = 0.5$                           | 25  |     |
| 36         | Attacker's CEP and Yield Requirements for $P_{K} = 0.9$                           | 26  |     |
| 4-1        | Bunker Configuration •  | 29  |     |
| 4-2        | Structural Response Analysis  | 31  |     |
| 4-3        | Comparison Between WES and Simplified Analysis for Elastic Bunker (300 psi, 1 MT) | 34  |     |
| 4-4        | Moment-Thrust Yield Criterion   | 36  |     |
| 4-5        | Bunker Response for 1 MT, 300 psi Overpressure                                    | 39  |     |
| 4-6        | Bunker Response for 10 KT, 300 psi Overpressure                                   | 40  |     |
| 4-7        | Bunker Response for 10 KT, 3000 psi Overpressu                                    | 41  |     |
| 4-8        | Failure Criterion Influence on Target Hardness · · ·                              | 43  |     |
| 4-9        | Target Hardness · · · · · · · · · · · · · · · · · ·                               | 48  |     |
| 4-10       | Attacker's CEP and Yield Requirements for $P_{K} = 0.5$ .                         | 49  |     |
| 4-11       | Attacker's CEP and Yield Requirements for $P_{K} = 0.9$                           | 50  |     |

# LIST OF TABLES

| Table No. | Title                                     | Page No. |
|-----------|---|----------|
| 4-1       | Bunker Modal Equations                    | . 32     |
| 4-2       | Systematic Bunker Uncertainties           | . 44     |
| 4-3       | Basis for Parameter Uncertainty Estimates | 46       |
| 4-4       | Bunker Random Variations                  | 51       |

1

#### 1. INTRODUCTION

A methodology for systematically evaluating the uncertainties associated with the prediction of vulnerability and hardness of ground facilities has been used for the past ten years to develop confidence statements, from the defender's point of view, for weapon systems survivability prediction. In this application, the defender theoretically has perfect intelligence of site conditions and structural configurations ani materials, and can establish, analytically or experimentally, the weapon effect environments and structural fragilities to within acceptable uncertainty bounds. Parametric sensitivity analyses are conducted to evaluate the impact of various uncertainties on the system survivability. This information proves to be invaluable for conducting design trade-offs, and for weapon effects research planning.

The present study represents the first attempt to apply such a methodology to analyzing the uncertainties in target vulnerability and hardness predictions. Target vulnerability and hardness and associated uncertainties, given good intelligence, were evaluated for two representative targets: (1) an above-surface industrial building; and (2) a shallow-buried command and control bunker.

The procedure for evaluating target hardness uncertainty was started with the formulation of mathematical models of the weapon effects and the target response and failure. Structural responses models for both targets were developed based on classical structural dynamics considerations. However, for the buried bunker, the structural model is more uncertain because of the inadequate data base. Uncertainties on model parameters were characterized and propagated through the analysis using a Taylor-series expansion method, to evaluate the best estimate hardness and the hardness uncertainty as a function of weapon yield. The results were evaluated and compared with the Physical Vulnetability System.

Uncertainties caused by potential systematic errors of analysis, i.e., bias between the mathematical model and the real world, were distinguished from nonsystematic, random variations of physical properties. When propagated through the weapon effect scaling laws, nonsystematic variations lead to variations in estimated damage distances. These nonsystematic damage distance variations are treated adequately by the Physical Vul nerability System, and for most cases do not cause significant errors in the estimated damage probabilities. However, systematic errors in target hardness estimates cause systematic errors in estimated damage probabilities which are not treated adequately by the Physical Vulnerability System at present. The emphasis of this study is focused upon the problems of quantifying the effects of the systematic errors of analysis.

From the results, it was concluded that for above surface targets such as industrial buildings, the current procedure for determining the best estimate of physical vulnerability is adequate. Uncertainty in the target physical vulnerability can be quantified based on the state-of-the-art knowledge of low level blast environments, material properties, and structural response modeling. Scale model tests of industrial targets would not be worthwhile unless the structural models were accurate representations of a high-value class of uniformly constructed structures.

For the shallow-buried targets such as command bunkers, procedures for physical vulnerability prediction to achieve the desired confidence levels remain to be validated. Of significant uncertainty is the structural response model and failure criteria. Testing of model structures to failure with the objective of developing more accurate prediction techniques is recommended. Additional sensitivity analyses, and research costbenefit trade-off studies should be conducted to provide a basis for setting priority for research projects pertinent to bunker vulnerability.

#### 2.0 STUDY APPROACH

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Target hardness predictions are generally derived analytically, and the uncertainties in these predictions are dependent upon the inputs to the analysis, and also upon the analytical model itself. In the present study, mathematical models for nuclear weapon environments ats and target structural loads and response were developed as a first step. The sources of uncertainties in the model parameters were then identified, and nominal values and uncertainty bounds for these parameters were determined. The analytical models were used to determine both target hardness predictions, and the sensitivity of these predictions to perturbations within the uncertainty bounds of the parameters input to the models. Next, the uncertainties of the parameters were propagated through the structural response model to determine the uncertainty bounds on the target hardness estimates. The final and most difficult step in the quantification of the target hardness uncertainty was to estimate the accuracy of the mathematical model itself, and incorporate this information into the target hardness estimate.

Section 2.1 discusses the distinction between random variations and uncertainties, and how these quantities are defined. The methodology for calculating hardness uncertainties, and the impact of these uncertainties on targeting is given in Section 2.2.

#### 2.1 STATISTICAL CONSIDERATIONS

If a number of cylinders, cast from the same batch of concrete and cured in a carefully controlled manner, were subjected to an unconfined compression test in the same loading device at essentially the same time, identical resistance would not be exhibited by each cylinder. Individual differences in the cylinders, such as air voids in the concrete, which occur randomly and can never be completely eliminated, would cause cylinder-to-cylinder strength variations. Thus, the strength of cylinders cast from a single batch of concrete is a random variable. Random variations are the actual physical variations that occur within a group, such as cylinders cast from the same batch of concrete, or from the same concrete mix specifications.

When a specific cylinder is considered, the unconfined compressive strength is not a random variable, since it has one specific, true value. This true value can be estimated, for example, based on a knowledge of the properties of the concrete batch from which it was cast. However, such an estimate will vary from the true value of strength of this particular cylinder because of random variation among cylinders from the batch. The true value of the unconfined compressive strength of a particular cylinder can be determined by subjecting the cylinder to a compression test.

Even after the test has been performed, there remains some uncertainty in the result. This determination of the true value of the compressive strength of the cylinder is only as good as the accuracy of the test will permit. Errors in the test measurements will cause an error in the determination of the true value of the cylinder strength. In many cases, this uncertainty can have more impact on the outcome of a study than the random variations. Effects of raudom variations tend to be self cancelling, while uncertainty often adds a significant error to the entire study result.

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Uncertainty refers to potential differences between the real world and the perceived world of analytical models and of model parameter values. There will be uncertainty in the measure of central tendency of a random variable (for example, the mean or median value), in the measure of its spread (for example, the standard deviation), and in the distribution function selected for the variable. The uncertainty in any of these statistical properties is the confidence, or the expectation, that the true parameter value is less than a specified value, defined over the range of possible values. (A 50% confidence value implies that the actual parameter value is as likely to be higher as it is to be lower). Uncertainty in the measure of central tendency of a targeting analysis variable, or in the analytical model chosen could introduce significant uncertainty in the targeting analysis results. However, uncertainties in the variance or in the distribution function of an analysis variable are generally not significant. The bounding values of a variable, or the limiting responses associated with the extremes of model assumptions, are the usual bases for estimating uncertainties. There is necessarily a degree of subjectivity on the confidence level ascribed to bounds. Reasonable engineering bounds are sometimes taken to represent 95% confidence bounds. Thus, if the uncertainty in the variable under consideration is normally distributed, these bounds will correspond to a region within plus-and-minus two standard deviations from the mean.

In targeting analysis both random variation and uncertainty effects should be properly treated. The response of a specific concrete structure, subjected to the effects as ociated with a particular nuclear device detonated at a specific location and time, is deterministic. There will be uncertainty, but not random variation, in the response. This is true even though some parameters of the analysis model might be described in terms of both a random variation and an uncertainty distribution, as for example, when the concrete mix specifications are known, but cylinder test data for the actual pour are not available. The random variance associated with the mix strength actually represents an uncertainty on the cylinder strength of the concrete in the specific structure of interest. The effect of this uncertainty is combined with that of other sources of uncertainty to determine the total uncertainty in the target hardness.

Where there are many similar structures constructed using the same mix specifications, the random variation in mix strength will have a smaller effect on the uncertainty in the median structure response if all structures are subjected to similar nuclear loading conditions. Random variation in concrete strength causes some structures to be stronger, and others to be weaker. This variation tends to average out when the median response of a large number of structures is considered. Thus, when analyzing many structures, random variation and uncertainty cannot be combined. Since the random variations tend to be dominated by the weapon miss distance (CEP) distribution, their effects are generally not critical.

An appreciation of the distinction between random variation and up ertainty leads to better understanding of the research options available, in addition to aiding the understanding of complex targeting situations. For example, the ratio of the standard deviations of the uncertainty and the random variations will influence the number of tests required to reduce the uncertainty to an acceptable level. In the present study, random variations in the analysis parameters are assumed to cause only random variations in the system response. However, it is possible that the eff.cts of random parameter variations around the median conditions will not average out, even after a large number of similar structures are considered. If this were the case, a systematic bias would be introduced in the response. For example, if a group of buried structures have a very stiff median foundation modulus, those sites with higher than median stiffness might exhibit nearly the same response as the median site, i.e., the response associated with a rigid foundation. However, those sites with much lower stiffness could have a markedly different response caused by a rigid body mode. The interaction between random variations and uncertainties can possibly be removed by categorizing sites, such that the response phenomenology is the same for all sites in a category, and by performing separate targeting analyses for each category. However, this interaction is treated in more formal statistical survivability modeling, such as in the FAST code (Reference 1).

Another approximation used in this study is that the uncertainty distributions are assumed to be lognormal. This assumption is commonly used in weapon effects analysis because ic conveniently approximates much of the data. If there are a number of uncertain parameters in the system, and these parameters are multiplicative factors affecting the system response, then the Central L.mit Theorem indicates that these uncertainties will be lognormally distributed (Reference 2).

Consistent with the assumption of a lognormal distribution, best estimate values of the parameters are taken as the median value of the distribution. For a continuous variable, the median is defined as the value of the variable at which 50% of the area under the distribution function lies above the median, and 50% below. For a lognormal distribution, the median corresponds to the mean of the logarithm of the variable. Uncertainties are defined in terms of  $2\sigma$  K-factors which represent approximately 95% bounds. This means there is approximately a 95% expectation that the true value of the parameter will be within a factor of K and 1/K of the median value of the parameter. The K-factor is related to the Bundard deviation of the distribution of the logarithm of the variable by

$$2\sigma = \ln K \tag{2-1}$$

# 2.2 EVALUATION OF UNCERTAINTIES

Two primary factors determine the importance of the uncertainties in various parameters affecting target response: (1) the sensitivity of the target hardness, p, to uncertainties in the input parameters,  $x_i$ ; and (2) the uncertainty in the median values of the parameters. If the target hardness is written as

$$\ln p = f(\ln x_{i})$$
 (2-2)

then from Reference 2 (Equation 4.39), the standard deviation in the uncertainty of In p,  $\sigma_{p_j}$ , which is caused by the uncertainty in the particular parameter,  $x_j$ , can be computed from the approximate relationship

$$\sigma_{\mathbf{p}_{j}} = \frac{\partial (\ln \mathbf{p})}{\partial (\ln x_{j})} \sigma_{j} \qquad (2-3)$$

where the partial derivative gives the sensitivity of the target hardness to variations in  $x_j$  and  $\sigma_j$  is the standard deviation in the uncertainty of  $\ln x_j$ . The approximation arises from neglecting terms higher than first order in the Taylor-series expansion of the function, f, used in calculating the standard deviation. Then, from Equation (2-1)

$$\ln K_{pj} = \frac{\partial (\ln p)}{\partial (\ln x_j)} \ln K_j \qquad (2-4)$$

where K and K are the uncertainty factors corresponding to  $\sigma$  and  $\sigma$ , respectively. Continuing with the approach of Reference 2 (Equation 4.44), an expression for the total target hardness uncertainty factor, K, can be derived as

$$K_{p} = \exp\left(\sum_{i=1}^{n} \sum_{j=1}^{n} \rho_{ij} \ln K_{pi} \ln K_{pj}\right)^{1/2}$$
(2-5)

where the  $\rho_{ij}$  are the correlation coefficients between the target hardness uncertainty factors.

Equations (2-4) and (2-5) make possible the evaluation of the relative impact of the parameter uncertainties on the target hardness uncertainty. In addition, the potential benefits in reduced target hardness uncertainty, resulting from reducing the uncertainties in the key parameters can be readily evaluated with these expressions. Whether these uncertainties can be reduced by test must be determined by a separate evaluation of the potential errors associated with the available test techniques. If no suitable techniques exist, then the requirements for simulation capability can be stated in terms of the noeded uncertainty reduction.

To determine the need for uncertainty reduction, the target hardness uncertainties are propagated through the scaling laws for weapon effects to determine damage-distance relationships, and the corresponding uncertainties, as a function of yield. Then, the impact of these uncertainties on the yield and accuracy requirements of the attacker can be evaluated.

For this study, each target was considered to be the object of a single attack aimed directly at that target. Each target was one of many constructed from the same designs and sited in the same generic geology. Therafore, random variations in as-built material properties are about 15 to 30%, and construction tolerances are usually small. For a given generic geology, weapon effect variations are also small, even considering the effect of weather conditions (which are partially known), and total random variation on target damage distances will usually have a standard deviation ( $\sigma_{\rm D}$ ) less that 20% of the median damage distance ( $\overline{\rm R}$ ). Based on the detailed analysis of Reference 3, (Appendix B), the probability of damage,  $P_{\rm K}$ , is very nearly the same as the cookie cutter equation:

$$P_{K} \approx 1 - P_{cc} \qquad (2-6)$$

if  $P_{K} \leq 0.9$  and  $\sigma_{D} \leq 0.2\overline{R}$ . The cookie cutter approximation is that there is zero probability of failure at ranges greater than  $\overline{R}$  and unity at ranges closer-in than  $\overline{R}$ . Then, convoluting this damage-distance model with the weapon miss distance Rayleigh distribution gives

$$P_{cc} = \exp(-0.5\overline{R}^2/\sigma^2)$$
 (2-7)  
 $\sigma = 0.849 \text{ CEP}$ 

A property of the Rayleigh distribution is that there is a 50% probability of the weapon landing within a distance from the target equal to the CEP (circular probable error).

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The random physical variations do not significantly affect the probability of damage computation in the above case. Their effect is much smaller than that caused by random variation of weapon miss distance (the CEP distribution). On the other hand, a systematic uncertainty in a damage distance estimate does affect the confidence in the calculated probability of damage.

The attacker's confidence (expressed as a function of damage distance) is the expectation that uncertainties in the targeting analysis would not result in a damage distance greater than a given value. Confidence estimates are thus based on the systematic uncertainty in damage distance, assuming a lognormal distribution of the uncertainty. The kill probability as a function of confidence level may be determined from Equation (2-6), using this damage distance versus confidence level distribution.

#### 3.0 INDUSTRIAL TARGET

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This section describes the analysis of a typical heavy steel frame industrial building, as shown in Figure 3-1, subjected to dynamic pressure loading. A generalized, single-degree-of-freedom model was utilized because of its tractability and because it has been widely accepted as a useful engineering tool. Work hardening of the structural material was taken into account to arrive at a realistic model for structural behavior. The analysis considers two loading conditions; side-on and end-on. Side-on loading deforms columns and crane rail beams, while end-on loading causes rotation and drop of the crane itself.

Uncertainties in the details of construction of the building, in the loads induced in the structure, and in the response of the structure to these loads are estimated, and propagated throughout the analysis. The magnitude of the uncertainties result in various levels of confidence in the success of an attack. Weapon yield and accuracy requirements are also determined.

## 3.1 TARGET DESCRIPTION

P. ....

The target selected for this study (Figure 3-1) is a typical medium size industrial building, as described in Reference 4. It is a single-story, single-span, multi-bay building with a heavy steel frame designed for an industry involving heavy equipment and the manufacture of large products. The typical span length for this class of building is 80 ft and the bay size is about 40 ft. A 50-ton, column-and-girder-supported travelling crane is mounted on beams, bridging the span of the buildings. Steel columns, built up from rolled steel sections, carry the roof and crane loads, and either concrete or steel roof trusses support the main roof load carrier. The joists on both the top and bottom chords are braced to prevent lateral buckling.

Precast reinforced concrete foundations provide seats for the columns. When the column is inserted and fastened, this joint can carry moments transferred from the columns. The foundations extend about six feet below the finished grade to prevent frost action during winter. The crane support girders are rolled steel beams or steel plate girders. The walls are supported by the foundation beams which bridge the span between column foundations. From the ground level up to about ten feet above the ground, the walls are made of brick or large concrete blocks. Above this height, the walls are made of light weight concrete panels or asbestos-cement corrugated sheeting. Commonly used roofing materials are steel sheet, asbestos-cement, and tile.



#### 3.2 STRUCTURAL RESPONSE ANALYSIS

When the building described in Section 3.1 is subjected to a nuclear attack, several vulnerabilities are possible. However, dynamic pressure and ground shock are the most critical weapon effects for above-the-surface structures. Overpressure and radiation would, of course, be lethal to personnel inside the building, but are not likely to cause permanent damage. Overpressure propagates to the inside of a building of this type faster than structural response, minimizing the effect of differential overpressure between the inside and the outside. At the overpressure levels of interest, structural materials are not affected by radiation, and cratering, debris, and thermal radiation are not significant threats to the building integrity. Electromagnetic pulse might damage electrical equipment and controls, but they can probably be repaired in a few days, or means can be found to operate without them.

Dynamic pressure acts on the side or end of the building like a hurricane, producing a lateral force proportional to the exposed area. Ground shock produces oscillating motions, much like an earthquake, which result in loads dependent on the building mass, and possibly cause differential settlement of columns. Dynamic pressure is the most significant loading mechanism for a building of this type, since there is a relatively large area exposed to the wind, and a relatively small roof mass. Building and crane-beam damage resulting from dynamic pressure are, therefore, the prime failure modes for the industrial building under consideration when subjected to a nuclear attack.

If parts of the roof or siding are blown off by nuclear wind pressure, emergency repairs could be made to permit resumed operations within a few days. However, structural damage to the crane or its supporting structure is likely to require materials which are in short supply in a war situation, and extensive is induction and erection of major structural members. It was assumed that use of the crane is essential to the industrial process carried on within the building, and that rendering the crane inoperative is a suitable measure of a successful attack.

One mode of failure for the overhead crane may be lateral displacement of the crane rail beams, just enough to allow the crane to fall, while not necessarily exceeding the beam yield stress. Another failure mode may involve substantial plastic deformation of the beams. Based on these failure modes, two limiting attack conditions can be considered: a low level attack of sufficient intensity to produce a lateral beam deflection just large enough to allow the crane to drop, provided it is parked at the location where the maximum lateral deformation occurs; or a higher level attack in which the maximum plastic deflection of the beam has been exceeded, causing the beam to full, and drop the crane from any parking position along the rail beam.

The geometry of a typical industrial building suggests that two primary loading orientations should be considered, mide-on and end-on. For the side-on case, the detonation point is located to the side of the building, but not necessarily centered between the columns. This orientation primarily loads the sides of the building, with a much smaller load on the ends of the building.

In the side-on loading case, the siding is assumed to fail first, since it is not designed to support large side pressures across the span between columns. This leaves the columns and crane rail beams exposed to the dynamic blast pressure. Parts of the siding, however, might stay on the beams and columns increasing the exposed area and, therefore, the load. Such loadings tend to move the rail beams out from under the crane, permitting the crane to fall.

For the end-on loading case, the detonation point is located in front of the building, not necessarily centered, so as to give the major loading to the end with a smaller loading acting on the sides of the building. In this loading case, blast pressures act over the side of the crane, Even though such loadings could subject the crane to pressures which are essentially uniform along its length, yaw moments would be created because of the unsymmetrical frontal area exposed to the blast wave. Such moments induced yaw rotation, which may be large enough to rotate the ends of the crane off the rail beams allowing the crane to fall to the floor.

The free field equations used to determine the loads are given in Appendix A. Uncertainties in the modeling, the loads, and the basic variables are discussed in Section 3.3. The models synthesized to represent side-on and end-on loading of the building are discussed in the following sections, with details given in Appendices B to D.

#### 3.2.1 Sida-on Loading

Side pressure on the industrial building will cause rotation of the foundations, bending of the columns as cantilevers, and lateral bending of the beams. The model of the building used in this study consists of two vertical columns attached to the ground through flexible torsional spring support. The rail beam spanning the distance between the tops of the columns has support stiffness between simple and clamped support conditions. The top of the column is assumed to carry a portion of the roof mass. When these structural elements are subjected to the dynamic pressure loading, bending and rotation are induced in the vertical columns and the horizontal rail beams.

The structural response is determined by first approximating the actual system

by the elasto-plastic, single-degree-of-freedom system model, whose governing equations are summarized in Appendix B. In this model, the generalized mass, stiffness, and force are related to the mass and elastic properties of the building, and the dynamic pressure loadings acting on it, by the equations developed in Appendix C.

After the system peak response to a given attack condition was determined, two failure boundaries were computed. The first failure boundary was based on the elastic lateral deformation of the beam just sufficient to allow the crane to fall, if it was located midway between the columns where the maximum lateral displacements occurred. The second failure boundary was based on the maximum stress in the beam exceeding the plastic failure stress, which would cause crane damage regardless of its location.

Figure 3-2 shows the damage distances for both failure criteria, based on the set of median analysis parameters, given in Appendix C. It is seen that there could be a significant enhancement in lethal radius if the crane were parked midway between columns. However, in the vulnerability analysis it was assumed that the crane was parked in its hardest position, near a column, and the plastic failure stress criterion governed. This assumption is reasonable for targeting since the parking position would not be expected to be known.

#### 3.2.2 End-on Loading

Figure 3-3 shows the approximate failure boundary for the end-on loading case. This loading case makes use of a simpler model to determine when crane drop will occur. In the development of the model, it was assumed that the end wall will be destroyed, and the dynamic pressure will act on the face of the crane itself. The dynamic pressure will move the crane along the rail support beams, and at the same time induce a yaw rotation which will tend to rotate the ends of the crane off the rail beams until it falls to the floor. This yaw rotation induced by the dynamic pressure is a consequence of the lack of symmetry in exposed area and loading on the crane. The failure boundary in this case could be based on the actual crane rotation, and the rotation required to cause the crane to fall between the two rail beams at each end of the crane. However, it takes only a relatively small rotation for the crane to be free to fail. Therefore, the failure boundary was approximated by assuming the peak disturbance yaw moment to be equal to the peak resistance yaw moment produced by forces at the crane-rail beam interfaces. A detailed description of the model and the median values of the parameters used is given in Appendix D.







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#### 3.3 STRUCTURAL RESPONSE UNCERTAINTIES

From the attacker's point of view, uncertainties leading to various degrees of confidence in a successful attack on the industrial target may be grouped as follows: uncertainties in weapons effects, uncertainties in intelligence data, and imperfect modeling. In the analysis which follows, these uncertainties are combined with the random distribution of miss distances to obtain probabilities of successful attacks at various confidence levels. To provide a better comparison with World War II damage from Japan, the building in this analysis was assumed to be oriented side-on where the damage criterion is structural failure of the rail beam.

### 3.3.1 Uncertainty in Loads

The empirical relationships between overpressure and range for a given weapon size have been well established from weapon tests at the Nevada Test Site and the Pacific Proving Ground. The dynamic pressure relationships are not so well defined empirically. The procedure used in this study was to estimate overpressures from the empirical Brode equation, and then compute the dynamic pressure by means of the Rankine-Hugoniot equation (See Appendix A). Overpressures were calculated with a  $2-\sigma$ uncertainty range of 1.2. A  $2-\sigma$  uncertainty range of 1.2 means that, if the overpressure uncertainties are lognormally distributed, about 95% of the overpressure values are expected to fall within the range of median/1.2 to median x 1.2.

#### 3.3.2 Uncertainties in Intelligence Data

Intelligence data on targets of interest is necessarily uncertain. This category includes the dimensions, thicknesses, and other properties of the various structural elements. The uncertainties due to intelligence limitations utilized in this study for purposes of illustration are as follows:

| Parameter         | 20 Uncertainty Factor |
|-------------------|-----------------------|
| Beam              |                       |
| Moment of Inertia | 2.0                   |
| Depth             | 1.2                   |
| Weight            | 1.3                   |
| Column            |                       |
| Moment of Inertia | 1.7                   |
| Frontal Depth     | 1.0                   |
| Weight            | 1.5                   |
| Roof              |                       |
| Mass              | 2.2                   |

# 3.3.3 Uncertainty in Target Model and Response

Even if the building's characteristics were perfectly known, response predictions to a given dynamic pressure would have large associated uncertainties. The single-degree-of-freedom approximation used here has been widely accepted as a useful engineering model. Predictions based on this model provide engineering estimates in the linear deformation range. Beyond the yield point, however, predictions are less accurate, since the behavior of engineering structures up to total collapse is difficult to predict. Other uncertainties included in this category are the degree of fixity afforded by joints, the drag coefficients associated with the various shapes, and the amount of wall siding that might stay attached to columns and beams, increasing their exposed area. The modeling uncertainties considered in this study are as follows:

| Parameter                | 20 Uncertainty Factor |
|--------------------------|-----------------------|
| Beam                     |                       |
| Drag Coefficient         | 1.8                   |
| End Fixity               | 3.2                   |
| Area Factor              | 2.2                   |
| Jolumn                   |                       |
| Drag Coefficient         | 1.8                   |
| End Fixity               | 3.2                   |
| Area Factor              | 2.2                   |
| Ductility                | 3.0                   |
| Work Hardening Parameter | 3.2                   |

#### 3.3.4 Uncertainty Propagation

Figure 3-4 shows the uncertainty in target hardness, in terms of the peak dynamic pressure that causes failure (Q), which results from the uncertainties discussed above. The data for this figure were obtained by expanding the expression for dynamic pressure at failure about the means of the various parameters in a Taylor series, and truncating the series to the linear terms. The resulting expression for the variance of Q involves its partial derivatives with respect to each parameter and the uncertainty of the parameter. The derivatives were calculated numerically, since there is no closed form expression for Q. Assuming a lognormal distribution for Q, 10 and 90% confidence levels were calculated for the range of weapon yields of interest and were plotted in Figure 3-4. A 90% attacker's confidence means that there is a 90% probability that the true (but unknown) hardness is equal to or less than the stated value.





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Figure 3-4. Target Hardness

Also shown in Figure 3-4 is a comparison between these analytically derived data, and the actual damage incurred by similar buildings due to the nuclear detonations in Japan during World War II. A recent DNA-sponsored study by Lulejian (Reference 5) has provided a detailed classification and analysis of the damage data compiled by survey teams at the detonation sites in Hiroshima and Nagasaki.

The structural damage to the Japanese buildings was reported in terms of the fraction of damage to the walls and to the roofs of the buildings and the distance of the building from the detonation. For this comparison, the structural damage to the building walls most nearly parallels the measure of building vulnerability used in the present study.

The data base includes 41 single-story heavy steel frame buildings, 38 in Nagasaki and only three in Hiroshima. These buildings are divided into two groups, those with cranes less than 25 tons, and those with cranes greater than 25 tons. The single-story buildings with heavy crunes comparable to the industrial target studied here existed only in Nagasaki, and only six data points are available.

As running a yield of 22 KT and a height of burst of 1640 ft for the Nagasaki detonation, as recommended by Lulejian, the distance from the burst of each of the six applicable data points was converted to a dynamic pressure and plotted in Figure 3-4. Each data point corresponds to a percent damage to the walls, as interpreted by the survey teams. It is  $\epsilon$  ected that the order of 50% damage can be used for general comparison with the analytical results of Figure 3-4.

The 10 and 90% confidence bounds on the data for 50% damage to the walls of the building, calculated using the maximum likelihood estimate method suggested by Lulejian, are also shown in Figure 3-4. These confidence bounds are seen to be in general agreement with the TRW calculated 10 to 90% confidence bounds on damage to the rail beams sufficient to drop the crane. However, since the damage definition and the buildings differ from those used in this study, much significance cannot be attached to the comparison without further study of the responses of the Nagasaki buildings.

3.4 COMPARISON WITH PVN SYSTEM

The Physical Vulnerability System (Reference 6) gives ratings of a range of turgets in terms of hardness at the reference yield of 20 KT, with a correction factor for other yields. The equivalent Physical Vulnerability Numbers (PVN) for the

industrial target under consideration were computed to provide a comparison with these data. Curves for targets with various sensitivities to yield, so-called K-factors, were calculated from the formulas given in Reference 6, and were matched with Figure 3-4. For the best estimate curve, the closest PVN number is 1209. For the curves at 10 and 90% confidence, the corresponding PVN numbers are 809 and 1609, respectively, where confidence is defined as the estimated likelihood that parameters having systematic uncertainty combine to bias the results to the extent indicated.

A comparison was also made between the procedures used in this work and the Newmark method (Reference 7). For a yield of 10 KT, the Newmark method gives a dynamic pressure hardness of 2.7 psi, which compares well with the value of 2.9 psi obtained by the method presented in this report for the same parameter values. The small difference is due mainly to the fact that work hardening was included in the present work, but is neglected in the Newmark method.

#### 3.5 ATTACKER'S REQUIREMENTS

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The weapon yields and accuracies required for kill probabilities  $(P_K)$  of 0.5 and 0.9 are shown in Figures 3-5 and 3-6. These figures are based on the target hardnesses shown in Figure 3-4, and on a Rayleigh-distributed miss distance. The uncertainty bands shown correspond to the effect of the systematic uncertainties on target hardness and damage distance.

For a 10-1 weapon, for example, a CEP of 1000 ft is needed for a 0.9 probability of soverely damaging the building (Figure 3-6). This is the best estimate, or 50% confidence value. To obtain 90% confidence that the 0.9  $P_K$  will actually be achieved, a more accurately guided weapon with a CEP of 750 ft is needed. If systematic uncertainties could be reduced, the CEP requirement for a 0.9 probability of kill with 90% confidence may be relaxed.

A reduction in systematic uncertainties has a similar effect on yield requirements. With a CEP of 1000 ft, for example, a 20-KT weapon is required for a  $P_{\rm X}$  of 0.9 with 90% confidence. This figure could be potentially reduced to meanly 10 KT if the uncertainties could be substantially reduced. It should be noted that if the yield is large enough, CEP requirements are easily met.



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#### 4.0 BURIED TARGET

Hardened buried structures design/analysis procedures typically consider only a moderate amount of yielding, within the domain of small deformations, and apply simplifying approximations which nearly always add conservatism. This background has generally led to significant underestimation of the overpressure (for a given yield) required to collapse such a structure. In fact, no fully buried concrete structure has collapsed when exposed to nuclear airblast loading; with a number of structures retaining their protective capability at overpressure higher than their predicted collapse load.

For targeting considerations the loading required to damage the structure beyond the point of its subsequent usefulness is of primary interest. In such cases, proper modeling of the large deformation response is of paramount importance and use of conservatisms which have been applied to the design process needs to be reconsidered.

In analyzing the response of a rectangular bunker the typical approach has been to model the roof as a single-degree-of-freedom slab with pinned or fixed supports. When the roof deflection reaches a specified factor (ductility ratio) times an elastic deflection, the structure is considered to have failed. However, the inability of the technical community to predict structural failure loads has led to the identification of several key issues regarding this approach, including (1) Is a single-degree-of-freedom model adequate?, (2) If so, what end conditions should be used?, (3) How is the response affected by loading on the bunker sides?, (4) How does structural damping influence the response?, (5) What is the effect of rigid body motions of the whole bunker?, (6) What is the large deflection force-deformation behavior of the structure?, and finally, (7) What is the failure criterion?

Since the analysis uncertainties have a major impact on the targeting uncertainties, some effort was placed in evaluating the analysis model to use in the study. It was determined, based on comparison with more detailed finite element calculational results, that a single-degree-of-freedom model would be adequate. However, tho model used in this study considered the primary response mode of the complete structure and first-order large deflection force-deformation structural behavior, which includes effects of in-plane stresses. Failure criterion is based on the structure achieving a specified ductility or deflection. Rigid body motions of the bunker have not been included directly in this study. However, a straightforward modification of this analysis is all that is required to incorporate inertial loading effects caused by these motions.

In order to evaluate the elastic response portion of this analysis model the bunker configuration studied is identical to that used in the Reference 8 finite element analysis performed by the Waterways Experiment Station (WES). Since little data are available to properly evaluate the large deformation portion of the response, this study primarily andresses the sensitivity of the response to possible variation in the primary model parameters. Estimates of the uncertainties in the model parameters and in the model itself, alorg with the sensitivity coefficients, determine the target hardness uncertainty.

#### 4.1 TARGET DESCRIPTION

A box-type reinforced concrete bunker structure is assumed to be sufficiently long so that the response can be considered two dimensional. Obviously, short structures with the same cross section will be stronger. Figure 4-1 shows the 40-ft wide by 25-ft high bunker to be fully buried under 5-ft of overburden. The 5-ft thick roof slab has 2.0% and 1.5% reinforcement at the slab bottom and top, respectively; while the 4-ft thick walls have 1.0% compression and tension steel. Other properties of the bunker structure and surrounding backfill are shown in the figure. The site medium is dry soil over shale with the depth to the soft rock located approximately at the bunker foundation level.

#### 4.2 STRUCTURAL RESPONSE ANALYSIS

The bunker is considered to be subjected to effects caused by the overpressure loading of a nuclear weapon detonation. The side loading on the walls is based on the stress attenuation associated with the overhead airblast loading. Thus, in this analysis, direct-induced and upstream-airblast-induced effects have been neglected. In layered stiffer geologies these effects could be quite significant and the possibility of the wall being more vulnerable than the roof should be considered.

If the primary response mode of the bunker is known, then "simplified" analysis procedures can be applied. These procedures, which assume the bunker to respond only in this primary mode, would be limited by (1) how closely the actual response corresponds to the assumed mode and (2) knowledge of the complete load-deformation relationships. The roof of the bunker considered in this study is expected to be the most critical vulnerability, which is consistent with the assumed response mode. For conditions where the wall response is most critical a different mode would be considered.

During the initial phase of the bunker response the structure behaves elastically, followed by various degrees of plastic deformation and finally collapse. The following subsections address each of the response phases.

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#### 4.2.1 Elastic Response Phase

The bunker is modeled as four rigidly connected one-way slabs whose motion are primarily due to bending effects, and therefore, only the midplane motion of these slabs is required to define the complete bunker response. The response of the ith slab is then approximated by (Figure 4-2)

$$y_i = \phi_i(x_i) n(t)$$
(4-1)

where  $\phi_i$  (positive inwards) is the normalized mode shape, n is the single-degree-of-freedom response amplitude and  $x_i$  is a coordinate along the midplane of the ith slab (roof, i = 1; walls, i = 2 and 3; floor, i = 4). This results in the undamped equations of motion

$$P_{1} \frac{\partial^{2} y_{1}}{\partial t^{2}} + D_{1} \frac{\partial^{4} y_{1}}{\partial x_{1}^{4}} = \sigma_{1}(x_{1}, t)$$
(4-2)

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$$D_{\pm} = \frac{E H_{\pm}^{3}}{12(1-v^{2})}$$
(4-3)

and  $\rho_i$ ,  $D_i$ ,  $H_i$  and  $\sigma_i$  are the mass density per unit midplane area, flexural rigidity, thickness and transverse applied stress, respectively, of the ith slab and E and V are Young's modulus and Poisson's ratio, respectively, of the concrete. Substituting the Equation (4-1) approximation into the equations of motion, multiplying by  $\phi_i$ , integrating with respect to  $x_i$  and summing over the four slabs yields

$$\dot{\eta} + \omega^2 \eta = \frac{1}{M_{eq}} \sum_{i=1}^{4} \int_0^{L_i} \sigma_i \phi_i dx_i$$
 (4-4)

where

and

$$M_{eq} = \sum_{i=1}^{4} \int_{0}^{L_{i}} \rho_{i} \phi_{i}^{2} dx_{i}$$
(4-5)

$$\omega^{2} = \frac{1}{M_{cq}} \sum_{i=1}^{4} \int_{0}^{L_{1}} D_{i} \frac{\partial^{2} \phi_{i}}{\partial x_{i}^{2}} dx_{i} \qquad (4-6)$$

are the generalized mass and frequency relationships.

Since this analysis considers a bunker configuration for which the roof vulnerability is most critical, the elastic mode shape (Table 4-1) was based on the deflected shape for a static uniform pressure (w) loading on the bunker roof. The moments  $M_A$  and  $M_B$ at the intersections between the walls and roof and walls and floor, respectively, are



Tuble 4-1 Bunker Modal Equations

A. Mode Shape

$$\begin{split} \phi_{1} &= 2 \left[ \left( \frac{x_{1}}{L_{1}} \right) - 2 \left( \frac{x_{1}}{L_{1}} \right)^{3} + \left( \frac{x_{1}}{L_{1}} \right)^{4} \right] + \frac{24M_{A}}{\omega L_{1}^{2}} \left[ - \left( \frac{x_{1}}{L_{1}} \right) + \left( \frac{x_{1}}{L_{1}} \right)^{2} \right] \\ \phi_{2} &= \frac{dM_{A}}{\omega L_{1}^{2}} \frac{D_{1}}{D_{2}} \frac{L_{2}^{2}}{L_{1}^{2}} \left[ -2 \left( \frac{x_{2}}{L_{2}} \right) + 3 \left( \frac{x_{2}}{L_{2}} \right)^{2} - \left( \frac{x_{2}}{L_{2}} \right)^{3} \right] - \frac{M_{B}}{M_{A}} \left[ - \left( \frac{x_{2}}{L_{2}} \right) + \left( \frac{x_{2}}{L_{2}} \right)^{3} \right] \\ \phi_{4} &= \frac{24M_{B}}{\omega L_{1}^{2}} \left[ - \left( \frac{x_{4}}{L_{4}} \right) + \left( \frac{x_{4}}{L_{4}} \right)^{2} \right] \\ where &= \frac{M_{A}}{\omega L_{1}^{2}} = \frac{1}{12} (1 - k_{1}) \quad , \quad \frac{M_{B}}{\omega L_{1}^{2}} = \frac{1}{24} k_{1}k_{2} \\ and &= k_{1} = \frac{k_{2}}{1 - \left( \frac{k_{3}}{L_{2}} \right)^{2}} \quad , \quad k_{2} = \frac{1}{1 + 2} \frac{L_{1}D_{2}}{L_{2}D_{1}} \quad , \quad k_{3} = 1 - k_{2} \end{split}$$

B. Generalized Mass

$$M_{eq} = \frac{Y_{c}}{g} \left\{ \left( H_{1} + \frac{Y_{R}}{Y_{c}} H_{g} \right) \left[ 0.197 - 3.89 \frac{M_{A}}{wL_{1}^{2}} + 19.2 \left( \frac{M_{A}}{wL_{1}^{2}} \right)^{2} \right] L_{1} + 9.75 \delta H_{2} \frac{D_{1}^{2}L_{2}^{4}}{D_{2}^{2}L_{1}^{4}} \left[ \left( \frac{M_{A}}{wL_{1}^{2}} \right)^{2} - 1.94 \frac{M_{A}M_{B}}{\left(wL_{1}^{2}\right)^{2}} + \left( \frac{M_{B}}{wL_{1}^{2}} \right)^{2} \right] L_{2} + 19.2\delta H_{1} \left( \frac{M_{B}}{wL_{2}^{2}} \right)^{2} \right] L_{1} \right\}$$

C. Frequency

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$$\omega^{2} = \frac{1}{M_{eq}} \frac{D_{1}}{L_{1}^{3}} \left\{ \left[ 19.2 - 384 \frac{M_{A}}{wL_{1}^{2}} + 2304 \left( \frac{M_{A}}{wL_{1}^{2}} \right)^{2} + 2304\delta \left( \frac{M_{B}}{wL_{1}^{2}} \right)^{2} \right] + 768\delta \frac{D_{1}L_{2}}{D_{2}L_{1}} \left[ \left( \frac{M_{A}}{wL_{1}^{2}} \right)^{2} - \frac{M_{A}M_{B}}{\left( \frac{wL_{1}^{2}}{wL_{1}^{2}} \right)^{2} + \left( \frac{M_{B}}{wL_{1}^{2}} \right)^{2} \right] \right\}$$

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determined from conditions of slope continuity at the corners, and the equivalent slab lengths,  $L_x$ , are measured from the midpoints of the corners.

In proceeding with the analysis the following simplifying approximations are made regarding the loading: (1) the mass of soil overburden is considered part of the roof which is loaded uniformly by a time varying overpressure, (2) the side loading is also uniform and time varying and is symmetrically applied on both sides of the bunker and (3) the effect of the bottom loading is neglected. Then Equation (4-4) becomes

$$\dot{H} + 2B\omega\dot{H} + \omega^{2}H = \frac{1}{M_{oq}} \left\{ p(t) \left[ \frac{2}{5} - \frac{1}{6} - \frac{M_{A}}{\omega L_{1}^{2}} \right] L_{1} + 2 \sigma_{2}(t) \left[ -\frac{2}{\omega L_{1}^{2}} - \frac{D_{1}L_{2}^{2}}{D_{2}L_{1}^{2}} (M_{A} - M_{B}) L_{2} \right] \right\}$$

$$(4-7)$$

where modal damping has been added with 8 being the percent critical damping and the generalized mass and frequency relationships are given in Table 4-1 (8 being a tracer parameter which is equal to 1.0 for the elastic portions of the response and  $\gamma_{e}$  and  $\gamma_{g}$  are the weight densities for concrete and soil overburden, respectively).

Equation (4-7) is directly integrated using the Brode HOB overpressure function (Reference 9) and an expression for lateral load based on the stress attenuation in a bilinear medium (linear loading wavespeed,  $c_0$ , and unloading wavespeed,  $c_1$ ) for a single exponential approximation ( $p_0 \exp - (t/t_{00})$ ) to the surface pressure (Reference 10, viz.,

$$\sigma(z_{0},t) = kp_{0} \left[ e^{-\frac{1}{t_{00}}(t - \frac{z_{0}}{c_{1}})} + 2\sum_{n} a^{n} e^{-\frac{n^{n}}{t_{00}}} \sinh\left(\frac{a^{n}z_{0}}{c_{1}t_{00}}\right) \right] H(t - \frac{z_{0}}{c_{0}}) \quad (4-8)$$

where

$$\alpha = \frac{1 - c_0 / c_1}{1 + c_0 / c_1}$$

and  $p_0$  is the peak surface overpressure,  $t_{000}$  is the initial slope time intercept, k is the lateral stress coefficient and  $Z_0$  is the mid-depth of the structure.

A purely elastic analysis of the baseline bunker configuration was performed to compare results of this simplified analysis with the more detailed WFS calculational results. This comparison, which is shown in Figure 4-3, indicates the peak deflection in the WES results to be slightly larger than in the simplified analysis (2.9 in. as compared to 2.7 in.). The waveform comparison is good for the first response evele; with the WES calculation exhibiting larger equivalent damping at later cycles, as might be expected. In both cases the response oscillates about the quasi-static solution (static deflection



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Figure 4-3. Comparison Between WES and Simplified Analysis for Elastic Bunker (300 psi, 1 MT) caused by the overpressure at a given time).

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A comparison between the simplified analysis and WES results for their stiffer backfill case does not show good agreement; probably caused by the approximations used in calculating the side loading (e.g. the side loading does not act until the backfill stress wave gets to the midpoint of the wall). Thus modeling of this loading needs further study. However, the baseline configuration of this study considers the soft backfill whose loading does not have a significant effect on the bunker roof response.

### 4.2.2 Small Deformation Plastic Response Phase

The elastic analysis is considered to be applicable until the fully plastic moment develops at the top or bottom corners or at the midpoint of the roof. This discussion assumes that the plastic hinge initially develops at the top corners (the case for the baseline bunker configuration), however, similar analysis methodology would be applicable if yielding initiated at the center.

Yielding is governed by the moment-thrust diagram (Reference 11) shown in Figure (4-4). The diagram is a linear fit between (1) the compression capability of the slab with zero moment,  $P_o$ , (2) the balance point  $M_b$ ,  $P_b$ , corresponding to the maximum moment, (3) the moment capability in pure bending,  $M_p$  and (4) the tension capability of the slab. For compactness the equations governing the points on the yield surface are given in the figure.

It is not known apriori whether yielding at the roof corners will occur within the roof or wall slabs and therefore both possibilities must be checked. The moment is determined from the curvature at the corner as

$$M_{A} = \frac{4D_{1}}{L_{1}^{2}} (1-k_{1})n(t)$$
(4-9)

while the axial load in the roof, determined from the shear (dM/dx) in the wall just below the corner, is given by

$$P_{A1} = \frac{4D_1}{L_2L_1^2} \left\{ 1 - k_1 + \frac{6k_2k_3}{1-k_2 - \frac{k_3^2}{2}} \right\} \eta(t)$$
(4-10)

and similarly for the axial loads in the walls

$$P_{A2} = \frac{24D_1}{L_1^3} n(t)$$
 (4-11)



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Figure 4-4. Moment-Thrust Yield Criterion

The ultimate moment is reached when  $M_A$ ,  $P_{A1}$  or  $M_A$ ,  $P_{A2}$  reach the yield surface.

When the yield hinge develops the stiffness and primary frequency of the structure decrease and the mode shape must be modified. The load, w, used as a basis for calculating the revised mode shape is taken as the average of the static loads required to (1) develop the yield moment,  $M_A^Y$ , at the corners and (2) develop the yield moment,  $M_A^Y$ , at the corners. The mode shape is then proportional to the static deflection caused by this load and by the corner moments  $M_A^Y$ . Thus, the equation for  $M_A$  in Table 4-1 is modified to be

$$\frac{\frac{M_{A}}{WL_{1}^{2}}}{\frac{1-k_{1}}{24}+4\left(1+\frac{M_{C}^{Y}}{\frac{W_{C}}{W_{A}}}\right)}$$
(4-12)

for purposes of calculating the mode shape  $\phi_1$ , and the corresponding equation of motion is

$$\ddot{n} + 2\beta\omega\dot{n} + \omega^{2}n = \frac{1}{M_{eq}} \left\{ p(t) \left[ \frac{2}{5} - 4 \frac{M_{A}}{wL_{1}^{2}} \right] L_{1} + 2M_{A} \left[ \frac{2}{L_{1}} - \frac{24M_{A}}{wL_{1}^{3}} \right] \right\}$$
(4-13)

where the generalized mass  $(M_{eq})$  and frequency ( $\omega$ ) are as defined in Table 4-1, with the tracer parameter  $\delta$  equal to zero. Continuity of displacement and velocity at the center of the roof is the basis for determining initial conditions for the plastic region. This small deformation plastic response formulation is considered to be applicable until the fully plastic moment is developed at the center of the roof.

Subsequent response can result in large deformation since three yield hinges have developed. However, resistance to this motion is still provided by the yield moments and by the in-plane tensile forces which develop as the roof deformation becomes large.

### 4.2.3 Large Deformation Plastic Response Phase

Once three yield hinges develop in the roof then bending effects between hinges will be neglected. The equation of motion of the link between two hinges is given by

$$\ddot{\theta} = \frac{1}{I_A} \left[ \frac{1}{8} p(t) L_1^2 - M_A^Y - M_C^Y - \frac{TL_1}{2} \theta \right]$$
(4-14)

with

$$I_{A} = \frac{1}{24} \frac{\gamma_{c}}{g} (H_{1} + \frac{\gamma_{s}}{\gamma_{c}} H_{s}) L_{1}^{3}$$

$$(4-15)$$

and

$$T = E_{S} (P_{1} + P_{1}^{*}) H_{1} \left[ (1 + \Theta^{2})^{1/2} - 1 \right]$$
(4-16)

is the tension in the roof slab, where  $E_S$  is the steel modulus, and  $\theta$  is the rotation for the link. During this phase of the response the yield moments  $M_A^Y$  and  $M_C^Y$  are reduced according to the moment - thrust criterion of Figure (4-4). When the maximum tension,  $T_c$ , is reached the moments go to zero and the tensile load thereafter is given by

$$T_{o} = f_{y}(p_{1} + p_{1})H_{1}$$
 (4-17)

where f, is the yield stress of the steel.

### 4.2.4 Structural Capability

Figure (4-5) shows the response of the baseline bunker configuration for a 1-MT/300-psi overpressure loading. Initial yielding occurs at the top of the wall at approximately 0.013 secs. If the subsequent response followed the yield surface of Figure (4-4) the wall moment would continue to increase since the axial load increases, from a state below the balance point. However, whether this additional capability is achievable after yielding occurs is questionable and the yield moment has been taken to be constant when the post-yielding axial compression increases (below the balance point). The yield surface is followed when the axial load decreases, as shown in Figure (4-4).

The moment in the center of the roof continues to increase in the plastic region until the yield criterion for the roof is satisfied. (The initial small moment decrease in this region is an inaccuracy in the model caused by the change in mode shape at the beginning of the first plastic region.) At 0.016 secs three yield hinges have developed in the roof and large deformations can occur which cause tension in the roof slab. The residual roof moment capability associated with this tension is indicated in Figure (4-5). A maximum roof deflection of 8.3 inches occurs at time of 0.084 secs, which corresponds to a ductility of 5.5, with respect to the three yield hinge deflection and a ductility of 8.3 with respect to the elastic deflection.

The bunker response to 10 kT/300 psi and 10 KT/3000 psi overpressure loadings is shown in Figures (4-6) and (4-7), respectively. In the latter case, the large roof tension and correspondingly large shear stresses at the top of the wall would actually tend to be relieved by cantilever yielding at the bottom of the wall (currently not included in the model). However, neglecting this effect would not be expected to have a significant effect on the results since (1) the moment caused by the roof tension was relatively small and (2) the shear loading was not considered to reduce the moment capability at the top of the wall (shown to be constant under large deformation).



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Figure 4-5. Bunker Response for 1 MT, 300 ps1 Overpressure

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Figure 4-7. Bunker Response for 10 KT, 3000 ps1 Overpressure

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A more general bunker model should include combined large deformation behavior of the roof and walls and yield conditions which include the effects of shear loading in addition to moment and thrust.

Figure (4-8) shows the damage function corresponding to a baseline ductility of 10 (with respect to three yield hinge deformation) and to a center deflection of 10% of the roof span (3.6 ft or  $\mu$  = 28.5). If the larger deformations were achievable, a significant increase in hardness over the  $\mu$  of 10 case would result, especially at the lower yields (factor of 2.7 for 10 KT compared with 1.6 for 1 MT). For purposes of further comparison damage functions are shown for a roof slab with simply supported and with fixed end conditions; both for ductilities of 10. The "lower bound" simply supported condition reflects hardnesses lower, by a factor of 1.7 at 10 KT and a factor of 1.5 at 1 MT, than determined from the analysis model used, with the corresponding factors for the "upper bound" fixed end conditions being 1.5 at 10 KT and at 1 MT. To investigate effects of wall yielding reducing the roof tension, additional analysis considered the maximum tension to be zero with the minimum moment equal to the roof capability under pure bending (M<sub>p</sub>). These assumptions resulted in a factor of 1.2 increase in hardness for 1 MT and 1.4 increase for 10 KT.

### 4.3 STRUCTURAL RESPONSE UNCERTAINTIES

Uncertainties in the bunker response are associated with uncertainties in (1) the structure-media interaction (SMI) response analysis, (2) the structure configuration, (3) the material properties and (4) the free field weapon effect environments. Since a generic bunker design has been considered, uncertainty sources associated with intelligence data have not been considered. However, even for a high degree of intelligence data (e.g., structural drawings, material specifications and average engineering site properties are known), uncertainties still exist in such parameters as structural material properties, since contractors typically use materials which have properties better than specification. For example, statistical analysis of two sets of test data for 40,000 psi steel reinforcing bars, summarized in Reference (12), shows the actual strength to be 1.19 and 1.27 times the nominal, respectively. Targeting vulnerability analysis can account for material properties which are better than design specification requirements.

Determination of the bunker hardness uncertainties is based on the methodology in Section 2.2. Sensitivities of the hardness to variation in the analysis parameters are determined as a function of yield from a series of response calculations, for specific yields, each of which consider variation of an individual parameter. Table 4-2 summarizes



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Peak Overpressure, pai

# Figure 4-8. Failure Criterion Influence on Target Hardness

| Category  | Parameter<br>×1  | Sensitivity<br><br>d(ln x <sub>1</sub> ) |                             | 20<br>X1<br>Uncert.      | 2c Target<br>Hardness<br>Uncertainty             |  |
|---|--|--|-----------------------------|--------------------------|--|--|
|   |  | 10 KT                                    | 1 MT                        | Factor                   | 10 KT  | 1 MT   |
| Structure<br>Media<br>Interaction<br>Response<br>Analysis | Mode Shape, $\phi^{(1)}$<br>Equiv. Roof Span, $L_1$<br>Failure Criteria, $v$<br>Bending Yield, $M_p$<br>Tensile Yield, $T_p$<br>Dumping, $v^{(2)}$<br>Analysis Model | 2+2<br>0+90<br>0+92<br>0+37              | 1.9<br>0.43<br>0.64<br>0.10 | 1.1<br>3.0<br>1.2<br>1.2 | 1.0<br>1.2<br>2.7<br>1.15<br>1.07<br>1.14<br>2.0 | 1.5<br>1.0<br>1.0<br>1.12<br>1.03<br>1.05<br>1.4 |
|   | Subtotal   | ···                                      |                             |                          | 3.7  | 2.1  |
| Structure<br>Configuration                                | Steel Aron, p & p*<br>Effective Roof Depth,H <sub>1</sub><br>Overburden Depth, H <sub>5</sub><br>Subtotal  | 1.45<br>2.0<br>0.22                      | 1.00<br>1.8<br>0.1)         | 3.1<br>1.07<br>1.5       | 1.15<br>1.14<br>1.09                             | 1.10<br>1.13<br>1.04                             |
| Material<br>Properties                                    | Steel Yield, f <sub>y</sub><br>Concrete Modulum, E <sub>C</sub><br>Backfill Stress<br>Speed, C <sub>O</sub> <sup>(1)</sup>   | 1,34<br>0,40                             | 1.00<br>0.20                | 1.2                      | 1,3<br>1,08<br>1,0                               | 1'<br>1.0/<br>1.5                                |
|   | Subtota1   | ļ  | <u> </u>                    |                          | 1.7  | 1.6  |
| Weapon<br>Effects   | Poak Overpressure, p <sub>o</sub><br>Initial Slope<br>Intercept, t <sub>o</sub>  | 0.73                                     | 0.97                        | 1.2                      | 1.14   | 1.19   |
|   | Subtotal   |  |                             |                          | 1.4  | 1.1  |
|   |  |  | Total                       |                          | 4.0  | 2.3  |

# Table 4-2 Systematic Bunker Uncertainties

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- (1) P = -0.5
- (2) Effect of 20% damping

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the key parameter sensitivities for 10 KT and 1 MT yields, the parameter  $2\sigma$  uncertainty factors and target hardness uncertainty component, corresponding to both yields, associated with each parameter and with the analysis model. The basis for estimating the  $2\sigma$ uncertainty factors in the analysis parameters is summarized in Table 4-3. の日本の非常

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Given reasonably good intelligence information on the structure, it is seen that the analysis parameter uncertainties, particularly the failure criteria, have the most significant effect on target hardness uncertainty. The analysis model uncertainty considers that even if all model parameters are known adequately, the analysis may not predict the response correctly. This uncertainty would be strongly dependent on the bunker being analyzed, i.e., if it were resting on soft soil such that rigid body motions were important or if the side loading had a significant effect on the response, the analysis model uncertainties would be larger than for the bunker considered in this study. Model uncertainties herein are considered to be largest where modeling of the structure yielding is most critical (low yields weapons) but are estimated to be not as large as the effects of failure criteria uncertainties. Additional test data for the large deformation response of model bunkers is required to better define the failure criteria and validate analysis, especially for the low yield weapon loading. Current uncertainties may have an important impact on weapon allocation.

Damping was not considered in the baseline analysis (although it may have been appropriate to consider the order of 20% damping) and it is seen that the effect of damping or its uncertainties is not very significant. The target hardness uncertainties shown in Table 4-2 are based on an analysis which considered 20% damping within the small deformation portion of the response. Rather large hardness increases are obtained if the same damping coefficient is used in both the elastic and plastic region. However, within the plastic region, the damping forces should not imply a total resistance which is significantly beyond the structure capability.

WES analysis results indicate that stiffer backfill media will reduce the roof response. It is estimated that this effect would not be greater than having the roof act as a fixed-end beam. Thus the effect of backfill uncertainty on target hardness is considered to be the same as the effect of mode shape uncertainty. In calculating total target hardness uncertainty this effect is counted only once, which is equivalent to having a correlation coefficient of -0.5 between the target hardness uncertainties corresponding to these parameters. Although used in this study, a lognormal distribution is not an accurate representation of the backfill related uncertainty since softening the backfill does not influence the response nearly as much as stiffening does. However, the approximation will not affect the total uncertainty.

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| Category   | Parameter<br>×i                  | Uncertainty Factor Basis  |  |  |
|--|----------------------------------|---|--|--|
| کی کر <sub>ی</sub> کر <sub>ی</sub> میں <sub>و</sub> ی برنے بانی میں ہیں۔ میں <sub>ا</sub> ی <sub>و</sub> ی م | Mode Shape, 4                    | Fixed-end and simply-supported<br>roof beam analysis results                            |  |  |
| Structure<br>Modia<br>Interaction<br>Response<br>Analysis  | Equiv. Roof Span. L <sub>1</sub> | Width of walls  |  |  |
|  | Failuro Critoria, p              | Difference between $\mu = 10$ and 0.10 $\mu_1$ roof deflection remults                  |  |  |
|  | Bending Yield, M <sub>p</sub>    | Roference (8)   |  |  |
|  | Tonsilo Viold, To                | Concrete tension capability   |  |  |
|  | Damping, A                       | Effect of 20% damping   |  |  |
|  | Analysia Model                   | Untimated banod on failure<br>criteria related uncorcaintion                            |  |  |
|  | Stool Aroa, p & p*               | Engineering entimate  |  |  |
|  | Effoctive Roof Depth, Ng         | Variation in steel nover  |  |  |
| Structure<br>Configuration   | Overburden Depth, U <sub>S</sub> | Design depth of 40 in.; nominal<br>of 60 in.; maximum of 90 in.<br>for the median value |  |  |
| <u></u>  | Stoel Yiold, f <sub>y</sub>      | Roforonco (8)   |  |  |
| Material<br>Proportion   | Concrete Modulus, E <sub>c</sub> | Reference (8): $E_{e} \sim \sqrt{f_{e}}$  |  |  |
|  | Backfill Stross Speed, Co        | Would not have greater effect<br>than fixed-end wlab                                    |  |  |
| Weapon<br>Effects  | Peak Overpressure, po            | Reference (9)   |  |  |
|  | Initial Slope Intercept, tw      | Same ав р <sub>о</sub>  |  |  |

# Table 4-3 Basis For Parameter Uncertainty Estimates

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The total target hardness uncertainty is considerably larger for 10 KT weapons (2 $\sigma$  factor of 4.0) than for 1 MT weapons (2 $\sigma$  factor of 2.3). This is primarily associated with the fact that at high yield the overpressure required to fail the structure is not nearly as dependent on the ductility as is the case for low yield. On the basis of these results damage function curves corresponding to 10%, 50% (best estimate) and 90% confidence are shown in Figure 4-9. The 50% confidence curve is calculated from the median values of all analysis parameters and, based on related analysis, is only very slightly different from the actual median hardness.

# 4.4 COMPARISON WITH PVN SYSTEM

For the best estimate curve, the closest PVN (Reference 6) is 37P7. For the 10 and 90% confidence curves, the corresponding numbers are 33P6 and 41P7, respectively. 4.5 ATTACKER'S REQUIREMENTS

Attacker requirements generally consider probabilities of kill,  $P_{\rm K}$ , of much greater than 50%. Random variations will have some influence on these requirements since the targeting may need to consider near-worst-site conditions. However, these are generally negligible compared to the random variations introduced by the GEP distribution.

Table 4-4 gives estimates of random variation effects for both the 10 KT and 1 MT yields. Based on these data and on the systematic uncertainty, the weapon yield and CEP accuracy required for  $P_K$  of 0.5 and 0.9 was calculated. The results are shown in Figures 4-10 and 4-11 for confidence levels of 10, 50 and 90%. Table 4-4 gives estimates of random variation effects for both the 10 KT and 1 MT yields. However, as discussed in Section 2, these effects do not significantly influence targeting requirements.

104 103 (11) 3727 33P6 Weapon Yield, KT It tracker's Confidence 102 And the second s Lene Lariane 102 101 100 103 102 102 104

Figure 4-9. Target Hardness

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CEP, ft

Figure 4-10. Attacker's CEP and Yield Requirement for  $P_{K}$  =0.5

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CEP, ft

Figure 4-11. Attacker's CEP and Yield Requirement for  $P_{K} = 0.9$ 

| Category                   | Parameter<br><sup>X</sup> i                | Sensitivity<br>$\frac{\partial(\ln p)}{\partial(\ln x_i)}$ |       | 2σ<br>× <sub>i</sub><br>Uncert. | 2ơ Target<br>Hardness<br>Uncertainty |      |
|----------------------------|--|--|-------|---------------------------------|--------------------------------------|------|
|                            |  | 10 KT  | 1 MT  | Factor                          | 10 KT                                | 1 MT |
| Structure<br>Configuration | Steel Area, p and p*                       | 1,45   | 1.00  | 1.0                             | 1.0                                  | 1.0  |
|                            | Effective Roof Depth,H                     | 2.0  | 1.8   | 1.04                            | 1.08                                 | 1.07 |
|                            | Overburden Depth, H <sub>S</sub>           | 0.22   | 0.11  | 1.5                             | 1.09                                 | 1.04 |
|                            | Subtotal                                   |  |       |                                 | 1.1                                  | 1.1  |
| Material<br>Properties     | Steel Yield, f <sub>v</sub>                | 1.34   | 1.00  | 1.2                             | 1.3                                  | 1.2  |
|                            | Concrete Modules, E                        | 0.40   | 0,20  | 1.2                             | 1.08                                 | 1.04 |
|                            | Backfill Stress Speed,<br>C.               |  |       |                                 | 1.6                                  | 1.5  |
|                            | Subtotal                                   |  |       |                                 | 1.7                                  | 1.6  |
| Weapon<br>Effects          | Peak Overpressure, p <sub>o</sub>          | 0.73   | 0.97  | 1.1                             | 1.07                                 | 1.10 |
|                            | Initial Slope Intercept,<br>t <sub>o</sub> | 1.52   | 0,77  | 1.1                             | 1,16                                 | 1.08 |
|                            | Subtotal                                   |  |       |                                 | 1.2                                  | 1.1  |
|                            |  |  | Tetal |                                 | 1.8                                  | 1.6  |

# Tatle 4-4 Bunker Random Variations

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### 5.0 CONCLUSIONS

This study demonstrates the use of an analytical methodology for the evaluation of the uncertainties associated with the prediction of target vulnerability. Two types of uncertainties are properly distinguished, namely: random variations, which represent actual physical variations; and uncertainties which represent potential bias between the real world and the perceived world of the analytical models and model parameters.

Random variations affect the damage probability of a given target (or the expected number of damaged targets for attacks on similar targets) and are treated adequately by the Physical Vulnerability System. However, the more significant systematic uncertainties are not treated adequately by this system. Where high confidence of high target damage probability is required, the targeting requirements for most cases are governed by the systematic uncertainties.

The targeting uncertainties for the industrial building studied are largely associated with the unique construction of a heavy crane building. These uncertainties could be reduced by experiments designed to provide force-versus-deflection data for structures similar to the 50-T crane building analyzed. However, the cost-to-benefit ratio for such experiments would be prohibitive unless correlations could be established between the representative structures tested and a number of industrial targets.

The targeting uncertainties for the buried bunker analyzed are largely associated with the fact that the analytical procedures have not been validated. The results of this study indicate that simplified analytical models which include the primary response mechanisms, such as the effect of wall flexibility and post-yielding resistance, are adequate. An advantage of a mechanistic model over a finite element analysis is that it is not restricted to a continuum description of the structure analyzed.

Additional test data are needed in several areas to reduce the uncertainties in the present analysis. Dynamic testing should consider representative construction techniques, backfill placement, and site media where possible. Initial testing should consider the smallest size models for which these conditions could be satisfied. The airblast loading should be simulated for times corresponding to structural failure. Both high and low yield testing is desirable, but the high pressure, low yield end of the hardness curve should be emphasized since the uncertainties are largest. Testing at the high yield end provides baseline hardness data.

Both the simplified analysis model and appropriate finite element analysis should be used for pre-test preduction and post-test evaluation. To support this effort, several features could be added to the simplified model, including (1) improved treatment of the bunker side loading, (2) rigid body motion of the bunker, and (3) effects of soil arching. The specific modeling of the soil-media-interaction phenomenology needs to be verified by comparison with results of finite element analysis and test data.

### 6.0 RECOMMENDATIONS

Based upon the results of this study, the following general recommendations are

1) Error analyses of target hardness estimates should be associated with each analytically derived target hardness estimate so that the potential users of such information can evaluate the associated risks.

made:

2) The  $\sigma_{\rm p}$  term of the Physical Vulnerability System should not be

used to include systematic errors of analyses. The  $\sigma_{\rm D}$  term very often has little effect upon the probability of damage. Hence including analytical uncertainty understates the risk of significant errors in estimated damage probabilities.

- 3) Additional testing of scale model bunkers should establish an empirical basis for predicting large deformation and establish failure criteria of this class of structure.
- 4) Simple analytical formulations for buried bunker response should be validated with test data.
- 5) Sensitivity analyses should be conducted to provide a data base to aupport weapon effectiveness evaluation and research planning.

### 7.0 REFERENCES

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### APPENDIX A FREE FIELD EQUATIONS

### A.1 Peak Overpressure

Equation (3-3) of Reference 11 expresses the peak overpressure  $p_{so}$ in psi in terms of the yield W expressed in MT and radial distance r expressed in kft created by a surface burst by the equation

$$P_{so} = \frac{3300W}{r^3} + \frac{192}{r} \sqrt{\frac{W}{r}}$$
 (A1)

# A.2 Peak Dynamic Pressure

Equation (3-10) of Reference 11 expresses the peak dynamic pressure  $q_0$  (psi) in terms of the peak overpressure  $p_{s0}$  (psi) and ambient pressure  $p_0$  (psi) by the equation

$$q_{o} = \frac{2.5 p_{so}^{2}}{p_{so}^{2} + 7 p_{o}}$$
 (A2)

### A.3 Time Variation of Dynamic Pressure

The peak dynamic pressure occurs shortly after the arrival of the blast pressure and subsequently decays approximately exponentially with time. It is assumed here the dynamic pressure q can be expressed by the equation

$$q = q_o e^{-\alpha t}$$
 (A3)

Approximations of the decay parameter  $\alpha$  or the time constant

$$\tau = 1/\alpha \tag{A4}$$

are obtained as follows:

Figure 3-21 of Reference 11 shows the variation of dimensionless dynamic pressure  $q/q_0$  with dimensionless time  $t/t_u$ . The time  $t_u$ represents the duration of the positive phase of the dynamic pressure. Figure 3-21 displays the variation of  $q/q_0$  for several peak overpressures



 $p_{so}$  (or peak dynamic pressures  $q_0$ ) for a 1 MT yield weapon. Values of  $t/t_u$  read from the curves at  $q/q_0 = 0.1$  are listed in Table Al in column 3. (The subscript is appended to denote the time when  $q/q_0 = 0.1$ .) The corresponding  $p_{so}$  and  $q_0$  values for these curves are listed in columns 1 and 2. Values of  $t_u$  are shown in column 4 which have been read from Figure 3-19 in Reference 11, corresponding to the value of  $p_{so}$ . Setting  $q = 0.1 q_0$  in Equation (A3) and solving for  $\alpha$  gives

$$\alpha = \frac{\ln(10)}{t_1}$$
(A5a)

or

$$\tau = \frac{t_1}{\ln(10)}$$
(A5b

Values of  $\tau$  listed in column (5) in Table A1 are obtained from Equation (A5b).

Figure Al shows the variation of  $\tau$  with  $q_0$ . The data indicate the variation can be approximated by a straight line on a log-log plot. This implies  $q_0$  can be approximated by the equation

$$lnq_{O} = ln(A) + B ln \tau$$
 (A6)

The straight line approximation in Figure Al corresponds to A = 0.25 and B = -2.72, so that (A6) becomes

$$lnq_{-} = -1.39 - 2.72 lu \tau$$

or

$$\tau = e^{-\left(\frac{1.39 + \ln q_o}{2.72}\right)}$$
(A7)

Equation (A7) represents the time constant for a 1 MT weapon. The time

| 1                                     | 2                                      | 3   | 4                                       | 3                  |
|---------------------------------------|--|---|---|--------------------|
| Overpressure<br>p <sub>so</sub> (psi) | Dynamic Press.<br>q <sub>o</sub> (psi) | t <sub>1</sub> /t <sub>u</sub><br>At q/q <sub>o</sub> = 0.1<br>Figure 3-21* | t <sub>u</sub><br>(sec)<br>Figure 3-19* | (sec)<br>Eq. (A5b) |
| 10                                    | 2                                      | 0.43  | 2.5                                     | 0.467              |
| 20                                    | 8                                      | 0.31  | 2.3                                     | 0.310              |
| 50                                    | 40                                     | 0.107   | 2.9                                     | 0.135              |
| 100                                   | 118                                    | 0.086   | 2.8                                     | 0.105              |

# Table A1: Dimensionless Time Corresponding to a Fixed Dynamic Pressure Ratio

constant for a weapon of yield W(MT) is

$$\tau = W^{1/3} e^{a_8 + \ln q_0}, a_8 = 1.39, a_9 = 2.72$$
(A8)

\*See Reference 11



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Figure A1: Variation of Time Constant with Peak Overpressure (W = 1MT).

# APPENDIX B ELASTO-PLASTIC ANALYSIS MODEL

Solutions of the single-degree-of-freedom elasto-plastic system considered are derived in this section for the idealized system illustrated in Figure B1.

# B.1 Elastic Range

The governing differential equation for the first elastic range  $0 \le y \le y_e$  is

$$m\ddot{y} + ky = P_{o}e^{-\alpha t}$$

$$(B1)$$

$$y(0) = \ddot{y}(0) = 0$$

and the solutions are

$$y = \frac{P_{o}}{m(\alpha^{2} + \omega_{1}^{2})} \left[ \frac{\alpha}{\omega_{1}} \sin\omega_{1}t - \cos\omega_{1}t + e^{-\alpha t} \right]$$

$$\dot{y} = \frac{P_{o}}{m(\alpha^{2} + \omega_{1}^{2})} \left[ \alpha\cos\omega_{1}t + \omega_{1}\sin\omega_{1}t - \alpha e^{-\alpha t} \right]$$
(B2)

where

$$\omega_1 = \sqrt{\frac{k}{m}}$$
(B3)

If the disturbance force is not large enough to cause the displacement to exceed the elastic limit  $y_e$ , (B2) are valid for all time in which case the displacement increases, reaches a maximum and then decreases. The time at maximum is denoted by  $t_2$  and is obtained by setting  $y(t_2) = 0$  in (B2) obtaining

$$e^{-\alpha t_2} = \cos \omega_1 t_2 + \frac{\omega_1}{\alpha} \sin \omega_1 t_2$$
 (B4)







The corresponding maximum value is obtained from (B2)

$$y_{\max} = \frac{\frac{P_o}{m(\alpha^2 + \omega_1^2)}}{\frac{1}{\omega_1}} \left[ \frac{\alpha}{\omega_1} \sin \omega_1 t_2 - \cos \omega_1 t_2 + e^{-\alpha t_2} \right]$$

which can be simplified by making use of (B4) to obtain

$$y_{\max} = \frac{P_o}{m\alpha\omega_1} \sin\omega_1 t_2$$
(B5)

Suppose the disturbance force is large enough to cause the displacement to exceed  $y_e$ . Then equations (B2) are valid until time  $t_1$  when  $y = y_e$ . This time is obtained from (B2) by substituting  $t = t_1$  and  $y = y_e$  to obtain

$$\frac{m(\alpha^2 + \omega_1^2)y_e}{P_o} - e^{-\alpha t_1} = \frac{\alpha}{\omega_1} \sin\omega_1 t_1 - \cos\omega_1 t_1$$
(B6)

The corresponding velocity at time  $t_1$  is

$$\mathbf{v}_{1} = \mathbf{\dot{y}}(t_{1}) = \frac{\mathbf{P}_{o}}{\mathbf{m}(\alpha^{2} + \omega_{1}^{2})} \left[ \alpha \cos \omega_{1} t_{1} + \omega_{1} \sin \omega_{1} t_{1} - \alpha e^{-\alpha t_{1}} \right]$$

This can be simplified by making use of (B6) to eliminate  $e^{-1}$  to obtain

$$v_1 = \frac{P}{m\omega_1} \sin\omega_1 t_1 - \alpha y_e$$
(B7)

# B.2 Plastic Range

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For those cases where  $P_o$  is large enough for the displacement to exceed  $y_e$ , the governing equations are  $(y > y_e)$ 

$$m\tilde{y} + k_1 y = P_0 e^{-\alpha t}$$
 (B8)

$$y(t_1) = y_e$$

$$y(t_1) = v_1 = \frac{P_o}{m\omega_1} sin\omega_1 t_1 - \alpha y_e$$
(B9)

The general solution of (B8) is

$$y = A \sin \omega_2 t + B \cos \omega_2 t + \frac{P_o e^{-\alpha t}}{m(\alpha^2 + \omega_2^2)}$$
(B10)

where

$$\omega_2 = \sqrt{\frac{k_1}{m}}$$
(B11)

The constants A and B obtained by imposing conditions (B9) are found to be

$$A = C_{1} \sin \omega_{2} t_{1} + \frac{C_{2}}{\omega_{2}} \cos \omega_{2} t_{1}$$

$$B = C_{1} \cos \omega_{2} t_{1} - \frac{C_{2}}{\omega_{2}} \sin \omega_{2} t_{1}$$
(B12)

where

$$C_{1} = y_{e} - \frac{\frac{P_{o}e^{-\alpha t_{1}}}{m(\alpha^{2} + \omega_{2}^{2})}}{\frac{-\alpha t_{1}}{m(\alpha^{2} + \omega_{2}^{2})}}$$
(B13)
$$C_{2} = v_{1} + \frac{\frac{P_{o}\alpha e}{m(\alpha^{2} + \omega_{2}^{2})}}{\frac{1}{m(\alpha^{2} + \omega_{2}^{2})}}$$

Let  $t_3$  represent the time when (B10) reaches its maximum value. This occurs when  $\dot{y}(t_3) = 0$  which gives

$$\frac{P_{o}^{\alpha e}}{m(\alpha^{2} + \omega_{2}^{2})} = A\omega_{2}\cos\omega_{2}t_{3} - B\omega_{2}\sin\omega_{2}t_{3}$$
(B14)

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The corresponding maximum displacement obtained from (B10) is

$$y_{max} = y(t_3) = Asin\omega_2 t_3 + Bcos\omega_2 t_3 + \frac{P_o e^{-\alpha t_3}}{m(\alpha^2 + \omega_2^2)}$$

As before, this can be simplified by making use of (B14) to obtain

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$$y_{max} = \left(A - B \frac{\omega_2}{\alpha}\right) \sin \omega_2 t_3 + \left(B + A \frac{\omega_2}{\alpha}\right) \cos \omega_2 t_3$$

Upon substituting for A and B and performing a few algebraic steps, one can also express  $y_{max}$  as

$$y_{max} = \left(y_{e} + \frac{v_{1}}{\alpha}\right) \cos\left[\omega_{2}(t_{3} - t_{1})\right] + \frac{1}{\alpha\omega_{2}} \left(v_{1}\alpha - \omega_{2}^{2}y_{e} + \frac{P_{o}e^{-\alpha t_{1}}}{m}\right) \sin\left[\omega_{2}(t_{3} - t_{1})\right]$$
(B15)

# APPENDIX C SIDE-ON LOADING ANALYSIS

### C.1 Introduction

In this derivation it is assumed the detonation point is located so as to subject the side of the building to the blast wave. The angle between the detonation point and a perpendicular to the rail beams which support the crane is denoted by  $\psi$ . When  $\psi = 0$ , the detonation point is centered on the side of the building and would not subject the front of the building to a dynamic pressure wave. When  $\psi$  is different from zero both the side and front of the building are subjected to dynamic pressure loading.

Failure is considered to be initiated when the crane support beam deforms sufficiently to allow the crane to fall to the floor below. The minimum deformation corresponds to the crane located where the maximum rail deformation occurs, nominally at the midpoint of the rail. Failure is considered to be a certainty if the deformations are large enough to exceed the plastic failure limit.

Equations relating weapon yield and range to the two failure limits are derived here for a single degree of freedom representation of the crane raii and its support columns. The single degree of freedom system is considered to be an elasto-plastic system which may have a work hardening behavior after yield begins in the rail beam.

The lateral displacement of the crane rail, denoted by y, corresponds to the total deformation that would be measured at the center of the rail beam. This displacement consists of the sum of the displacement of the top of the vertical columns (where the rail beams are attached) plus the displacement at the center of the rail beam relative to its end attachment points. The single degree of freedom approximation begins with a determination of the linear spring rates and static mode shapes. The spring rate is determined by applying a load P at the center of the crane support beams. One half of this load is reacted by each of the two columns. Equations are derived for various end fixity conditions both at the beam-column interface and also at the

column-ground interface. Once the deformed shapes have been determined, the generalized mass, spring constant and aerodynamic forces are determined. The column-support beam system and its single degree of freedom approximation are illustrated in Figures 1 and B1. Figure C1 shows an idealization of the elastic members when subjected to the load P. A list of the parameters used in this analysis is given in Table C-1.

# C.2 Beam Deformed Shape and Spring Constant

The moment at any point  $x_2$  (in the range  $0 \le x_2 \le (\ell_2/2)$  due to the load P is

$$M = M_0 - \frac{P}{2} x_2$$
 (C1)

The root moment  $M_{O}$  corresponds to the moment reacted by the column. If the beam is simply supported, the moment  $M_{O}$  is zero. By assuming the beam is uniform one can write

$$EI_{2} \frac{d^{2}y_{2}}{dx_{2}} = M_{0} - \frac{P}{2}x_{2}$$
 (C2)

and then by assuming

$$y_2(x_2 = 0) = 0$$
 and  $\frac{dy_2}{dx_2}\left(x_2 = \frac{\ell_2}{2}\right) = 0$  (C3)

Equation (C2) integrates to give

$$EI_{2}y_{2} = \left(\frac{Pk_{2}^{2}}{16} - \frac{M_{o}k_{2}}{2}\right)x_{2} + \frac{M_{o}}{2}x_{2}^{2} - \frac{Px_{2}^{3}}{12}$$
(C4)

The support moment  $M_0$  can be eliminated by introducing a fixity factor  $F_2$  defined as

$$F_{2} = \frac{M_{o} \text{ (corresponding to actual system)}}{M_{o} \text{ (corresponding to a clamped beam)}}$$
(C5)

# Table C1 Parameters for Side-on Loading Case

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| PARAMETER   | MEDIAN            |  |
|---|-------------------|--|
| neutral axis to outer fibers of beam (in)           | 6.0               |  |
| column drag coefficient (dimensionless)             | 1.3               |  |
| beam drag coefficient (dimensionless)               | 1.3               |  |
| column frontal depth (in)                           | 14.8              |  |
| beem frontal denth (in)                             | 36                |  |
| Young's modulus for column (net)                    | 3v10 <sup>7</sup> |  |
| Young's modulus for beam (psi)                      | 3×10 <sup>7</sup> |  |
| column and finity factor (dimensionless)            | 0.5               |  |
| been and fighty factor (dimensionless)              | 0.5               |  |
| beam end fixity factor (dimensionless)              | 0.5               |  |
| acceleration of gravity (in/sec)                    | 300               |  |
| column area moment of inertia (in )                 | 1700              |  |
| beam area moment of inertia (in )                   | 300               |  |
| column length (ft)                                  | 33                |  |
| beam length (ft)                                    | 40                |  |
| lumped roof mass (lb sec <sup>2</sup> /in)          | 25                |  |
| ambient pressure (psi)                              | 14.7              |  |
| range to ground zero (kft)                          | 5                 |  |
| weapon yield (KT)                                   | 20                |  |
| column weight/unit length (1b/ft)                   | 140               |  |
| beam weight/unit length (1b/ft)                     | 170               |  |
| minimum displacement of beam for crane<br>drop (in) | 6                 |  |
| work hardening parameter (dimensionless)            | 0.25              |  |
| column area factor (dimensionless)                  | 2                 |  |
| beam area factor (dimensionless)                    | 2                 |  |
| ductility parameter (dimensionless)                 | 6                 |  |
| beam yield stress (ksi)                             | 36                |  |
|   |                   |  |

and the governing differential equation is

$$EI_{1} \frac{d^{2}y_{1}}{dx_{1}^{2}} = \frac{PL_{1}}{2} - \frac{Px_{1}}{2}$$
(C13)

Solutions of (C13) subject to the conditions

$$y_1(x_1 = 0) = 0, \quad \frac{dy_1}{dx_1}(x_1 = 0) = \theta_1$$
 (C14)

$$EI_{1}y_{1} = (EI_{1}\theta_{1})x_{1} + \frac{P\ell_{1}}{4}x_{1}^{2} - \frac{Px_{1}^{3}}{12}$$

$$EI_{1}\frac{dy_{1}}{dx_{1}} = EI_{1}\theta_{1} + \frac{P\ell_{1}}{2}x_{1} - \frac{Px_{1}^{2}}{4}$$
(C15)

By letting  $\delta_1$  represent the deflection at the top of the column  $(x_1 = \ell_1)$  (C15) gives

$$\delta_1 = \theta_1 \ell_1 + \frac{P \ell_1^{3}}{6EI_1}$$
(C16)

Note that if the base was pinned, the system would be unstable so one cannot introduce a fixity factor based on root moment that can be considered to be zero in the limit. The column fixity factor  $F_1$  is defined as

$$F_1 = 1 - \frac{\delta_1 \text{ (due to rotation of base)}}{\delta_1 \text{ (due to bending of column)}}$$
(C17)

With this definition  $F_1 = 1$  corresponds to a clamped base, similar to  $F_2 = 1$  corresponding to a clamped support for the beam. However,  $F_1 = 0$  corresponds to a root support in which the top of the column
With this definition  $F_2 = 0$  corresponds to a simply supported beam, and  $F_2 = 1$  corresponds to a clamped beam. For a clamped beam

$$M_{oc} = \frac{P_{2}}{8}$$
(C6)

Introducing (C6) into (C5) and solving for  $M_{o}$  gives

$$M_{o} = F_2 M_{oc} = \frac{P_2^2}{8} F_2$$
 (C7)

which introduced into (C4) gives

$$EI_{2}y_{2} = \frac{P\ell_{2}^{3}}{16} (1 - F_{2}) \left(\frac{x_{2}}{\ell_{2}}\right) + \frac{P\ell_{2}^{3}}{16} F_{2} \left(\frac{x_{2}}{\ell_{2}}\right)^{2} - \frac{P\ell_{2}^{3}}{12} \left(\frac{x_{2}}{\ell_{2}}\right)^{3}$$
(C8)

By letting  $\delta_2$  represent the deflection at the center (where the load P is applied) relative to the top of the column, (C8) gives

$$\delta_2 = \frac{P L_2^3}{E I_2} \left( \frac{4 - 3F_2}{192} \right)$$
 (C9)

The corresponding beam spring constant  $k_{\rm b}$  equal to  $P/\delta_2$  is

$$k_{\rm b} = \frac{192\alpha_2}{4 - 3F_2} \tag{C10}$$

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$$a_2 = \left(\frac{EI}{\ell^3}\right)_{\text{beam}}$$
 (C11)

## C.3 Column Deformed Shape and Spring Constant

The moment at any point  $x_1$  of the column shown in Figure CL due to the end load P/2 is

$$M = \frac{P_{1}}{2} - \frac{P_{1}}{2}$$
(C12)

has equal displacements contributions due to column bending and column rotation. By making use of (C16) Equation (C17) becomes

 $F_1 = 1 - \frac{\theta_1 \ell_1}{P \ell_1^{3/6EI_1}}$ 

or

$$\theta_1 \ell_1 = (1 - F_1) \frac{P \ell_1^3}{6EI_1}$$
 (C18)

which substituted into (C16) gives

$$S_1 = (2 - F_1) \frac{P \ell_1^3}{6EI_1}$$
 (C19)

Hence, the column spring constant  $k_c = (P/2)/\delta_1$  becomes

$$k_{c} = \frac{3\alpha_{1}}{2 - F_{1}}$$
(C20)

where

$$\alpha_1 = \left(\frac{EI}{\ell^3}\right)_{\text{column}} \tag{C21}$$

By making use of (C19) Equation (C15) gives the deflection at any point to be

$$y_1 = \frac{P}{12\alpha_1} \left[ 2(1 - F_1) \frac{x_1}{x_1} + 3\left(\frac{x_1}{x_1}\right)^2 - \left(\frac{x_1}{x_1}\right)^3 \right]$$
 (C22)

# C.4 Effective Spring Constant

The effective spring constant of the single degree of freedom approximation is composed of a parallel combination of two column springs in series with the beam spring. Hence, the effective spring constant k is

$$k = \frac{(2k_c)(k_b)}{2k_c + k_b}$$
 (C23)

where  $k_{c}$  and  $k_{h}$  are given in (C10) and (C20).

C.5 Relation Between Beam Yield Stress and Deformation

The deflection y that produces a bending stress equal to the yield stress in the outermost fibers of the beam is denoted by  $y_e$ . The corresponding relative deformation in the beam is denoted by  $\delta_{2e}$ . The relation between the yield stress  $\sigma_y$  and the deformation of single degree of freedom system  $y_e$  is derived in this section. The total deformation of the single degree of freedom system y is related to column and beam deformations by the equation

$$y = \delta_1 + \delta_2 \tag{C24}$$

Since the same force exists in the equivalent system, the beam and in the columns, one can write

$$ky = 2k_c \delta_1 = k_b \delta_2$$
 (C25)

From (C23), (C24) and (C25) one obtains the relative deformation of the beam center  $\delta_2$  and the deformation of the top of the column  $\delta_1$  for a given total deformation y to be

$$\delta_{1} = \left(\frac{k}{2k_{c}}\right) y$$

$$\delta_{2} = \left(\frac{k}{k_{b}}\right) y$$

$$(C26)$$

The maximum bending moment occurs at the center of the beam and according

to Equation (C8) is (sign was changed to make moment positive)

$$M_{max} = EI_2 \frac{d^2 y_2}{dx_2^2} \bigg|_{x_2 = \ell_2/2} = \frac{P\ell_2}{8} (2 - F_2)$$
(C27)

The maximum moment can be expressed in terms of  $\delta_2$  by solving (C9) for F and introducing the result in (C27) to obtain

$$M_{max} = \frac{24EI_2\delta_2}{\ell_2^2} \left(\frac{2-F_2}{4-3F_2}\right)$$
(C28)

and the corresponding maximum bending stress  $\sigma_{max}$  in the outer fibers located a distance C from the neutral axis is

$$\sigma_{\max} = \frac{M_{\max}C}{I_2} = \frac{24EC}{\ell_2^2} \left(\frac{2-F_2}{4-3F_2}\right) \delta_2$$
(C29)

At a deflection which produces the yield stress one writes  $\delta_2 = \delta_2$ and  $\sigma_{max} = \sigma_y$  and (C29) gives

$$\sigma_{y} = \frac{24EC}{\ell_{2}^{2}} \left( \frac{2 - F_{2}}{4 - 3F_{2}} \right) \delta_{2e}$$
(C30)

By making use of (C26) the deflection  $y_e$  can also be expressed as

$$v_{y} = \frac{24EC}{k_{2}^{2}} \left( \frac{2 - F_{2}}{4 - 3F_{2}} \right) \left( \frac{k}{k_{b}} \right) y_{e}$$
 (C31)

## C.6 Generalized Mass

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The generalized mass m of the equivalent single degree of freedom system is obtained from the equation

$$my^{2} = 2\int_{0}^{l} y_{1}^{2} dm_{1} + 2\int_{0}^{l} (y_{2} + \delta_{1})^{2} dm_{2}$$
 (C32)

which is based on an energy equivalence criteria. The factor 2 in the first integral accounts for 2 columns, while the factor 2 on the second integral is introduced because the integration extends over one half the beam.

In Equation (C32) y represents the displacement of the equivalent single degree of freedom system,  $y_1$  represents the shape of the deformed column under the end load P/2,  $y_2$  represents the deformed shape of the beam under the central load P, and  $\delta_1$  represents the displacement of the beam supports. Upon replacing y by the expressions (C24) (and then introducing (C19) and (C9)), and substituting (C8) for  $y_2$  and (C22) for  $y_1$  and finally performing the integrations, the results can be put in the form

$$m = K_1 m_1 + (K_2 + \Delta K_2) m_2$$
 (C33)

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where

$$K_{1} = \frac{512}{105} \frac{\left[470 - 511F_{1} + 140F_{1}^{2}\right]\left(\frac{\alpha_{2}}{\alpha_{1}}\right)^{2}}{\left[4 - 3F_{2} + 32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}}\right]^{2}}$$

$$K_{2} = \left(\frac{1}{35}\right) \frac{\left[272 - 427F_{2} + 168F_{2}\right]^{2}}{\left[4 - 3F_{2} + 32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}}\right]^{2}}$$

$$K_{2} = 32 \frac{\left(2 - F_{1}\right)\left[32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}} + 5 - 4F_{2}\right]\left(\frac{\alpha_{2}}{\alpha_{1}}\right)}{\left[4 - 3F_{2} + 32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}} + 5 - 4F_{2}\right]\left(\frac{\alpha_{2}}{\alpha_{1}}\right)}$$

$$\left[4 - 3F_{2} + 32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}}\right]^{2}$$

$$\left[4 - 3F_{2} + 32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}}\right]^{2}$$

$$\left[4 - 3F_{2} + 32(2 - F_{1})\frac{\alpha_{2}}{\alpha_{1}}\right]^{2}$$

# C.7 Generalized Force

The generalized force of the equivalent single degree of freedom system is obtained from the relation

$$Py = 2 \int_{0}^{2} y_{1} dP_{1} + 2 \int_{0}^{2} (y_{2} + \delta_{1}) dP_{2}$$
 (C35)

which is based on a work equivalence principle. The differential work terms arriving from the dynamic pressure loading are expressed as

$$dP_{1} = q(n_{1}d_{1}C_{D1}) dx_{1}$$
 (C36)

(C36)

 $dP_2 = q(n_2 d_2 C_{D2}) dx_2$ 

where q is the dynamic pressure loading,  $C_D$  is the drag coefficient, d represents the width of member exposed to the blast pressure. The n factors are introduced to account for a possible increase in effective frontal area resulting from some effect such as the outer skin of the building wrapping around the member and increasing the area exposed to the air blast. Upon introducing the expressions for y, y<sub>1</sub> and y<sub>2</sub> and performing the integrations, as was done for the generalized mass, the generalized force P becomes

$$\mathbf{P} = \mathbf{P}_1 + \mathbf{P}_2 \tag{C37}$$

where

$$P_{1} = \frac{1}{2} q \left\{ \frac{16(n_{1}d_{1})\ell_{1} C_{D1}(7 - 4F_{1}) \frac{\alpha_{2}}{\alpha_{1}}}{32(2 - F_{1}) \frac{\alpha_{2}}{\alpha_{1}} + 4 - 3F_{2}} \right\}$$

$$P_{2} = \frac{1}{2} q \left\{ \frac{(n_{2}d_{2})\ell_{2} C_{D2} \left[ 5 - 4F_{2} + 64(2 - F_{1}) \frac{\alpha_{2}}{\alpha_{1}} \right]}{32(2 - F_{1}) \frac{\alpha_{2}}{\alpha_{1}} + 4 - 3F_{2}} \right\}$$
(C38)



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Figure C1: Idealized System Elements. (a) Elements Subjected to Pressure Loading (b) Elements Subjected to Centerload P

#### APPENDIX D. END-ON LOADING ANALYSIS

## D.1 Introduction

In this derivation it is assumed the detonation point is located such that it subjects the end of the building to a blast wave pressure loading. The angle between the detonation point and a perpendicular to the crane is denoted by  $\psi$ . When  $\psi = 0$ , the detonation point is directly in front of the end of the building and the blast wave impinges on the end of the building without impinging on the side of the building. When  $\psi$  is different from zero a component of the blase wave impinges on the sides of the building.

From Figure D-1, it can be seen that even if  $\psi = 0$  and the blast wave subjects the crane to a uniform pressure loading along its length, a yaw moment would exist about the mass center because of an unsymmetrical frontal area exposed to the pressure wave. The rotation produced by the yaw moment is resisted by contact forces between the wheels and rail beam. If the disturbance moment is large enough compared to the resistive moment, the crane will rotate and fail the the floor below. Equations governing this type of failure are derived in this appendix. A list of the parameters used in the derivation are given in Table D-1.

# D.2 Single Degree of Freedom Approximation

In general the impinging blast wave produces both a translation and rotation of the crane. Translational motion of even several feet may not necessarily be critical with regard to the crane leaving its track and falling to the floor. On the other hand, only a few degrees of rotation would be sufficient to allow the crane to fall. In this section translational motion is neglected and only the yaw rotational degree of freedom is considered. By letting  $\theta$  represent the yaw angle, the differential equations governing the motion for the system illustrated in Figure D1 is

$$I_o^{\ddot{o}} = M_o - \left[ (a - b) F_2 + bF_1 \right] \operatorname{sgn} M_o$$
 (D1)

where  $I_0$  is the yaw mass moment of inertia about the mass center and  $F_1$  and  $F_2$  are the rail reaction forces.







# **ELEVATION VIEW**

Figure D1: Crane Yaw Motion Model.

# Table D-1. Parameters for End-on Loading Case

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| PARAMETER  | MEDIAN |
|--|--------|
| crane length (ft)  | 78     |
| distance from left rail to crane center of mass (ft)             | 39     |
| minimum inward motion of crane before failure (ft)               | 0.5    |
| maximum inward motion of crane before failure (ft)               | 1.0    |
| drag coefficient (dimensionless)                                 | 1.3    |
| frictional resistance at left rail (1b)                          | 1000   |
| frictional resistance at right rail (1b)                         | 1000   |
| moment of inertia about center of mass (ft 1b sec <sup>2</sup> ) | 25000  |
| depth of crane (ft)  | 0.8    |
| depth of operator housing (ft)                                   | 2.5    |
| distance from center of mass to housing (ft)                     | 10     |
| length of housing (ft)   | 4      |
| range to ground zero (kft)                                       | 5      |
| crane weight (1b)  | 2000   |
| weapon yield (KT)  | 20     |
| area factor (dimensionless)                                      | 2      |

The blast pressure disturbance moment M is

 $M_{o} = q \cos \psi \int_{-b}^{a-b} (\eta h C_{D}) u du$   $M_{o} = q_{o} e^{-ut} \eta C_{D} I_{2} \cos \psi$ (D2)

where

$$I_{2} = \int_{-b}^{a-b} u h \, iu = \frac{1}{2} h_{1}a(a-2b) + \frac{1}{2} h_{2}(\Delta \ell) \left[ \Delta \ell + 2\ell_{1} \right]$$
(D3)

In (D2) the change in the normal pressure due to a rotation is neglected because one only needs to consider small changes in 0 from its original zero value. Here again the n factor is included to account for a possible increase in effective area upon which the blast pressure acts.

Equation (D1) can be written in the form

$$b = Ae^{-\alpha t} - B$$
 (D4)

where

$$\Lambda = \frac{M_o}{I_o} = \frac{q_o \eta C_b \cos \psi}{I_o}$$

$$B = \frac{M_R}{I_o} = \frac{(a - b) F_2 + bF_1}{I_o} \operatorname{sgn}(M_o)$$
(D5)

The solution of (D4) for zero initial conditions is

$$\dot{\vartheta} = \frac{A}{\alpha} \left( 1 - e^{-\alpha t} \right) - Bt$$

$$0 = \frac{A}{\alpha^2} \left( \alpha t + e^{-\alpha t} - 1 \right) - \frac{1}{2} Bt^2$$
(D6)

The time when  $\theta$  reaches a maximum is denoted by  $t_1$  and is obtained by setting  $t = t_1$  and  $\dot{\theta} = 0$  in (D6) to obtain

$$\frac{A}{B}\left(1-e^{-x_1}\right)-x_1=0$$
(D7)

where

$$\mathbf{x}_1 = \alpha t_1 \tag{D8}$$

One observes that a maximum will only occur when A > B, i.e., the disturbance moment must be larger than the resistive moment. When A > B the maximum yaw angle is

$$\theta_{\max} = \frac{A}{a^2} \left( \alpha t_1 + e^{-\alpha t_1} - 1 \right) - \frac{1}{2} B t_1^2$$

which can be written more simply by making use of (D7) and (D8) to obtain

$$\theta_{\max} = \left[\frac{2A - (2 + \alpha t_1)B}{2\alpha^2}\right] \left(\alpha t_1\right)$$
(D9)

## D.3 Two Degree of Freedom Approximation

The yaw motion  $\theta$  considered in the preceding section is generalized here to account for crane motion along the rail beams. Let  $y_1$  and  $y_2$  represent the downtrack motion of the left and right ends of the crane. One can describe the crane motion in terms of any two of the three coordinates  $y_1$ ,  $y_2$  or  $\theta$ . In this derivation motion is described in terms of  $y_1$  and  $\theta$ .

The kinetic energy of the system T is

$$\Gamma = \frac{1}{2} m \dot{y}_1^2 + \frac{1}{2} I \dot{\theta}^2 + m b \dot{y}_1^{\dot{\theta}} com\theta$$
(D10)

where I is the yaw mass moment of inertia about the left end. The dynamic pressure force is taken to be

$$q = q_0 e^{-\alpha t} \cos(\psi + \theta)$$
 (D11)

and the track reaction forces are

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$$F_{1} = f_{1} \operatorname{sgn}(\dot{y}_{1})$$

$$F_{2} = f_{2} \operatorname{sgn}(\dot{y}_{2})$$
(D12)

where

$$y_{2} = y_{1} + a \tan \theta$$

$$\dot{y}_{2} = \dot{y}_{1} + a \dot{\theta} \sec^{2}\theta$$
(D13)

The generalized forces associated with the disturbance and reaction forces are

$$Q_{y_1} = -F_1 - F_2 + q nC_D I_1$$

$$Q_0 = -F_2 \operatorname{asec}^2 \theta + q nC_D I_2$$
(D14)

where  $I_2$  is given in (D3) and

$$I_{1} = \int_{0}^{\ell} hdu = h_{1}a + h_{2}(\Delta \ell)$$
 (D15)

By making use of the preceding relations and substituting into Lagrange's equations, one obtains

$$m\ddot{y}_{1} + (mbcos0)8 = G_{1}$$
(mbcos0) $y_{1} + I\ddot{0} = G_{2}$ 
(D16)

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where

$$G_{1} = mb\theta^{2} \sin\theta - F_{1} - F_{2} + \eta C_{D}I_{1}q_{0}e^{-\alpha t} \cos(\psi + \theta)$$

$$G_{2} = -F_{2} \operatorname{asec}^{2}\theta + \eta C_{D}I_{2}q_{0}e^{-\alpha t} \cos(\psi + \theta)$$
(D17)

The solution of Equations (D16) for  $\dot{y}_1$  and  $\ddot{\theta}$  gives

$$\dot{y}_{1} = \frac{IG_{1} - (mb\cos\theta)G_{2}}{m[I - mb^{2}\cos^{2}\theta]}$$

$$\ddot{\theta} = \frac{G_{2} - (b\cos\theta)G_{1}}{I - mb^{2}\cos^{2}\theta}$$
(D18)

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Equations (D18) are to be solved subject to the initial conditions

$$y_1(0) = \dot{y}_1(0) = \theta(0) = \dot{\theta}(0) = 0$$
 (D19)

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