APPENDIX 08

B2113 Draft Blast Analysis 21MAY1999

DEPARTMENT OF DEFENSE EXPLOSIVES SAFETY BOARD
2461 EISENHOWER AVENUE
ALEXANDRIA, VIRGINIA 22331-0600

DDESB-KO 21 MAY 1999

MEMORANDUM FOR HQ AIR FORCE SAFETY CENTER
(ATTN: SEW)

SUBJECT: Request for Preliminary Design Review

References: (a) HQ AFSC(SEW) Memorandum, 24 Mar 1999, Same Subject

(b) Ease, Inc. Technical Report, "Draft Blast Analysis Report, Hill AFB Project Number WR41611, Building 2113, Bays 2 and 3", Dec 1998

The subject site plan forwarded by the references has been reviewed with respect to explosives safety criteria. Based on the information furnished, the site plan is approved provided personnel who are unrelated to explosives operations are afforded the equivalent public transportation route (PTR) protection as they are conveyed to and from Building 2113.

Retain a copy of the complete site plan package and this letter of approval as a permanent record at the installation. Update the master planning documents and installation drawings to show these clear zones.

Point of contact is Dr. Chester E. Canada, DDESB-KT1, PH: 703-325-1369, FAX: 703-325-6227, E-MAIL: canadce@hqda.army.mil.

James T. Drake, GS-15
DANIEL T. TOMPKINS
Colonel, USAF
Chairman

SEW COPY

DEPARTMENT OF THE AIR FOR(
HEADQUARTERS OGDEN AIR LOGISTICS CENTER (AFMC)
HILL AIR FORCE BASE, UTAH

MEMORANDUM FOR AFMC/SEW
ATTENTION: JIM STATON

FROM: OO-ALC/SEW
7290 8th Street
Hill AFB UT 84056-5003

SUBJECT: Preliminary Review of Design Concept to Reduce Explosive Safety Quantity-Distance

1. Request the Air Force Safety Center and Department of Defense Explosive Safety Board (DDESB) review and comment on the attached design concept to reduce explosive safety Quantity-Distance (Q-D) by target hardening. Request expeditious processing of this request.

2. Our proposal is to harden an existing explosive operating building to provide equivalent inhabited building protection so that DoD or private operations not related to explosives can be conducted within the explosive clear zone. This is based on ***DoD Ammunition and Explosive Safety Standards*** 6055.9, Chapter 5, paragraph 5.D.3. We could find no precedence for our approach; AF and DDESB concept review and comment will provide us with a foundation from which to proceed in project design. When AF and DDESB comments are received, the formal design and siting planning process will be implemented.

3. A preliminary analysis, Attachment 1, titled *Draft Blast Analysis Report Hill AFB Project Number WR41611 Building 2113; Bays 2 and 3* was conducted to determine the feasibility, cost, and design strategy to harden Building 2113. The results of the analysis provided responsible managers an acceptable cost estimate and design strategy to harden the building.

4. BACKGROUND: The Technology and Industrial Support Directorate (TI) located at Hill AFB conducts non-destructive inspections of various commodities including DoD Titled X ammunition and explosives (A&E). The equipment used to inspect A&E is located in the explosive safety clear zone in a properly sited operating location. This equipment is unique in that it employees an industrial Computed Tomography System (CT) scanner. It is similar to CT systems used in medical applications only much more powerful. It has unmatched capabilities within the Air Force and civilian industrial sector. The CT system is not 100% utilized. It has been identified as having excess capacity available, and as such under public law and technology innovation legislation the system is an ideal candidate for other use by DoD and private sector non-explosive commodities for strategic and compelling reasons. As an example, aircraft parts and components could be inspected to determine serviceability and flight integrity. This system is considered, for aircraft parts, as the only inspection method capable of meeting requirements for engineering studies redetermine parts failure and structural fatigue. Thus having a strategic and compelling impact on safety of flight. In addition, other United States government agencies have identified this system as a potential source for inspection of commodities not available within their agency. Private companies whose missions involve both government and non-government workloads would benefit from the use of this system. Currently the use of this

system is restricted to A&E because DDESB explosive safety standards do not allow DoD or private operations, not related to explosives, to be conducted within an explosive clear zone.

5. A number of alternatives were considered and rejected prior to the proposed. The following is a summary of those alternatives.

Alternative 1: Refuse workload – Defer to private industry.

Reason for Rejection: There is no equivalent system available in either private or government organizations.

Alternative 2: Purchase duplicate system and locate outside the clear zone.

Reason for Rejection: The cost to purchase the system and to modify or construct a building would be in the order \$3.5 million. The expected generated revenue could not offset this investment cost.

Alternative 3: Relocate existing CT system outside explosive clear zone.

Reason For Rejection: Loss of primary mission capability to inspect Minuteman missiles. In addition, a facility would be required to safety house the 9 MEV radiation source. Currently, there is no such facility on the base nor is there land available to build such a facility.

Alternative 4: Rewarehouse explosive to eliminate violations.

Reason For Rejection: The explosive in 3 of 11 locations identified, as being in violation can not be rewarehouse to eliminate the violation.

Alternative 5: Construct new facility in explosive clear zone that meets Quantity-Distance.

Reason For Rejection: There is no land within the explosive clear zone to accommodate a new facility that would meet all the Q-D requirements without effecting or eliminating other explosive operations. Also the cost of \$1.75 million is prohibitive.

6. The point of contract is Mr. George Stratman, DSN 777-1425

Ray W. Tidwell

RAY W. TIDWELL
Chief of Weapons Safety
OO-ALC Safety Office

Attachment:
1. Draft Blast Analysis Report Hill AFB Project Number WR4111
Building 2113; Bays 2 and 3

cc:
TIE

EASE, Inc.
Professional Engineering Services

DRAFT BLAST ANALYSIS REPORT

HILL AFB PROJECT NUMBER WR41611

BUILDING 2113; BAYS 2 AND 3

Submitted to:

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Civil Engineering, 75CEG/CECM-B
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Prepared by:

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DECEMBER 1998

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1. Ammunition and Explosives Safety Standards, Department of Defense (DOD 6055.9-STD), Assistant Secretary of Defense (Production and Logistics), October 1992

2. Blast Analysis of Reinforced Concrete Slabs (BARCS)Computer Program, J.M. Ferrito, Naval Facilities Engineering Command, July 1977 original publication; updated November 1988 by Naval Civil Engineering Laboratory, Port Hueneme, CA

3. Explosives Safety Standards, Department of the Air Force Manual (AFM 91-201), HQ USAF/SE; October 1994

4. Explosives Safety Standards (DRAFT), Department of the Air Force Manual (AFM 91-201); HQ USAF/SE, November 1998

5. FRANG, Version 1.0, Computer Program, Naval Civil Engineering Laboratory, Port Hueneme, CA, August 31, 1988

6. Prediction of Building Debris for Quantity-Distance Siting, Technical Paper #13; Department of Defense Explosives Safety Board; Alexandria, VA; Date ???

7. SHOCK, Version 1.0, Computer Program, Naval Civil Engineering Laboratory, Port Hueneme, California, January 1988

8. Structures to Resist the Effects of Accidental Explosions (Volume 1, 2 & 3), Department of the Army Technical Manual (TM 5-1300), Department of the Nay Publication (NAVFAC P-397), Department of the Air Force Manual (AFR 88-22), Washington, D.C., December 1991

LIST OF ACRONYMS

AFB	Air Force Base
AFMAN	Air Force Manual
AFR	Air Force Regulation
CT	computed tomography system
DDESB	Department of Defense Explosive Safety Boards
DOD	Department of Defense
ES	exposed site
IBD	inhabited building distance
IL	intraline distance
IM	intermagazine distance
PES	potential explosive site
Q-D	Quantity-Distance
MCE	maximum credible event
NEW	net explosive weight
ROM	rough-order-of-magnitude
TI	Technology and Industrial Support Directorate
TM	Tri-Service Manual

1.0 BACKGROUND

The Technology and Industrial Support Directorate (TI), located at Hill AFB, conducts non-destructive inspections of various commodities including DOD ammunition and explosives. The main component of the specialized system of equipment utilized to inspect ammunition and explosives is a unique industrial computed tomography system (CT) scanner. The CT system is similar to those systems used by the medical industry, but much more powerful. The system has unmatched capabilities within the U.S. Air Force and the civilian industrial sector.

The equipment is located within BAY 2 of BLDG 2113B. Personnel operating and controlling the equipment are located in the adjacent BAY 3. Originally, the use of the system was restricted to DOD related explosive workloads in accordance with the Department of Defense Explosive Safety Board's (DDESB) explosive safety standards. BLDG 2113B is properly sited in relationship to other potential explosive sites (PES) at Hill AFB based on intraline distance (IL).

Hill AFB's mission does not currently utilize the CT system to its full capacity. In accordance with public law and technology innovation legislation, the CT system is an ideal candidate for use in the inspection of other DOD and private sector non-explosive commodities. For example, aircraft parts and components could be inspected utilizing the CT system to evaluate serviceability and structural integrity. For the inspection of aircraft parts, the CT system is considered to be the only method capable of supporting required engineering studies. The studies determine if parts have suffered failure due to fatigue. Therefore, the capability to use the CT system has a "strategic and compelling impact" on flight safety. Other U.S. government agencies have identified the CT system as a potential tool for inspection that is not currently available within their agency.

FIGURE 1: SITE PLAN

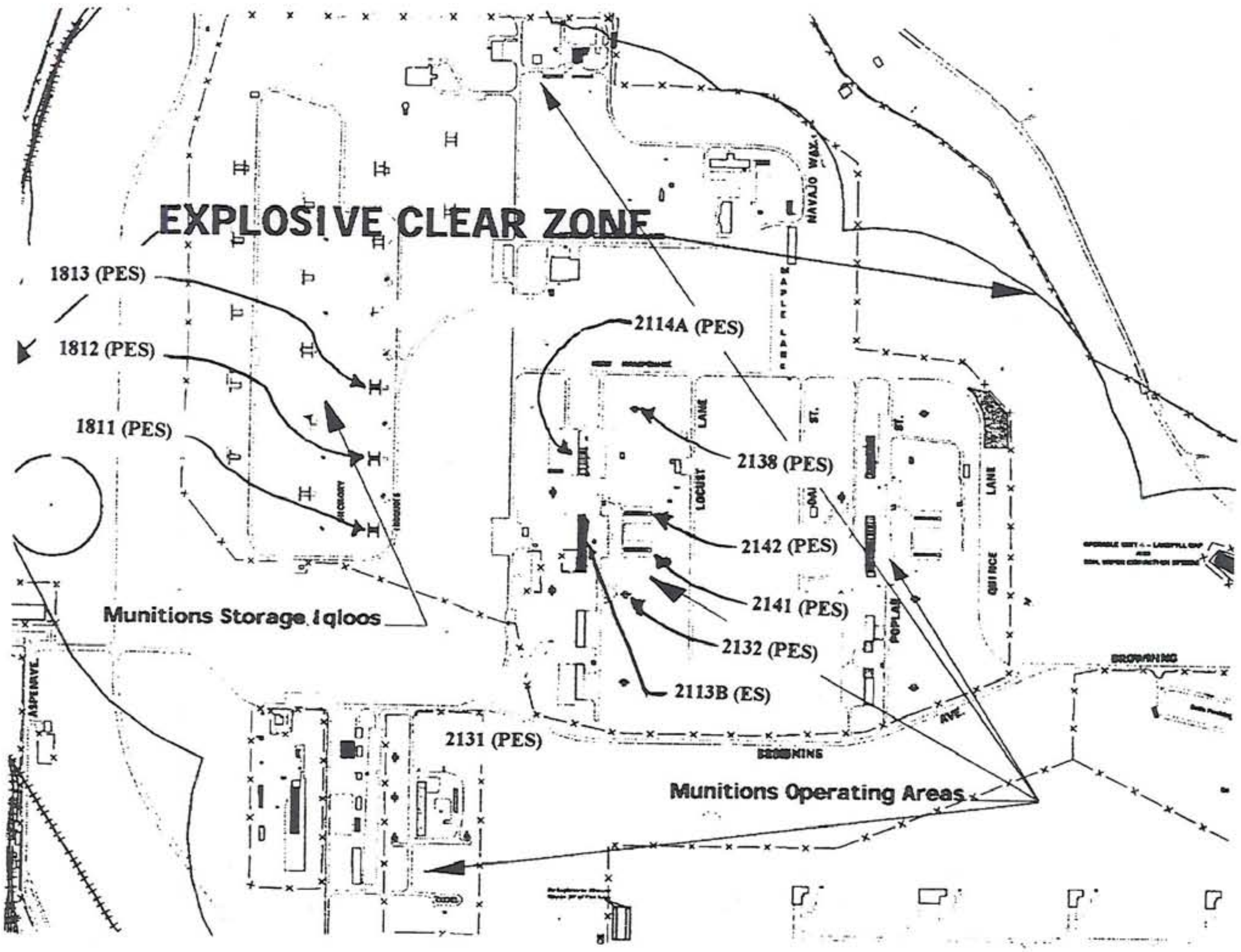
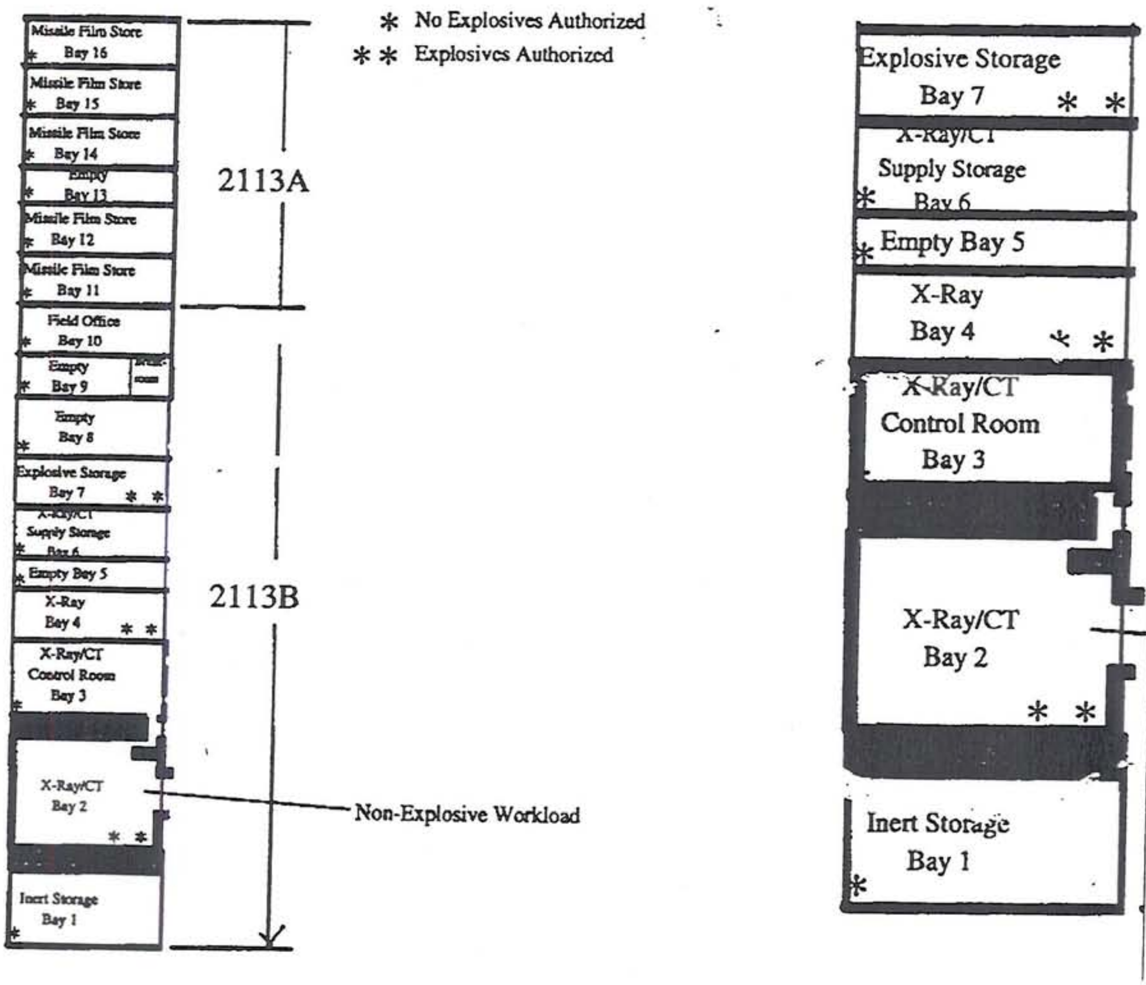


FIGURE 2: BLDG 2113 PLAN



The cost to purchase the system and to modify or construct a building to house the system is prohibitive. Similarly, private companies with missions involving both government and non-government workloads would benefit from the capability to utilize the CT system.

Allowing non-DOD and DOD non-related non-explosive workloads to be operated on within BLDG 2113B would violate the DDESB explosive safety standards. In this case, all personnel would need to be provided with a minimum level of protection equivalent to the level of protection provided if the building were separated from the PES by the DDESB's inhabited building distance (IBD) and default minimum fragment distances. Hill AFB has carefully considered the alternatives, control measures, and the potential for corrective action. Hill AFB proposes to harden the structural elements of the existing BAYS 2 and 3 of BLDG 2113B to ensure personnel, equipment, and non-explosive and/or non DOD items being worked on within the facility are protected from the overpressure and fragment effects of an accidental detonation at one of the PES's. This blast analysis has been prepared in support of Hill AFB's submittal of a new explosive site plan package.

2.0 BLAST ANALYSIS OBJECTIVES

The objectives of this blast analysis are to:

- *identify specific PES's which currently violate the IBD and/or default fragment distances;*
- *predict worst case overpressure loading scenarios based on the methodology outlined in the Tri-Service Manual TM 5-1300 (AFR 88-22);*

- *predict dynamic response of ES structural elements to overpressure based on methodology outlined in AFR 88-22;*
- *develop recommendations to upgrade structural elements of ES as required;*
- *predict the effect of fragments on ES, assuming recommendations for structural upgrades have been implemented; and,*
- *develop rough-order-of-magnitude (ROM) prediction of cost to implement the recommendations for structural upgrade.*

3.0 IDENTIFICATION OF PES'S

The buildings within this area of Hill AFB, including BLDG 2113B, are sited at least IL distance based on operations which are related and support functions that are DOD controlled. The IL distance is determined calculated by taking the cube root of the net exlosive weight (NEW) of the PES multiplied times "18" (i.e., K18). Buildings for which intermagazine (IM) and intraline distance (IL) apply, do not require minimum fragment distances to be considered in their siting. If non-DOD and DOD non-related and/or non-explosive operations are conducted in BLDG 2113B, personnel within BAYS 2 and 3 must be provided with IBD or minimum fragment distances, whichever is greater. IBD is determined based on Table 3.3 of AFMAN 91-201 and the default minimum fragment distance for NEW greater than 100 lbs (i.e., 1,250'). TABLE 1 of this document contains a summary of the buildings which would be in violation. Each of these buildings are PES's which will be considered in the evaluation of the effects of the maximum credible event (MCE) on the ES's structural elements.

TABLE 1: SUMMARY OF PES's

BLDG NO.	FACILITY/OPERATION DESCRIPTION	SITED NEW HC/D	DISTANCE ACTUAL	DISTANCE REQUIRED	VIOLATION IAW Table 3.3 of AFMAN 91-201
2114A	SHP MSL SVC Munitions Operating Facility 6 M AFMC/OO-ALC/LM	10,408 lbs 1.1	505'	1250'	minimum default fragment distance and K40
2141	Segregated Storage Magazine, Aboveground 0 AFMC/OO-ALC/649 MUNS	425 lbs per bay 1.1	237'	1250'	minimum default fragment distance and K40
2142	Segregated Storage Magazine, Aboveground 0 AFMC/OO-ALC/649 MUNS	425 lbs per bay 1.1	332'	1250'	minimum default fragment distance
1811	Storage, Igloo Igloo, Double Front (Hill) 0 AFMC/OO-ALC/LMX	135,776 lbs 1.1	1207'	2025'	minimum default fragment distance and K35/K50 ($0.3955W^{0.7227}$)
1812	Storage, Igloo Igloo, Double Front (Hill) 0 AFMC/OO-ALC/LMX	149,404 lbs 1.1	1315'	2171'	K35/K50 = ($0.3955W^{0.7227}$)
1813	Storage, Igloo Igloo, Double Front (Hill) 0 AFMC/OO-ALC/LMX	130,925 lbs 1.1	1544'	1973'	K35/K50 = ($0.3955W^{0.7227}$)
2131	ABG Mag , Earth Cvd Magazine, Aboveground 0 AFMC/OO-ALC/649 Muns	28,958 lbs 1.1	711'	1250'	minimum default fragment distance and K40
2132	ABG Mag , Earth Cvd Magazine, Aboveground 0 AFMC/OO-ALC/649 Muns	431 lbs 1.1	271'	1250'	minimum default fragment distance and K40
2138	ABG Mag , Earth Cvd Magazine, Aboveground 0 AFMC/OO-ALC/649 Muns	4,357 lbs 1.1	896'	1250'	default fragment distance controls

Buildings 2148 and 2108 are not included because the are currently empty and scheduled for demolition.

4.0 PREDICT BLAST LOADINGS

This section of the analysis focuses on the prediction of the worst case blast loading to be used to evaluate the existing capacity of the ES structure and as the basis of design for any required structural upgrades.

The ES must have the structural capacity to withstand the shock front as it traverses the structure. Blast wave parameters that will be developed for each PES, based on the methods outlined in AFR 88-22, include: peak incident pressure, dynamic pressure, reflective pressures, impulse, duration, particle velocity, and shock wave length. These parameters are a function of many variables, some of which are listed here:

- *quantity, formulation, and configuration of explosive involved,*
- *whether blast is considered air or surface burst*
- *whether the blast is unconfined, partially confined, or confined.*
- *distance between PES and ES*
- *ES geometric configuration*

Initially, Hopkinson or cube-root scaling can be used to relate the characteristic properties of the blast wave from an explosion of one energy level to that of another energy level. According to cube-root scaling, a given pressure will occur at a given distance from an explosion that is proportional to the cube root of the energy yield. This has been proven true experimentally for explosive weights ranging from a few ounces to hundreds of tons. Using cube-root scaling, if R is the distance from a reference explosion of weight W pounds, parameters such as overpressure, dynamic pressure, and particle velocity for the reference explosion would be equivalent for an explosion of weight W_2 pounds at a distance R_2 given by:

$$R/R_2 = (W/W_2)^{1/3} \quad or \quad R/W^{1/3} = R_2/W_2^{\ 1/3}$$

The term for scaled distance, $R/W^{1/3}$, is represented by Z in this analysis. Cube-root scaling implies that all quantities with units of pressure (psi) and velocity (ft/ms) are unchanged in the scaling. The advantage to scaling is that a large amount of the data associated with the blast wave parameters can be shown on relatively simple plots. The other blast wave parameters of impulse, duration, and wave length must be multiplied by $W^{1/3}$ to get the absolute value to be used in design. The scaling relations apply when there are: (a) identical ambient conditions, (b) identical charge shapes, and (c) identical charge-to-surface area geometries. However, reasonable values can be obtained using the scaling relations even when only similar conditions exist.

DOD facilities are sited in accordance with the DDESB's Quantity-Distance (Q-D) criteria. Q-D refers to the protection requirements in terms of distance from a PES to an ES. Distance requirements are typically specified by the value of K, using the terminology K9, K11, K18, to mean K equals 9, K equals 11, K equals 18, etc. Calculation of the required separation distance is an application of cube root scaling:

$$D = K\,W^{1/3}$$

At any distance D, calculated using the same K factor, the peak incident pressure will be the same. In assessing the risk of damage/injury to facility, equipment, and personnel injury, K factors and the associated peak incident pressures are often utilized.

The buildings identified in TABLE 1, were originally sited to provide at least K18 level of protection. With out a detailed engineering risk assessment, an ES at K18 can expect to experience a peak incident pressure of 3.5 psi and the following effects:

- *Damage to unstrengthened buildings will be of a serious nature and approximately 50 percent or more of the facility will be lost. Sensitive electronic equipment is expected to stop functioning.*
- *There is a 1-percent chance of eardrum damage to personnel.*
- *Personnel injuries of a serious nature (including some fatalities) are likely from fragments, debris, firebrands, or other objects, including the structural failure of buildings walls and roofs.*

If the non-DOD and/or DOD non-related non-explosive work is performed in BAYS 2 and 3 of BLDG 2113B, the required level of protection is defined by Table 3.3 of AFMAN 91-201. At this distance the peak incident pressure is anticipated to be 1.2 psi/0.9 psi and the effects (without more detailed engineering risk assessment) are anticipated to be:

- *Unstrengthened buildings can be expected to sustain damage; with up to about 5 percent of the building having to be replaced.*
- *Personnel in buildings are provided a degree of protection from death or fatal injury. Personnel injuries from projectile fragments and the failure of the exposed facility (including the possibility of fatalities) will depend upon the PES structure, the amount of ammunition, their fragmentation characteristics, and the strength of the ES structure.*
- *Personnel in the open are not expected to be injured directly by the blast.*

It has been established that the actual distances between the PES's and ES, in this case, are not sufficient to provide the required level of protection by distance. If protection can not be provided by distance, DDESB allows for the hardening of the ES structure based on a detailed engineering evaluation of dynamic loads and expected structural response using the methods of AFR 88-22.

A prediction of the worst case loading scenario is developed in TABLE 2 of this document. The PES's which currently violate the Q-D requirements are listed *in Column (1)*. Simplifications leading to safety conservative structural designs were made in the development of the procedures outlined in AFR 88-22. However, unknown factors can still cause an overestimation of a structure's capacity to resist the effects of an explosion. Unexpected shock wave reflections, construction methods, quality of construction materials, etc. vary for each facility. To compensate for such unknowns, it is recommended that the TNT equivalent weight be increased by 20 percent. This increased charge weight is the "effective charge weight" to be used in design. The sited NEW for each building multiplied by a factor of 1.2 is provided in *Column (2)*.
The cube root of the effective charge weight is calculated for each PES in *Column (3)*. This value is utilized throughout the analysis process to convert scaled values to actual values.

The actual distance between the closest exterior point of the structure where the PES is located to the closest exterior point of the ES structure is listed in *Column (4)*. An actual scaled distance (i.e, K-factor) is calculated in *Column (5)* by dividing the value in *Column (4)* by the value in *Column (3)*.

TABLE 2: PREDICTION OF WORST CASE LOADING

(1) BLDG NO.	(2) 1.2 X SITED NEW (LBS)	(3) $W^{1/3}$ (LBS $^{1/3}$)	(4) DISTANCE ACTUAL (FT)	(5) ACTUAL K-FACTOR	(6) PEAK INCIDENT PRESSURE (PSI)	(7) SCALED IMPULSE (PSI-MS / $W^{1/3}$)	(8) IMPULSE (PSI-MS)
2114A	**12,499**	**23.21**	**505**	**21.76**	**2.641**	**3.979**	**92.35**
2141	510	7.99	237	29.66	1.725	2.948	23.55
2142	510	7.99	332	41.55	1.121	2.122	16.95
1811	**162,931**	**54.62**	**1207**	**22.10**	**2.583**	**3.920**	**214.11**
1812	179,285	56.39	1315	23.32	2.394	3.723	209.94
1813	157,110	53.96	1544	28.61	1.809	3.053	164.74
2131	**34,750**	**32.63**	**663**	**20.32**	**2.918**	**4.248**	**138.57**
2132	517	8.02	271	33.79	1.455	2.596	20.82
2138	5,228	17.36	896	51.61	0.855	1.717	29.81

Column (6) peak incident pressure values and *Column (7)* scaled impulse values for a surface burst at the scaled distance in *Column (5)* were determined utilizing Figure 2-15 of AFR 88-22. The actual impulse for a detonation involving the NEW in *Column (2)* is calculated by multiplying the scaled impulse by the scaled NEW. These values are an estimation of the free-field blast parameters at the moment the shock wave strikes the exposed wall face. The actual forces acting on a structure associated with a plane shock wave are a function of the peak pressure, the impulse, the dynamic pressures, and reflected pressures. For design purposes, it is necessary to establish the variation or decay of both the incident and dynamic pressures with time since the effects on the structure subjected to a blast loading depend upon the intensity-time history of the loading as well as on the peak intensity. The design loads will vary as the shock front traverses a structure. The effect of the shock front on structural elements will also vary based on the direction the shock front approaches BLDG 2113B from for the following reasons:

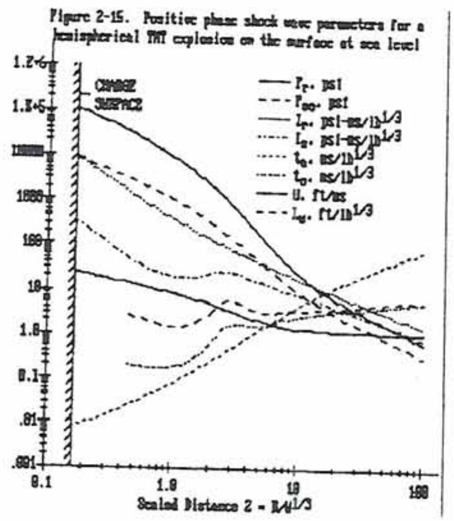
- BLDG 2113 is rectangular in plan view; therefore, the effects of blast loading on the ES will be different from a shock wave travelling in a north-south direction versus a shock wave travelling in the east west direction.
- The exposed walls of BAYS 2 and 3 face east and west. A shock wave traveling east-west may have a greater effect on these walls than a shock wave traveling north-south may have.
- Bays 2 and 3 are located closer to the south end of BLDG 2113 than to the north end. A shock wave traveling south to north may have a greater effect on the roof loading than a shock wave traveling from the north, east or west.

To accurately determine the overall loading on an exposed surface, a step-by-step analysis of the wave propagation across the surface should be made. This analysis includes an integration of the pressure at points of interest on the surface and at various times to determine the equivalent uniform incident pressure acting on a span, *L*, as a function of time.

To reduce the scope of the detailed analysis to a conservative cross-section of PES's, the magnitude of the free field incident impulses in *Column (8)* can be compared. The field was narrowed down to the three highlighted PES's for more detailed evaluation.

For each of these three PES's the complete set of free-field blast wave parameters was developed utilizing figure 2-15 of AFR 88-22.

FIGURE 3: COPY OF *FIGURE 2-15* FROM AFR 88-27



P_{so}	**Peak Positive Incident Pressure**	psi
P_r	**Peak Positive Normal Reflected Pressure**	psi
$i_s/W^{1/3}$	**Scaled Unit Positive incident Impulse**	psi-ms/lb$^{1/3}$
$i_r/W^{1/3}$	**Scaled Unit Positive Normal Reflected Impulse**	psi-ms/lb$^{1/3}$
$t_A/W^{1/3}$	**Scaled Time of Arrival of Blast Wave**	ms/lb$^{1/3}$
$t_o/W^{1/3}$	**Scaled Positive Duration of Positive Phase**	ms/lb$^{1/3}$
U	**Shock Front Velocity**	ft/ms
W	**Effective Charge Weight**	lbs
$L_w/W^{1/3}$	**Scaled Wave Length of Positive Phase**	ft/lb$^{1/3}$

TABLE 3: PES @ 2114A, BAY 2; W = 12,499 lbs, $W^{1/3}$ = 23.21, $\alpha = 0°$

	R_g	Z	P_{so}	P_r	$i_s/W^{1/3}$	$i_r/W^{1/3}$	$t_A/W^{1/3}$	$t_o/W^{1/3}$	U	$L_w/W^{1/3}$
①	260	11.21	7.724	18.61	7.321	15.95	5.273	2.769	1.345	3.725
②	505	21.76	2.641	5.691	3.979	7.731	13.75	3.508	1.199	4.204
③	542	23.35	2.389	5.119	3.718	7.171	15.07	3.589	1.191	4.274

Predict BAY 3 roof and side wall loading;

$P_{sof} = 2.641\ psi$
$L = 37'$
$L_w/W^{1/3} = 4.204\ ft/lb^{1/3}$ $\qquad L_w = 97.57'$

$L_w/L = 97.57'/37' = 2.64$	
$C_e = 0.7491 \quad C_eP_{so} = 0.7491 \times 2.641\ psi = 1.98\ psi$	*figure 2-196*
$t_d/W^{1/3} = 1.2\ ms/lb^{1/3} \qquad t_d = 27.85\ ms$	*figure 2-197*
$t_{of}/W^{1/3} = 4.77\ ms/lb^{1/3} \qquad \mathbf{t_{of} = 110.71\ ms}$	*figure 2-198*
$q_o = 0.1038\ psi$	*figure 2-3*
$C_D = -0.4 \qquad \mathbf{C_eP_{sof} + C_Dq_o = 1.94\ psi}$	
$C_{e^-} = 0.2724 \quad \mathbf{C_{e-}P_{so} = 0.2724 \times 2.641\ psi = 0.72\ psi}$	*figure 2-196*
$t_{of-}/W^{1/3} = 10.89\ ms/lb^{1/3} \qquad \mathbf{t_{of^-} = 252.76\ ms}$	*figure 2-198*
$0.25\ t_{of-} = 0.25 \times 252.76 = 63.19\ ms$	
$t_{of} + t_{o-f} = 110.71\ ms + 252.76\ ms = 363.47\ ms$	

FIGURE 4: COPY OF *FIGURE 2-196* FROM AFR 88-27

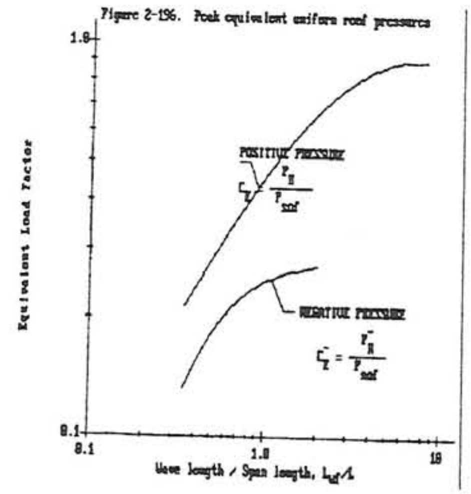


FIGURE 5: COPY OF *FIGURE 2-197* FROM AFR 88-27

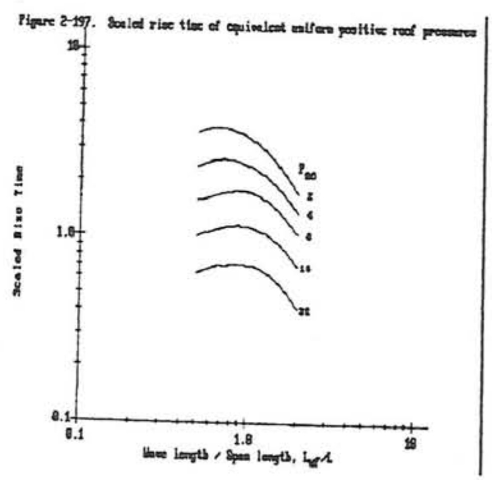


FIGURE 6: COPY OF *FIGURE 2-198* FROM AFR 88-27

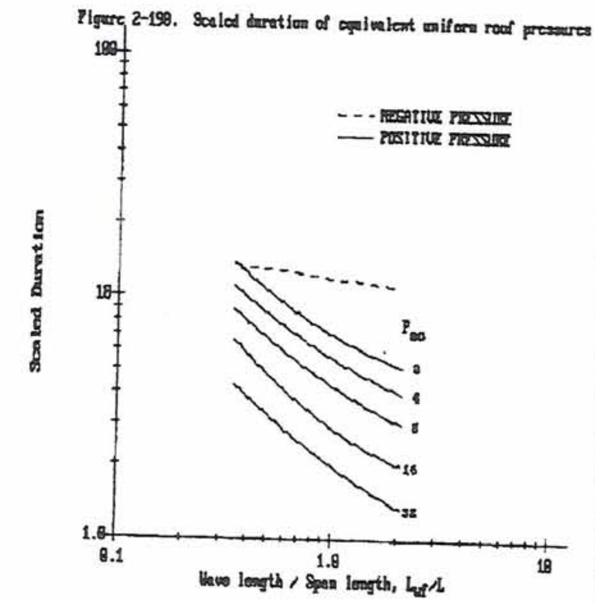
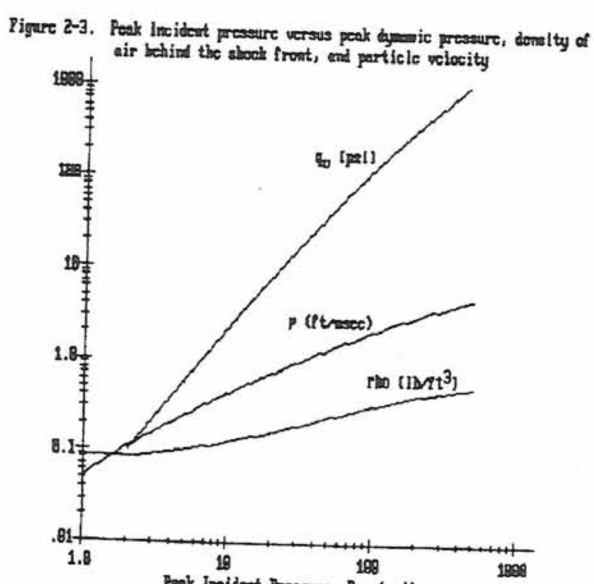


FIGURE 7: COPY OF *FIGURE 2-3* FROM AFR 88-27



Predict BAY 2 roof and side wall loading;

$P_{sof} = 2.389\ psi$
$L = 50'$
$L_w/W^{1/3} = 4.274\ ft/lb^{1/3}$ $\quad L_w = 99.11'$
$L_w/L = 99.11'/50' = 1.98$
$C_e = 0.6652 \quad C_eP_{so} = 0.6652 \times 2.389\ psi = 1.59\ psi$ *figure 2-196*
$t_d/W^{1/3} = 1.66\ ms/lb^{1/3}$ $\quad t_d = 38.53\ ms$ *figure 2-197*
$t_{of}/W^{1/3} = 5.01\ ms/lb^{1/3}$ $\quad \mathbf{t_{of} = 116.28\ ms}$ *figure 2-198*
$q_o = 0.084\ psi$ *figure 2-3*
$C_D = -0.4$ $\quad \mathbf{C_eP_{sof} + C_Dq_o = 1.56\ psi}$
$C_{e}- = 0.2721 \quad \mathbf{C_{e-}P_{so} = 0.2721 \times 2.389\ psi = 0.65\ psi}$ *figure 2-196*
$t_{of-}/W^{1/3} = 10.91\ ms/lb^{1/3}$ $\quad \mathbf{t_{of-} = 253.22\ ms}$ *figure 2-198*
$0.25\ t_{of-} = 0.25 \times 252.76 = 63.31\ ms$
$t_{of} + t_{o-f} = 116.28\ ms + 253.22\ ms = 369.50$

TABLE 4: PES @ 1811; W = 162,931 lbs, $W^{1/3}$ = 54.6

	R_g	Z	P_{so}	P_r	$i_s/W^{1/3}$	$i_r/W^{1/3}$	$t_A/W^{1/3}$	$t_o/W^{1/3}$	U	$L_w/W^{1/3}$
①	1207	22.10	2.583	5.558	3.920	7.604	14.03	3.525	1.197	4.219
②	1257	23.01	2.439	5.232	3.771	7.284	14.79	3.572	1.193	4.259
③	1275	23.34	2.391	5.122	3.720	7.175	15.06	3.588	1.191	4.274

Predict BAY 2 and BAY 3 west wall loading;

$P_{so} = 2.583\ psi$
$i_s/W^{1/3} = 3.920\ psi\text{-}ms/lb^{1/3}$ $\quad i_s = 214.03\ psi\text{-}ms$
$t_o/W^{1/3} = 3.525\ ms/lb^{1/3}$ $\quad t_o = 192.47\ ms$
$\mathbf{P_r = 5.558\ psi}$
$i_r/W^{1/3} = 7.604\ psi\text{-}ms/lb^{1/3}$ $\quad \mathbf{i_r = 415.18\ psi\text{-}ms}$
$t_r = (2i_r)/P_r = \mathbf{149.4\ ms}$

Predict BAY 2 and BAY 3 roof loading;

$P_{so} = 2.583\ psi$
$L = 50'$
$L_w/W^{1/3} = 4.219\ ft/lb^{1/3}$ $\quad L_w = 230.36$

$L_w/L = 230.36/50' = 4.61$
$C_e = 0.8715 \quad C_eP_{so} = 0.8715 \times 2.583\ psi = 2.25\ psi$ — *figure 2-196*
$t_d/W^{1/3} = 0.6\ ms/lb^{1/3} \qquad t_d = 32.8\ ms$ — *figure 2-197*
$t_{of}/W^{1/3} = 4.0\ ms/lb^{1/3} \qquad \boldsymbol{t_{of} = 218\ ms}$ — *figure 2-198*
$q_o = 0.123\ psi$ — *figure 2-3*
$C_D = -0.4 \qquad \boldsymbol{C_eP_{sof} + C_Dq_o = 2.2\ psi}$
$C_{e}\text{-} = 0.2724 \quad \boldsymbol{C_{e}P_{so} = 0.2724 \times 2.389\ psi = 0.67\ psi}$ — *figure 2-196*
$t_{of}/W^{1/3} = 10.9\ ms/lb^{1/3} \qquad \boldsymbol{t_{of\text{-}} = 595\ ms}$ — *figure 2-198*
$0.25\ t_{of\text{-}} = 0.25 \times 595 = 149\ ms$
$t_{of} + t_{o\text{-}f} = 218\ ms + 595\ ms = 813$

Predict BAY 2 east wall loading;

$P_{so} = 2.391\ psi$
$L = 41'$
$L_w/W^{1/3} = 4.274\ ft/lb^{1/3} \qquad L_w = 233.36$
$L_w/L = 233.36/41' = 5.69$
$C_e = 0.8954 \quad C_eP_{so} = 0.8954 \times 2.391\ psi = 2.14\ psi$ — *figure 2-196*
$t_d/W^{1/3} = 0.41\ ms/lb^{1/3} \qquad t_d = 22.39\ ms$ — *figure 2-197*
$t_{of}/W^{1/3} = 4.0\ ms/lb^{1/3} \qquad \boldsymbol{t_{of} = 218\ ms}$ — *figure 2-198*
$q_o = 0.1114\ psi$ — *figure 2-3*
$C_D = -0.4 \qquad \boldsymbol{C_eP_{sof} + C_Dq_o = 2.1\ psi}$
$C_{e}\text{-} = 0.2724 \quad \boldsymbol{C_{e}P_{so} = 0.2724 \times 2.389\ psi = 0.67\ psi}$ — *figure 2-196*
$t_{of}/W^{1/3} = 10.9\ ms/lb^{1/3} \qquad \boldsymbol{t_{of\text{-}} = 595\ ms}$ — *figure 2-198*
$0.25\ t_{of\text{-}} = 0.25 \times 595 = 149\ ms$
$t_{of} + t_{o\text{-}f} = 218\ ms + 595\ ms = 813$

TABLE 5: PES @ 2131; W = 34,750 lbs, $W^{1/3}$ = 32.63

	R_g	Z	P_{so}	P_r	$i_s/W^{1/3}$	$i_r/W^{1/3}$	$t_A/W^{1/3}$	$t_o/W^{1/3}$	U	$L_w/W^{1/3}$
①	625	19.16	3.184	6.938	4.490	8.861	11.60	3.366	1.215	4.088
②	663	20.32	2.918	6.323	4.248	8.319	12.56	3.431	1.207	4.140
③	713	21.86	2.624	5.651	3.962	7.693	13.83	3.513	1.198	4.208

Predict BAY 2 roof and east/west wall loading;

$P_{sof} = 2.918\ psi$		
$L = 50'$		
$L_w/W^{1/3} = 4.140\ ft/lb^{1/3}$	$L_w = 135'$	
$L_w/L = 135'/50' = 2.7$		
$C_e = 0.7552 \quad C_eP_{so} = 0.7552 \times 2.918\ psi = 2.2\ psi$		*figure 2-196*
$t_d/W^{1/3} = 1.1\ ms/lb^{1/3}$	$t_d = 35.89\ ms$	*figure 2-197*
$t_{of}/W^{1/3} = 4.1\ ms/lb^{1/3}$	$\mathbf{t_{of} = 134\ ms}$	*figure 2-198*
$q_o = 0.1177\ psi$		*figure 2-3*
$C_D = -0.4$	$\mathbf{C_eP_{sof} + C_Dq_o = 2.15\ psi}$	
$C_{e}- = 0.2724 \quad \mathbf{C_{e-}P_{so} = 0.2724 \times 2.918\ psi = 0.79\ psi}$		*figure 2-196*
$t_{of-}/W^{1/3} = 10.9\ ms/lb^{1/3}$	$\mathbf{t_{of-} = 356\ ms}$	*figure 2-198*
$0.25\ t_{of-} = 0.25 \times 356 = 89\ ms$		
$t_{of} + t_{o-f} = 134\ ms + 356\ ms = 490\ ms$		

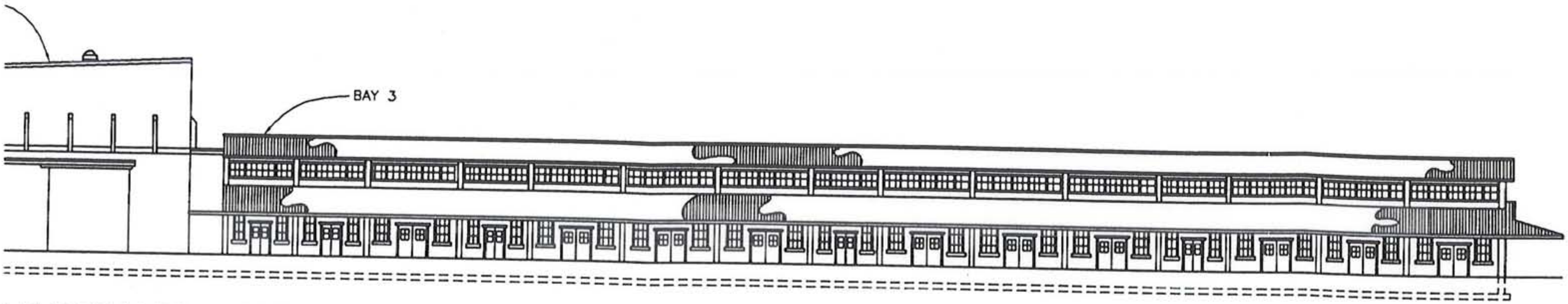
Predict BAY 3 roof and east/west wall loading;

$P_{sof} = 2.624\ psi$		
$L = 38'$		
$L_w/W^{1/3} = 4.208\ ft/lb^{1/3}$	$L_w = 137'$	
$L_w/L = 137'/38' = 3.6$		
$C_e = 0.8261 \quad C_eP_{so} = 0.8261 \times 2.624\ psi = 2.17\ psi$		*figure 2-196*
$t_d/W^{1/3} = 0.7\ ms/lb^{1/3}$	$t_d = 28.33\ ms$	*figure 2-197*
$t_{of}/W^{1/3} = 4.0\ ms/lb^{1/3}$	$\mathbf{t_{of} = 131\ ms}$	*figure 2-198*
$q_o = 0.1145\ psi$		*figure 2-3*
$C_D = -0.4$	$\mathbf{C_eP_{sof} + C_Dq_o = 2.12\ psi}$	
$C_{e}- = 0.2724 \quad \mathbf{C_{e-}P_{so} = 0.2724 \times 2.624\ psi = 0.71\ psi}$		*figure 2-196*
$t_{of-}/W^{1/3} = 10.9\ ms/lb^{1/3}$	$\mathbf{t_{of-} = 356\ ms}$	*figure 2-198*
$0.25\ t_{of-} = 0.25 \times 356\ ms = 89\ ms$		
$t_{of} + t_{o-f} = 131\ ms + 356\ ms = 346.46$		

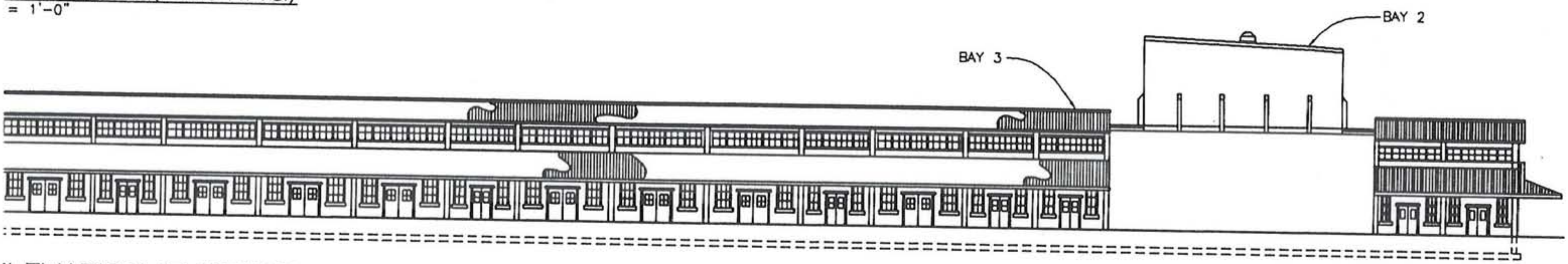
5.0 PREDICT DYNAMIC RESPONSE OF ES

In this section an evaluation of the existing structural elements is made to determine if the element has the capacity to withstand the dynamic loading predicted in the previous section. BLDG 2113 is made up of 16 bays (refer to FIGURE 2). The specialized equipment is located in BAY 3 and the operators of equipment are in BAY 2. These two bays have significantly different structural structural systems. BLDG 2113 was originally constructed in 1940/41. BAY 2 was reconstructed in 1958 to accomodate x-ray equipment. BLDG 2113 elevations are illustrated in FIGURE 8.

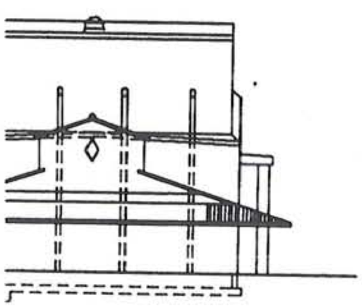
BAY 2 has interior dimensions of 46'-4" by 41'-8" in plan by 41' in height. The north and south walls are composite (i.e., one foot concrete; 7'-8" sand fill; one foot concrete) to a height of 22'-10.". From 22'-10" above finished grade up to the roof line the north and south walls are 12" thick conventionally reinforced concrete. The east and west walls are 24" thick conventionally reinforced up to a height of 22'-10:" 12" thick conventionally reinforced from 22'-10" to the roof line. There is only one significant opening on the east wall (15' wide x 18' high). The opening is covered by a substantial 24" thick conventionally reinforced rolling door. Personnel entry is through a serpentine type entrance in the north-east corner. The roof structure is made up of open web steel joists at approx. 9'-3" o.c. spanning north-south. 20 gage 1-1/2" steel type B deck panels span between joists. The roof deck is protected from weather by a gravel surfaced five ply built up roofing system. Based on a review of the as-built drawings, the connection of the deck to the joists, in addition to the gage of the decking and the relatively substantial roofing system, the roof panels were not designed to be "blow-out" panels. The bay interior is open to the structure above; i.e., no suspended ceiling system. There



LEVATION (EXISTING)
= 1'-0"



LEVATION (EXISTING)
= 1'-0"



ELEVATION (EXISTING)
= 1'-0"

is a bridge crane installed at 30' above the finished floor. The bridge spans north-south and rails run east-west.

Check Light Gauge Steel Roofing Panels in BAY 2

20 Gauge 1-1/2" B Deck
Span = 9'-3"
A446, Grade A steel
$E = 30x\ 10^6$ psi
$f_y = 33{,}000$ psi
$f_{dy} = 1.21 \times 1.1 \times 33{,}000 = 44{,}000$ psi
$S_{(+)} = 0.235\ in^3$
$S_{(-)} = 0.248\ in^3$
$I = 0.216\ in^4$
$w_{panel} = 2.3$ psf
$w_{5ply\ w/gravel\ roofing} = 6.5$ psf
$M_{up} = (44{,}000\ psi \times 0.235\ in^3)/12 = 862\ lb/ft$
$M_{un} = (44{,}000\ psi \times 0.248\ in^3)/12 = 909\ lb/ft$
$r_u = 3.6\ (M_{un} + 2M_{up})/L^2 = ((3.6)(690\ lb/ft + 2 \times 803\ lb/ft))/(9.25\ ft)^2 = 111\ lb/ft$
$K_E = EI/(0.0062 \times L^3) = (30 \times 10^6\ psi \times 0.216\ in^4)/(0.0062 \times 9.25\ ft^3 \times 144) = 9170$ lb/ft
$w/g = ((2.3\ psf + 6.5\ psf)(10^6)(9.25\ ft))/32.2 = 25.27 \times 10^5\ lb\text{-}m^2/ft$
$T_N = 2\pi((0.74 \times 25.27 \times 10^5\ lb\text{-}m^2/ft)/9170\ lb/ft)^{1/2} = 90\ ms$
$P = p \times b = 2.2\ psi \times 12" \times 12" = 317\ lb/ft$ *(BLDG 1811 loading)*
$P/r_u = (317\ lb/ft)/(111\ lb/ft) = 2.86$
$T/T_n = 218\ ms/90\ ms = 2.42$
$X_m/X_E = 150$ ***No Good; panel is expected to fail***
$P = p \times b = 2.15\ psi \times 12" \times 12" = 310\ lb/ft$ *(BLDG 2131 loading)*
$P/r_u = (310\ lb/ft)/(111\ lb/ft) = 2.79$
$T/T_n = 134\ ms/90\ ms = 1.49$
$X_m/X_E = 65$ ***No Good; panel is expected to fail***
$P = p \times b = 1.94\ psi \times 12" \times 12" = 280\ lb/ft$ *(BLDG 2114 loading)*
$P/r_u = (280\ lb/ft)/(111\ lb/ft) = 2.52$
$T/T_n = 111\ ms/90\ ms = 1.23$
$X_m/X_E = 25$ ***No Good; panel is expected to fail***

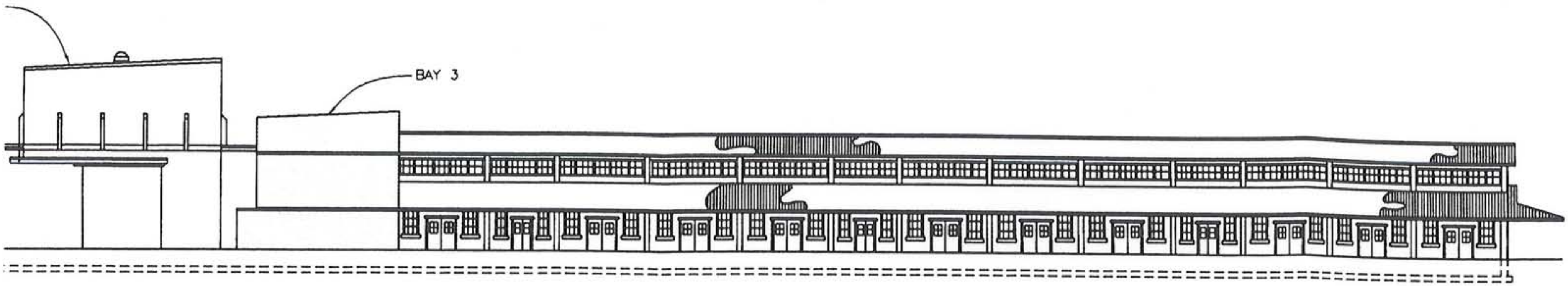
What if additional support were provided at each midspan: *Span = 4.63'*
$M_{up} = (44{,}000\ psi \times 0.235\ in^3)/12 = 862\ lb/ft$
$M_{un} = (44{,}000\ psi \times 0.248\ in^3)/12 = 909\ lb/ft$

$r_u = 3.6\ (M_{un} + 2M_{up})/L^2 = ((3.6)(690\ lb/ft + 2 \times 803\ lb/ft))/(4.63\ ft)^2 = 386\ lb/ft$
$K_E = EI/(0.0062 \times L^3) = (30 \times 10^6\ psi \times 0.216\ in^4)/(0.0062 \times 4.63\ ft^3 \times 144) =$
$73{,}127\ lb/ft$
$w/g = ((2.3\ psf + 6.5\ psf)(10^6)(4.63\ ft))/32.2 = 12.65 \times 10^5\ lb\text{-}m^2/ft$
$T_N = 2\pi((0.74 \times 12.65 \times 10^5\ lb\text{-}m^2/ft)/73{,}127\ lb/ft)^{1/2} = 23\ ms$
$P = p \times b = 2.2\ psi \times 12'' \times 12'' = 317\ lb/ft$ *(BLDG 1811 loading)*
$P/r_u = (317\ lb/ft)/(386\ lb/ft) = 0.82$
$T/T_n = 218\ ms/23\ ms = 9.48$
$X_m/X_E = 2.25$
$X_e = r_u \times L/K_E = (386\ lb/ft)\ (4.63')/73{,}127\ lb/ft = 0.02'$
$X_m = 0.02' \times 2.25 = 0.05'$
$\theta = 1.36°$ ***O.K.***

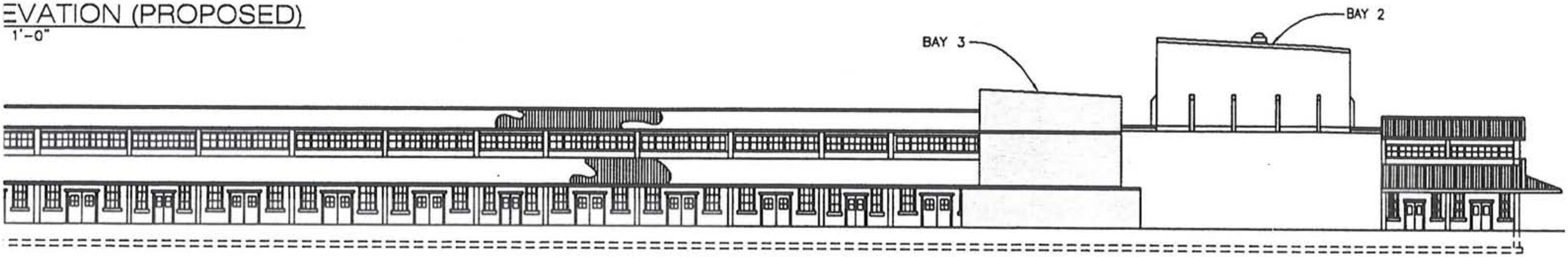
Check Open Web Steel Joists in BAY 2

U28L12 (28" deep)
Span = 41'-7"
Spacing of joists = 9.25'
Weight of decking and roof materials = 8.8 psf
f_y *chords = 50,000 psi*
f_y *web = 36,000 psi*
Dynamic increase factor for chords only: $C = 1.19$
Dynamic design stress, f_{ds} *chords* $= 1.19 \times 1.1 \times 50{,}000\ psi = 65{,}450\ psi$
Maximum allowable ductility ratio: 4.0
Maximum allowable end rotation: 2 degrees
Assumed DLF = 0.62
Equiv static live load on joist: $w_1 = 0.62\ (2.2\ psi \times 144) \times 9.25' = 1817\ lbs/ft$
Service live load on joist: $w_2 = (1817\ lb/ft)/(1.7 \times 1.19 \times 1.1) = 817\ lb/ft$
Using standard loading tables for 28LH12, total load carrying capacity = 837 lb/ft; live load carrying capacity = 520 plf < 817 plf (doesn't look like its going to work; but check anyway...)
approximate weight of joist and decking = 27 plf + 81.4 plf = 108 plf
total load-carrying capacity (excluding dead load = 837 plf - 108.4 plf = 728 plf
$r_u = 1.7 \times 1.19 \times 1.1 \times 728\ lb/ft = 1620\ lbs/ft$
$I = 26.767\ (520\ plf)(42')^3(10^{-6}) = 1031\ in^4$

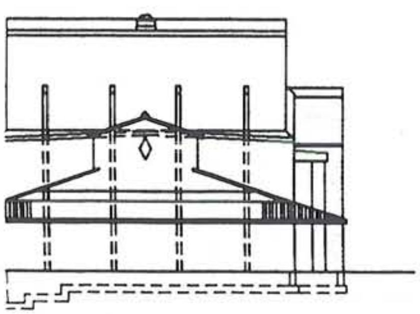
$K_E = 384\ EI/5L^3 = (384 \times 30 \times 10^6 \times 1031\ in^4)/(5)\ (504'')^3 = 18{,}555\ lb/in$
$X_E = r_u L/K_E = 1620\ lb/ft\ (42')/18{,}555\ lb/in = 3.67''$
$M = 108.4\ lb/ft \times 42' \times 10^6/386.4 = 11{,}782{,}609\ lb\text{-}ms^2/in$

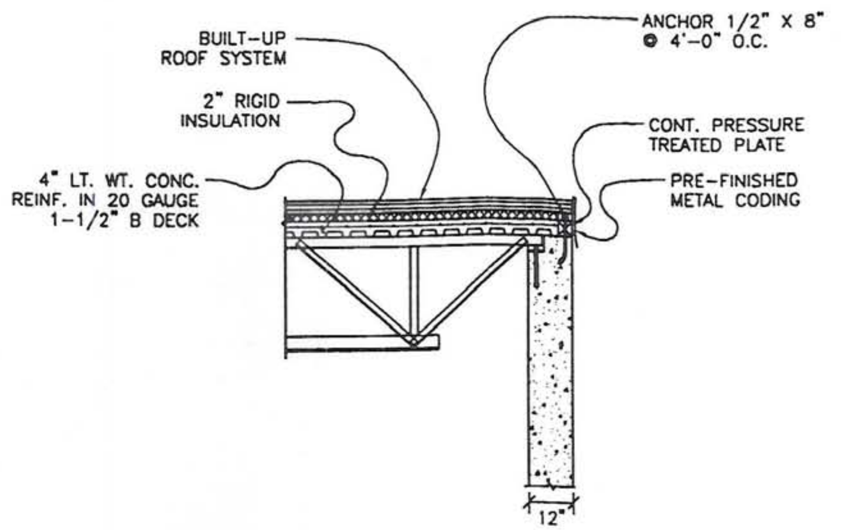


EVATION (PROPOSED)
1'-0"

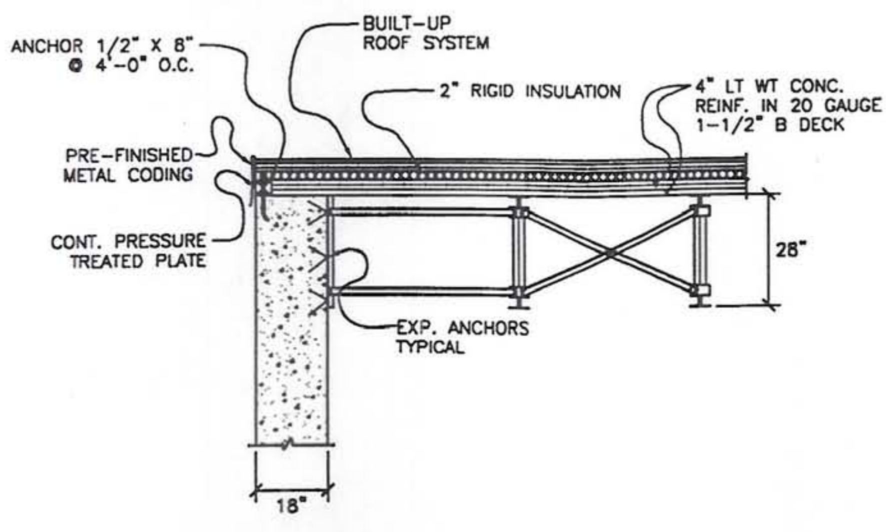


.EVATION (PROPOSED)
1'-0"





C FRAME WORK
SCALE: 1/4" = 1'-0"



D FRAME WORK
SCALE: 1/4" = 1'-0"

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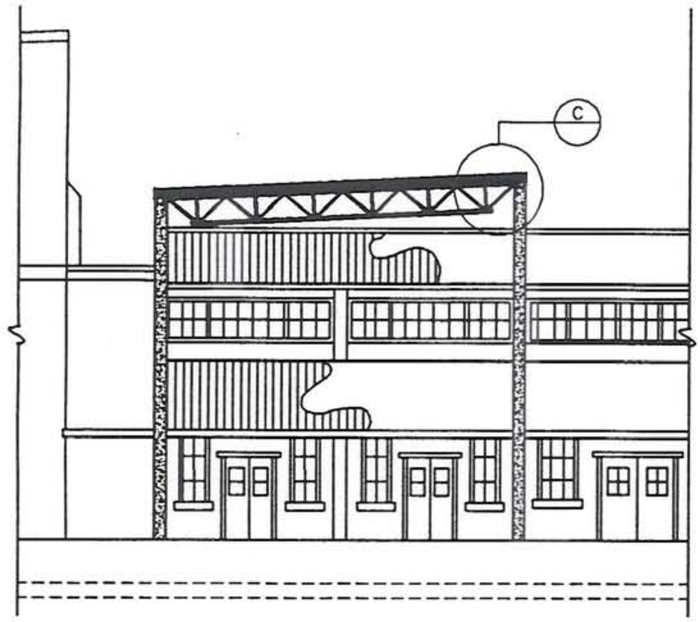
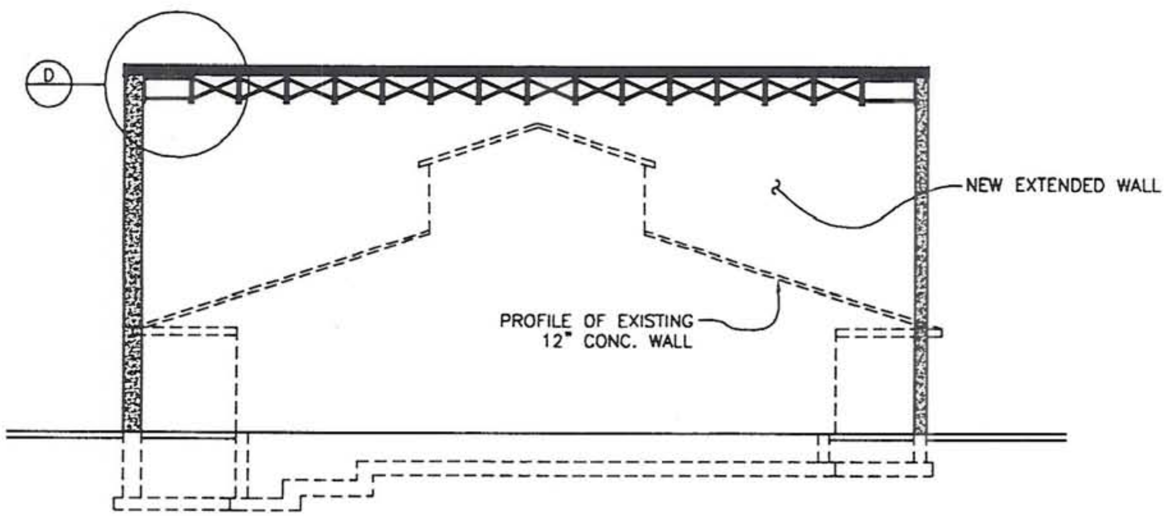
SHEET NO:	DRAWING TITLE:	DATE:	DRAWN BY:	LOCATION:	PROJECT TITLE:
FIG. 11	BUILDING SECTIONS	12-11-98	TSC	HILL AIR FORCE BASE BLDG 2113, BAYS 2 & 3	BLAST ANALYSIS DRAFT REPORT
	NO XREF.	REV. NUMBER:	CHECKED BY:		
	SCALE: VARIES				



B EAST SECTION
SCALE: 1/16" = 1'-0"



B SOUTH SECTION
SCALE: 1/16" = 1'-0"

- add (5) five open web steel joists, (Vulcraft 28LH12, or equal) centered between each existing set of joists
- install new 20 gage Type B roof deck with 4" light weight concrete slab
- install new insulated roofing system

It will be critical to plan and specify in construction documents the means of protecting the equipment within BAY 2 during the construction activities.

7.2 BAY 3 UPGRADES

The upgrade requirements to this bay are more significant. The existing structure was not designed to withstand blast loading. The combination of corrugated roof panels, minimal roof framing members, hollow masonry units, and and wire mesh glazing, all contribute to the prediction of structural failure and/or severe personnel injury in the event of a detonation at any of the PES's. In developing the following recommendations, it was assumed that the objective of the upgrades is to provide a structure that can withstand the predicted blast loading and that will protect personnel and equipment within from fragments.. Architecturally, the changes should blend in with the BAY 2's existing construction and meet Hill AFB' standards. Also, the recommendations should be designed to minimize impact on operations within BAYS 2 and 3.

- construct conventionally reinforced concrete walls (and footings) on the east (12") and west (18") sides
- extend the existing 12" concrete walls on the north and south will in plan to the east and west to meet the new walls.

- extend the north and south walls in height so that the top of the wall is at one elevation (i.e., in the same manner the north wall of BAY 2 was extended to form the X-Ray bay wall in the past)
- install open web steel trusses (Vulcraft 28LH12 or equivalent) at 4' 0.c. and spanning north - south.
- install new 20 gage Type B roof deck with 4" light weight concrete slab
- install new insulated roofing system

Provide a serpentine type egress for personnelon both the east and west sides. The new walls will require footings. Open web steel joists at 4'-0" on center, spanning the 30' between newly extended north and south walls, will be required. As a control room, BAY 3 does not require that roof panels be able to blow out due to a detonation from within. Therefore, 20 gage, 1-1/2", type B steel deck will span between joists. A new insulated standing seam roofing system can be applied to the structural roof deck. Interface between the the existing and structure and the new enclosing structure will be closed off and sealed from weather.

7.3 ESTIMATE OF COST TO CONSTRUCT

A *Rough-Order-of-Magnitude* (ROM) cost estimate was developed to make a prediction of the cost to construct the recommended structural upgrades. The estimated cost at this conceptual level of design is $300,000. A relatively detailed break down of costs is included in the following tables.

TABLE 7: ROUGH ORDER OF MAGNITUDE COST ESTIMATE

COST ESTIMATE SUMMARY	[X] ROM [ ] PRELIMINARY [ ] FINAL [ ] OTHER	PROJECT NUMBER WR41611
INSTALLATION HILL AFB, UT	USING AGENCY	
PROJECT TITLE BLDG 2113 BAYS 2 & 3 UPGRADE FOR BLAST	DRAWING NUMBER	
PREPARED BY EASE, INC. — DATE 12/11/98	APPROVED BY	DATE

DIVISION AND ITEM		QUANTITY AND UNIT	UNIT COST [ ] DIRECT [ ] CONTRACT MT'L	LABOR	TOTAL	EXTENSIONS MT'L	LABOR	TOTAL
02 SITE WORK						$3,510	$3,806	$7,316
03 CONCRETE						$50,343	$44,107	$94,450
05 METALS						$27,853	$27,636	$55,489
07 THERMAL & MOISTURE PROTECTION						$34,273	$29,799	$64,072
08 DOORS AND WINDOWS						$8,000	$3,000	$11,000
09 FINISHES						$576	$760	$1,336
16 ELECTRICAL						$4,820	$3,065	$7,885
SUB-TOTAL						$129,375	$112,174	$241,549
GENERAL CONDITIONS	8.0%					$10,350	$8,974	$19,324
OVERHEAD & PROFIT	10.0%					$12,937	$11,217	$24,155
DESIGN CONTINGENCY	10.0%					$6,469	$5,609	$12,077
TOTAL						$159,131	$137,974	$297,105

COST ESTIMATE DETAIL	[X] ROM [ ] PRELIMINARY [ ] FINAL [ ] OTHER	PROJECT NUMBER WR41611
INSTALLATION *HILL AFB, UT*	USING AGENCY	
PROJECT TITLE *BLDG 2113 BAYS 2 and 3 UPGRADE FOR BLAST*		

DIVISION AND ITEM	QUANTITY AND UNIT	UNIT COST [ ] DIRECT [ ] CONTRACT MT'L	LABOR	TOTAL	EXTENSIONS MT'L	LABOR	TOTAL
02 SITE WORK							
Remove Concrete Paving	350 SF	0.00	0.68	0.68	0	238	238
Sawcut Concrete Ramp	16 LF	0.00	18.00	18.00	0	288	288
Demo Concrete Ramp	1 JB	0.00	750.00	750.00	0	750	750
Excavate	80 CY	12.00	12.00	24.00	960	960	1,920
Backfill & Compact	40 CY	24.00	12.00	36.00	960	480	1,440
Concrete Paving/Slab on Grade	600 SF	1.40	1.40	2.80	840	840	1,680
Asphalt Repair	1 LS	750.00	250.00	1000.00	750	250	1,000
TOTAL 02					3,510	3,806	7,316
03 CONCRETE							
4" Lt Wt Conc. Roof Deck	3,847 SF	1.20	1.00	2.20	4616	3,847	8,463
Concrete Footing	15 CY	108.00	100.00	208.00	1620	1,500	3,120
Concrete 18" Wall	1,300 SF	11.88	10.44	22.32	15444	13,572	29,016
Concrete 12" Wall	3,619 SF	7.92	6.96	14.88	28662	25,188	53,851
TOTAL 03					50,343	44,107	94,450
05 METALS							
Open Web Steel Joists, Bay 2	5,640 LB	1.00	1.50	2.50	5,640	8460	14,100
Open Web Steel Joists, Bay 3	12,528 LB	1.00	0.65	1.65	12,528	8143.2	20,671
1/2" Steel Plate	834 SF	7.00	10.00	17.00	5,838	8340	14,178
Metal Deck, 20 gage, 1-1/2"	3,847 SF	1.00	0.70	1.70	3,847	2692.9	6,540
TOTAL 05					27,853	27,636	55,489
07 THERMAL & MOISTURE PROTECTION							
Roofing System	430 SF	65.00	65.00	130.00	27950	27,950	55,900
Pre-finished Metal Cap	170 LF	5.60	1.40	7.00	952	238	1,190
Pre-finished Roof Gutter	380 LF	3.20	1.00	4.20	1,216	380	1,596
Rigid Insulation, 2"	3,847 SF	1.08	0.32	1.40	4,155	1,231	5,386
TOTAL 07					34,273	29,799	64,072
08 DOORS AND WINDOWS							
Low Pressure Blast Door, Frame w/ Hardware	4 EA	2000.00	750.00	2750.00	8,000	3,000	11,000
TOTAL 08					8,000	3,000	11,000
09 FINISHES							
Suspended Acoustical Ceiling System, 2' x 4'	576 SF	1.00	1.32	2.32	576	760	1,336
TOTAL 09					576	760	1,336
16 ELECTRICAL							
Duplex Receptacle	4 EA	14.50	26.92	41.42	58	108	166
Light Switch, Single Pole	4 EA	12.52	24.38	36.90	50	98	148
Type 1, lighting	4 EA	150.00	45.00	195.00	600	180	780
Type 2 lighting	12 EA	85.00	45.00	130.00	1,020	540	1,560
Exit Light	4 EA	148.00	35.00	183.00	592	140	732
Branch Circ, Conduit & Wire	1 JB	2500.00	2000.00	4500.00	2,500	2,000	4,500
TOTAL 16					4,820	3,065	7,885

BARCS·

HILL AFB BLDG 2113 BAY 2 XRAY FACILITY WALL

BLAST WALL HEIGHT	41.00	FT
BLAST WALL LENGTH	41.70	FT
DURATION OF LOAD	131.00000	MSEC
FICTITIOUS PEAK PRESSURE	2.10000	PSI
EFFECTIVE IMPULSE	139.00	PSI MS

O.K. FOR 2131 LOADING and FOR 1811 east wall 2114A & 1811 But FAILS FOR 1811 West wall

HEIGHT 492.00 LENGTH 500.40

DYNAMIC CONCRETE STRENGTH		3570.00
DYNAMIC STEEL STRESS		51480.00
THICKNESS CONCRETE	INCHES	12.0000
THICKNESS OF SAND	INCHES	0.0000
THETA ALLOWABLE	DEGREES	2.0000

12" constant thickness

#4's @ 13" o.c. EWEF

AREA VERT TOP STEEL/FT	0.1800	COVER	1.5000
AREA VERT BOT STEEL/FT	0.1800	COVER	0.8000
AREA HORIZ TOP STEEL/FT	0.1800	COVER	1.5000
AREA HORIZ BOT STEEL/FT	0.1800	COVER	0.8000

TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION

CONCRETE MODULUS PSI	3079249.
RATIO MOD STEEL/CONCRETE	9.42
GROSS MOMENT INERTIA	144.00
AVE CRACKED MOM INERTIA	13.51
AVE MOMENT INERTIA	78.75
AVERAGE PERCENT STEEL	0.0014
D FACTOR MU=1/6	249433920.
D FACTOR MU= 0.3	266483120.

ALLOW SHEAR UNREINFORCED WEB	105.00	PSI	1139.24	LBS/IN WIDTH
ALLOW SHEAR AT SUPPORT	565.49	PSI	6135.55	LBS/IN WIDTH

UNREINFORCED CONCRETE THETA LE 2 DEG

POSITIVE VERTICAL MOMENT	8550.39
NEGATIVE VERTICAL MOMENT	8009.85
POSITIVE HORIZONTAL MOMENT	8550.39
NEGATIVE HORIZONTAL MOMENT	8009.85

SUPPORT ON 3 SIDES

YIELD LINE Y ABOVE FLOOR

LOCATION YIELD LINE LENGTH	250.20	
LOCATION YIELD LINE HEIGHT	321.48	
ULTIMATE LOAD CAPACITY RU	0.8012	
HORIZ SHEAR LOAD AT SUPPORT	151.46	LB/IN WIDTH
VERT SHEAR LOAD AT SUPPORT	154.54	LB/IN WIDTH
HORIZ SHEAR AT DIST FROM SUPPORT	13.36	PSI
VERT SHEARAT DIST FROM SUPPORT	13.67	PSI
ALLOWABLE MAX DEFLECTION	8.7518	

LOAD MASS FACTOR 0.6509
MASS CONCRETE ONLY 1754.80

FIRST YIELD POINT AT PT2
ELASTIC LIMIT RE PSI 0.38
ELASTIC DEFLECTION XE 0.3473

SECOND YIELD AT PT 3
ELASTO PLASTIC LIMIT 0.49
ELASTO-PLASTIC DEFLECTION 0.6375
ULTIMATE RESISTANCE 0.80
PLASTIC DEFLECTION 1.6604

ULTIMATE RESISTANCE RU 0.80
ELASTIC DEFLECTION LIMIT XE 1.1901
STIFFNESS KE 0.67

MASS 1754.797
LOAD 2.100
DURATION 131.000
RESISTANCE 0.801
STIFFNESS 0.673

NATURAL PERIOD 320.788971

MAXIMUM DEFLECTION **5.740591**

TIME TO MAXIMUM DEFLECTION 202.920700

DURATION/NATURAL PERIOD 0.408368

LOAD/RESISTANCE 2.621150

ELASTIC DEFLECTION LIMIT 1.190091

MAX FRAGMENT SPALL VELOCITY FT/SEC 3.695972

BARCS ·

HILL AFB BLDG 2113 BAY 2 XRAY FACILITY WALL

FOR 1811 LOADING
O.K. assuming wall was 24" all the way to top

BLAST WALL HEIGHT	41.00 FT	
BLAST WALL LENGTH	41.70 FT	
DURATION OF LOAD	149.00000	MSEC
FICTITIOUS PEAK PRESSURE	5.60000	PSI
EFFECTIVE IMPULSE	415.00PSI MS	

HEIGHT 492.00 LENGTH 500.40

DYNAMIC CONCRETE STRENGTH		3570.00
DYNAMIC STEEL STRESS		51480.00
THICKNESS CONCRETE	INCHES	24.0000
THICKNESS OF SAND	INCHES	0.0000
THETA ALLOWABLE	DEGREES	2.0000

AREA VERT TOP STEEL/FT	0.3800	COVER	2.0000
AREA VERT BOT STEEL/FT	0.3800	COVER	0.8000
AREA HORIZ TOP STEEL/FT	0.3800	COVER	2.0000
AREA HORIZ BOT STEEL/FT	0.3800	COVER	0.8000

TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION

CONCRETE MODULUS PSI	3079249.
RATIO MOD STEEL/CONCRETE	9.42
GROSS MOMENT INERTIA	1152.00
AVE CRACKED MOM INERTIA	124.19
AVE MOMENT INERTIA	638.10
AVERAGE PERCENT STEEL	0.0014
D FACTOR MU=1/6	2021043580.
D FACTOR MU= 0.3	2159185150.

ALLOW SHEAR UNREINFORCED WEB	105.04 PSI	2374.00	LBS/IN WIDTH
ALLOW SHEAR AT SUPPORT	565.49 PSI	12780.03	LBS/IN WIDTH

UNREINFORCED CONCRETE THETA LE 2 DEG

POSITIVE VERTICAL MOMENT	37382.75
NEGATIVE VERTICAL MOMENT	35426.51
POSITIVE HORIZONTAL MOMENT	37382.75
NEGATIVE HORIZONTAL MOMENT	35426.51

SUPPORT ON 3 SIDES

YIELD LINE Y ABOVE FLOOR

LOCATION YIELD LINE LENGTH	250.20	
LOCATION YIELD LINE HEIGHT	321.48	
ULTIMATE LOAD CAPACITY RU	3.5225	
HORIZ SHEAR LOAD AT SUPPORT	665.91	LB/IN WIDTH
VERT SHEAR LOAD AT SUPPORT	679.44	LB/IN WIDTH
HORIZ SHEAR AT DIST FROM SUPPORT	26.81	PSI
VERT SHEARAT DIST FROM SUPPORT	27.53	PSI
ALLOWABLE MAX DEFLECTION	8.7518	

LOAD MASS FACTOR	0.6509
MASS CONCRETE ONLY	3509.59

FIRST YIELD POINT AT PT2

ELASTIC LIMIT RE PSI	1.68
ELASTIC DEFLECTION XE	0.1896

SECOND YIELD AT PT 3

ELASTO PLASTIC LIMIT	2.18
ELASTO-PLASTIC DEFLECTION	0.3480
ULTIMATE RESISTANCE	3.52
PLASTIC DEFLECTION	0.8978

ULTIMATE RESISTANCE RU	3.52
ELASTIC DEFLECTION LIMIT XE	0.6427
STIFFNESS KE	5.48

MASS	3509.594
LOAD	5.600
DURATION	149.000
RESISTANCE	3.522
STIFFNESS	5.480

NATURAL PERIOD	159.001480
MAXIMUM DEFLECTION	3.470819
TIME TO MAXIMUM DEFLECTION	137.493271
DURATION/NATURAL PERIOD	0.937098
LOAD/RESISTANCE	1.589793
ELASTIC DEFLECTION LIMIT	0.642738
MAX FRAGMENT SPALL VELOCITY FT/SEC	3.024612

BARCS.

HILL AFB BLDG 2113 BAY 2 XRAY FACILITY WALL

BLAST WALL HEIGHT	41.00	FT
BLAST WALL LENGTH	41.70	FT

DURATION OF LOAD 131.00000 MSEC

FICTITIOUS PEAK PRESSURE	2.10000	PSI
EFFECTIVE IMPULSE	218.00	PSI MS

HEIGHT 492.00 LENGTH 500.40

DYNAMIC CONCRETE STRENGTH	3570.00
DYNAMIC STEEL STRESS	51480.00
THICKNESS CONCRETE INCHES	12.0000
THICKNESS OF SAND INCHES	0.0000
THETA ALLOWABLE DEGREES	2.0000

AREA VERT TOP STEEL/FT	0.1800	COVER	1.5000
AREA VERT BOT STEEL/FT	0.1800	COVER	0.8000
AREA HORIZ TOP STEEL/FT	0.1800	COVER	1.5000
AREA HORIZ BOT STEEL/FT	0.1800	COVER	0.8000

TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION
TYPE 1 CONSTRUCTION

CONCRETE MODULUS PSI	3079249.
RATIO MOD STEEL/CONCRETE	9.42
GROSS MOMENT INERTIA	144.00
AVE CRACKED MOM INERTIA	13.51
AVE MOMENT INERTIA	78.75
AVERAGE PERCENT STEEL	0.0014
D FACTOR MU=1/6	249433920.
D FACTOR MU= 0.3	266483120.

ALLOW SHEAR UNREINFORCED WEB	105.00	PSI	1139.24	LBS/IN WIDTH
ALLOW SHEAR AT SUPPORT	565.49	PSI	6135.55	LBS/IN WIDTH

UNREINFORCED CONCRETE THETA LE 2 DEG

POSITIVE VERTICAL MOMENT	8550.39
NEGATIVE VERTICAL MOMENT	8009.85
POSITIVE HORIZONTAL MOMENT	8550.39
NEGATIVE HORIZONTAL MOMENT	8009.85

SUPPORT ON 3 SIDES

YIELD LINE Y ABOVE FLOOR

LOCATION YIELD LINE LENGTH	250.20	
LOCATION YIELD LINE HEIGHT	321.48	
ULTIMATE LOAD CAPACITY RU	0.8012	
HORIZ SHEAR LOAD AT SUPPORT	151.46	LB/IN WIDTH
VERT SHEAR LOAD AT SUPPORT	154.54	LB/IN WIDTH
HORIZ SHEAR AT DIST FROM SUPPORT	13.36	PSI
VERT SHEARAT DIST FROM SUPPORT	13.67	PSI
ALLOWABLE MAX DEFLECTION	8.7518	

LOAD MASS FACTOR 0.6509
MASS CONCRETE ONLY 1754.80

FIRST YIELD POINT AT PT2
ELASTIC LIMIT RE PSI 0.38
ELASTIC DEFLECTION XE 0.3473

SECOND YIELD AT PT 3
ELASTO PLASTIC LIMIT 0.49
ELASTO-PLASTIC DEFLECTION 0.6375
ULTIMATE RESISTANCE 0.80
PLASTIC DEFLECTION 1.6604

ULTIMATE RESISTANCE RU 0.80
ELASTIC DEFLECTION LIMIT XE 1.1901
STIFFNESS KE 0.67

MASS 1754.797
OAD 2.100
URATION 131.000
ESISTANCE 0.801
TIFFNESS 0.673

NATURAL PERIOD 320.788971

MAXIMUM DEFLECTION 5.740591

TIME TO MAXIMUM DEFLECTION 202.920700

DURATION/NATURAL PERIOD 0.408368

LOAD/RESISTANCE 2.621150

ELASTIC DEFLECTION LIMIT 1.190091

MAX FRAGMENT SPALL VELOCITY FT/SEC 3.695972

BARCS

HILL AFB BLDG 2113 BAY 3 CONTROL ROOM WEST WALL

```
    BLAST WALL HEIGHT          26.00 FT
    BLAST WALL LENGTH          41.70  FT

    DURATION OF LOAD       149.00000  MSEC

    FICTITIOUS PEAK PRESSURE         5.60000  PSI
    EFFECTIVE IMPULSE                415.00PSI MS

  HEIGHT      312.00    LENGTH      500.40

DYNAMIC CONCRETE STRENGTH      3570.00
DYNAMIC STEEL STRESS          51480.00
THICKNESS CONCRETE  INCHES     18.0000
THICKNESS OF SAND   INCHES      0.0000
THETA ALLOWABLE    DEGREES      2.0000

AREA VERT TOP STEEL/FT          0.4000  COVER    1.5000
AREA VERT BOT STEEL/FT          0.4000  COVER    0.8000
AREA HORIZ TOP STEEL/FT         0.4000  COVER    1.5000
AREA HORIZ BOT STEEL/FT         0.4000  COVER    0.8000
      TYPE 1 CONSTRUCTION

CONCRETE MODULUS PSI               3079249.
RATIO MOD STEEL/CONCRETE          9.42
GROSS MOMENT INERTIA            486.00
AVE CRACKED MOM INERTIA          70.11
AVE MOMENT INERTIA              278.06
AVERAGE PERCENT STEEL           0.0020
D FACTOR MU=1/6                        880686080.
D FACTOR  MU= 0.3                     940882368.

ALLOW SHEAR UNREINFORCED WEB    106.49  PSI        1794.30 LBS/IN WIDTH
ALLOW SHEAR AT SUPPORT          565.49  PSI        9528.47  LBS/IN WIDTH
UNREINFORCED CONCRETE  THETA LE 2 DEG

POSITIVE VERTICAL MOMENT     29030.01
NEGATIVE VERTICAL MOMENT     27828.81
POSITIVE HORIZONTAL MOMENT     29030.01
NEGATIVE HORIZONTAL MOMENT     27828.81

SUPPORT ON  3 SIDES

YIELD LINE  Y  ABOVE FLOOR

LOCATION YIELD LINE LENGTH      250.20
LOCATION YIELD LINE HEIGHT      274.04
ULTIMATE LOAD CAPACITY  RU     3.7857
HORIZ SHEAR LOAD AT SUPPORT          622.32  LB/IN WIDTH
VERT SHEAR LOAD AT SUPPORT           622.46  LB/IN WIDTH
HORIZ SHEAR AT DIST FROM SUPPORT      34.21  PSI
VERT SHEARAT DIST FROM SUPPORT        34.22  PSI
ALLOWABLE MAX DEFLECTION       8.7518

LOAD MASS FACTOR                0.6074
MASS CONCRETE ONLY             2456.51
```

ELASTIC DEFLECTION XE	0.2699

SECOND YIELD AT PT 3

ELASTO PLASTIC LIMIT	1.78
ELASTO-PLASTIC DEFLECTION	0.4188
ULTIMATE RESISTANCE	3.79
PLASTIC DEFLECTION	1.6925

ULTIMATE RESISTANCE RU	3.79
ELASTIC DEFLECTION LIMIT XE	1.2877
STIFFNESS KE	2.94

MASS	2456.511
LOAD	5.600
DURATION	149.000
RESISTANCE	3.786
STIFFNESS	2.940
GAS PRESSURE	0.00
DURATION	0.00

NATURAL PERIOD	181.621902
MAXIMUM DEFLECTION	4.916589
TIME TO MAXIMUM DEFLECTION	132.517014
DURATION/NATURAL PERIOD	0.820386
LOAD/RESISTANCE	1.479237
ELASTIC DEFLECTION LIMIT	1.287683
MAX FRAGMENT SPALL VELOCITY FT/SEC	4.545382

$Me = 0.72\ M = 8{,}483{,}478\ lb\text{-}ms^2/in$
$T_N = 2\ \pi\ ((8{,}483{,}478\ lb\text{-}ms^2/in)/18{,}555\ lb/in)^{1/2} = 134\ \ ms$
$P/r_u = (2.2\ psi \times 144 \times 9.25')/\ 1621\ lb/ft = 1.81$ (BLDG 1811 loading)
$T/T_N = 218\ ms/134\ ms = 1.63$
$X_m/X_E > 27$ **No Good**
$P/r_u = (2.15\ psi \times 144 \times 9.25')/\ 1621\ lb/ft = 1.77$ (BLDG 2131 loading)
$T/T_N = 134\ ms/134\ ms = 1.0$
$X_m/X_E > 10$ **No Good**
$P/r_u = (1.94\ psi \times 144 \times 9.25')/\ 1621\ lb/ft = 1.6$ (BLDG 2114 loading)
$T/T_N = 111\ ms/134\ ms = 0.83$
$X_m/X_E > 6.5$ **No Good**

Try adding open web steel joist, centered between each existing, to reduce loading on joists and to cut roof deck span in half

Span = 41'-7"
Spacing of joists = 9.25'/2 = 4.63'
approximate weight of joist and decking = 27 plf + 40.74 plf = 68 plf
total load carrying capacity (excluding dead load = 837 plf - 68 plf = 769 plf
$r_u = 1.7 \times 1.19 \times 1.1 \times 769\ lb/ft = 1711\ lbs/ft$
$I = 26.767\ (520\ plf)(42')^3(10^{-6}) = 1031\ in^4$
$K_E = 384\ EI/5L^3 = (384 \times 30 \times 10^6 \times 1031\ in^4)/(5)\ (504")^3 = 18{,}555\ lb/in$
$X_E = r_uL/K_E = 1711\ lb/ft\ (42')/18{,}555\ lb/in = 3.87"$
$M = 68\ lb/ft \times 42' \times 10^6/386.4 = 7{,}391{,}304\ lb\text{-}ms^2/in$
$Me = 0.72\ M = 5{,}321{,}739\ lb\text{-}ms^2/in$
$T_N = 2\ \pi\ ((5{,}321{,}739\ lb\text{-}ms^2/in)/18{,}555\ lb/in)^{1/2} = 106\ \ ms$
$P/r_u = (2.2\ psi \times 144 \times 4.63)/\ 1711\ lb/ft = 0.86$ (BLDG 1811 loading)
$T/T_N = 218\ ms/106\ ms = 2.0$
$X_m/X_E = 2$
$X_m = 2\ (3.87") = 7.74"$
$\theta = 1.76°$ **O.K.**
$t_e/T = 0.14;\ \ t_e = 0.14 \times 218\ ms = 30.52\ ms$
$E = f_{ds}/E_st_e = 65.45\ ksi/30 \times 10^3\ ksi \times 0.031\ sec = 0.07\ in/in/sec$
$DIF = 1.17 \cong 1.19$ *for this level of design*

Check Concrete Walls in BAY 2

assume $f'c = 3000\ psi$
assume $f_y = 40{,}000\ psi$
$E = 2.9 \times 10^6$

24" thick to 22'-10" then 12" thick to 41'
24" wall reinforced w/#6's at 14" o.c. each way each face
12" wall reinforced w/#4's at 13" o.c. each way each face
BARCS program utilized to predict structural response. (output is included in appendices). 24" wall full height would be sufficient to resist the worst case loading. 12" wall would not resist worst case loading on west; but O.K. on east; recommend adding 1/2" steel plate to interior upper portion of west 12" wall anchorage to concrete would need to be designed and detailed to provide composite action between concrete and steel. Convert composite wall section to transformed section:

assume $Es/Ec = n = 9$

$A_1 = 15.96\ in^2$	$y_1 = 6.5"$	$A_1y_1 = 103.74\ in^3$	$I_1 = 191.5\ in^4$
$A_2 = 6\ in^2$	$y_2 = 0.25"$	$A_2y_2 = 1.5\ in^3$	$I_2 = 0.13\ in^4$

Sum of $A = 21.96\ in^2$
Sum of $Ay = 105.24\ in^3$
Sum of $I = 191.63\ in^4$
Sum $Ay/$*Sum* $A = 4.79"$
$I = 191.62\ in^4 + 21.96\ in^2\ (4.79")^2 = 696\ in^4$
$S = 696\ in^4/(12.5" - 4.79") = 90.3\ in^3$
Span $= 41'\text{-}7"$
$E = 30 \times 10^6\ psi$
$f_{ds} = 47{,}120\ psi$
$S_{(+)} = 90.3\ in^3$
$I = 696\ in^4$
$w = 170\ plf$
$M_{up} = (47{,}120\ psi \times 90.35\ in^3)/12 = 355\ k\text{-}ft$
$R_u = 8\ M/L = 8\ (355\ k\text{-}ft)/\ (41.6\ ft) = 68\ k$
$K_E = 384EI/(5 \times L^3) = (384 \times 30 \times 10^3\ psi \times 696\ in^4)/(5 \times 41.6\ ft^3 \times 144) = 154\ k/ft$
$w/g = (170\ plf)(10^3)(41.6\ ft))/32.2 = 219{,}627\ k\text{-}ms^2/ft$
$T_N = 2\ \pi\ ((0.72 \times 219{,}627\ k\text{-}ms^2/ft)/154\ k/ft)^{1/2} = 201\ ms$
$P = p \times b = 5.6\ psi \times 47.6' \times 144" = 33.6\ k$ *(BLDG 1811 loading)*
$P/R_u = (33.6\ k)/(68\ k) = 0.49$
$T/T_n = 149\ ms/201\ ms = 0.74$
$X_m/X_E = 0.8$
$X_E = R_u/K_E = 68\ k \times 12/154\ k/ft = 5.3"$
$X_m = 0.8\ (5.77") = 4.61"$
$\theta = 1.06$ *degrees* ***O.K.***

The equipment is controlled from BAY 3. Originally BAY 3 was an operating bay. North and south walls are 12" conventionally reinforced concrete which extend up to the roof line. The profile of the roof line varies in the east-west direction. East and west walls are hollow clay block units with a height of 12.' There are large window openings and a personnel door in both east and west walls. In addition, there are clerestory windows, set back, facing east and west. The roof structure is made up of steel purlins at aproximately 52" o.c. spanning 30.' Corrugated roof panels span between steel purlins. These panels were originally intended to blow out in the event of an detonation during operations within the bay. The inside of the bay has non-bearing partition walls and suspended ceiling.

Check Corrugated Roofing Panels of BAY 3

Span = 52"
Corrugated Panels
$E = 30x\ 10^6$ *psi (assumed)*
$f_y = 33{,}000$ *psi (assumed)*
$f_{dy} = 1.21 \times 1.1 \times 33{,}000 = 44{,}000$ *psi*
$S = 0.0429\ in^3$
$I = 0.0112\ in^4$
$w_{panel} = 2.0$ *psf*
$M = (44{,}000\ psi \times 0.0429\ in^3)/12 = 157\ lb/ft$
$r_u = 3.6\ (M_{un} + 2M_{up})/L^2 = ((3.6)(157\ lb/ft + 2 \times 157\ lb/ft))/(4.33\ ft)^2 = 90.37\ lb/ft$
$K_E = EI/(0.0062 \times L^3) = (30 \times 10^6\ psi \times 0.0112\ in^4)/(0.0062 \times 4.33\ ft^3 \times 144) =$
$4632\ lb/ft$
$w/g = ((2\ psf)(10^6)(4.33\ ft))/32.2 = 268{,}944\ lb\text{-}m^2/ft$
$T_N = 2\ \pi\ ((0.74 \times 268{,}944\ lb\text{-}m^2/ft)/4632\ lb/ft)^{1/2} = 42\ ms$
$P = p \times b = 2.2\ psi \times 12" \times 12" = 317\ lb/ft$ *(BLDG 1811 loading)*
$P/r_u = (317\ lb/ft)/(90.37\ lb/ft) = 3.51$
$T/T_n = 218\ ms/42\ ms = 5.19$
$X_m/X_E > 500$ **No Good; panel is expected to fail**
$P = p \times b = 2.15\ psi \times 12" \times 12" = 310\ lb/ft$ *(BLDG 2131 loading)*
$P/r_u = (310\ lb/ft)/(90.37\ lb/ft) = 3.43$
$T/T_n = 134\ ms/42\ ms = 3.19$
$X_m/X_E = 400$ **No Good; panel is expected to fail**

$P = p \times b = 1.94$ psi x 12" x 12" = 280 lb/ft *(BLDG 2114 loading)*
P/r_u = (280 lb/ft)/(90.37 lb/ft) = 3.1
T/T_n = 111 ms/42 ms = 2.64
X_m/X_E = 170 ***No Good; panel is expected to fail***

Check Steel Purlins BAY 3

6 x 12 beams at 52" o.c.
Span = 30'
$S = 7.31\ in^3$
$I = 22.1\ in^4$
A36 steel
f_{ds} = 1.1 x 1.29 x 36 ksi = 51.1 ksi
$M = 7.31\ in^3$ x 51.1 ksi / 12 = 31.13 k-ft
d/t_w =26.2 O.K.
b_f/t_f = 7.1 O.K.
m = ((2 psf x 4.33') + 12 plf) x 30' x 10^3 /32.2 = 19,248 k-ms^2/ft
K_E = (384 x 30 x 10^3 x 22.1 in^4)/5 (30')^3 (144) = 13.10 k/ft
$T_N = 2\pi$((0.72 x 19,248 k-ms^2/ft)/13.10 k/ft)$^{1/2}$ = 204 ms
T/TN = 218 ms/204 ms = 1.07
P = 2.2 psi x 30' x 4.33' x 144/1000 = 41.2 kips
Ru = 8 x 31.13 k-ft/30'=8.3 kips
P/Ru = 41.2 kips /8.3 kips = 5
Xm/XE = 100 ***NO GOOD; Purlin is expected to fail***

Check Unreinforced Hollow Clay Block Units in BAY 3

8" thick hollow, 35 psf, f'm 700 psi
Height = 12'
Allowable rotations to be limited to 0.5 degrees
r_u = (2/144"^{2})1350 psi (4"-0.5")^2 = 1.6 psi ***NO GOOD; Wall expected to fail***

Predict Window Pane Response in BAY 3

The existing window panes are assumed to fail without further quantification of capacity, since the masonry wall is predicted to fail. Window glazing should be of heat-treated, fully-tempered glass in fixed or non-operable frames; the existing glazing does not meet this requirement. The design criteria and

> *installation criteria of the frames (i.e, sealants, gaskets, anchorage, edge bite of framing) is of as much importance as the glazing panel itself. Typically poor framing detailing and installation practice is a the cause of failure. The windows are not required for operational purposes, therefore it is recommended to not include windows in the structural upgrades.*

A summary of the predicted dynamic response of the structural elements to blast loading from the three PES's is presented in TABLE 6 below. Proposed upgrades to harden the structure are provided in the last column of the table and further detailed in the next section of this document.

TABLE 6: SUMMARY OF PREDICTED STRUCTURAL RESPONSE TO SHOCK LOADING

ES	STRUCTURAL ELEMENT	PES	PREDICTED STRUCTURAL RESPONSE	PROPOSED STRUCTURAL UPGRADE
BAY 2	Roof Panel	BLDG 1811	FAIL	*supporting joists are predicted to fail also; recommend leaving panel/roofing in place and adding joists to cut panel span in half*
		BLDG 2131	FAIL	
		BLDG 2114A	FAIL	
BAY 2	Roof Joist	BLDG 1811	FAIL	*add five open web steel joists to reduce the on center spacing; this will also improve structural response of roof panels*
		BLDG 2131	FAIL	
		BLDG 2114A	FAIL	
BAY 2	West Wall	BLDG 1811	FAIL	*add 1/2" steel pl. to interior upper half of wall*
		BLDG 2131	O.K.	
		BLDG 2114A	O.K.	
BAY 2	East Wall	BLDG 1811	O.K.	*no upgrades necessary*
		BLDG 2131	O.K.	
		BLDG 2114A	O.K.	
BAY 3	Roof Panel	BLDG 1811	FAIL	*Only the north and south walls are predicted to withstand blast loading; recommend adding concrete walls to east and west; extending north ex. concrete wall to match south ex. concrete wall; new roof structure.made up of open web steel joists and steel roof decking*
		BLDG 2131	FAIL	
		BLDG 2114A	FAIL	
BAY 3	Roof Purlin	BLDG 1811	FAIL	
		BLDG 2131	FAIL	
		BLDG 2114A	FAIL	
BAY 3	East/West Wall	BLDG 1811	FAIL	
		BLDG 2131	FAIL	
		BLDG 2114A	FAIL	

6.0 PREDICT EFFECT OF FRAGMENTS ON ES

Significant damage from accidental detonations can be caused not only by the shock loads but also by the impact of the fragments which were generated during the explosions and hurled against the ES at high speeds. Typically, fragments generated during a detonation can be divided into two main types: primary and secondary. Primary fragments are produced by the shattering of the explosive container and are characterized by very high initial velocities, a large number of fragments, and relatively small sizes in comparison to secondary fragments. Primary fragments initially travel at velocities of thousands of feet per second. Secondary fragments can be produced due to the blast wave interaction with objects located near the explosive source and/or the donor structure. They are typically larger than primary fragments and initially travel at velocities of hundreds of feet per second.

Protection from fragment impact is typically provided by requiring the PES and the ES to be separated by the DDESB's default distance. These distances are specified in AFMAN 91-201. For a PES with a NEW greater than 100 lbs, the minimum default distance to an ES that requires IBD distance protection is 1250.' DDESB approved analyses may be used to determine reduced distances for both primary and secondary fragments. The destructive potential of a fragment is a function of the fragment's shape, material, momentum and kinetic energy distribution. Each of those functions listed is also a function of many other variables. Accurate prediction of the effects of fragments is extremely complex. There is less data from full scale tests to back up prediction methodology's for fragments than there is for backing up the prediction of blast wave parameters.

In this case, protection can not be assured based on distance without significantly reducing quantities at the ES. The best approach is to provide protection by ensuring that the ES structure has the capacity to withstand penetration by fragments and to provide protection against the effects of firebrands. The focus of this evaluation is on providing the protection within the ES. Personnel outside of the ES structure (i.e., in the parking lot, in vehicles, in other portions of BLDG 2113, etc.) will be exposed to the fragment hazard.

Based on the results of the previous section, it is assumed that the worst case scenario of all the PES's is a detonation in BLDG 1811 of 135,776 lbs NEW. BLDG 1811 is a an earth covered steel arch magazine with a reinforced concrete head wall on the east and west ends. Earth covered magazine are not designed to resist the damaging effects of an internal detonation: it is accepted that the magazine will be demolished. Many assumptions are made in the following calculations to develop a realistic and conservative estimate of the threat due to fragments that should be considered in the design of the structural upgrades. The calculations are in accordance with AFR 88-22.

spherical charge
casing thickness = 0.5"
$R_f = 1207'$
$W = 135{,}776 \text{ lbs} \times 1.2 = 162{,}931 \text{ lbs}$
density of explosive $= 0.0558 \text{ lb/in}^3$
$d = 14' = 168''$
$W_c = 4/3\pi(168.5''^3 - 168''^3) \times 0.283 \text{ lb/in3} = 50{,}310 \text{ lbs}$
$W/W_c = 162{,}931 \text{ lbs}/50{,}310 \text{ lbs} = 3.24$
$(2E')^{1/2} = 8000$ *(for TNT)*
$v_o = (2E')^{1/2}((W/W_c)/(1 + (3W/5W_c)))^{1/2} = 8393 \text{ ft/sec}$
$M_A = Bt_c^{5/6}\, d^{1/3}(1 + (t_c/d)) = 0.312\,(0.5)^{5/6}\,(168'')^{1/3}(1 + 0.5'',/168'') = 0.68$
$W_{f\,average} = 2M_A^2 = 0.92 \text{ oz}$
$N_T = 8\,(50{,}310 \text{ lbs})/(0.68)^2 = 870{,}415$ *fragments*
$W_{f\,design} = M_A^2\, ln^2(1\text{-}CL) = (0.68)^2\, ln^2(1-0.95) = 4.17 \text{ oz}$
$v_s = v_o\, e^{-0.004Rf/Wf1/3} = 418 \text{ ft/sec}$

Determine maximum penetration into ES concrete wall with f'c = 3,000 psi
$W_f = 4.17\ oz = 0.26\ lbs$
$d = ((3/4)(1/\pi)(0.26\ lbs)(1/0.283))^{1/3}(2) = 1.21"$
$k_{xf} = 0.7\ (1.92 \times 10^{-3}\ W_f^{\ 0.37}\ v_s^{\ 0.9}) = 0.52"$
$\boldsymbol{x'_f = k_{xf}(4{,}000\ psi\ /3{,}000\ psi)^{1/2} = 0.60" < 12"}$ ***O.K.***
minimum concrete thickness to prevent perforation: $\boldsymbol{T_{pf} = 2.28" < 12"}$ ***O.K.***
minimum concrete thickness to prevent spalling: $\boldsymbol{T_{sp} = 3.42" < 12"}$ ***O.K.***

Determine maximum penetration into mild steel

$x = 0.21\ W_f^{\ 0.33}\ v_s^{\ 1.22} = 0.12"$

This doesn't bode well for cold formed steel roof deck panels. Recommend utilizing light weight (110 pcf) concrete (approx. 4" depth) with deck panels. This will provide protection against fragment penetration and a one-hour fire rating to the roof assembly. Need to go back and check open web steel joists to make sure they can handle the additional dead load associated with the concrete.

Check Open Web Steel Joists in BAY 2 due to additional dead load

U28L12 (28" deep)
Span = 42'
Spacing of joists = 4.63
Weight of decking and roof materials= 37 psf
f_y *chords = 50,000 psi*
f_y *web = 36,000 psi*
Dynamic increase factor for chords only: C = 1.19
Dynamic design stress, f_{ds} *chords = 1.16 x 1.1 x 50,000 psi = 63,800 psi*
Maximum allowable ductility ratio: 4.0
Maximum allowable end rotation: 2 degrees
Assumed DLF = 0.62
Equiv static live load on joist: $w_1 = 0.62\ (2.2\ psi \times 144) \times 4.63' = 909\ lbs/ft$
Service live load on joist: $w_2 = (909\ lb/ft)/(1.7 \times 1.16 \times 1.1) = 419\ lb/ft$
Using standard loading tables for 28LH12, total load carrying capacity = 837 lb/ft; live load carrying capacity = 520 plf > 409 plf
approximate weight of joist and decking = 27 plf + 171 plf = 198 plf
total load-carrying capacity (excluding dead load = 837 plf - 198 plf = 639 plf
$r_u = 1.7 \times 1.16 \times 1.1 \times 639\ lb/ft = 1386\ lbs/ft$
$I = 26.767\ (520\ plf)(42')^3(10^{-6}) = 1031\ in^4$
$K_E = 384\ EI/5L^3 = (384 \times 30 \times 10^6 \times 1031\ in^4)/(5)\ (504")^3 = 18{,}555\ lb/in$
$X_E = r_u L/K_E = 1386\ lb/ft\ (42')/18{,}555\ lb/in = 3.14"$

$M = 198\ lb/ft \times 42' \times 10^6/386.4 = 21{,}521{,}739\ lb\text{-}ms^2/in$
$Me = 0.72\ M = 15{,}495{,}652\ lb\text{-}ms^2/in$
$T_N = 2\pi((15{,}495{,}652\ lb\text{-}ms^2/in)/18{,}555\ lb/in)^{1/2} = 182\ ms$
$P/r_u = (2.2\ psi \times 144 \times 4.63')/ 1386\ lb/ft = 1.06$ *(BLDG 1811 loading)*
$T/T_N = 218\ ms/182\ ms = 1.20$
$X_m/X_E = 2.25$
$X_m = 2.25\ (3.14'') = 7.07''$
$\theta = 1.61°$ **O.K.**
$t_e/T = 0.22;\ t_e = 0.22 \times 218\ ms = 48\ ms$
$E = f_{ds}/E_s t_e = 63.8\ ksi/30 \times 10^3\ ksi \times 0.048\ sec = 0.04\ in/in/sec$
$DIF = 1.16 = 1.16$ O.K.

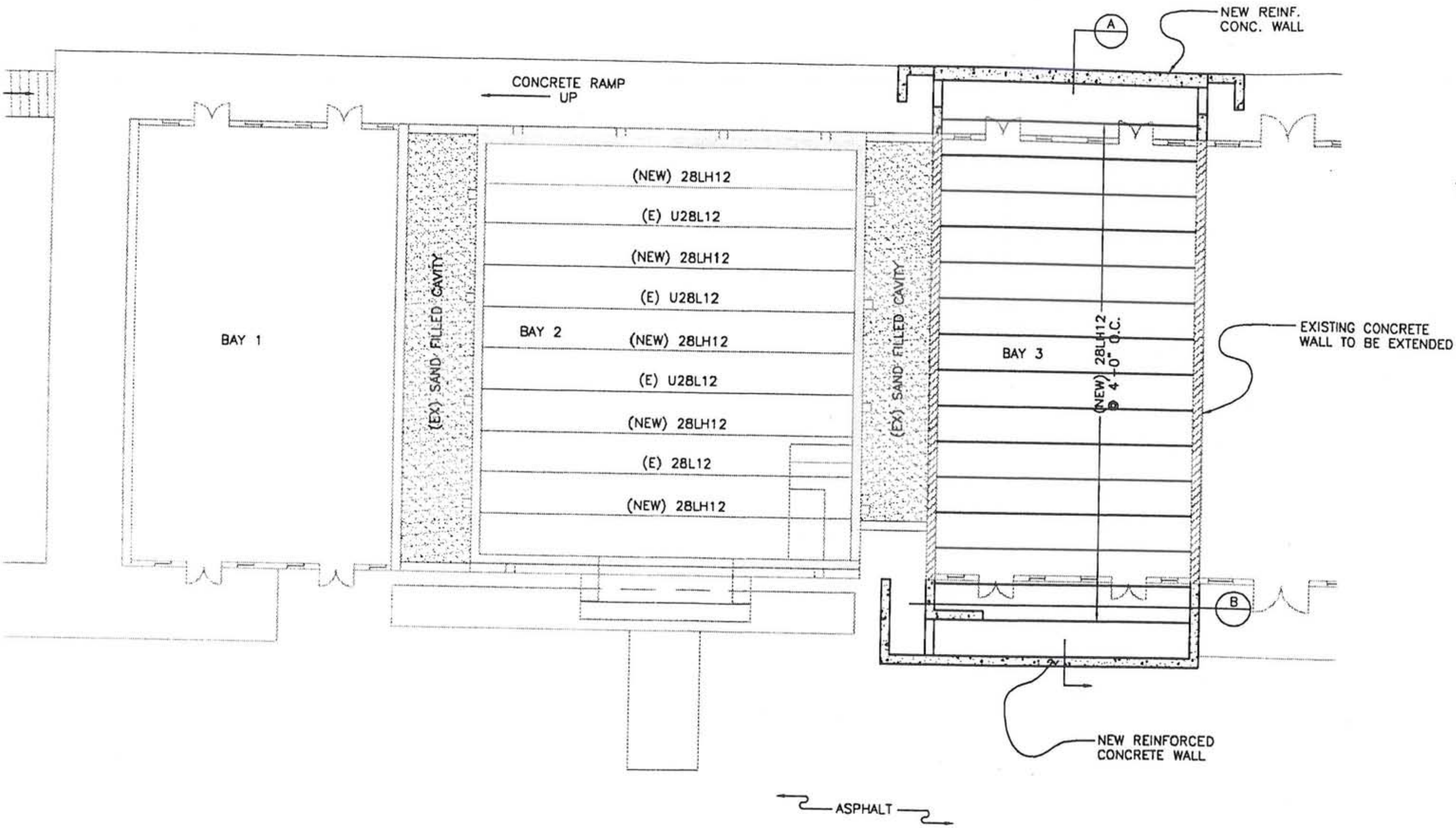
7.0 SUMMARY AND RECOMMENDED ACTIONS

The recommended upgrades described in the next two sections are illustrated in FIGURES 9 and 10.

7.1 BAY 2 UPGRADES

The existing roof deck is spanning 9'-3" and is predicted to fail under "worse-case" blast loading scenario. The open web steel joists are also predicted to fail. Both of these issues could be eliminated by adding joists at mid deck span. However, cold formed steel roof panels on their own are not substantial enough to prevent penetration of fragments. Roof deck capacity to protect against fragments can be improved by adding light weight concrete slab. The narrower upper portion of concrete west wall was predicted to fail under the blast loading. The addition of a 1/2" steel plate to the interior of the 12" thick wall would result in the required capacity as long as the anchorage of the steel plate to the existing concrete was designed to effectively provide composite action between the two materials. The recommended necessary changes to BAY 2 include:

- remove existing 5-ply built-up roof, insulation, and roof deck panels



PLAN VIEW (PROPOSED)
SCALE: 1/16" = 1'-0"