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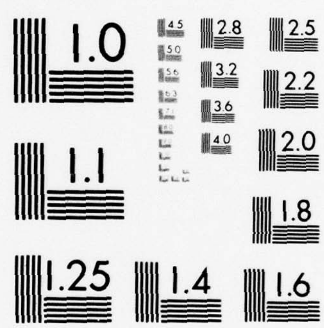

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LEVEL II

SURVIVABILITY ON THE FRINGE OF HIGH RISK AREAS

FINAL REPORT

DCPA Contract DCPA01-76-C-0324

Work Unit 1621G

October 1978

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This report contains the results of a study aimed at exploring the sheltering potential for people in low risk areas, i.e., in areas with overpressures less than 2 psi (13.79 kPa) caused by megaton range nuclear weapon detonations. Two basement shelters and eight special purpose shelters were evaluated and a "people survivability function" was developed for each. A survivability function relates percent survivors in a given shelter to a range of overpressures at the shelter site. Casualty mechanisms considered with this set of shelters include blast, i.e., dynamic pressures and debris from the breakup of the shelter

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) structure. Results included in this report allow the civil defender to estimate the people survivability potential in and near the fringe of high risk areas when shelters of the type considered in this study are used to protect the population.

SURVIVABILITY ON THE FRINGE OF HIGH RISK AREAS

DCPA Contract DCPA01-76-C-0324

Work Unit 1621G

FINAL REPORT

by

A. Longinow

for

Defense Civil Preparedness Agency
Washington, D.C. 20301

August 1978

DCPA REVIEW NOTICE

This report has been reviewed in the Defense Civil Preparedness Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Defense Civil Preparedness Agency.

PREFACE

This is the final report on IIT Research Institute (IITRI) J6389 entitled, "Survivability on the Fringe of High Risk Areas". The study was performed for the Defense Civil Preparedness Agency (DCPA) under Contract DCPA01-76-C-0324. Work was initiated August 5, 1976 and completed June 15, 1978.

The study was performed in the Structural Analysis Section, Engineering Division of IITRI by A. Longinow. Mr. D. A. Bettge of DCPA monitored the program. The numerous suggestions provided by Mr. Bettge in the course of this study are gratefully acknowledged.

Respectfully submitted,
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AL/al

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1.1 Statement of the Problem

The primary concern in United States civil defense is the short-term and the long-term survival of the population. At the present time various population centers are at-risk with respect to a nuclear weapon attack. At any given time and location the level of risk is variable and reaches its potentially highest level during a crisis period.

Current United States thinking, relative to a national civil defense posture, includes crisis relocation planning (CRP). This would result in moving a significant fraction of the high risk urban area population into the surrounding low level of risk areas.

Obviously, not all of the people will or can leave the high risk urban areas. Many of the currently existing life support facilities (LSF), i.e., food processing plants, food storage warehouses, and medical supply manufacturing plants are located in or near potentially high risk urban areas. This is also true of numerous vital industries not immediately or directly related to life support but whose continued operation at some level of production is vital to the viability of the nation. This would include among others the materials processing and equipment manufacturing industries. Therefore certain groups of people will be required to remain behind to staff and operate designated LSF and other critical industries. Before CRP can be effectively implemented, two basic questions need to be answered:

1. What level of shelter is required in host (low level of risk) areas?
2. What level of protection is required in fringe areas i.e., areas in direct vicinity to high risk areas?

The objective of the study reported was to produce data on the basis of which question such as 1 and 2 could be answered. The study was concerned with the *development of survivability functions for people in regions with overpressures 2 psi (13.79 kPa) and less, caused by megaton range nuclear weapon detonations. Shelters for which people* *survivability functions were developed included community centers, individual homes and special purpose shelters. All casualty mechanisms pertinent to low risk areas were considered.*

1.2 Shelters in Host Areas

Regions of low risk that would act as host areas for the displaced population would include all feasible population centers and adjoining areas able to provide shelter and capable of being reached by the (assigned) population within a stipulated time period.

Since shelters against the effects of nuclear weapons (in the true definition of the term) do not exist, the displaced population in host areas would need to be housed in all available accommodations capable of providing some level of protection. This would include community centers such as town halls, armories, churches, schools, theaters and shopping centers. Other candidates would be large office buildings, certain parking garages and basements of residences. When these sources are exhausted, it will be necessary to construct expedient shelters and make use of other resources such as mines, caves and tunnels if such are available in the vicinity.

Shelter for the displaced (and host) population is imperative. People need to be protected against the effects of blast, thermal and fallout radiation, near and in fringe of high risk areas. They may require protection against postattack fires. In areas further removed from high risk areas they will require protection against fallout radiation. In all cases they need to be protected against the elements which depending on the season of the year may include snow, rain, local storms, etc.

1.3 Weapon Induced Environment

The surface or near surface detonation of a megaton range nuclear weapon will produce a variety of direct and indirect effects which can produce casualties to a resident population. The nature and intensity of these weapon-generated environments in and near the fringe of high risk areas must therefore be explicitly examined and defined. The high risk area boundary is defined here as the 2 psi (13.79 kPa) overpressure contour. It would appear desirable to examine the environments at overpressure levels somewhat in excess of 2 psi (13.79 kPa), as well as below 2 psi (13.79 kPa), in order to more accurately determine casualty functions and their trends.

The blast overpressure and wind (dynamic pressure) effects will be significant effects for the production of casualties in the fringe areas. The 2 psi (13.79 kPa) blast overpressure wave will have a positive phase duration of 3 to 10 seconds depending upon the weapon yield and other weapon related factors. The wind environment will have a peak intensity of approximately 100 fps, about 70 mph (112.65 km/h) and a positive phase duration which is slightly greater than that of the overpressure wave. The shock wave will arrive at the corresponding range approximately 20 to 60 seconds after the time-of-burst, hence some warning time may be available to, at least, alert individuals. Other weapon effects will exist at these distant ranges; however, many of them, such as prompt nuclear radiation, may not be significant.

The free field overpressure and wind environments will produce or induce a number of secondary environments which have the potential of producing casualties. These are:

- blast overpressures within shelters
- wind fields within shelters
- dust and debris environments external of shelters
- debris within shelters due to structural failure and collapse
- debris within shelters due to internal wind effects
- window glass debris and the penetration of external debris into shelters
- others, such as fires, etc.

Proceeding from high overpressure levels downward, i.e., from high to low level of risk areas, probable damage and casualties experienced in fringe areas can be described as follows.

In the 5 to 2 psi (34.77 to 13.79 kPa) range, most framed buildings less than 10 stories in height are expected to remain standing. Those with weak, nonarching walls will for the most part have lost them, depositing debris in the buildings and in the street below. Windows, window frames, interior partitions and doors will be lost. Most furniture and contents will be displaced by the blast winds and be spread throughout the building. Some, on the far side of the building, will be ejected to the street below. What applies to furnishings also applies to people. There will be numerous injuries among people located in the upper stories and some fatalities. Thermal radiation, blast winds and debris (including window glass) will be significant casualty mechanisms for people located in unshielded upper story areas. However, people in basements are expected to survive relatively uninjured, except possibly for those near basement entrances.

Most family residences located in such regions will have suffered structural damage ranging from severe damage to collapse. The debris environment within and in the vicinity of such buildings is expected to be extremely hazardous during the passage of the blast wave. Approximately one-half of the occupants in the upper story spaces will have experienced fatality. The remainder will have experienced injuries. Those located in basements will mostly survive, though impact injuries are expected.

In the 2 to 1 psi (13.79 to 6.89 kPa) range, large framed structures will experience broken windows, doors, and damage to light interior partitions. Flying debris (glass and other objects) will produce a hazardous environment for people in the upper stories. Injuries will be numerous; however, most people are expected to survive. The environment in the upper stories of residences should be similar to that in the upper stories of framed buildings, i.e., a hazardous debris environment will dominate casualties. Residences are expected to experience moderate to severe damage, i.e., broken windows, doors, cracked studs, rafters and damage to roofs. Basements of residences should provide fairly adequate protection in this region. The debris environment external to shelters is briefly described.

Tree branches are more resistant to dynamic pressures in the winter when defoliated than in the summer. For purposes of relating various types of damage to overpressure, it may be said that great numbers of tree branches will be torn off at a range corresponding to an overpressure of 2 psi (13.79 kPa) (wind velocity 70 mph (112.65 km/h) and perhaps 30 percent of the trees will be blown down at 3 psi (20.68 kPa) (wind velocity nearly 100 mph (160.93 km/h). Fences, telephone poles, street signs, framed car garages are expected to be severely damaged and thus would produce a hazardous environment during the passage of a blast wave in the fringe of high risk areas.

The free field wind environment will generate dust clouds and otherwise loft and transport debris over significant ranges. The relatively low intensity of the wind will limit the size of debris which will be lofted from relatively flat terrain. The 100 fps (30.48 m/sec) velocity associated with the 2 psi (13.79 kPa) overpressure condition will just loft natural particles such as stone and gravel (i.e., small debris) which are approximately 1 mm in diameter. At an overpressure level of 4 psi (27.58 kPa) the limiting size increases to about 1 cm. Other types of debris, both large and small, will exist due to the rupture of "soft" structures. The debris can be accelerated to horizontal velocities approaching the peak wind velocity depending on the time of free flight, the injection point, and various characteristics of the debris such as their weight, shape and size.

The state of utilities in the fringe of high risk areas in the postevent period is expected to be as summarized in Table 1 (Ref. 4).

TABLE 1. SUMMARY OF POSTEVENT PERIOD

Overpressure Region	Utility Damage
1 to 2 psi (6.89 to 13.79 kPa)	Electricity - Overhead lines disrupted over a wide area. Minor damage to control equipment. Light damage to unhoused substations in wooded areas. Many service lines down. Gas - Exposed regulators and meters suffer light damage from debris. Many residential service connections broken. Gasholders suffer light damage. Power loss commonplace. Water - Widespread breakage of residential services-laboratory equipment and softening and chlorination equipment suffer light damage from debris. Power loss commonplace. Sewerage - Widespread but minor damage to service connections. Pumps stop due to power loss. Light damage to filters and chlorination equipment. Power loss commonplace. Communications - Overhead lines disrupted over wide areas. Telemetering and control equipment suffer light damage from light structural debris. Power loss commonplace.
2 to 5 psi (13.79 to 34.77 kPa)	Electricity - Virtually all overhead lines and poles down. Severe damage to substations near wooded areas and in buildings. Severe damage to control panels. Pole mounted transformer down. Circuit breakers, capacitators, condensers, relays, etc., in buildings damaged. Gas - Exposed regulators and meters severely damaged. Widespread damage to regulators, meters and service connections in buildings. Gasholders destroyed. Moderate to severe damage to control station. Water - All residential service connections and many major building connections damaged. Widespread debris damage to hydrants. Many storage tanks damaged. Treatment plants and laboratory equipment suffer moderate to severe damage. Sewerage - Widespread damage to service connections. Moderate to severe damage to pumping and treatment equipment. Communications - Most overhead lines and poles down. Communications hardware items in buildings moderate to severe damage from building debris. Some antennas down.

1.4 Personnel Shelters and People Survivability Functions

To provide a means for estimating the survivability potential for the population in the fringe of high risk areas and in their vicinity away from high risk areas, people survivability functions were developed for several types of shelters. These include the following.

1. Basement with a Precast/Prestressed Overhead Floor System
2. Basement of a Wood Framed, Single Family Residence
3. Special Purpose, Basement Shelters
   - Concrete Block Shelter
   - Lean-to Shelter
   - Rigid Frame Shelter
   - Reinforced Concrete Block Shelter
4. Special Purpose, Outside Shelters
   - Aboveground A-Frame Shelter
   - Plywood Box Shelter
   - Wood Grate Roof Shelter
   - Gable Roof Shelter

Each shelter was analyzed when subjected to the prompt effects of a single, megaton range nuclear weapon exploded near the ground surface. Since the study dealt with shelters which were either in basements or were closed and mounded with soil, it was possible to neglect thermal radiation and prompt nuclear radiation as significant casualty mechanisms. Effects considered include diffraction and drag loading produced by the blast wave on the shelter structure and dynamic pressure and debris (from the breakup of the structure) on the shelter occupants.

The degree of protection provided by a shelter is described here by means of a survivability function. A typical survivability function is shown in Figure 1. A casualty function would be similar except that instead of the probability of survival, P_s, the ordinate would be in terms of the probability of casualty, P_c where $P_c = 1.0 - P_s$.

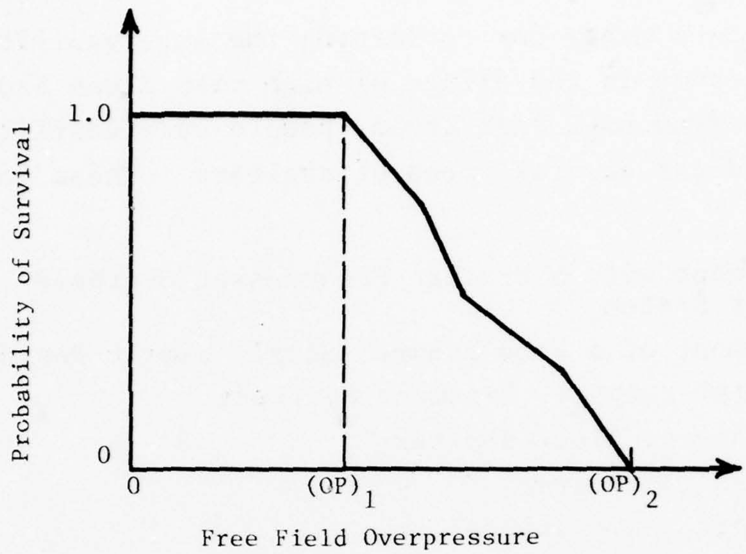


Figure 1. People Survivability Function

Obviously, when dealing with blast effects in the range of overpressures where the ground shock can be neglected, the survival of people in a shelter is strongly dependent on the survival of the shelter itself. Thus, prior to any damage to the shelter, i.e., prior to overpressure $(OP)_1$, see Figure 1, blast effects are kept to a minimum and the probability of survival of individuals against the effects of blast is essentially 1.0. When total structural collapse of the shelter is experienced, the probability of survival is generally very low. Considering the low likelihood of rescue and medical aid in the immediate postevent period, the probability of survival at the overpressure producing total shelter collapse, i.e., $(OP)_2$ shown in Figure 1, is assigned a value of zero. In the transition range between $(OP)_1$ and $(OP)_2$, casualties are produced by debris and dynamic pressures when local areas of the shelter are breached by the blast. The manner in which casualties are estimated against these casualty mechanisms is described in Reference 5. A brief description of the type of structural analysis performed here for determining overpressures $(OP)_1$, $(OP)_2$, etc. is given next.

A shelter is a three-dimensional network using some combination of the following structural components, i.e., beams, columns, stressed skin panels, reinforced or unreinforced concrete block, etc. When loaded, the internal force and bending moment distributions in each component are coupled, i.e., statically indeterminate. Although it is possible to solve the complete interaction problem by considering all components in the network simultaneously, both in space and in time, the extent of effort involved is not warranted due to uncertainties in the distribution of blast loading, the extent of variation in material properties, the response of timber when subjected to dynamic loads, etc. Instead, the structural network is uncoupled by making certain simplifying assumptions regarding the restraint conditions at the junction points of each component. Upon decomposing the system into a series of smaller systems in the manner described, each component can be analyzed as a separate unit and the weakest link in the network determined. For a preliminary analysis, which this study was intended to provide, results obtained in this fashion are sufficiently accurate.

It is believed that results included in this report allow the civil defender to estimate the people survivability potential in and near the fringe of high risk areas when shelters of the type considered in this study are used to protect the population.

This report is a companion volume to "Survivability in Crisis Upgraded Shelters", Contract DCPA01-76-C-0321, Work Unit 1619B by the same author. It deals with the problem of the sheltering potential for people (key workers) remaining in high risk areas.

2. PEOPLE SURVIVABILITY ESTIMATES FOR SHELTERS ON THE FRINGE OF HIGH RISK AREAS

This chapter contains results of analyses that were performed to determine the protective capabilities of certain categories of buildings that would be located in the fringe of high risk areas. Such buildings would act as personnel shelters. They were evaluated both as built and upgraded. The hazard environment is assumed to be produced by the effects of a single, megaton range nuclear weapon exploded near the ground surface. Buildings and upgrading techniques considered in the analyses were selected on the basis of ground rules discussed in the previous chapter.

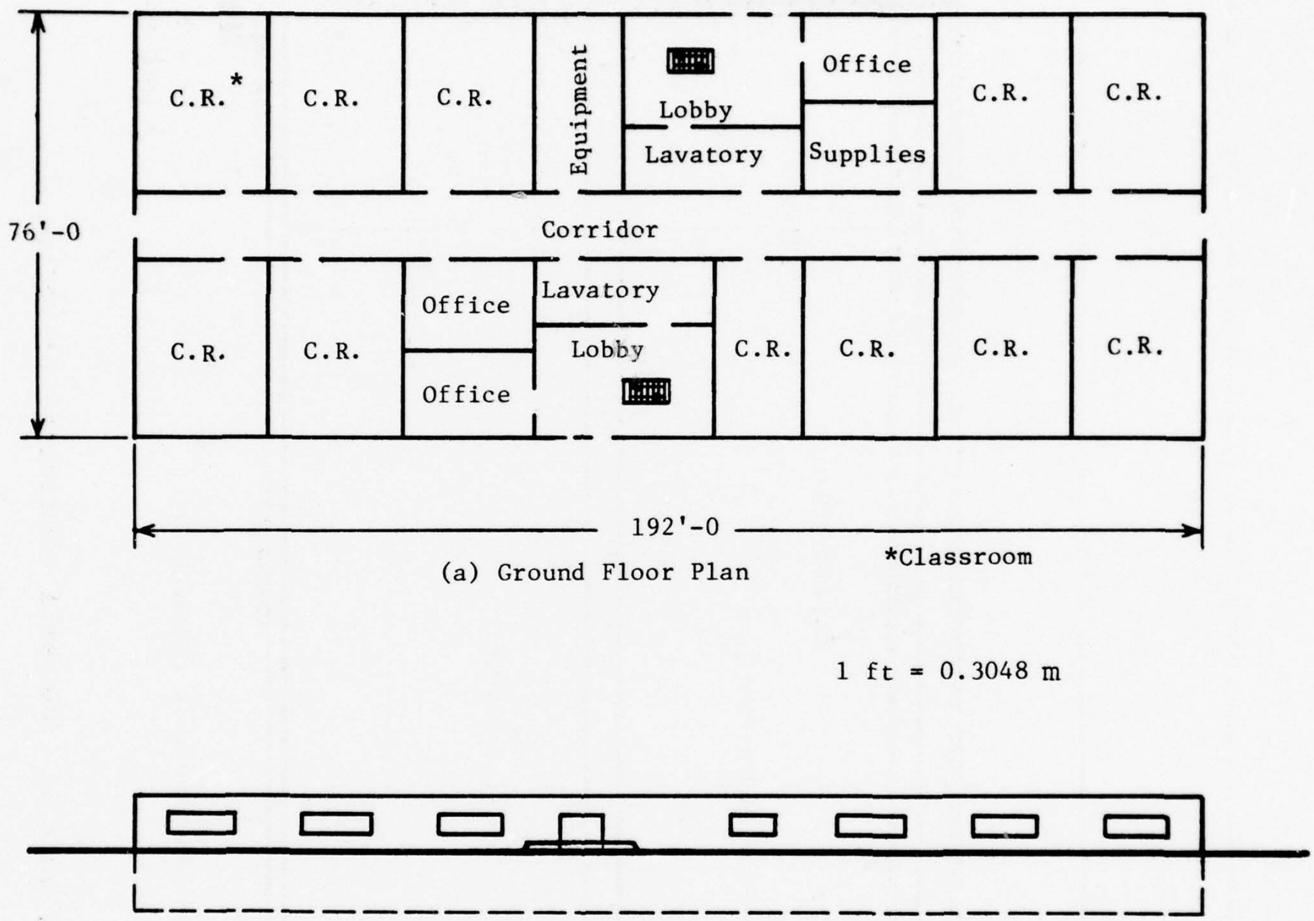
2.1 Basement Shelter with a Precast/Prestressed Overhead Floor System

2.1.1 Building Description - The building considered herein is one classroom building of a moderate sized secondary school. Other portions of the school such as the gymnasium, cafeteria, health services, shop, laboratory and recreational facilities are housed in neighboring buildings whose sheltering potential is not being considered. These buildings are therefore not shown.

The classroom building is a one-story structure with a full basement. It contains approximately 29,000 sq ft (2694.19 sq m) of floor space. Its floor plan and elevation are shown in Figure 2.

The roof and the ground floor consists of precast, prestressed hollow core deck sections (see Figure 3 and 4). Exterior walls are of brick and are 9 inches (0.229 m) thick. Longitudinal corridor walls on the ground floor are also of brick and have a thickness of 9 inches (0.229 m). Exterior walls and interior corridor walls are load-bearing. Corridor walls in the basement as well as the peripheral walls are of reinforced concrete and are 9 inches (0.229 m) thick. All interior partitions are of 6 inch (0.15 m) concrete masonry. They are nonload-bearing.

The building has an occupancy under conventional use of approximately 460 people. This includes students, faculty, administrative and maintenance personnel. Typical occupancy would probably be less.

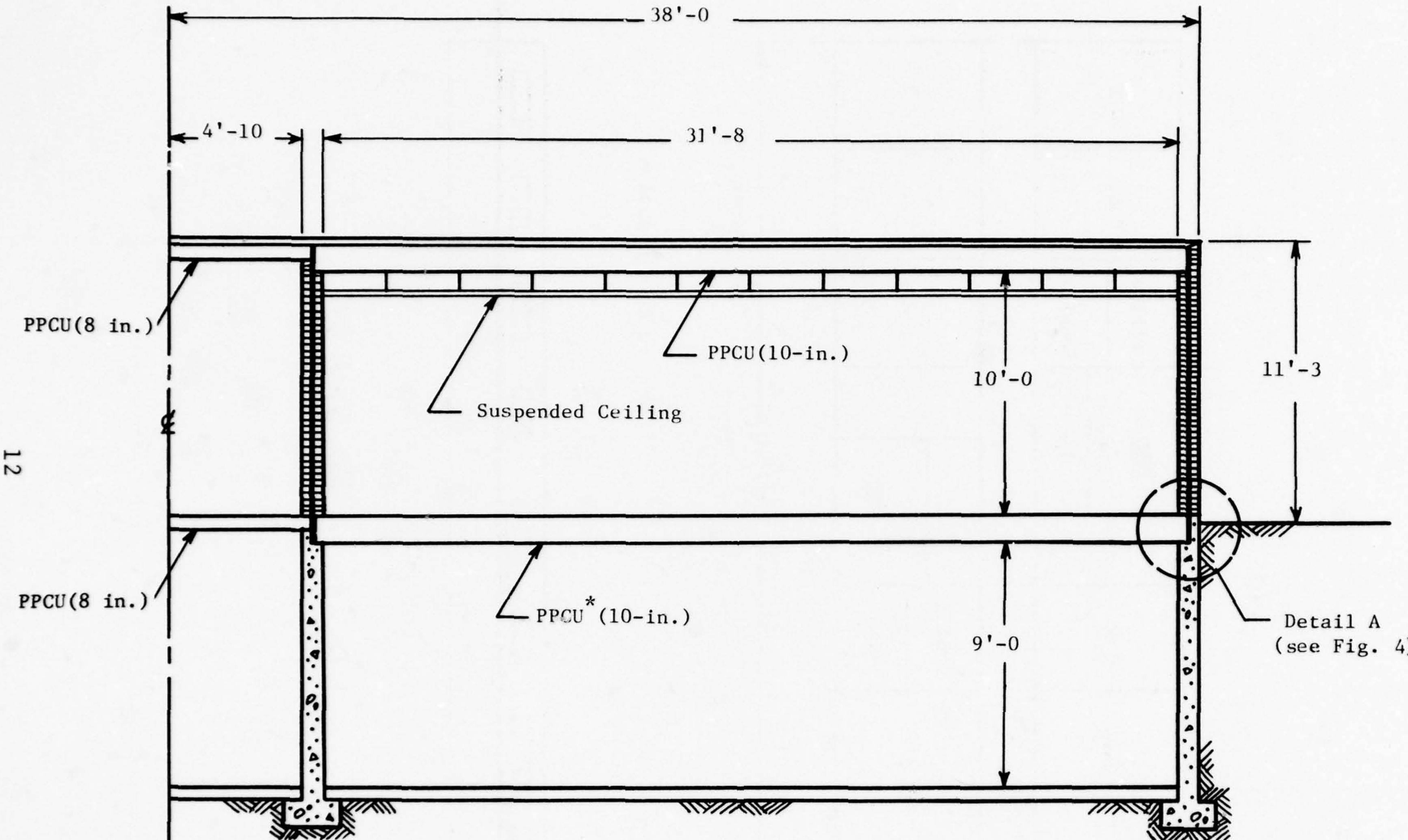


(a) Ground Floor Plan

1 ft = 0.3048 m

(b) Longitudinal Elevation

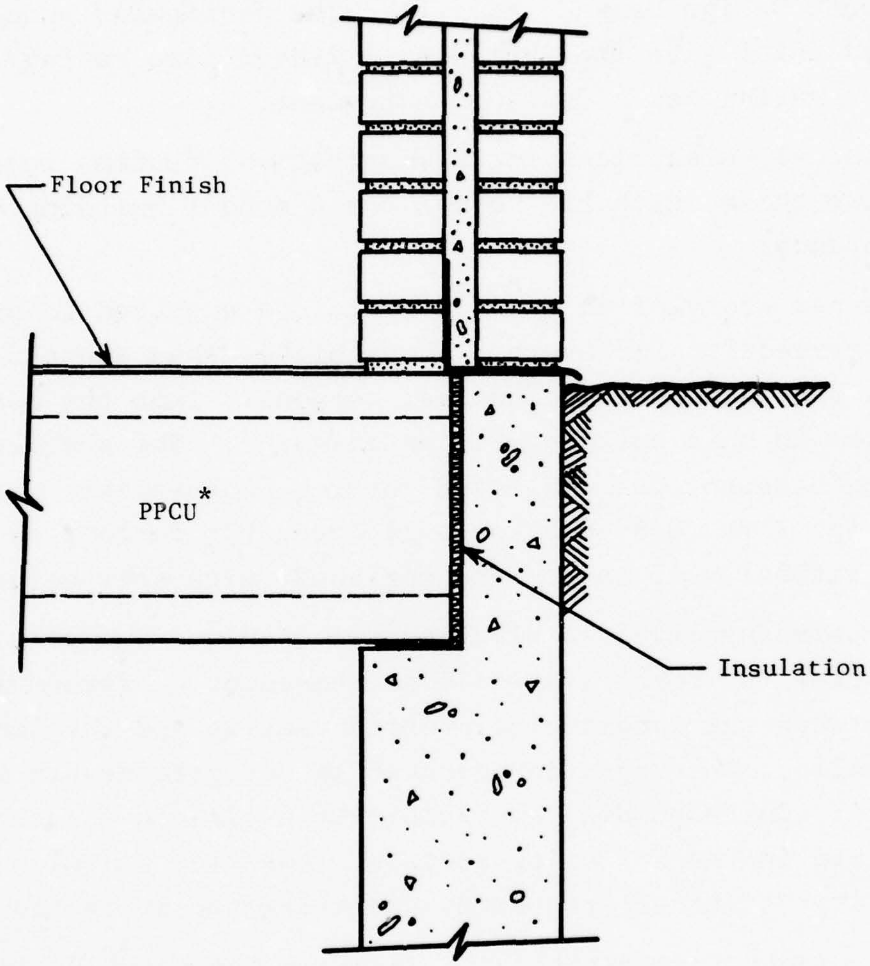
Figure 2. Floor Plan and Elevation



* Precast, prestressed concrete unit (SPANCRETE)

1 ft = 0.3048 m
1 inch = 25.4 mm

Figure 3. One-half Transverse Elevation



* Precast, prestressed concrete unit (SPANCRETE)

Figure 4. Detail A from Figure 3

Structural aspects of this building were designed in accordance with the ACI (318-71) Building Code (Ref. 6) and Building Code Requirements for Engineered Brick Masonry (Ref. 7). Precast, prestressed units were selected using procedures recommended in the Spancrete Design Manual (Ref. 8). The particular precast, prestressed units give this building a 2-hour fire rating. The 2-hour fire rating was a design requirement.

Mechanical, electrical and communications systems in this building are those which are normal for a school building with the given occupancy.

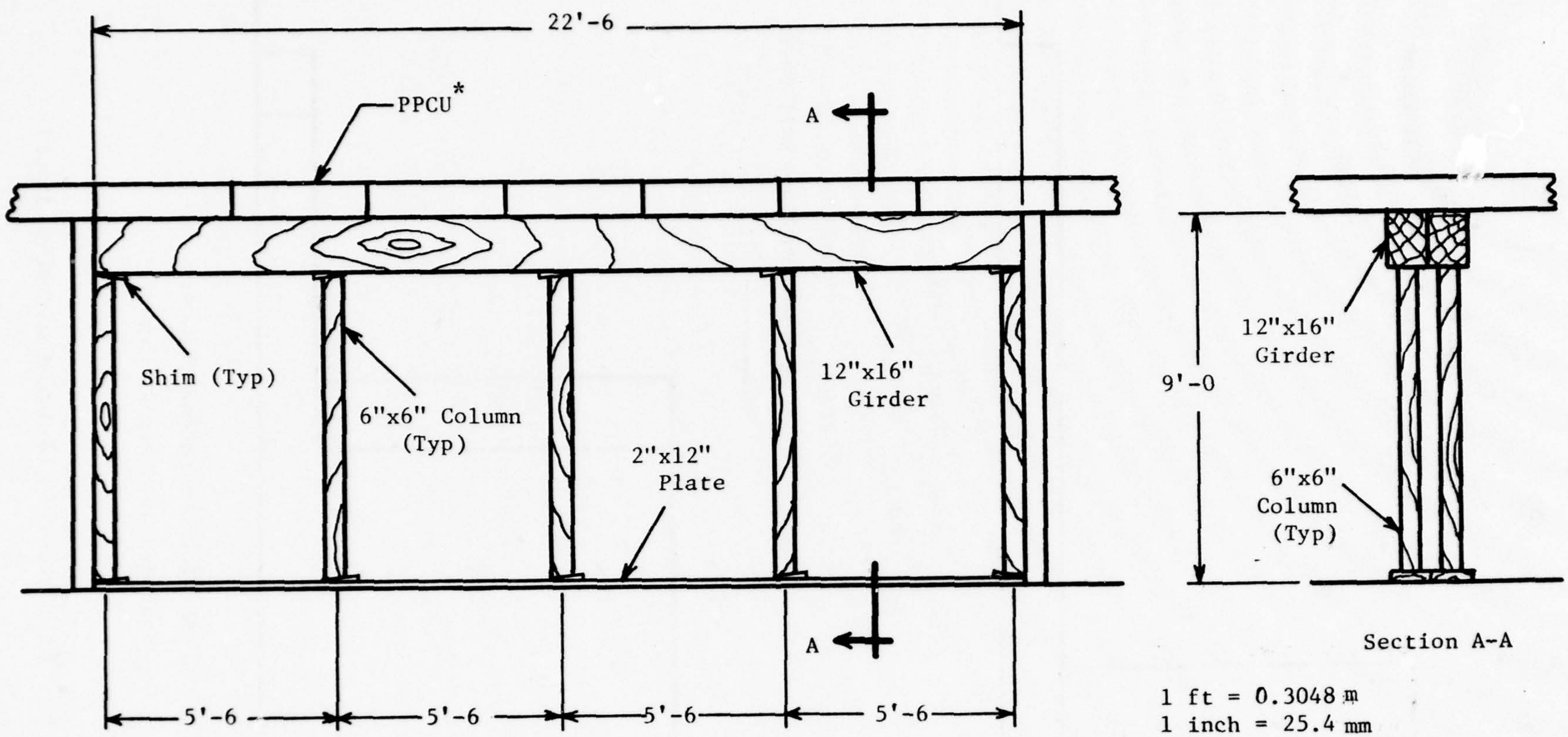
The upper story of this building is not expected to provide any blast protection for overpressures higher than about 0.5 psi (3.45 kPa) for megaton range nuclear weapons. Thus the most viable shelter area in this building is the basement. The sheltering potential of the basement was evaluated for four conditions: as-built; as-built plus 1 ft (0.3048 m) of soil cover for fallout protection; upgraded, without soil cover; and upgraded, with soil cover.

The upgrading involved blocking all openings and placing a timber bracing (support) system in all basement classroom areas halfway between the longitudinal corridor walls and the longitudinal basement walls. The upgrading concept as designed herein is shown in Figure 5. Corresponding survivability estimates for this shelter are described in the following section. Analyses on which these survivability estimates are based, are presented in Section 2.1.3.

2.1.2 People Survivability in Basement Shelter - Estimates of the protective capabilities of the basement portion of this structure are summarized in Figure 6. They are based on the effects of blast resulting from a single, megaton-range nuclear weapon exploded near the ground surface.

The primary casualty mechanism was found to be debris from the breakup of the overhead floor system. The mode of failure was found to be shear of the individual precast, prestressed concrete units used in the construction of this building.

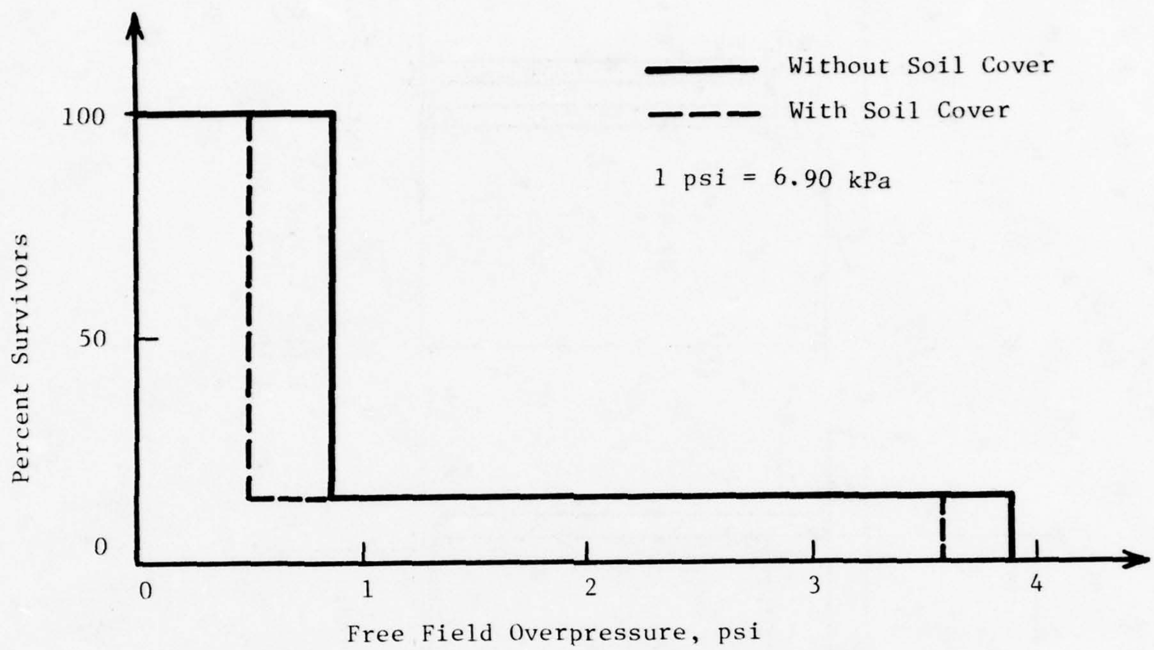
In making these estimates it was assumed that people are uniformly distributed in all classroom and corridor areas. Failure



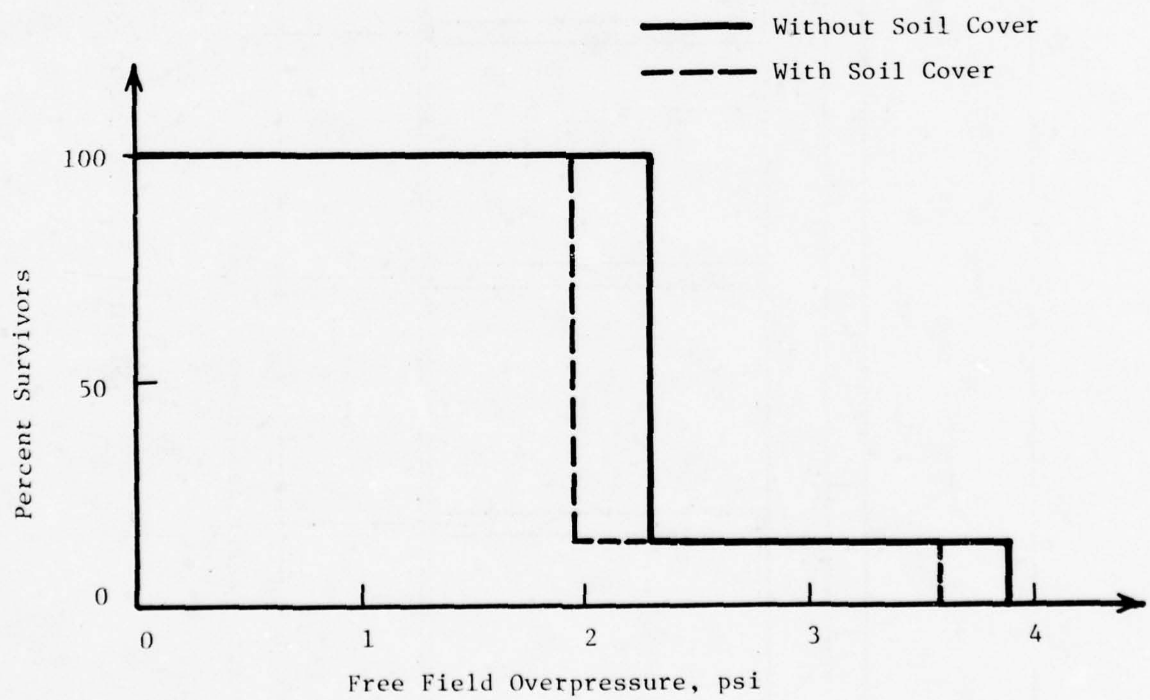
1 ft = 0.3048 m
1 inch = 25.4 mm

* Precast, prestressed concrete unit

Figure 5. Basement Shelter Upgrading



(a) Structure as Built



(b) Upgraded as Shown in Figure 5

Figure 6. Estimates of People Survivability

is first produced in the classroom areas. When this occurs, 84 percent of the shelter floor area is affected by debris and therefore 84 percent of the occupants are casualties. This then is followed by the failure of the corridor overhead floor system, resulting in all occupants being casualties.

In the upgraded structure the sequence of failure is the same. The effect of this upgrading concept is to essentially double the blast protective capabilities of the basement shelter.

Note that if only the corridor area is used without any upgrading of the classrooms or the corridor, the people survivability function becomes a "cookie cutter" type and survivability is extended to 3.89 psi (26.82 kPa). This can be further improved with upgrading.

2.1.3 Analysis of Basement Shelter - The precast/prestressed overhead floor system provides the primary protection to the basement area. With entranceways blocked off and protected, no blast casualties are expected until the overhead floor system fails and/or the entranceway closures fail. When the overhead floor fails (collapses), casualties will be due primarily to debris impact from the failed overhead floor system. When the entranceway closures fail, but the floor system remains, casualties will be due primarily to tumbling impact of individuals produced by dynamic pressures entering shelter areas. The former effects are expected to be the more severe.

This section contains calculations on the resistance of the overhead floor system and entranceway closures against the effects of blast. Estimates of people survivability are made on the basis of these results. Four sheltering cases described in Section 2.1.1 are considered.

2.1.3.1 Members and Material Properties: The structural system consists of unrestrained precast/prestressed, hollow core concrete units. Those over the classroom areas span 32 ft (9.75 m) and are 10 inches (0.25 m) deep. Those over the corridor area span 10 ft (3.05 m) and are 8 inches (0.20 m) deep. Construction details are shown in Figure 3 and Figure 4.

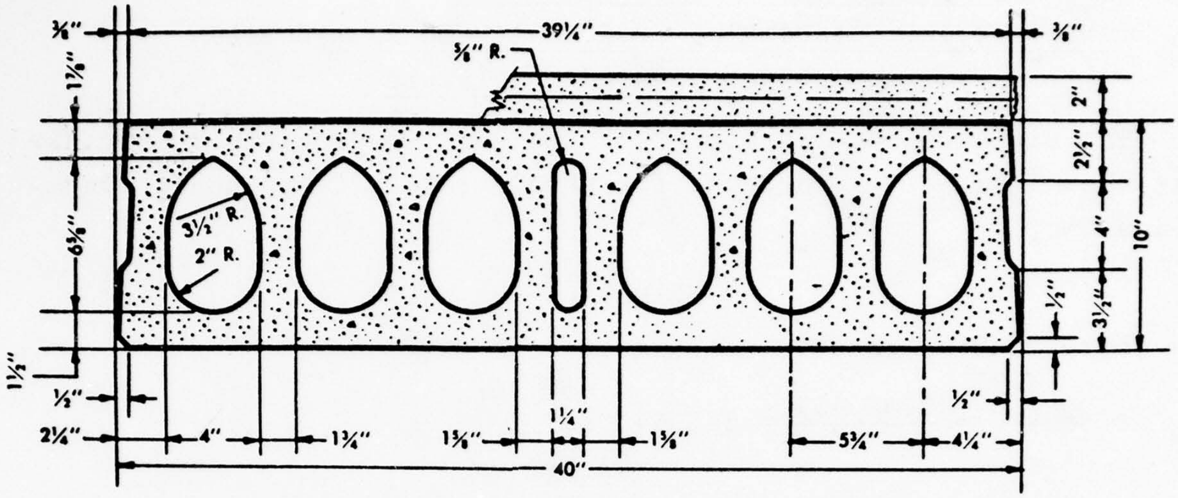
Cross section details of the units are illustrated in Figure 7. Note, although 2-inch (0.05 m) concrete topping is indicated in Figure 7, no topping was used in this building. Some pertinent properties are given in Table 2.

TABLE 2. MEMBER AND MATERIAL PROPERTIES

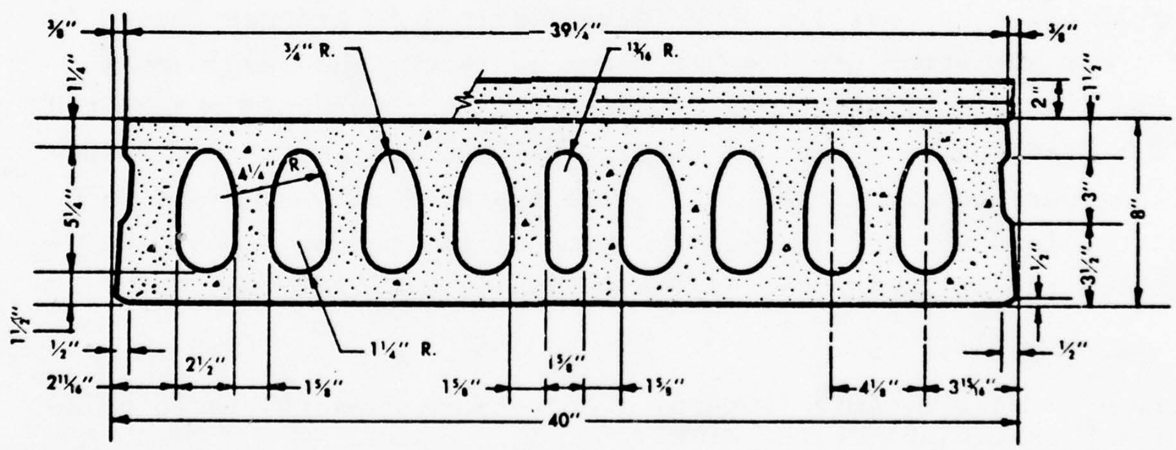
	10 inch Unit (2-10708)	8 inch Unit (2-8506)
Cross-Sectional Area, A	257 $(\text{inch})^2$	218 $(\text{inch})^2$
Moment of Inertia, I	2933 $(\text{inch})^4$	1515 $(\text{inch})^4$
Distance from the Center of Gravity Axis to the Top Fiber, y_t	4.81 inches	4.02 inches
Distance from the Center of Gravity Axis to the Bottom Fiber, y_b	5.19 inches	3.98 inches
Width of Solid Section at the Center of Unit, b	13.30 inches	17.00 inches
Effective Depth, d	7.78 inches	5.84 inches
Weight of Unit, w	75 psf	64 psf
Compressive Strength of Concrete, f'_c	4000 psi	4000 psi
Tensile Strength of Steel, f'_s	250 ksi	250 ksi
Number of Strands	8	6
Strand Cross-Sectional Area	0.108 $(\text{inch})^2$	0.058 $(\text{inch})^2$
Strand Cover	2 inches	2 inches
Ultimate Bending Moment, M_u	33.96 k-ft/ft	10.81 k-ft/ft

1 $(\text{inch})^2$ = 645.16mm^2 1 psi = 6.89 kPa 1 $(\text{inch})^4$ = 416231mm^4
1 inch = 25.4mm 1 k-ft/ft = 4.45 kN·m/m 1 psf = 47.88 Pa

Prestress was taken as 65 percent of ultimate, or 0.65 f'_s = 162.5 ksi (1120.40 MPa). This resulted in an initial prestress force, F_i = 140.4 kips (624.53 kN) for the 10 inch (0.25 m) unit and F_i = 56.55 kips (251.55 kN) for the 8 inch (0.20 m) unit. Prestress loss from all causes was taken at 20 percent. The final prestress forces in the 10 inch (0.25 m) and the 8 inch (0.20 m) units were 112.32 kips (499.62 kN) and 45.24 kips (201.24 kN) respectively. Based on information contained in Reference 9 under dynamic loading the strength of concrete and steel increases on the average by 25 percent. This value was therefore used in subsequent analyses.



(a) 10 inch unit (2-10708)



(b) 8 inch unit (2-8506) 1 inch = 25.4 mm

Note: Although this illustration shows 2 inches of topping concrete, no topping concrete was used in the construction of this building.

Figure 7. Precast Prestressed Concrete Units (SPANCRETE)

2.1.3.2 Analysis of As-Built Structure:

Load Carrying Capacity of 32 ft Span (No Soil Cover)

Load Carrying Capacity Based on Flexure

The ultimate bending moment of the 2-10708 precast, prestressed unit when increased to account for strain-rate effects under dynamic loading is

$$M_o = 1.25(33.96) = 42.45 \text{ kip-ft/ft of width}$$

The corresponding, static uniform load is

$$w_o = \frac{8M_u}{\ell^2} = \frac{8(42.45)}{(32)^2} = 331.64 \text{ psf}$$

$$\text{dead load (d.l.)} = \underline{75. \quad \text{psf}}$$

$$w_o \text{ (net)} = 256.64 \text{ psf} = \underline{1.78 \text{ psi}}$$

w_o (net) is the static, uniform live load carrying capacity of the unit.

For megaton range blast loadings with peak overpressures less than 10 psi, the positive phase duration (t_d) is greater than 1.6 sec. For the structural member being analyzed, the fundamental period (T) is approximately 0.195 sec. With this, $t_d/T > 8.21$ and $t_m/T \sim 0.49$, where t_m is the time to maximum response. For loads of long duration in comparison to the fundamental period and such that the load variation up to the time of maximum response is negligible, the following expression is applicable (Ref. 10)

$$p_m = w_m(1-1/2\mu) \qquad (1)$$

where p_m = peak dynamic flexural overpressure capacity (step pulse loading) of the member

w_m = static resistance of the member (bilinear resistance function)

$\mu = y_m/y_e$ = ductility ratio, i.e., ratio of maximum to yield deflections

Since we are dealing with a prestressed member, the ultimate resistance cannot be much beyond the yield point because high strength prestressing steel does not exhibit a pronounced plastic range.

Collapse is expected soon after the ultimate bending moment is reached. A ductility ratio of $\mu = 1.3$ is therefore assumed.

$$p_m = w_m(1-1/2\mu) = 1.78\ (1-\frac{1}{2.6}) = \underline{1.10\text{ psi}}$$

Load Carrying Capacity Based on Shear

Ultimate shear stress for a prestressed member is governed, according to ACI 318-71, by equation 11-10, or 11-11 and 11-12. Equation 11-10 was chosen as sufficiently adequate for the problem at hand. This equation is given as

$$v_c = 0.6\sqrt{f'_c} + 700\ \frac{V_u d}{M_u} \tag{2}$$

and such that

$$2\sqrt{f'_c} \le v_c \le 5\sqrt{f'_c}\ , \qquad \frac{V_u d}{M_u} \le 1 \tag{3}$$

where

v_c is the nominal permissible shear stress carried by concrete
V_u and M_u are the applied design shear force and design load moment at the cross section in question,
d is the effective depth of the member

For a uniformly loaded, simply supported beam V_u and M_u can be expressed as follows, where w is the uniform load.

$$\begin{aligned} V_u(x) &= \frac{w}{2}\ (\ell-2x) \\ M_u(x) &= \frac{wx}{2}\ (\ell-x) \end{aligned} \tag{4}$$

Substituting these in equation (2) and increasing f'_c by 25 percent to account for strain-rate effects under dynamic loading, the following expression for v_c is obtained.

$$v_c = 0.6\sqrt{1.25(f'_c)} + 700\ \frac{d}{x}\ \frac{(\ell-2x)}{(\ell-x)} \tag{5}$$

This equation is plotted in Figure 8 subject to restrictions stipulated by equation (3).

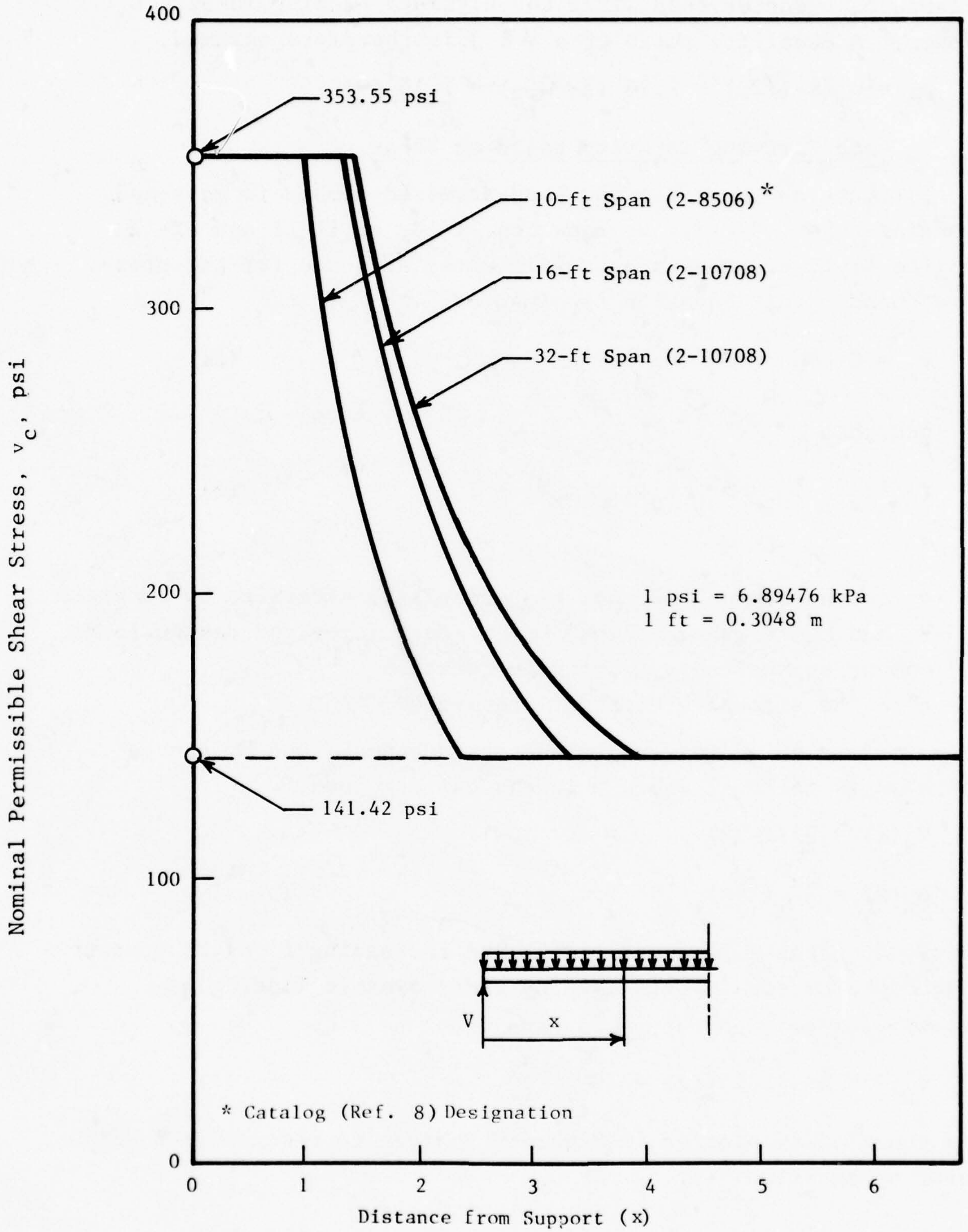


Figure 8. Variation of Nominal Permissible Shear Stress

Since v_c is not uniform over the length of the member, then for a given loading the shear stress needs to be checked at an interior point such as x = 4.0 ft. At this location v_c=141.42 psi.

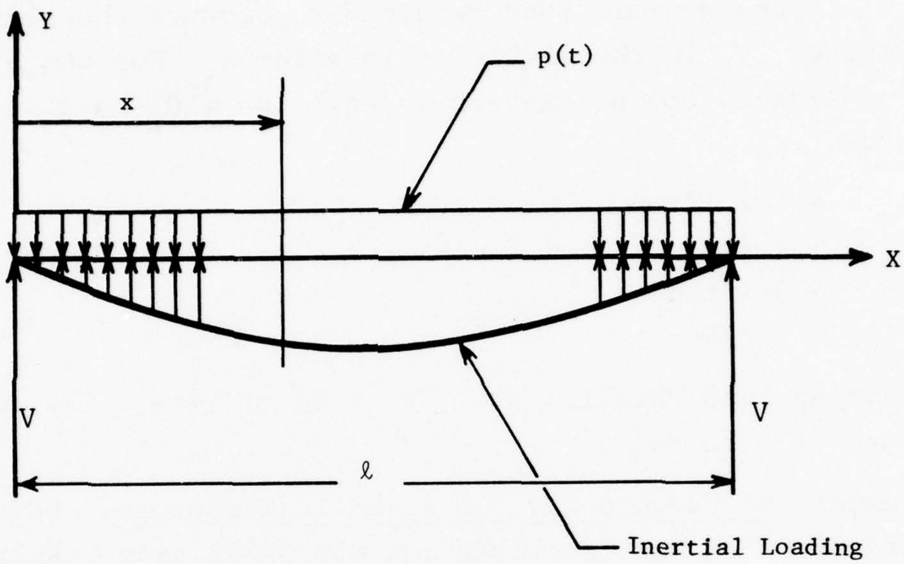


Figure 9. Determination of Dynamic Shear

For the beam loaded as shown in Figure 9 the shear at any position x is

$$V(x) = V - p(t)x + m\ddot{y}(t)c(x) \qquad (6)$$

Assuming a parabolic distribution of inertial force,

$$c(x) = b\left(\frac{x}{\ell}\right)^2\left(\frac{1}{2} - \frac{x}{3\ell}\right) \qquad (7)$$

From Ref. 5. The dynamic end shear V is

$$V = 0.39R + 0.11F + W/2 \qquad (8)$$

where R is the total flexural resistance of the member (kips)
F is the applied dynamic load (kips), and
W is the dead load (kips)

Considering peak response in the elastic range of the member than $F = \frac{R}{2}$ and $\ddot{y} = -\frac{F}{m}$. Substituting into equation (6) we have

$$V(x) = v_c A = 0.89F + W/2 - (F + W)\frac{x}{\ell} - Fc(x) \qquad (9)$$

from which

$$F_s = \frac{v_c A - W\left(\frac{1}{2} - \frac{x}{\ell}\right)}{0.89 - c(x) - \frac{x}{\ell}} \qquad (10)$$

where F_s is the dynamic load required to produce the critical shear stress, v_c in the member at location x. For the structural member analyzed, the parameters in equation (10) have the following values:

v_c = 141.42 psi
x = 4 ft
W = 8.0 kips
c(x) = 11/256

Substituting into equation (10), F_s = 13.07 kips. The corresponding unit load is 0.85 psi.

Summary of Results for the As-Built Structure: Using the same procedure as was described for the 32-ft span, analyses were performed for the following additional cases

- 32 ft span with soil (radiation) cover
- 10 ft span with and without soil cover

The load produced by the soil cover was taken at 100 psf. Results are summarized in the following table.

TABLE 3. SUMMARY OF RESULTS (As-Built Structure)

Span	Flexural Failure Overpressure psi	(kPa)	Shear Failure Overpressure psi	(kPa)
32 ft (9.75 m) (No Soil Cover)	1.10	(7.58)	0.85	(5.86)
32 ft (9.75 m) (With Soil Cover)	0.67	(4.62)	0.49	(3.38)
10 ft (3.05 m) (No Soil Cover)	4.35	(29.99)	3.89	(26.82)
10 ft (3.05 m) (With Soil Cover)	3.92	(27.03)	3.59	(24.75)

2.1.3.3 Design of Upgrading: The shelter was upgraded by providing an intermediate timber support system in the classroom areas halfway between the longitudinal corridor walls and the longitudinal, external basement walls (shown in Figure 10). Since

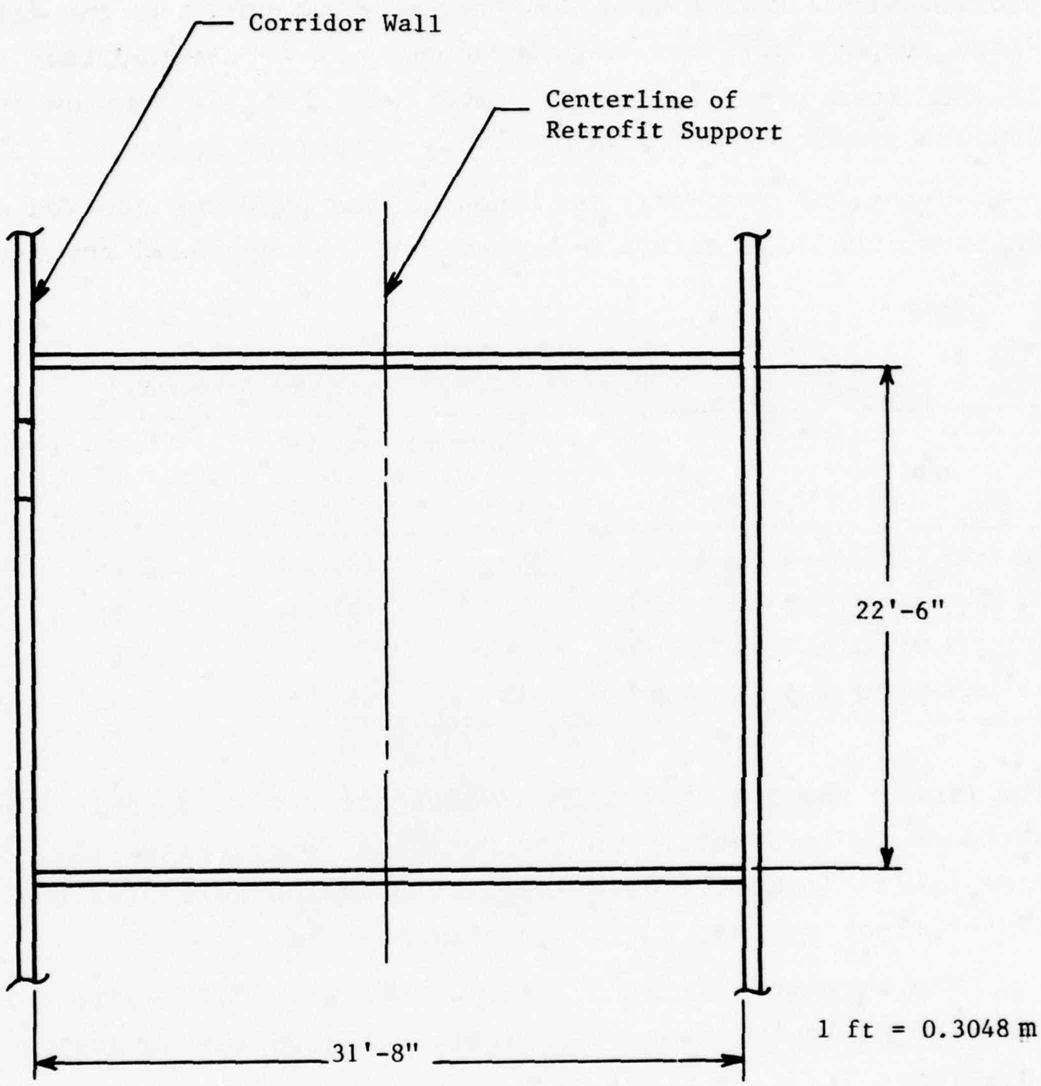


Figure 10. Classroom Plan

the only steel in the member is the prestressing steel located approximately 2 inches from the bottom, continuity at the intermediate support will not be maintained. It is assumed that once the dynamic load is applied, the member will fail in tension over the support producing two equal, simply-supported spans.

Using the procedure outlined in the previous section of this chapter, the load carrying capacity of the upgraded structure is given.

TABLE 4. SUMMARY OF RESULTS (Upgraded Structure)

Span	Flexural Failure Overpressure		Shear Failure Overpressure	
	psi	(kPa)	psi	(kPa)
32 ft(9.75 m)(No Soil Cover)	5.35	(36.89)	2.29	(15.79)
32 ft(9.75 m)(With Soil Cover)	4.92	(33.92)	1.97	(13.58)
10 ft(3.05 m)(No Soil Cover)	4.35	(29.99)	3.89	(26.82)
10 ft(3.05 m)(With Soil Cover)	3.92	(27.03)	3.59	(24.75)

The timber support system is designed to resist dynamic shears when the dynamic load acting on the overhead floor system is 1.97 psi (see Table 4) and the dead load includes the soil plus the weight of the floor system, i.e., 175 psf.

The support system is assumed to consist of Longleaf Pine, with the following properties (Ref. 11) used in the design calculations.

Fiber stress at proportional limit (static bending)	- 9,300 psi
Maximum shearing strength (shear parallel to grain)	- 1,500 psi
Compression parallel to the grain at proportional limit	- 6,150 psi
Modulus of Elasticity	- 1,990,000 psi

These values were increased by 25 percent to account for strain rate effects under dynamic loading.

The design load was computed as

$$V = 0.39R + 0.11F + \text{dead load} \qquad (11)$$

For elastic response R = 2F

$$V = 0.89F + d.1. = 0.89\ (1.97)(40)(16)(12) + 175\left(\frac{40}{12}\right)(8) = 18.132k/unit$$

$$w = 5.44\ k/ft\ of\ support$$

The size of the girder is governed by the maximum shearing strength of the material. Assuming a four-span continuous girder, the required cross section based on horizontal shear is 12 inches by 16 inches. Columns are 6 inches by 6 inches and are selected based on instability. The final retrofit concept was shown in Figure 5.

The entranceway closure is shown in Figure 11. It consists of 6 inch by 6 inch planks supported on three 6-inch by 6 inch stringers. This closure is in turn supported by the stairs on one end and by three 6-inch by 6 inch columns at the other. Additional support for the slab is provided in the back of the stairs. Assuming Longleaf Pine as the upgrading material, the closure is stronger than the basement overhead floor system.

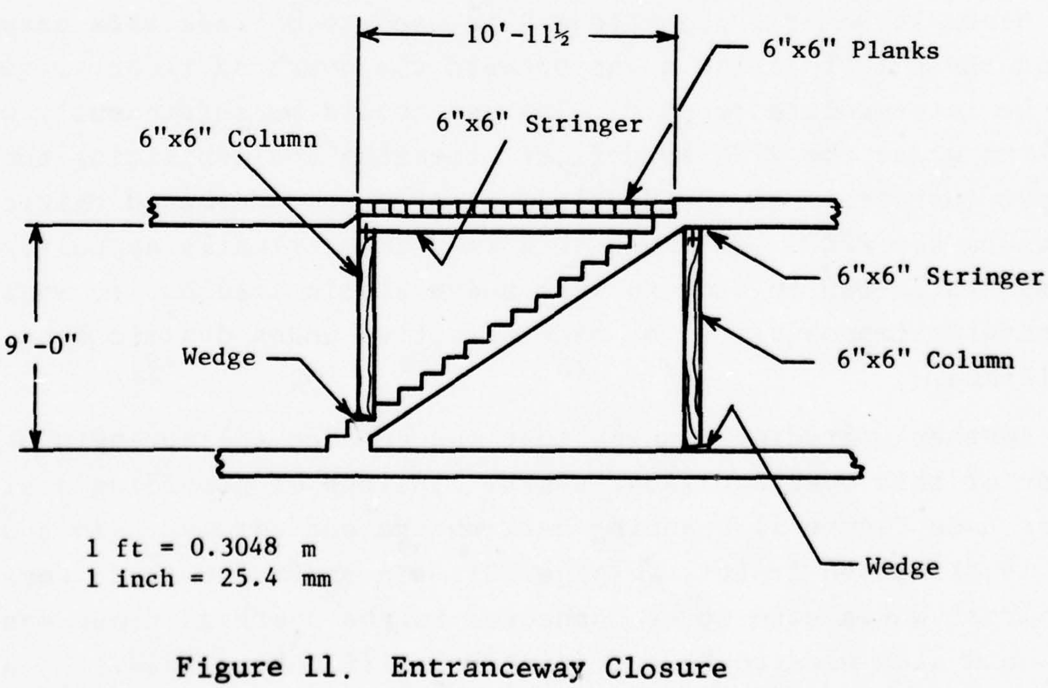


Figure 11. Entranceway Closure

2.1.4 Discussion - This basement shelter was chosen for analysis because precast and precast/prestressed concrete members are entering the existing building inventory. Currently such elements comprise approximately 10 percent of the construction market. Such units are strong, durable, versatile and due to currently high labor costs, they compete favorably with cast-in-place construction. They are found in retailing stores, schools, municipal buildings, office buildings and multistory and multifamily dwellings. It is, therefore, important to know the extent of their protective capabilities.

The upgrading concept considered assumes that the PPCU will rupture directly over the intermediate support over its center, resulting in two simply-supported spans. Since the floor system is 32 ft (9.75 m) wide, the blast wave propagation velocity about 1088 fps (331.62 m/s) and the fundamental period of the unit about 0.20 sec, then the floor system will be loaded essentially instantaneously and will, therefore, respond to a uniformly applied load. Thus the initial rupture producing two spans should occur in close proximity to the center of the intermediate support. This is an important assumption which should be verified experimentally.

Other structural concepts may be used to upgrade this basement. One of these is to allow a gap between the overhead floor system and the intermediate support. The gap should be sufficiently wide so as to allow the PPCU to deflect, engaging and mobilizing the support just prior to the development of a predetermined critical stress in the PPCU. Although this is a theoretically appealing concept which can be made to work under static loading, it would be essentially impossible to be made effective under dynamic load conditions.

Another upgrading concept that may enhance the strength behavior of this overhead floor system consists of providing a single girder (see Figure 5) spanning between the end columns. This concept is discussed in Ref. 12 (page 20). In order for it to work, the girder would need to be connected to the overhead floor system and would also need to be sufficiently stiff not to load it. All of these concepts need testing and evaluation.

2.2 Residential (Single Family) Building

2.2.1 Building Description - The structure considered (see Figure 12 and 13) (Ref. 13) is a two-story clapboard house of conventional wood frame construction with a full basement. Plan dimensions are 24 ft 8 inches (7.52 m) by 33 ft 4 inches (10.16 m). The first floor level is 2 ft (0.61 m) above grade. Openings into the basement include six windows, one exterior door and one interior stairwell opening. Basement windows are in wells with about half of the window exposed above grade. The basement walls are of cinder block construction with common brick masonry at locations where the first floor girders are supported. The basement floor is of concrete and is 4 inches (0.10 m) thick. The ceiling heights of the basement (to the bottom of the joists) is about 7 ft 4 inches (2.24 m).

There is a fireplace on one of the shorter walls which necessitates more detailed framing around it and a stairwell in the center of the house with its special framing.

The first floor is supported by two 6 by 8 (0.15 m by 0.20 m) inch wood girders. These extend the width of the house, continuous over two pipe columns to form three spans of about 8 ft 3 inches (2.51 m) each. Joists are 2 by 8 inches (0.05 m by 0.20 m) spanning 13 ft 4 inches (4.06 m) between basement walls and girder and 6 ft 8 inches (2.03 m) between the two girders. Joists are 16 inches (0.41 m) on centers with diagonal sheathing and finished flooring installed above.

The structural design-analysis considered examines two houses of identical geometry and construction but built with different grades of lumber, both of which are in common use. The stronger wood is longleaf pine and sugar pine is assumed for the cheaper grade.

Reference 14 recommends that the minimum quality to be used for framing and structural purposes in the dry and protected locations usual in buildings should be as follows:

1 ft = 0.3048 m

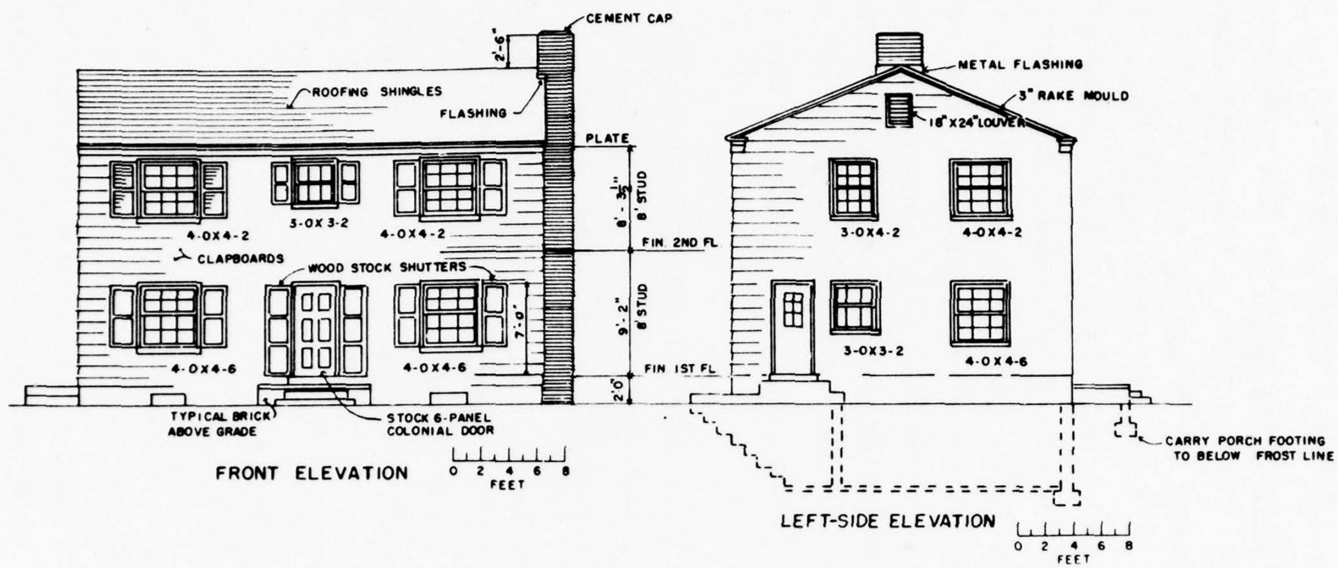
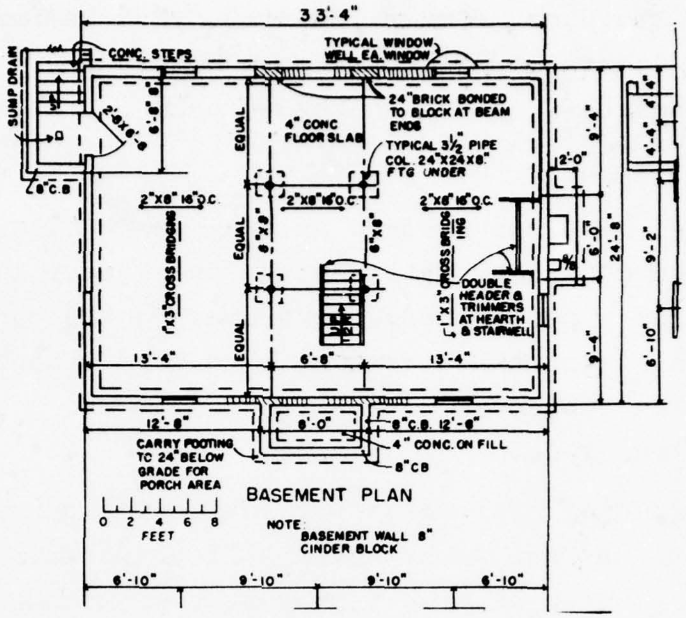
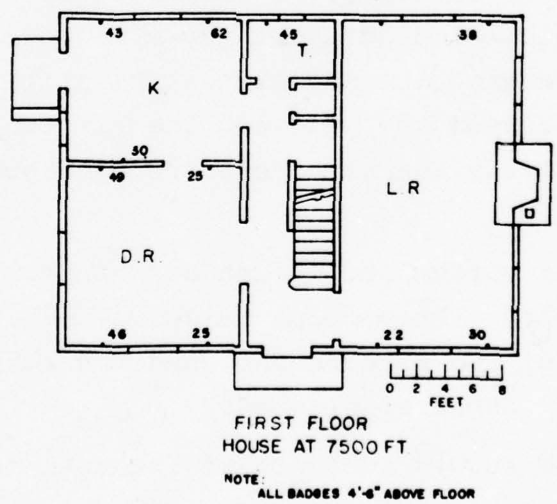


Figure 12. Single Family Residence (Ref. 13)



a) Basement Plan



b) First Floor Plan

Figure 13. Floor Plans

(a) For lumber less than 2 inches (0.05 m) thick and for all studding, Number 1 common, yard lumber

(b) For joists and rafters, common structural

(c) For girders, posts and heavy beams, structural, prime structural or structural square edge and sound, depending upon the species.

Longleaf pine, also referred to as long-leaf yellow pine or L.L.Y.P., is used for girders and posts and wherever heavy loads are to be carried. Sugar pine is graded according to the basic provisions for structural material as common. Ordinary studding, joists and rafters, unless subjected to special loads, are usually taken from Number 2 common grade.

When upgrading residences for sheltering purposes, it is reasonable to assume that lumber used will be what is available. Longleaf pine and sugar pine represent two extremes in material properties and were chosen primarily for this reason.

2.2.2 Upgrading Concepts - Upgrading this basement for sheltering purposes involves increasing the resistance of the overhead floor system and adequately blocking up all openings. The strength of the overhead floor is increased by providing additional supports for the joists which cut their unsupported length in half. This involves placing additional girders parallel to the existing ones and halfway between the wall and the existing girders. Two variations of basically the same retrofit concept are considered (Ref. 15).

The first is a girder and column and girder support system consisting of timber. The concept is shown in Figure 14. In the building considered, two girders are used and each is supported by four columns making three equal spans.

The girder, to supply adequate resistance, must be quite large, however, timbers of larger dimension (e.g., 6 by 8 inches (0.15 m by 0.20 m)) may not be readily available. Therefore, the girders herein are assumed to be built up of smaller dimension lumber. Splices within the cross section for adequate length are staggered and should not be placed over or near a column.

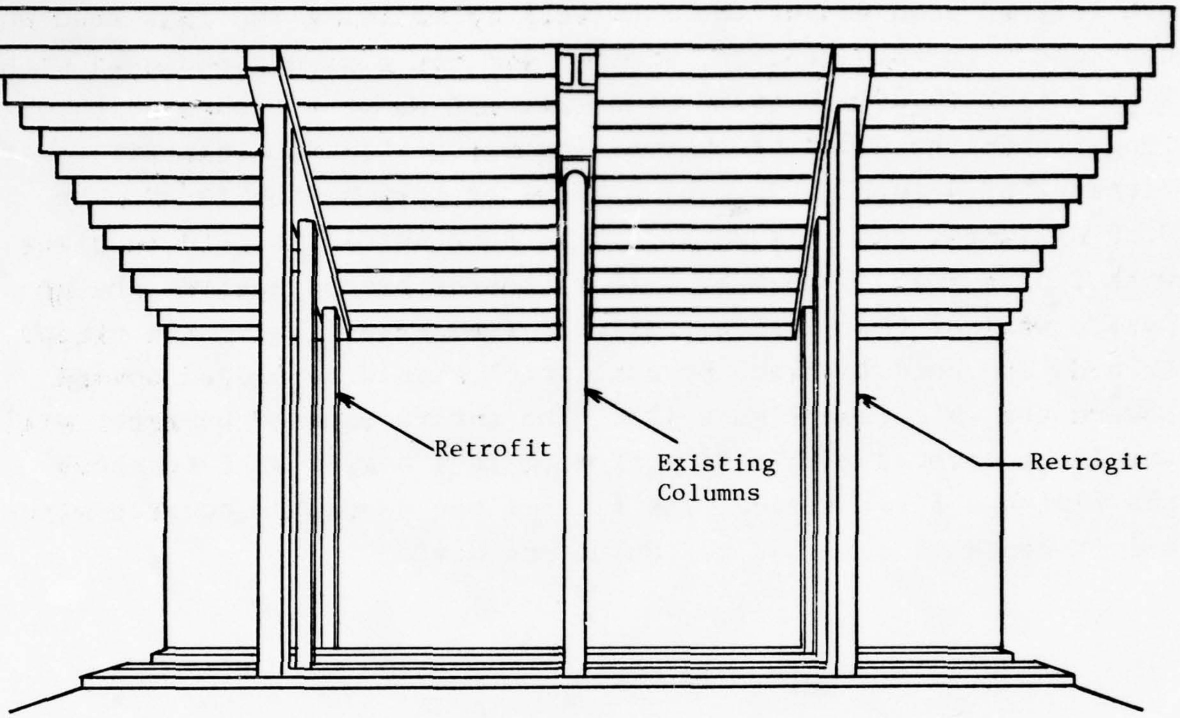


Figure 14. Girder and Column Upgrading Concept

The columns are built up from 2 by 4 inch (0.05 m by 0.10 m) boards and each board should be the full column length with no splices. These built-up cross sections should be nailed together securely to assure their structural action as a unit. Commercial house jacks may be used if available. Each joist should be secured to the girder by nails and/or nailed strapping and the columns should be wedged tightly in place before securing with nailed strapping to the girder. Each column at its base should be secured to the floor or braced in both directions with 1 by 4 inch (0.03 m by 0.10 m) boards butted against the walls and columns and between columns. Another alternative is to install 1 by 4 inch (0.03 m by 0.10 m) boards as sills on the floor and secure columns to the sill.

The second type of joist support system is a stud wall constructed of 2 by 4 inch (0.05 m by 0.10 m) boards. The midpoint of each joist is supported by a single 2 by 4 inch (0.05 m by 0.10 m) stud as seen in Figures 15 and 16. Openings for accessibility are left at each end of the stud wall by omitting the last stud on each end. In this concept, windows and the door are enclosed with 2 by 4 inch (0.05 m by 0.10 m) boards cut about 6 inches (0.15 m) longer than the width of the opening and nailed to cross pieces of either 2 by 4 inch (0.05 m by 0.10 m) or 1 by 4 inch (0.03 m by 0.10 m) lumber (see Figure 17). The door shield is held in place with 2 by 4 inch (0.05 m by 0.10 m) boards braced against the opposite wall of the outside stairwell (see Figure 18). The window shields are held in place by soil which should be sloped upward toward the wall (see Figure 17). The entire exposed basement wall should be covered with this soil berm to a height of 1 ft above the finished first floor. The first floor should be covered with a 1 ft depth of soil for radiation protection.

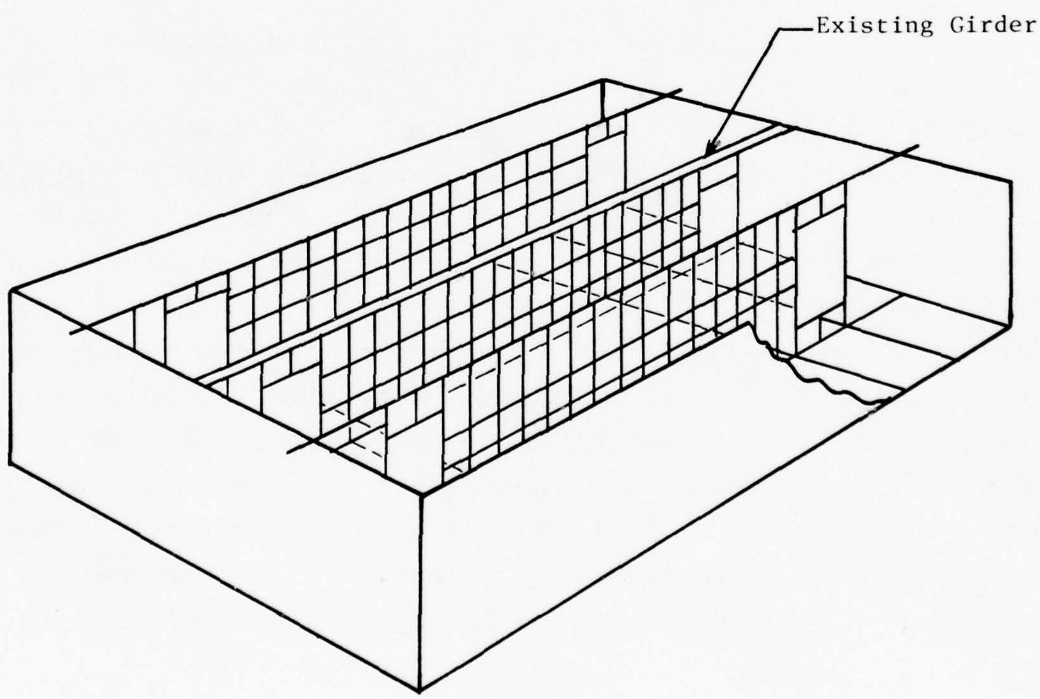


Figure 15. Studwall Upgrading Concept
(Overall View of Basement)

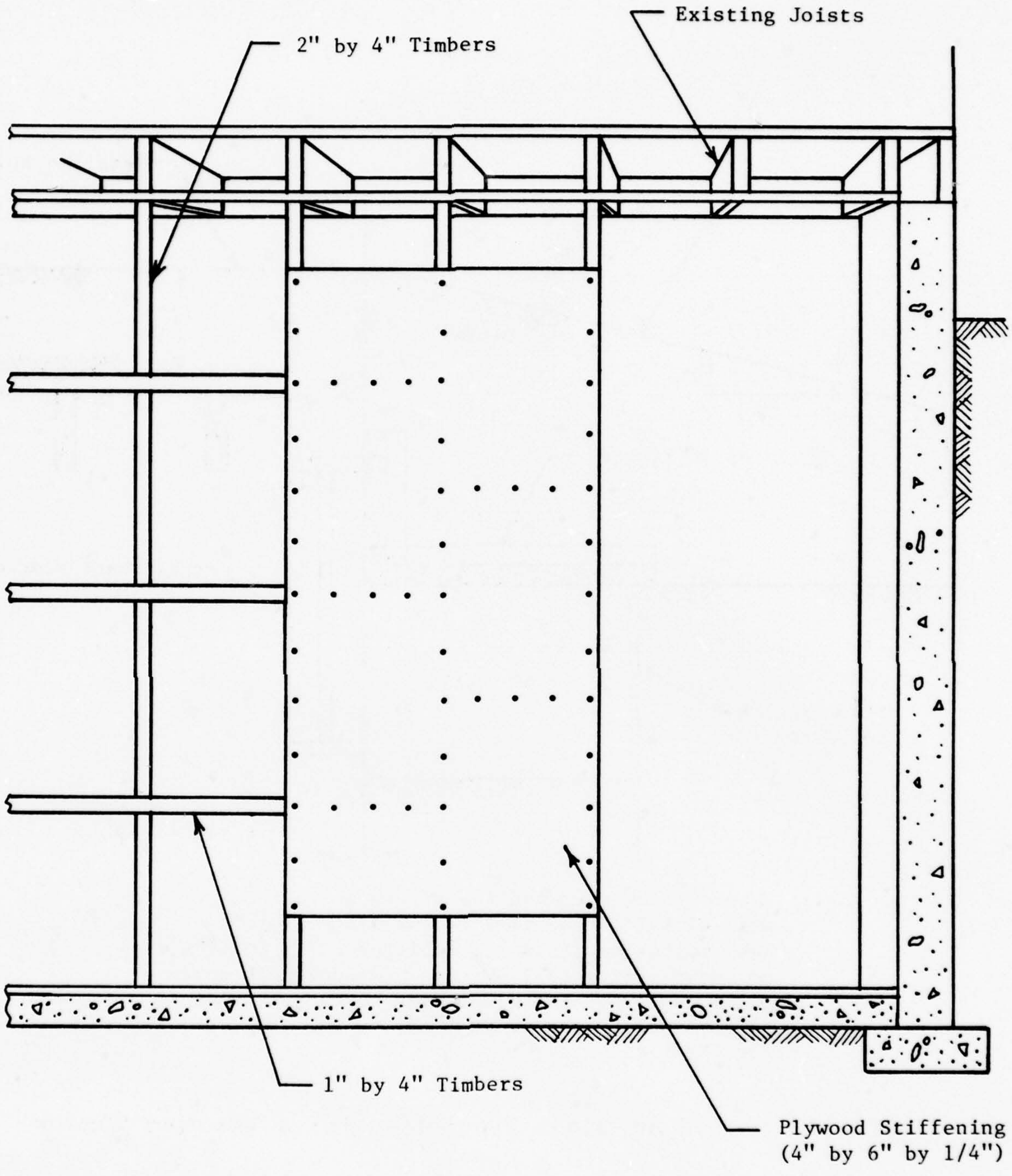
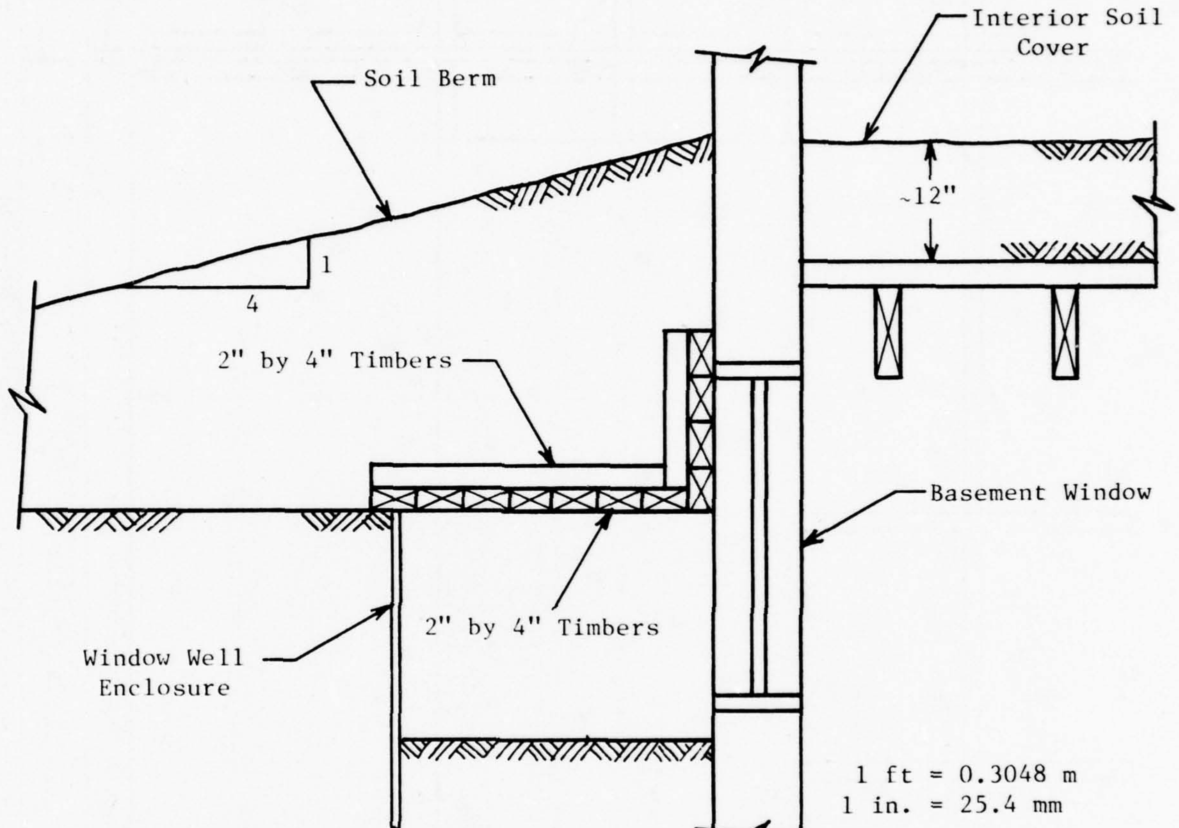


Figure 16. Studwall Upgrading Concept
(End Detail)

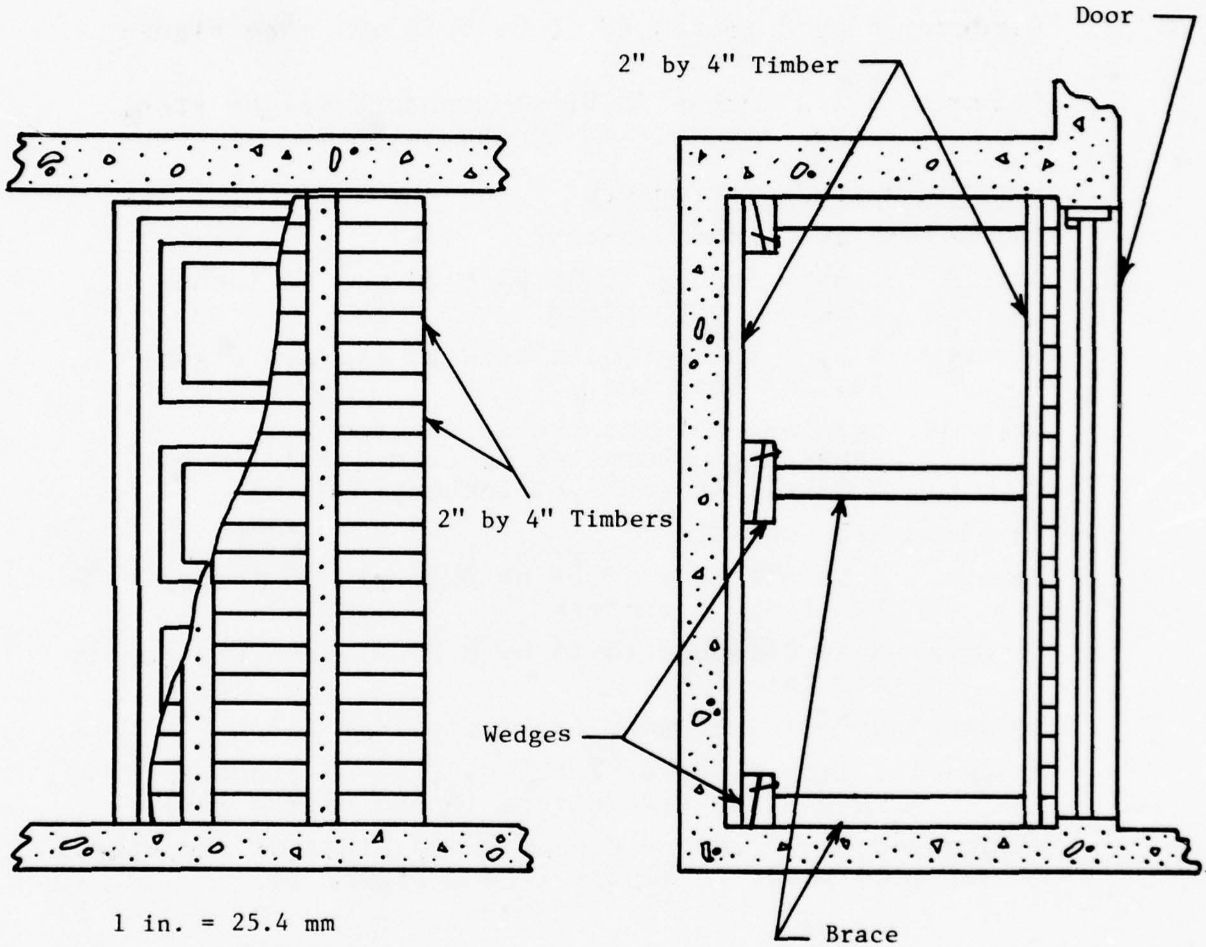


Note: Cover window area and top of window well with 2" by 4" timbers cut 4 ft long. Nail together with shorter pieces of 2" by 4"'s, as shown. Pile soil on top of window well cover and around the entire building to the same level as the soil inside or at least to the level of the first floor.

Figure 17. Expedient Blast Protection for a Basement Window

Cut 2 by 4's to lengths of 6" wider than door.
Nail to two 2 by 4's cut about 3" longer than door height.

Hold door cover in place by bracing 2 by 4's cut to length against opposite wall.



(a) Front View

(b) Side View

Figure 18. Expedient Blast Protection for the Basement Door

Structural and material data are summarized as follows.

Structural Data

1. For the as-built structure

Joists: 2 by 8 inches (0.05 by 0.20 m), 16 inches (0.41 m) on centers

Girders: 6 by 8 inches (0.15 by 0.20 m), see Figure 13a for location

Columns: 3-1/2 inches (0.09 m) standard weight steel pipe, length - 80 inches (2.03 m)

2. For the upgraded structure

(a) Column and Girder Concept

Joists: 2 by 8 inches (0.05 by 0.20 m), 16 inches (0.41 m) on centers

Girders: 6 by 8 inches (0.15 by 0.20 m), see Figure 19a for location

Columns: original columns are as in item 1. upgrading columns: 6 by 6 inches (0.15 by 0.15 m), length - 80 inches (2.03 m)

(b) Studwall Concept

Joists: 2 by 8 inches (0.05 by 0.20 m), 16 inches (0.41 m) on centers

Girders: 6 by 8 inches (0.15 by 0.20 m), see Figure 13a for location

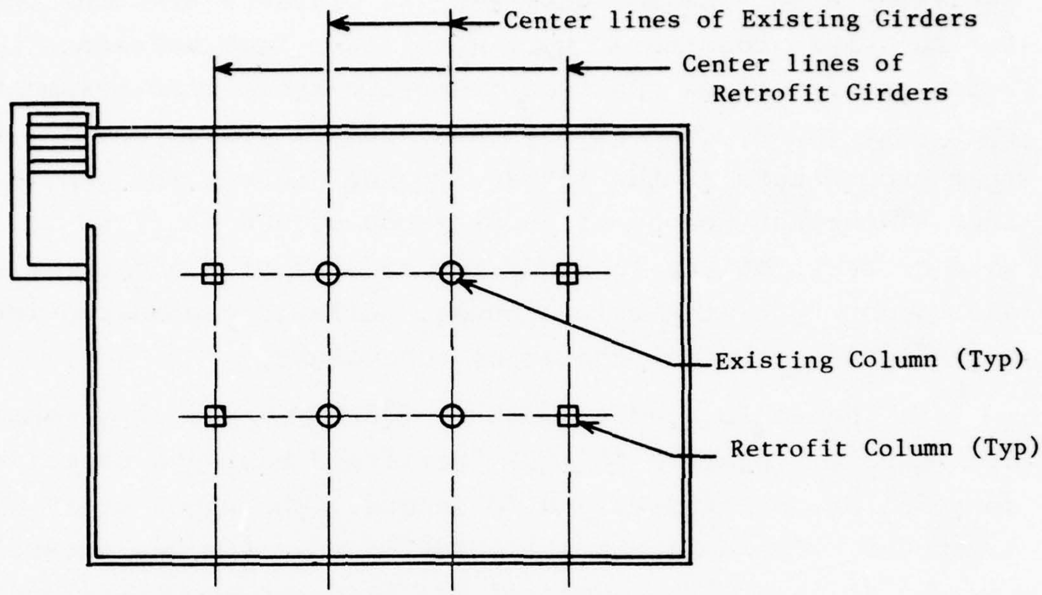
Columns: original columns are as in item 1.

Studwalls: 2 by 4 inches (0.05 by 0.10 m) studs at each joist (see Figure 16 and Figure 19b)

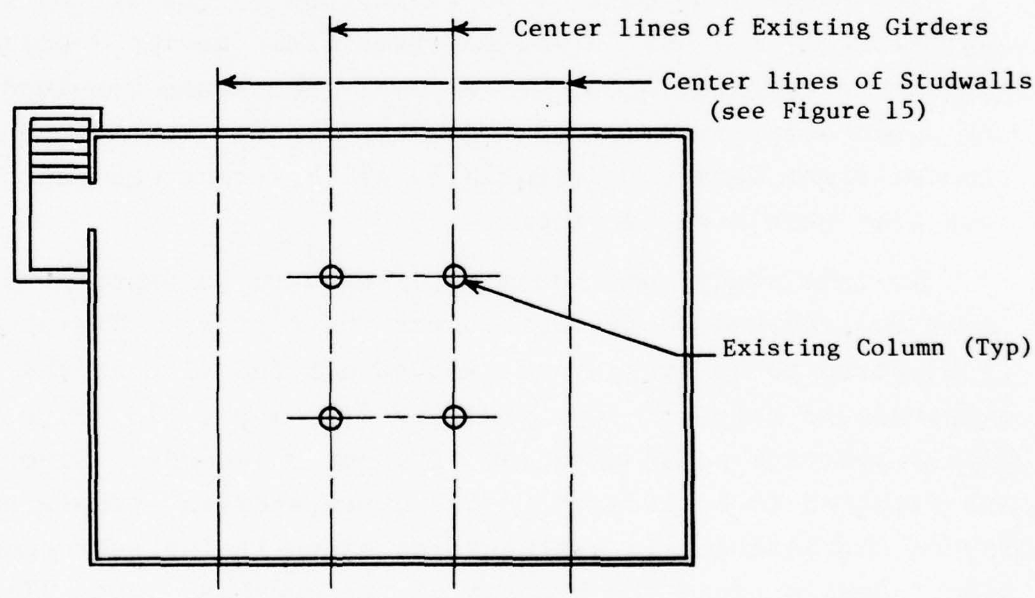
Window and door closure concepts and sizes of structural members are shown in Figure 17 and Figure 18.

TABLE 5. MECHANICAL PROPERTIES OF WOOD

	Longleaf Pine	Sugar Pine
Modulus of Rupture, σ_t, psi, (MPa)	14,700 (101.35)	8,000 (55.16)
Maximum Crushing Strength (Compression Parallel to the Grain), σ_c, psi, (MPa)	8,440 (58.19)	4,770 (32.89)
Maximum Shearing Strength (Shear Parallel to the Grain), τ_m, psi, (MPa)	1,500 (10.34)	1,050 (7.24)
Modulus of Elasticity, E, psi, (MPa)	$1.99(10)^6$(13,720)	$1.2(10)^6$(8274)



(a) Girder and Column Upgrading



(b) Studwall Upgrading

Figure 19. Upgrading Concepts (Plan View)

2.2.3 Response of a Wood Frame House to the Effects of a Nuclear Weapon - As indicated in the previous section, the plans for the house considered here were taken from Reference 13. This reference describes two wood-frame two-story with-basement houses that were constructed at 3500 and 7500 ft (88.90 and 190.50 m) from ground zero at the Nevada Proving Grounds and exposed to a 16.4 KT nuclear weapon at an altitude of 300 ft (7.62 m). The purpose of the test was to study the effects of a nuclear explosion on one common type of American house. Effects considered included gamma radiation, thermal radiation, and blast.

The house located at 3500 ft (88.90 m) was at a range of approximately 5.25 psi (27.58 kPa) free field having a positive phase duration of approximately 0.70 second. The upper story was totally destroyed. The basement overhead floor system was severely damaged, however at least 50 percent of the basement occupants would have been uninjured survivors.

The house located at 7500 ft (190.50 m) was at the range of approximately 1.9 psi (8.96 kPa) free field having a positive phase duration of approximately 0.9 second. The house remained standing. The upper story experienced moderate damage. The basement experienced light damage. Essentially all basement occupants would have been uninjured survivors.

In this study, the house is assumed to be exposed to a megaton-range nuclear weapon detonated near the surface. The upper story is expected to be broken and removed off the site at the free field overpressure range of approximately 2 to 3 psi (13.79 to 20.68 kPa) with a positive phase duration of about 3 seconds. Since no people are expected to be located in the upper stories, people survivability in the basement is evaluated as shown in the subsequent discussion. Debris, from the breakup of the overhead joist floor is considered to be the only relevant casualty mechanism. Probability of survival is estimated using three steps:

1. Determine the collapse overpressure for the joist floor system.
2. Identify basement areas affected by falling debris and estimate debris sizes.

3. Relate debris size and impact velocity to the extent of casualty and estimate the probability of survival.

2.2.4 Collapse Overpressure of the Joist Floor System - The primary load resisting structure is taken to be a framework consisting of joists, girders and columns as illustrated in Figure 20. Joists are assumed to be continuous over the girders and simply-supported at their ends. The influence of sheathing and floor boards is considered to the extent that they help in distributing the load. Structural interaction between the joists and the flooring is considered to be negligible (Ref. 16). The joists are assumed to be braced to the extent sufficient to preclude lateral buckling. Girders are assumed to be continuous over the columns and simply-supported at their ends.

The loading is uniformly distributed over the floor area. Partial failure (collapse) is assumed to be produced when a span (joist or girder) results in a mechanism due to plastic hinge formation or when the shearing strength (see Table 5) is exceeded. Total collapse is assumed to occur when a sufficient number of hinges and/or shear failures are produced to create a mechanism or shear failure in each span. Ultimate plastic bending moments for joists and girders were computed using the expression (Ref. 17,18).

$$M_u = \frac{n\, b\, h^2\, \sigma_c (45n + 64)}{12(3n + 4)^2} \qquad (12)$$

where

σ_c = longitudinal ultimate compressive stress

$n = \frac{\sigma_t}{\sigma_c}$

σ_t = longitudinal ultimate tensile stress

b = width of beam cross section

h = height of beam cross section

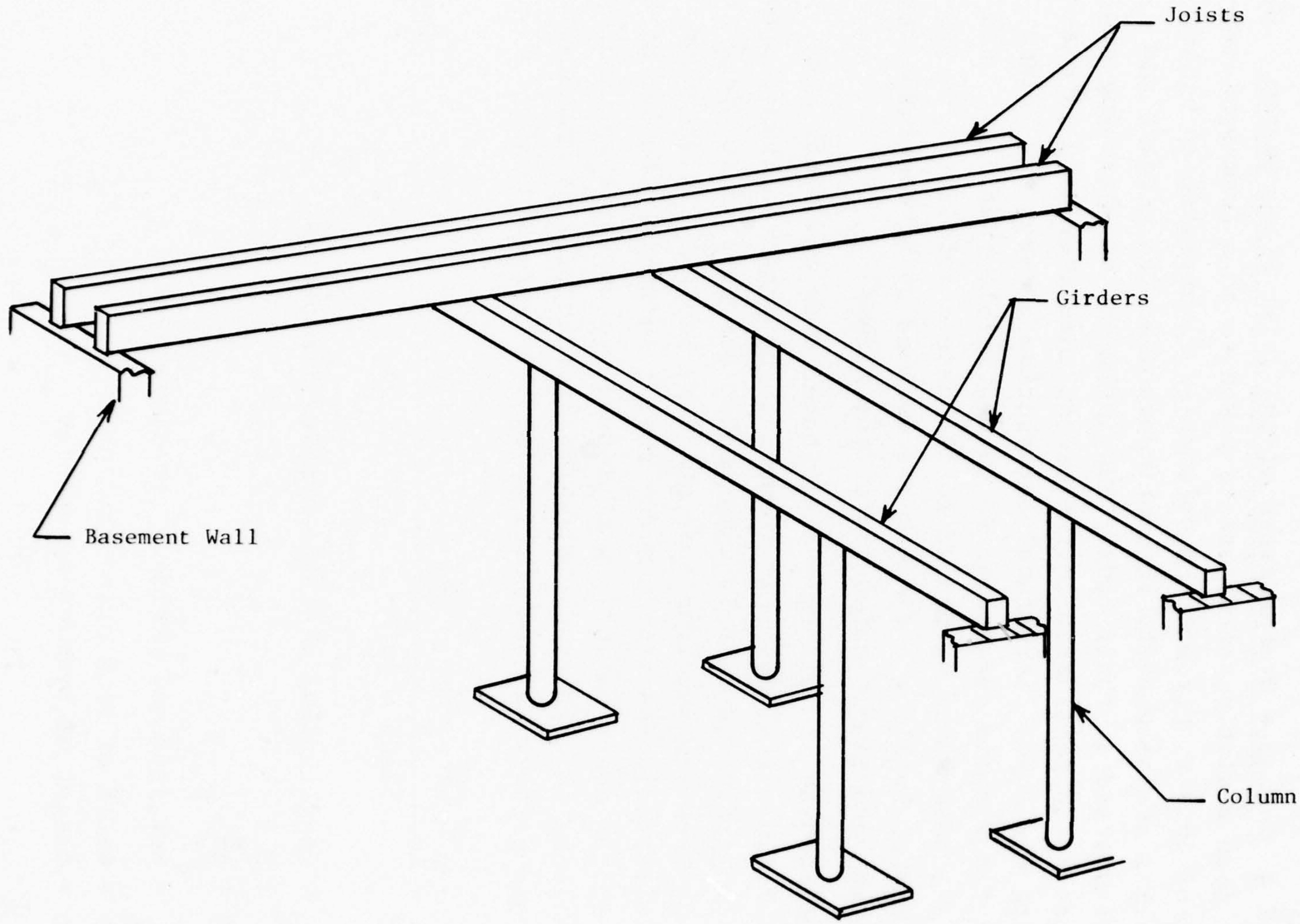


Figure 20. Basement Framing

The ultimate shear in the member of the framework was computed using the expression

$$V_u = \frac{\tau_m g h}{\alpha} \qquad (13)$$

where

τ_m = maximum shearing strength, shear parallel to the grain

α = form factor for the cross section

The basement framework (Figure 20) is statically indeterminate. Standard methods of continuous structures (Ref. 19) were used to determine the static stress distribution in the elastic range. Static collapse loads were estimated using procedures given in Reference 20. The corresponding dynamic load was determined using equation (1) with a ductility factor of 10.

2.2.4.1 As-Built Structure: Collapse overpressures for the as-built wood joist floor system are summarized in Table 6 with reference to Figure 21.

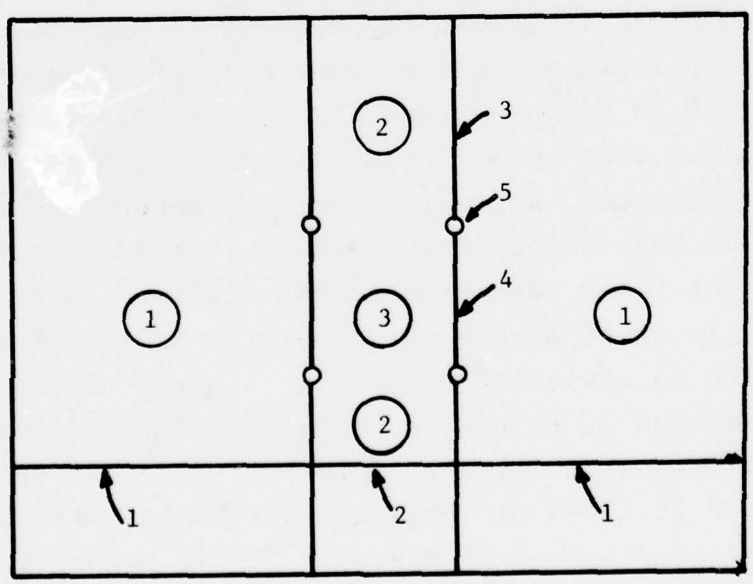


Figure 21. Basement Floor Plan, As Built-Structure

This figure identifies the primary structural members and basement areas (circled numbers) which are different due differences in the strength of corresponding portions of the overhead floor system. The collapse (instability) overpressure for the steel columns (item 5, Figure 21) is estimated at 26.0 psi (179.26 kPa).

TABLE 6. PREDICTED COLLAPSE OVERPRESSURES, AS-BUILT STRUCTURE

	Component		Flexure psi (kPa)	Shear psi (kPa)
Longleaf Pine	1	Joist, long span	5.01 (34.54)	7.88 (54.33)
	2	Joist, short span	28.91 (199.33)	7.88 (54.33)
	3	Girder, external span	6.52 (44.95)	5.83 (40.20)
	4	Girder, internal span	9.07 (62.54)	5.83 (40.20)
Sugar Pine	1	Joist, long span	2.21 (15.24)	5.42 (37.37
	2	Joist, short span	13.52 (93.22)	5.42 (37.37)
	3	Girder, external span	2.91 (20.06)	3.98 (27.44)
	4	Girder, internal span	4.12 (28.41)	3.98 (27.44)

Referring to Table 6 and considering sugar pine as the building material then partial failure is produced at 2.21 psi (15.24 kPa). Long span joists collapse affecting 18 percent of floor area with lethal debris, (all in region 1 (see Figure 21)). At 2.91 psi (20.06 kPa) both the long span joists and the external spans of girders collapse affecting approximately 71.6 percent of floor area. At 4.12 psi (28.41 kPa) the entire system collapses affecting 87 percent of basement floor area with lethal debris. Based on this reasoning, an estimate of people survivability is illustrated in Figure 22 for both construction materials. Lethal debris is that which weighs more than 10 pounds (4.54 kg) and impacts with a velocity greater than 20 fps (6.1 m/sec) (see Ref. 5). A free fall from a height of 8 ft (2.44 m) (basement height) results in a terminal velocity of 22.7 fps (6.92 m/sec). Debris velocities under blast conditions producing failure are expected to be higher. Debris sizes were estimated based on the results of Reference 13.

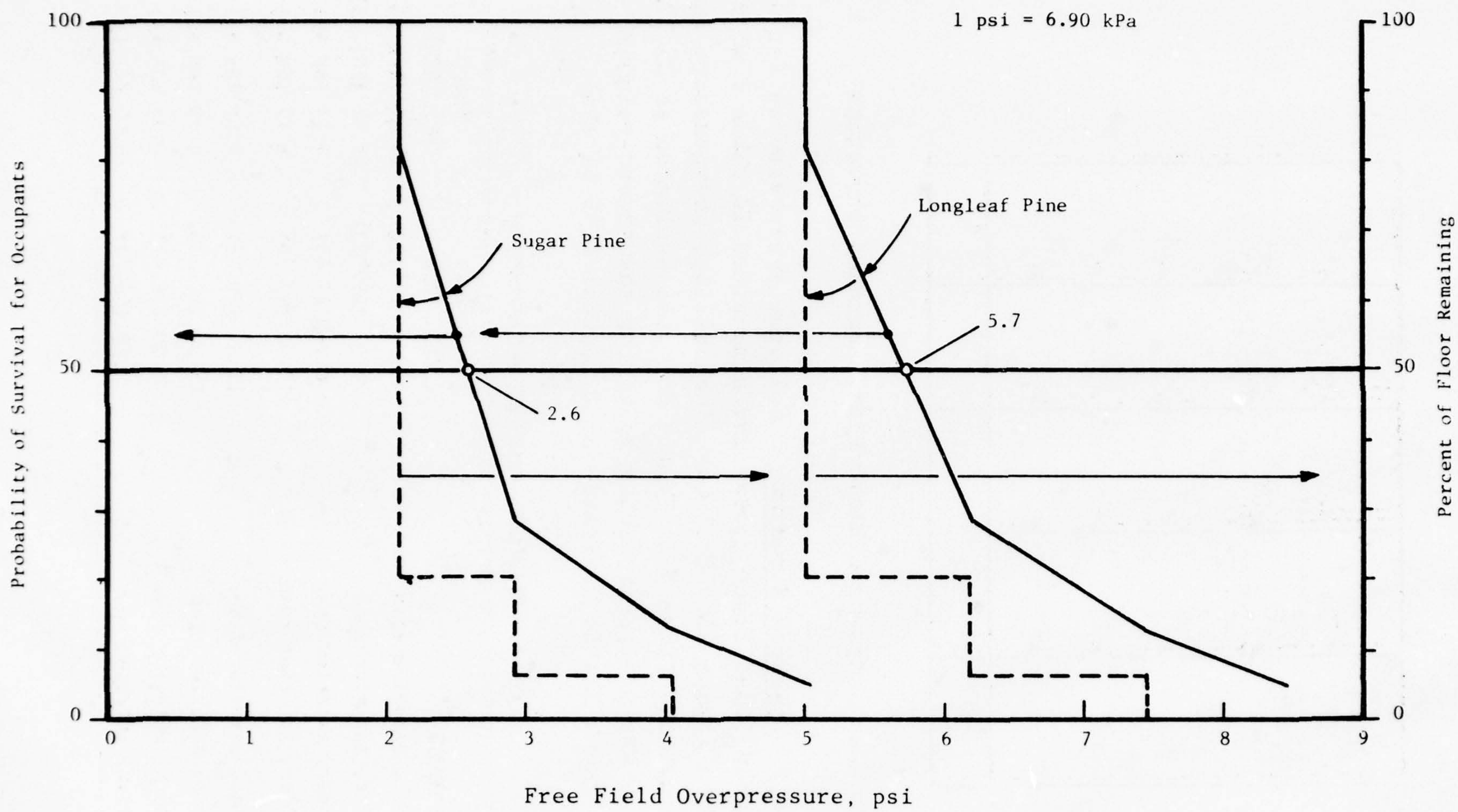


Figure 22. Survivability Estimate for the As-Built Structure

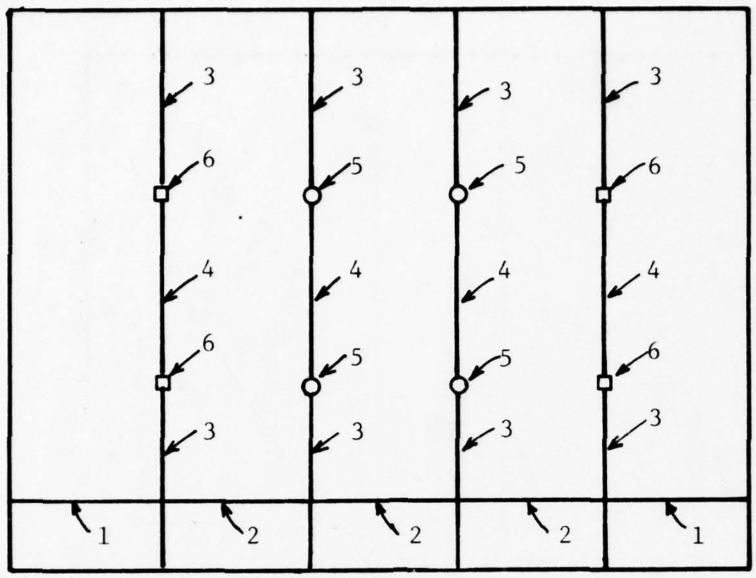


Figure 23. Basement Floor Plan, Upgraded Structure

2.2.4.2 Upgraded Structure: Collapse overpressures for the upgraded wood joist floor system are summarized in Table 7 with reference to Figure 23. Collapse (instability) overpressure for the steel columns (item 5 in Figure 23) were estimated at 25.70 psi (177.20 kPa). Corresponding value for the timber columns (item 6, Figure 23) is estimated at 27.20 psi (189.61 kPa).

TABLE 7. PREDICTED COLLAPSE OVERPRESSURE

		Component	Flexure psi (kPa)	Shear psi (kPa)
Longleaf Pine	1	Joist, exterior span	20.99 (144.72)	15.41 (106.25)
	2	Joist, interior span	28.91 (199.33)	15.41 (106.25)
	3	Girder, exterior span	9.78 (67.43)	8.77 (60.47)
	4	Girder, interior span	13.54 (93.36)	8.77 (60.47)
Sugar Pine	1	Joist, exterior span	11.51 (79.36)	9.69 (66.81)
	2	Joist, interior span	15.92 (109.76)	9.69 (66.81)
	3	Girder, exterior span	5.29 (36.47)	6.04 (41.64)
	4	Girder, interior span	7.38 (50.88)	6.04 (41.64)

Using these results and the reasoning described in connection with the as-built structure a people survivability estimate was made and is presented in Figure 24 for both construction materials.

2.2.5 Discussion of Results - Wood joist floor systems are constructed of joists with large differences in strength and stiffness characteristics. To overcome this difficulty in the survivability analysis it was decided to bracket the results within the limits of low and high strength. It was for this reason that longleaf pine and sugar pine were chosen as the construction materials, with longleaf pine representing the upper bound and sugar pine the lower bound on strength.

Results obtained are considered to be within the state of the art. However, they should be verified by means of experimental techniques. At the present time there is virtually no usable information on failure (collapse) loads of structural lumber either for static or dynamic loads. This has imposed a significant limitation on this study effort.

2.3 Special Purpose Shelters

Over the past two decades a number of different special purpose shelter concepts have appeared in the civil defense literature. Since such concepts represent potential sheltering options, several representative types are presented here with the object of investigating and categorizing their protective capabilities when used to protect individuals in a direct effects nuclear weapon environment. Eight shelters were analyzed in previous studies with results included in References 21 and 22. These results were reviewed and substantially updated in the course of this effort. Shelters considered are identified as:

A. Basement Concrete Block Shelter (Ref. 24)
B. Lean-to Shelter (Ref. 25)
C. Rigid Frame Shelter (Ref. 25)
D. Reinforced Concrete Block Shelter (Ref. 25)
E. Aboveground A-Frame Shelter (Ref. 26)
F. Plywood Box Shelter (Ref. 27)

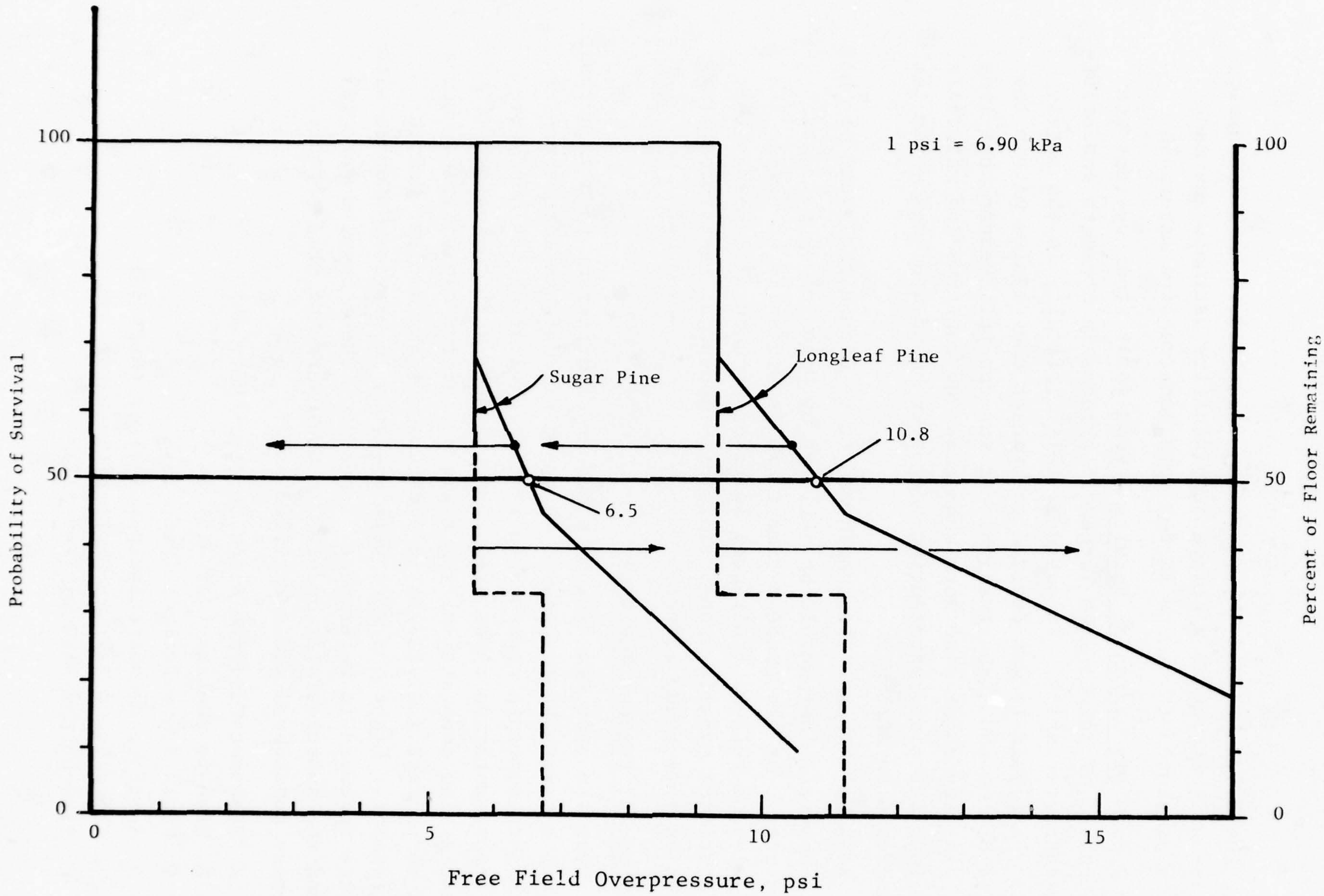


Figure 24. People Survivability Estimate for the Upgraded Structure

G. Wood Grate Roof Shelter (Ref. 28)

H. Gable Roof Shelter (Ref. 29)

Each was designed in detail and plans (engineering drawings or sketches) are available in the respective references. Some, such as the wood grate roof shelter were constructed with the object of determining time, labor and equipment necessary for construction. One shelter similar to the rigid frame shelter was constructed in the basement of a two-story wood-frame house tested in Nevada (Ref. 13).

A simplified structural analysis was performed on each shelter assuming that weapon effects were produced by a megaton-range weapon exploded near the ground surface. The shelters would, therefore, be located in the Mach region of the weapon. Since each shelter is either located in a basement or is closed and mounded, thermal radiation is not a significant weapon effect. Also, since overpressures of interest are less than 22 psi (151.68 kPa), then prompt nuclear radiation is also not considered to be a significant weapon effect. These two effects were neglected. Effects considered include diffraction and drag loading produced by the blastwave on the shelter structure and dynamic pressure and debris (from the breakup of the structure) on the shelter occupants.

In analyzing the shelters, the wood in each case was assumed to be a pine whose mechanical properties are an average between longleaf pine and sugar pine (see Table 5). Results of the analyses are summarized in Figure 25 and are discussed in Section 2.3.1. Basic characteristics of these shelters are illustrated in Figures 26 through Figure 35 which are included at the end of section 2.3.2.2. The shelters are described in terms of framing systems, material properties, and blast resistance.

2.3.1 <u>Discussion of Results</u> - The degree of protection provided by each of the eight shelters against the effects of blast is indicated in Figure 25. It reflects the magnitude of the design load and/or the choice of the structural system.

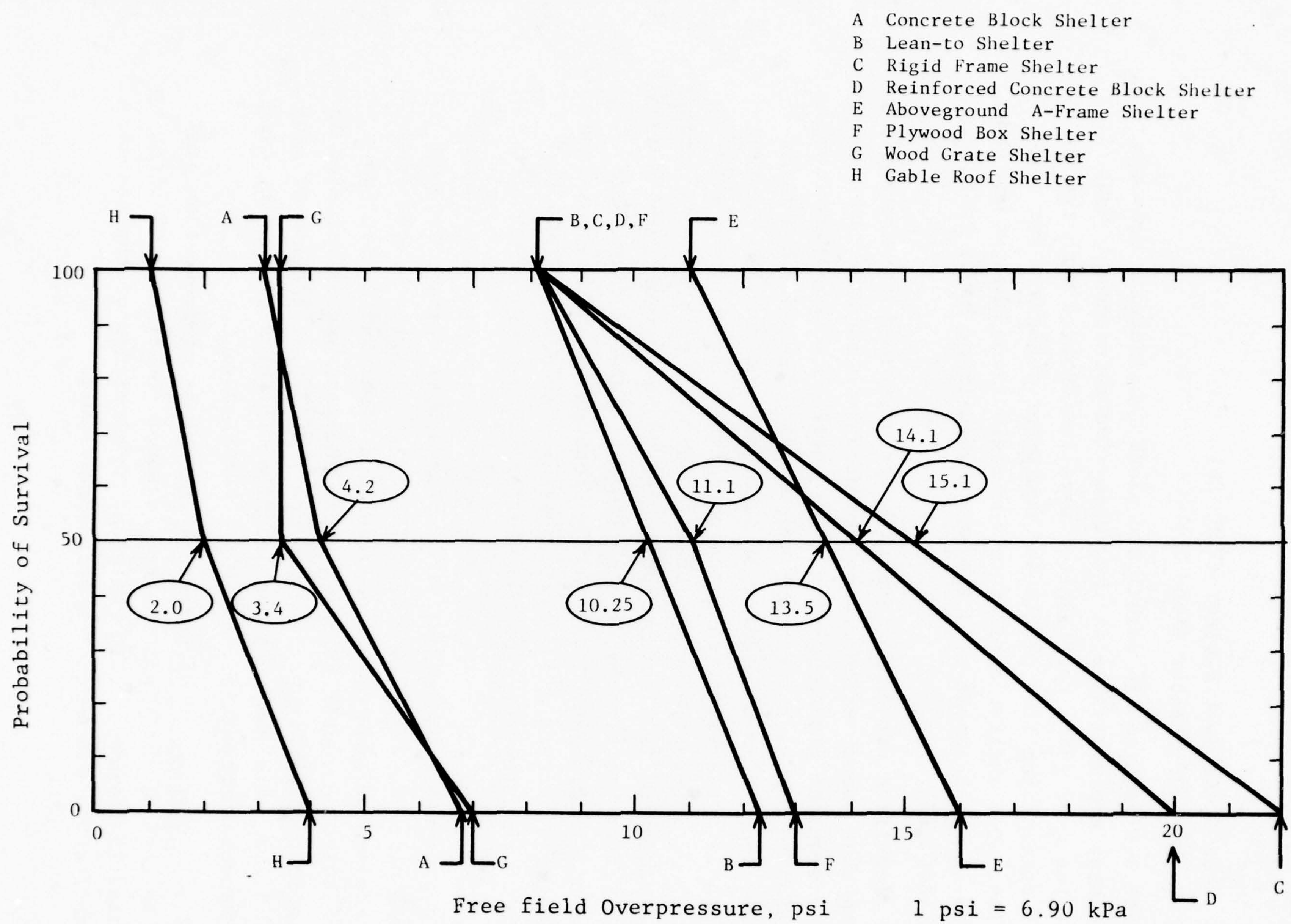


Figure 25. People Survivability Estimates, Special Purpose Shelters

These results clearly indicate that simple family type shelters can be constructed which will provide a substantial (greater than 20 psi (137.90 kPa) amount of blast protection.

Shelters B, C and D were designed to resist an equivalent static load of 10 psi (68.95 kPa). As indicated in Figure 25, these shelters are capable of resisting at least this much and mostly more. Shelters E and F were designed primarily for fallout radiation protection. However, due to their framing systems and member sizes they both possess a significant level of blast protection. Shelters A, G and H represent poorer choices in member sizes, spacing of members and connections for blast resistance, but are certainly adequate for their intended function, i.e., fallout radiation protection.

There is no question that each of these shelters can be strengthened to provide substantially more blast resistance than they currently possess. In fact, this can be done without any great difficulty. For example, consider the basement lean-to shelter (shelter B). By simply doubling up on the stringers, i.e., two stringers where there is one now, doubling the thickness of the plywood and increasing the size of connections, raises the MLOP from 10.25 psi (70.67 kPa) to about 20.50 psi (139.62 kPa). This modification is an obvious one and not necessarily optimum. However, even if it is accepted, it does not call for anything out of the ordinary as far as materials are concerned or the ability to construct the shelter with a great deal of confidence in its performance. Materials considered here are readily available to the general homeowner and the construction procedure is believed not to be beyond the capabilities of the homeowner as well.

These shelters demonstrate that expedient family blast resistant shelters can be designed and constructed to resist fairly high overpressure levels, in excess of 20 psi (137.90 kPa).

Special purpose family shelters can be built in basements or outside. Basements, where they exist, offer certain advantages. First, there is no need to dig a trench since the basement is generally below grade.

A shelter can be constructed ahead of time and left alone or put to dual use until needed. Maintenance requirements are minimal when compared to outside shelters in temperate climates.

Basements of wood frame structures are mostly unreinforced. At high overpressure levels, basement walls may need to be braced to assure the survival of the shelter. Bracing the basement walls can be done in several different ways. One useful way would be to develop a bracing system which also serves to provide resistance to the overhead floor. This would aid in the survival of the basement longer and would preclude some of the debris from falling into the basement at overpressures when buildings in the area are destroyed. A building wall framed with 2 by 4 inch (0.05 m) by 0.10 m) studs at 16 inches (0.41 m) on center, sheathed with 1/2 inch (12.7 mm) plywood, will resist a peak overpressure of 0.34 psi (2.34 kPa). The floor system will resist 3.46 psi (23.86 kPa) Reference 25. Thus it is not likely that debris from the house demolished by the blast will fall into the basement, especially for megaton range weapons. It is more likely that it will be blown off site and become distributed over a large area. Some of this debris is likely to be deposited in or on basements located in their paths.

Basements whose overhead floors are not at grade but instead are several feet (2 to 3 ft (0.61 to 0.91 m)) abovegrade are likely to lose the floor at about the same time the house is destroyed because the exposed basement walls are weak. However, it should not be assumed that significant quantities of debris from the house will be deposited in the basement.

2.3.2 Framing Systems, Materials, and Blast Resistance -

2.3.2.1 Basement Shelters: Shelters A, B, C and D are intended to be used in basements of single-family residences. Shelter A is illustrated in Figure 26. Its primary intent is to provide protection against the effects of fallout radiation. According to Reference 24, this shelter is capable of being constructed within 20 man-hours or less. It provides 52 sq ft (4.83 sq m) of floor area and 260 cu ft (7.36 cu m) of space. At 65 cu ft (1.84 cu m) per person it is capable of accommodating four persons.

Since the masonry is unreinforced, the strength of shelter walls is on the order of 2 to 3 psi (13.79 to 20.68 kPa) and therefore, this shelter will fail as the overhead floor of the building fails. Its people survivability estimate is shown in Figure 25 and is approximately the average of the two results given in Figure 22 for a basement without a shelter. Therefore, this shelter provides virtually no blast protection over that provided by the conventional floor system over the basement. If the building survives, this shelter will provide fallout radiation protection which otherwise is minimal. If the building collapses, having such a shelter and being in it may be more hazardous than having no shelter at all.

If this shelter is modified by replacing the masonry walls with wooden walls consisting of 2 by 4 inch (0.05 by 0.10 m) timbers side by side, then the 50 percent probability of survival would be approximately at 20 psi (137.90 kPa).

<u>Shelter B</u> is illustrated in Figure 27. This is a lean-to shelter with a framing system consisting of stringers and plywood sheeting. A single module (see Figure 27) is a 4 by 8 ft (1.22 by 2.44 m) unit. This module, set against a wall at a 45 degree angle will result in an inside clear dimension of 5 ft 8 inches (1.73 m). The floor area, using a single module, is 23.33 sq ft (2.17 sq m) and the volume 68.06 cu ft (1.93 cu m). The addition of the entrance module provides additional volume. The 4 ft (1.22 m) modules are required to provide sufficient space for three persons.

The module would be constructed as a continuous unit. It was designed, as a plywood stressed-skin panel to resist a static load of 10 psi (68.95 kPa). It is capable of resisting approximately 20 psi (137.90 kPa) in static bending, but is limited by the shear capacity of the plywood.

The door unit to be used in conjunction with the shelter module is shown in Figure 27. Its size is such as to accommodate an opening 18 inches (0.46 m) wide and 30 inches (0.76 m) high. A door constructed of 3/4 inch (19.05 mm) plywood and reinforced with an adequate number of 2 by 4 inch (0.05 by 0.10 m) members is assumed to be provided.

Based on the structural analysis performed, the following results are reported. Exterior plywood sheeting is estimated to fracture at 8.20 psi (56.54 kPa). The stringers are estimated to yield at 7.30 psi (50.33 kPa) and collapse at 12.30 psi (84.81 kPa). Although the connections will fail, their failure is not expected to appreciably enhance shelter collapse since the lean-to is pushed into the corner by the blast loading. For this reason the resistance of connections was not evaluated.

People survivability estimate is shown in Figure 25 and is based on the following reasoning. Up to about 8.20 psi (56.54 kPa) no significant structural damage is experienced and therefore, all shelter occupants are uninjured survivors. At about 8.20 psi (56.54 kPa), the outer sheet of plywood fractures between the stringers, causing the lower (interior) sheet to be pushed into the shelter area. Some of the energy is absorbed by the yielding of the stringers. Casualties are produced by the interior sheet of plywood and the sandbags, driven into the shelter due to the fracture of the outer sheet. The intensity of this casualty mechanism increases with increasing overpressure. At about 12.3 psi (84.81 kPa) the entire module is expected to collapse resulting in essentially no survivors. Casualties are due to the interaction of shelterees with failed or failing structural components.

The ability of this shelter to provide protection against the effects of blast in a closed condition can be substantially improved with little difficulty. To accomplish this would require redesigning the connectors, increasing the thickness of the plywood and the size of the stringers.

Shelter C is illustrated in Figure 28. The design incorporates the use of prefabricated rigid frames for the main structural members. The main elements of the shelter are designed as 4 ft (1.22 m) modules, similar to the lean-to shelter, so that standard 4 ft (1.22 m) sheets of plywood can be used. A single module results in 20 sq ft (1.86 sq m) of floor area and 80 cu ft (2.27 cu m) of volume. Incorporating the entrance module provides additional volume. Two such modules, plus the entranceway module would be required to provide sufficient space for three persons.

Structurally, the shelter consists of 2 by 12 inch (0.05 by 0.30 m) roof and wall members connected by 1/2 inch (12.7 mm) plywood gussets, nail-glued at the joint between wall and roof to form a rigid frame. The frames are spaced approximately 16 inches (0.41 m) on center and faced on both sides with 3/4 inch (19.05 mm) plywood. Both wall and roof elements were designed to withstand 10 psi (68.95 kPa) static overpressure.

As the rest of the shelter, the end wall is made up of 2 by 12 inch (0.05 by 0.30 m) vertical members with 3/4 inch (19.05 mm) plywood sheets and sandbag fillers. The 2 by 12 inch (0.05 by 0.30 m) members are connected to the floor bearing plate and to the roof section of the entrance module frame. The sheet metal connectors used elsewhere in the shelter are also used at these connections. The entrance unit is of similar construction to the one used in the lean-to shelter.

The exterior plywood sheeting is estimated to fracture at about 8.20 psi (56.54 kPa). Roof stringers and wall stringers have failure overpressures of 17.60 psi (121.35 kPa) and 13.90 psi (95.84 kPa) respectively. The mode of failure in each case is shear. Overpressure resistance for the connections was estimated at 12.0 psi (82.74 kPa) for the upper connection and 17.8 psi (122.73 kPa) for the lower connection.

A people survivability estimate is shown in Figure 25 and is based on the following reasoning. Up to about 8.20 psi (56.54 kPa) no structural damage is experienced and therefore, all occupants are uninjured survivors. At about 8.20 psi (56.54 kPa) the outer plywood fractures between the stringers. This is expected to be followed closely by the separation of the interior sheet of plywood (see Figure 28 for construction details). Casualties are produced at this stage by the interior plywood and sandbags driven into the shelter area. The intensity of this casualty mechanism increases with increasing overpressure. Collapse of the entire shelter due to failure of stringers and connections is estimated at about 22 psi (151.68 kPa), resulting in no survivors.

Shelter D is shown in Figure 29. This is a rectangular shelter constructed of reinforced concrete block walls with a stringed-skin panel roof. Dimensions of the shelter are similar to that of the rigid frame shelter (Figure 28). Using the 16 inch (0.41 m) dimension of the block as a module, the shelter interior size is about 4 ft (1.22 m) high by 5 ft (1.52 m) wide and 10 ft (3.05 m) long. The resulting floor area is 50 sq ft (4.65 sq m) and the volume 200 cu ft (5.66 cu m). Six courses of block work would be required to construct the shelter. As with the two previous shelters, this one was also designed to resist a static overpressure of 10 psi (68.95 kPa). The entranceway is located on the short side of the shelter rather than on the long side as in the previous two shelters. The entranceway is shown in Figure 30.

In the results of a structural analysis performed, it is assumed, as with previous shelters discussed, that the blast load environment is produced by a megaton-range nuclear weapon detonated at the ground surface.

The plywood sheeting is estimated to fracture at 8.20 psi (56.54 kPa). The roof/basement wall connection is estimated to fail at about 10 psi (68.95 kPa). The wall is estimated to rupture at 13.5 psi (93.08 kPa) and collapse at 20 psi (137.90 kPa). The collapse overpressure for the roof stringers is estimated at 17.6 psi (121.35 kPa).

A people survivability estimate for this shelter is given in Figure 25. It is based on reasoning which is similar to that used with shelter C (rigid frame shelter), since both shelters are similar.

2.3.2.2 Outdoor Shelters - Shelter E is shown in Figure 31. This is an aboveground A-frame shelter with the roof at 45 deg with the horizontal. The roof and the rear wall consist of 2 by 6 inch (0.05 by 0.15 m) boards nailed together with 20-penny nails. The front wall consists of 2 by 4 inch (0.05 by 0.10 m) boards which are also nailed together. The shelter is covered with soil to a depth of 2 ft (0.61 m) at the sides and in the rear. A sandbag or masonry barrier 2 ft (0.61 m) thick would be provided up against the front wall outside the shelter and up against the closed door inside the shelter.

The shelter has a floor area of approximately 205 sq ft (19.05 sq m) and a volume of 689 cu ft (19.51 cu m). At 65 cu ft (1.84 cu m) per person this shelter has room for approximately 10 persons.

Based on a simplified dynamic structural analysis, it was estimated that the front wall is capable of resisting approximately 11 psi (75.84 kPa) and the roof approximately 16 psi (110.32 kPa). Based on these results it is estimated that all shelter occupants are survivors up to 11 psi (75.84 kPa) and are fatalities for overpressures greater than about 16 psi (110.32 kPa). Between these overpressures casualties will be produced by debris from the breakup of the front wall and door and due to people being moved about by the blast winds. The people survivability estimate is shown in Figure 25.

Shelter F, the "plywood box shelter" is shown in Figure 32 which includes some of its construction details. For purposes of this discussion, this shelter is assumed to consist of a framework of 2 by 4 inch (0.05 by 0.10 m) boards overlaid with 3/4 inch (19.05 mm) plywood. The framework is assumed to be nailed using 0.135 by 1-1/2 inch (3.43 by 38.10 mm) helically threaded nails. Four such nails are assumed to exist at each corner connection of the frames. The frames are at 16 inch (0.41 m) centers. The shelter is semimounded with about 2 ft (0.61 m) of soil over the roof. Sandbags would be used to partially protect the entranceway.

The dimensions of the shelter, out to out are: length, 8 ft (2.44 m); width, 3 ft 11-1/4 inches (1.20 m); height, 4 ft 4-3/8 inches (1.33 m). The shelter has approximately 27.22 sq ft (2.53 sq m) of clear floor area and 108.9 cu ft (3.08 cu m) of volume. This size is adequate for, at most, two persons.

Based on the structural analysis performed, it was estimated that the plywood sheating would rupture at about 8.2 psi (56.54 kPa), the corner connections of the frame at about 11.1 psi (76.53 kPa) and the frame itself at about 12.5 psi (86.18 kPa). It is therefore, reasonable to estimate that all shelter occupants are survivors up to 8.20 psi (56.54 kPa) and are fatalities after about 12.5 psi (86.18 kPa). Between these overpressures casualties would be produced by debris from the breakup of the shelter roof. The people survivability estimate is shown in Figure 25.

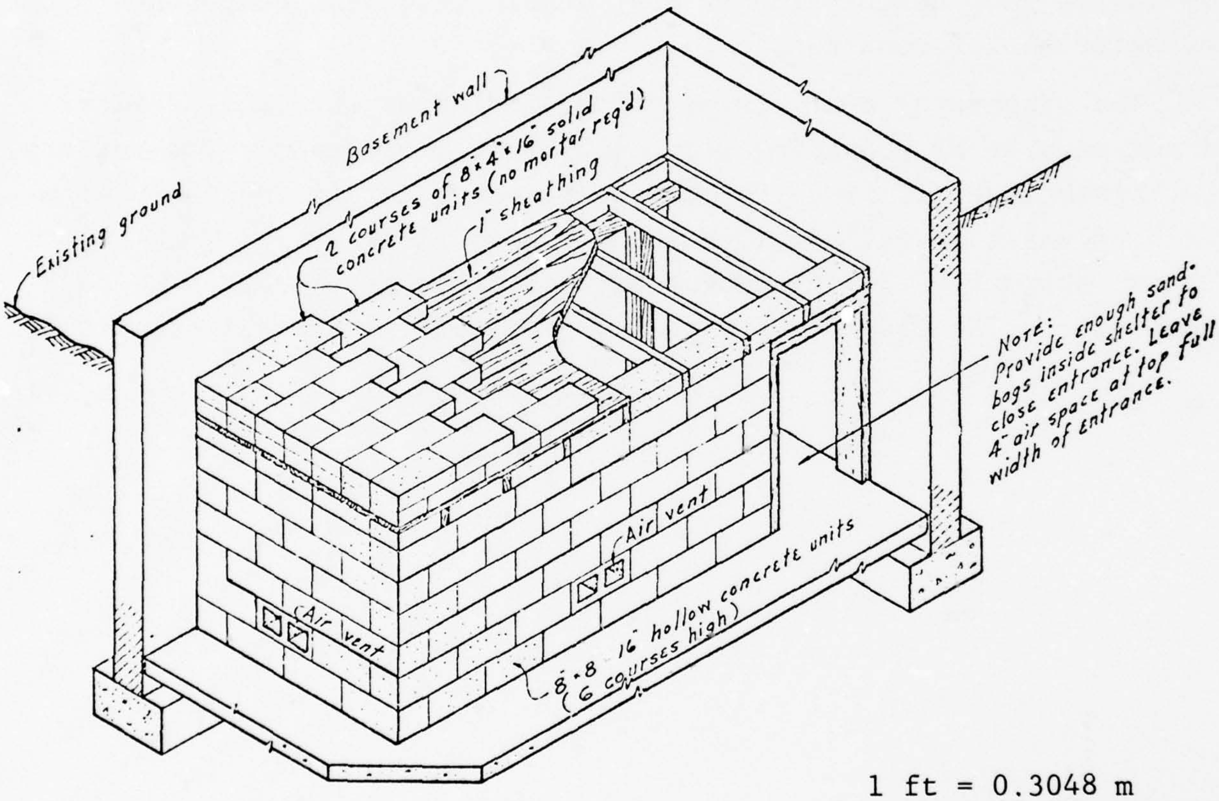
Shelter G is shown in Figure 33. This is basically a roofed over trench shelter whose walls are lined with plywood sheating. Nominal shelter dimensions are 20 ft (6.1 m) wide, 28 ft (8.53 m) long and 6 ft (1.83 m) deep. Shielded entrances are provided at each end of the 28 ft (8.53 m) long trench. The flat shelter roof is at grade and consists of 2 by 4 inch (0.05 by 0.10 m) wood joists 3-1/4 inches (82.55 mm) on centers supported on a longitudinal wood girder at midspan and at the trench walls on a continuous mud sill placed directly on earth. The girder consists of two 2 by 12 inch (0.05 by 0.30 m) boards and is supported on columns which are at 7 ft (2.13 m) centers. The columns are 4 by 4 inch (0.10 by 0.10 m) boards. Earth protective fill on the shelter roof is supported on asphalt roll roofing placed over the joists. Earth filled endwalls consist of vertical standing stud reinforced plywood panels forming inner and outer wall faces to support earth filling.

A structural analysis of the shelter indicates that the columns will reach the limit of their elastic stability at about 3.4 psi (23.4 kPa). The roof is estimated to collapse at about 6.3 psi (43.4 kPa). Based on these values it is estimated that all personnel are survivors up to 3.4 psi (23.4 kPa) with 50 percent fatalities at 3.4 psi (23.4 kPa). No survivors are expected beyond 6.3 psi (43.4 kPa). The survivability estimate is shown in Figure 25.

Shelter H, gable roof expedient shelter, is illustrated in Figures 34 and 35. It is approximately 49 ft (14.94 m) long, 12 ft (3.66 m) wide and has an average interior height of about 6 ft (1.83 m). It was designed to provide protection against fall-out radiation and accommodate approximately 50 persons.

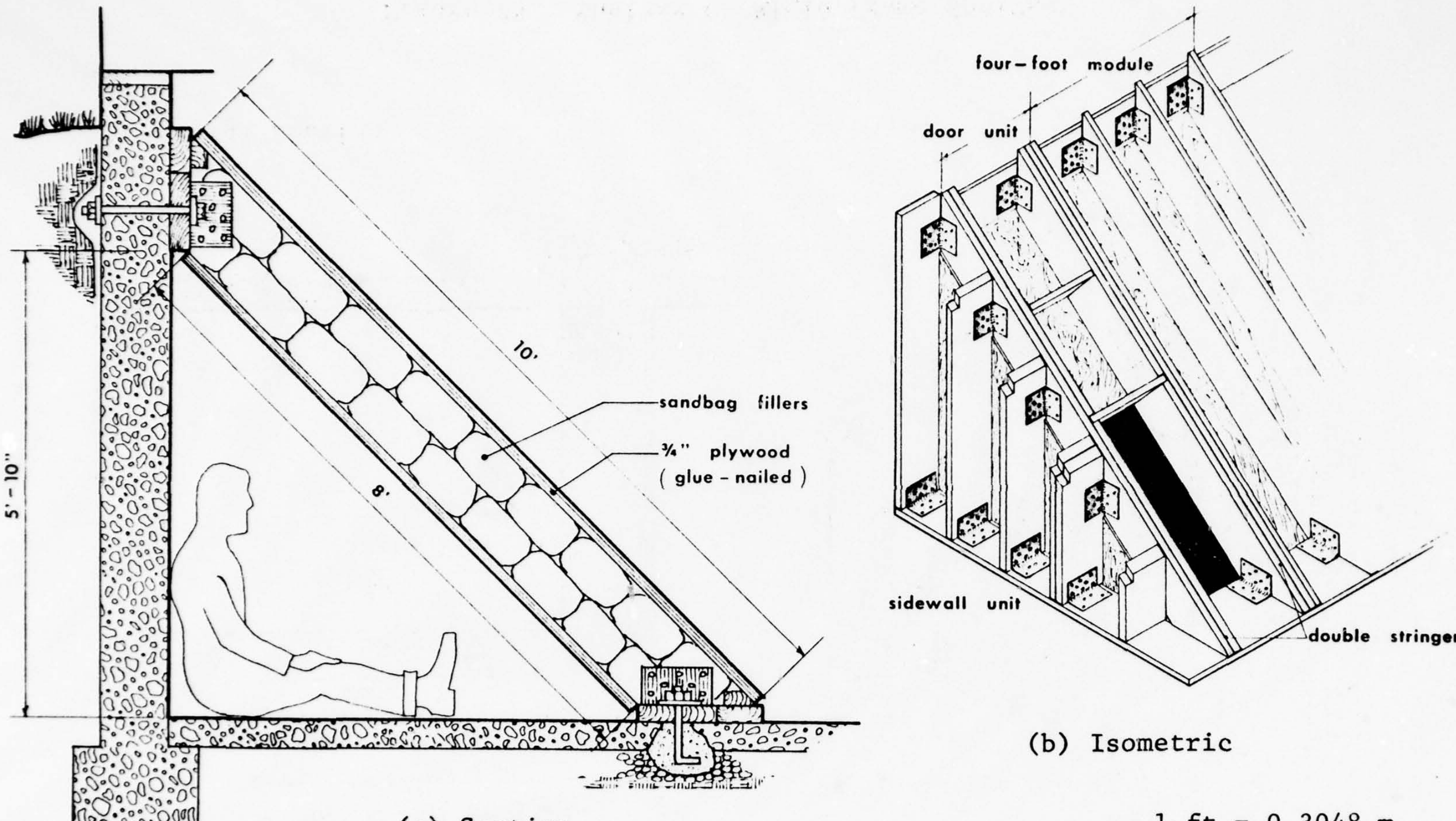
The shelter is of wood frame construction with sloping rafters supported on a center ridge beam and at ground level. Figure 35 provides a good description of most major structural components and materials of construction.

The structural analysis performed indicates that this shelter is not capable of providing very much blast protection. The rafters are fairly long, 12 ft (3.66 m) and are spaced at 16 inches (0.41 m). Their collapse is estimated at about 2 psi (13.79 kPa). When this occurs, approximately 50 percent of shelterees are expected to be casualties. The people survivability estimate is shown in Figure 25.



1 ft = 0.3048 m
1 inch = 25.40 mm

Figure 26. Shelter A, Basement Concrete Block Shelter



(a) Section

(b) Isometric

1 ft = 0.3048 m
1 inch = 25.4 mm

Figure 27. Shelter B, Lean-to Shelter

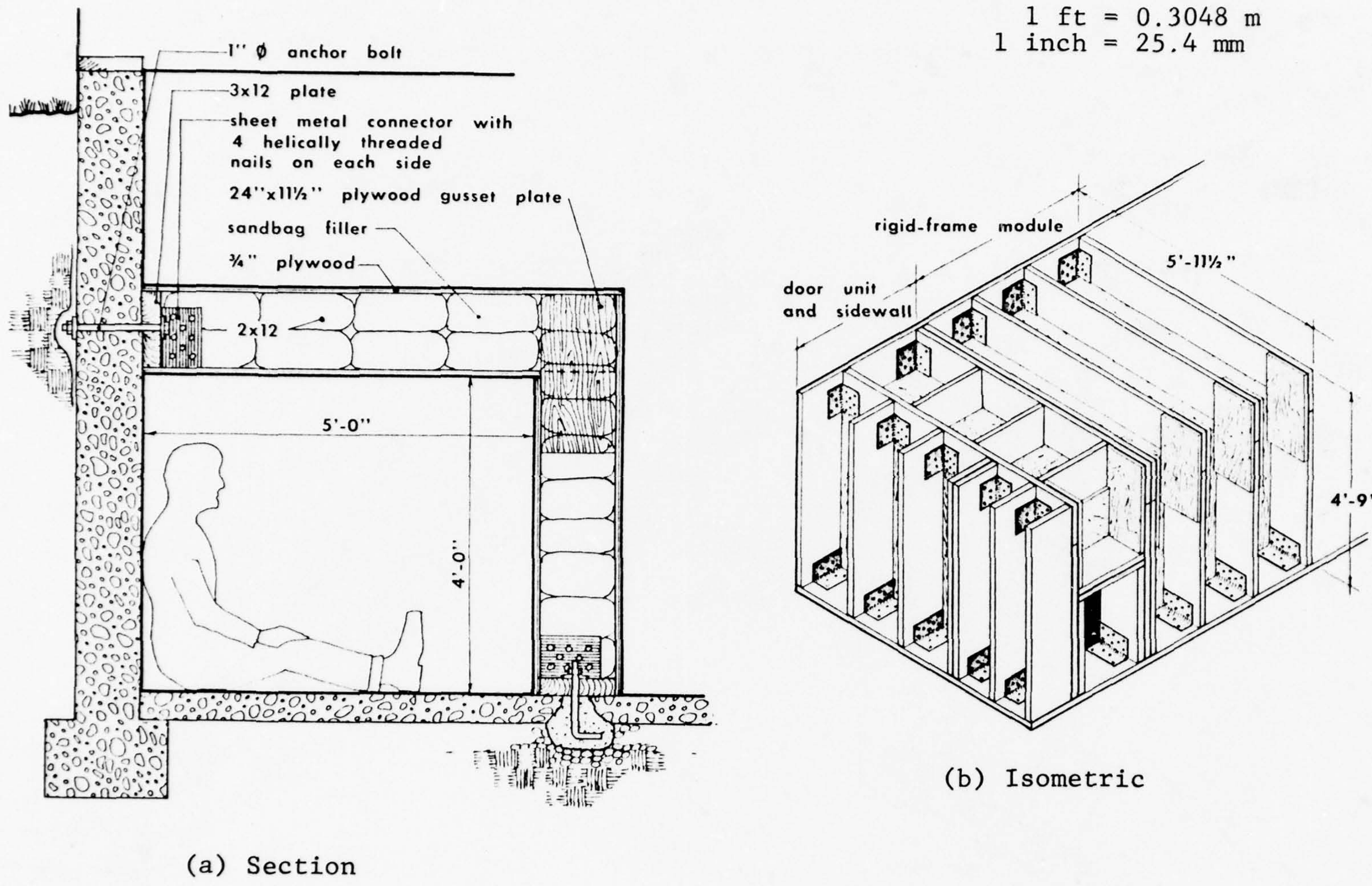


Figure 28. Shelter C, Rigid Frame Shelter

1 ft = 0.3048 m
1 inch = 25.4 mm

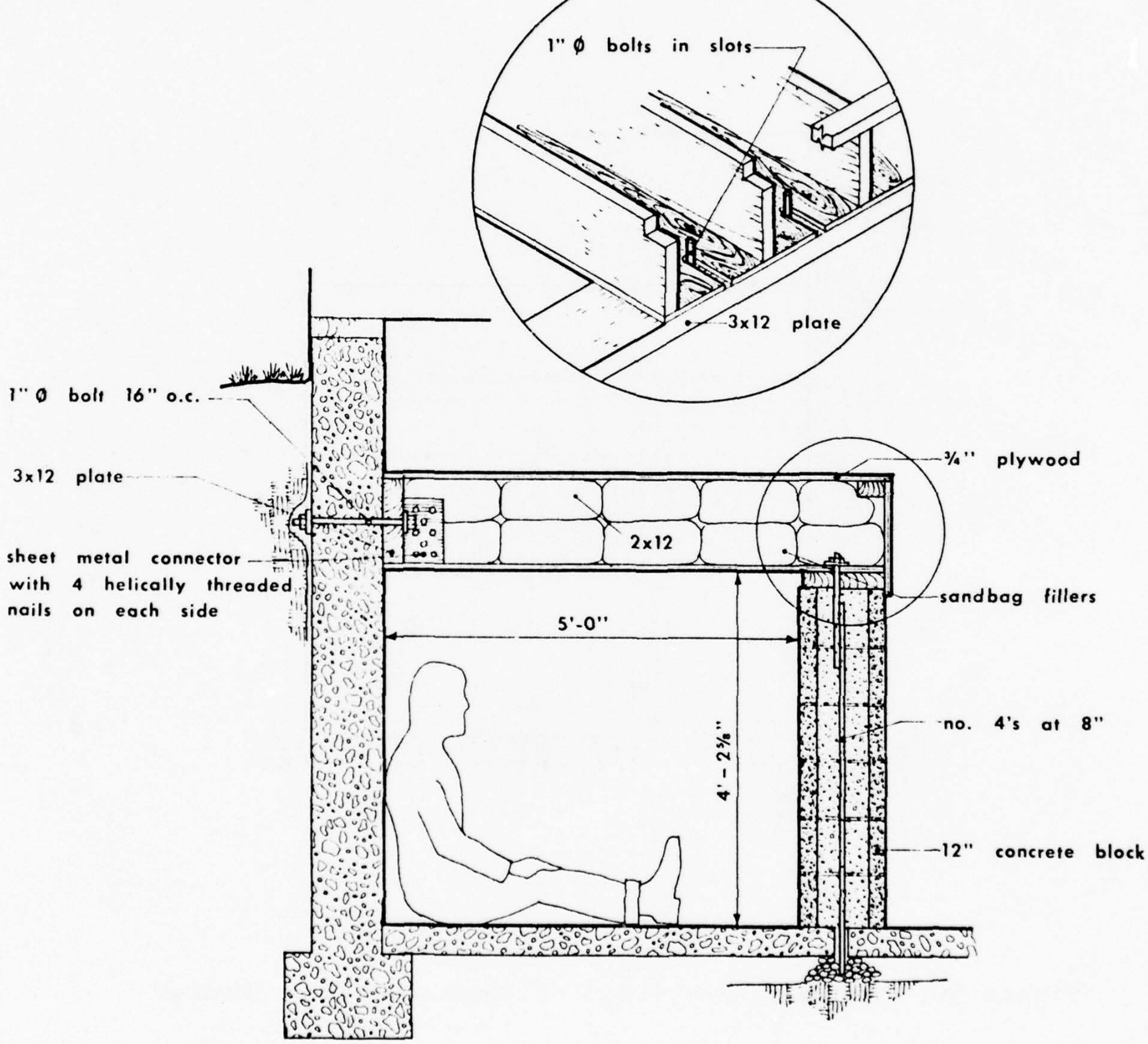


Figure 29. Shelter D, Reinforced Concrete Block Shelter

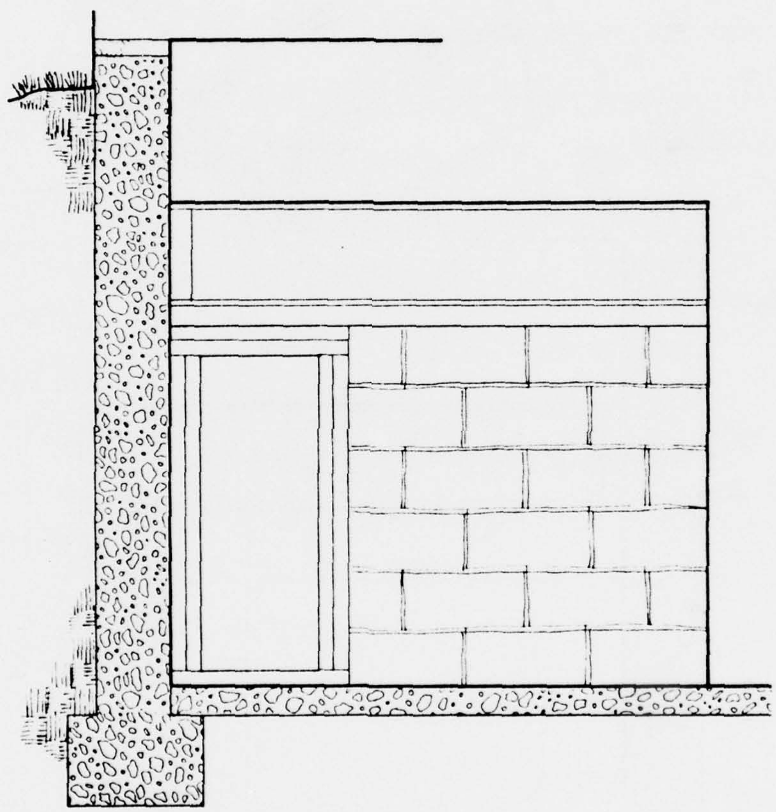
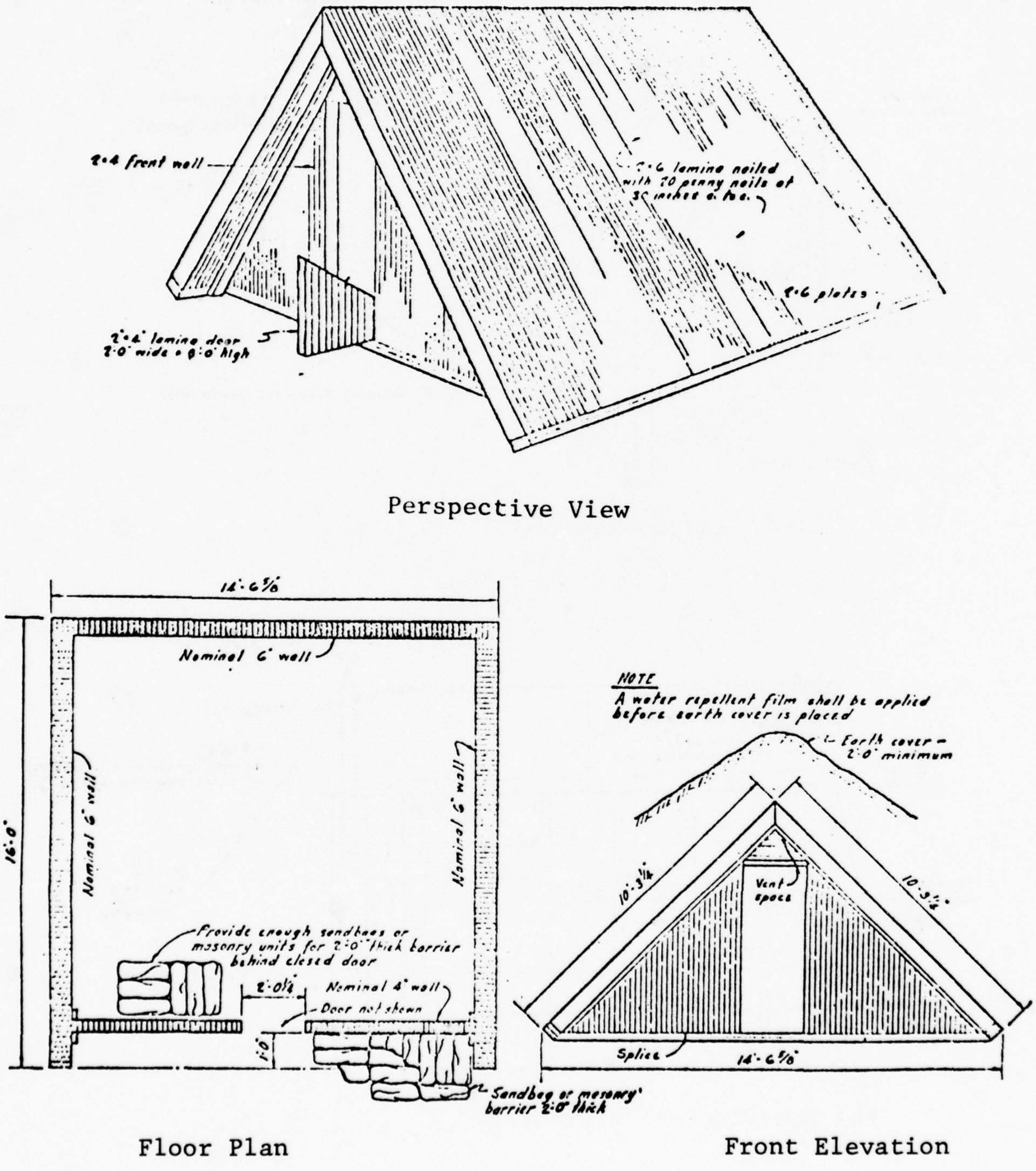


Figure 30. Sidewall Elevation of Concrete Block Shelter

1 ft = 0.3048 m
1 inch = 25.4 mm



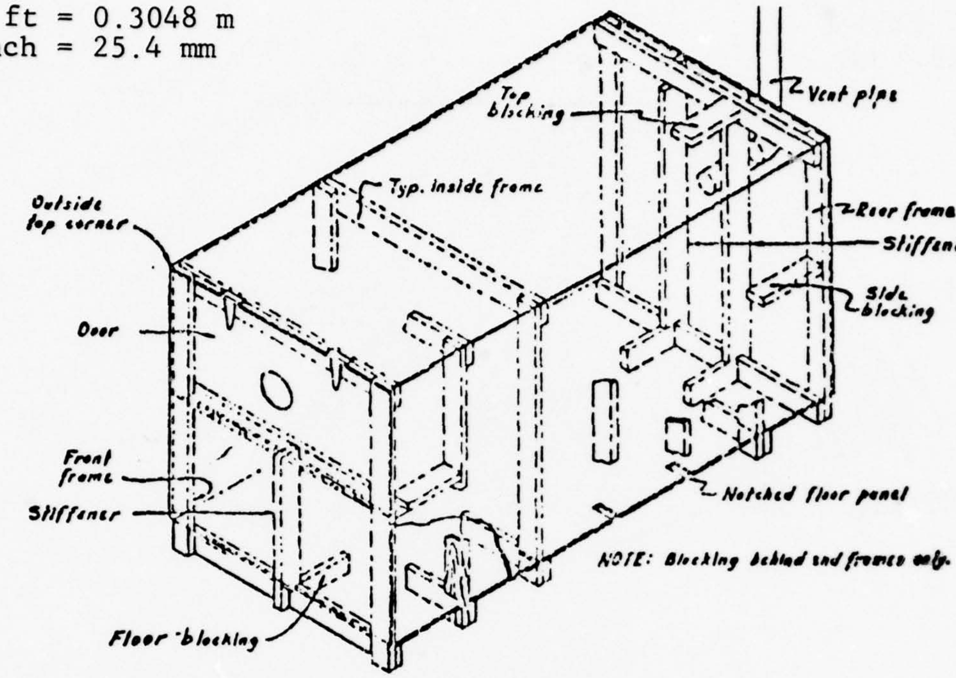
Perspective View

Floor Plan

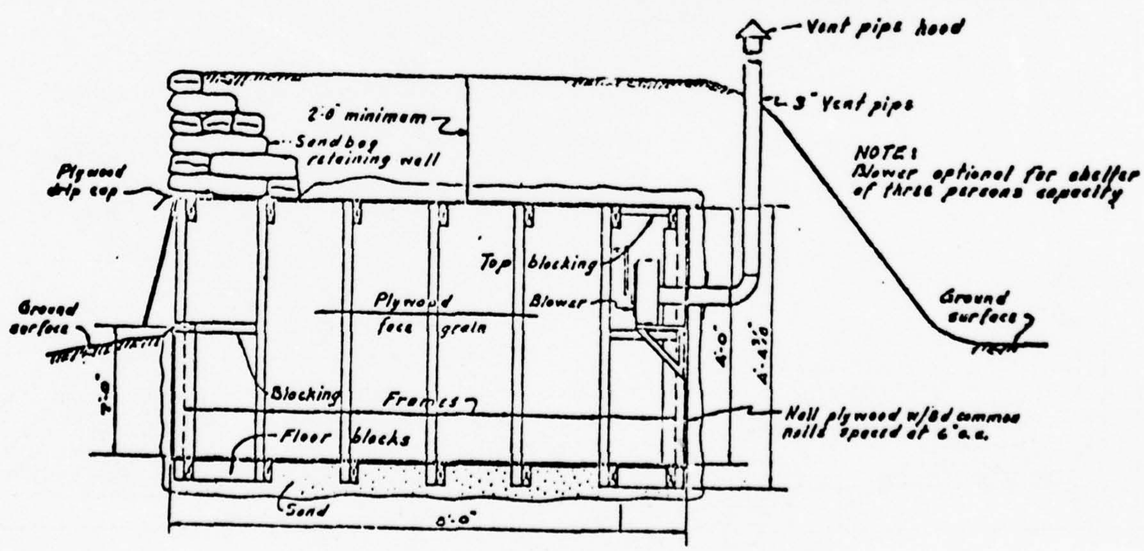
Front Elevation

Figure 31. Shelter E, Aboveground A-Frame Shelter

1 ft = 0.3048 m
1 inch = 25.4 mm



(a) Isometric



(b) Longitudinal Elevation

Figure 32. Shelter F, Plywood Box Shelter

1 ft = 0.3048 m
1 inch = 25.4 mm

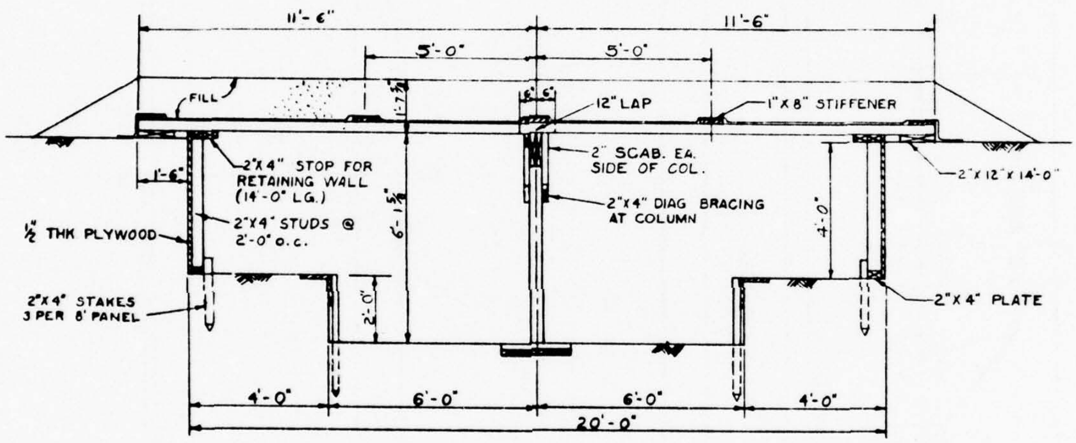
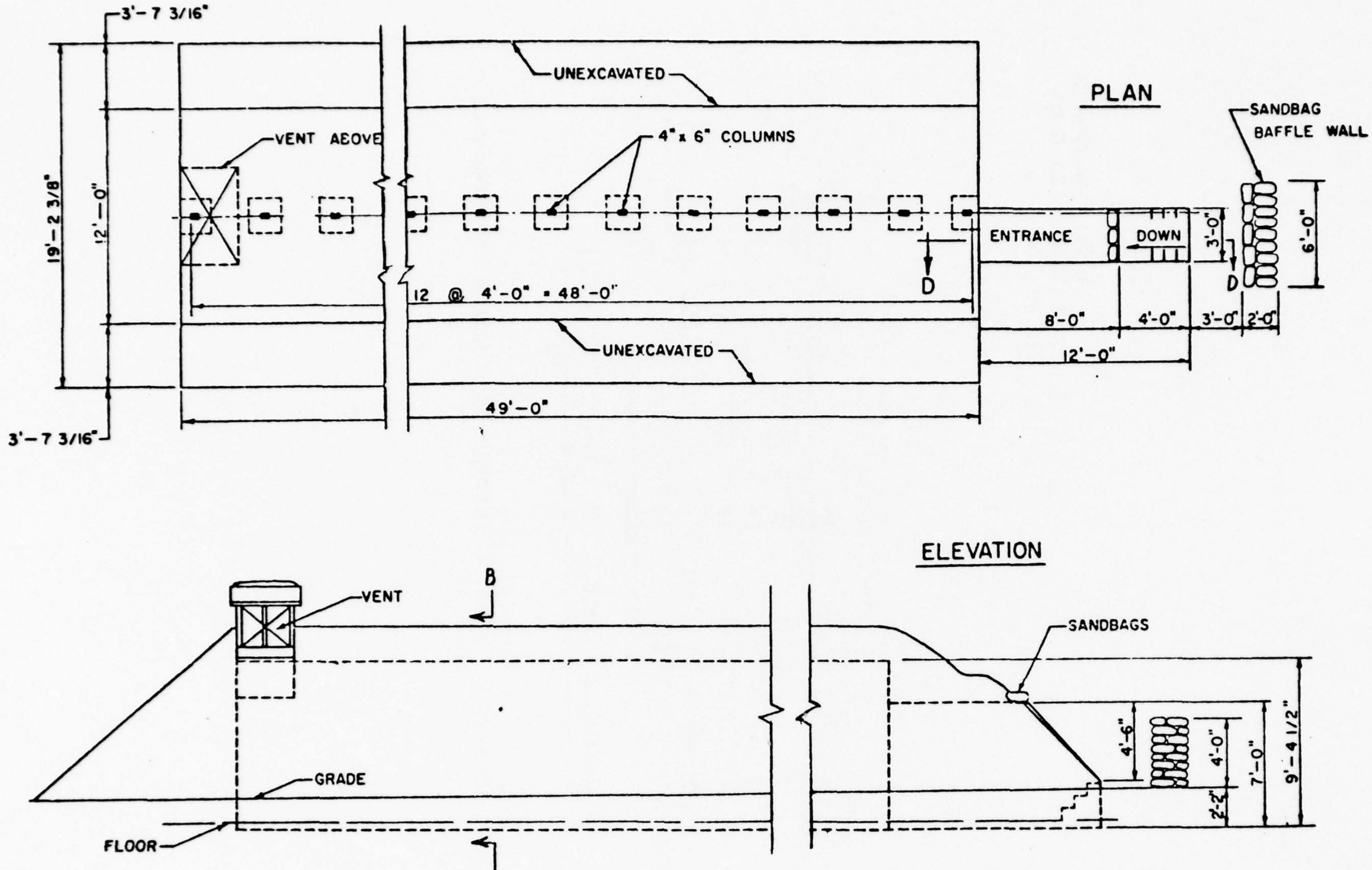


Figure 33. Shelter G, Section Through Wood Grate Roof Shelter



1 ft = 0.3048 m
1 inch = 25.4 mm

Figure 34. Shelter H, Gable Roof Shelter (plan and elevation)

1 ft = 0.3048 m
1 inch = 25.4 mm

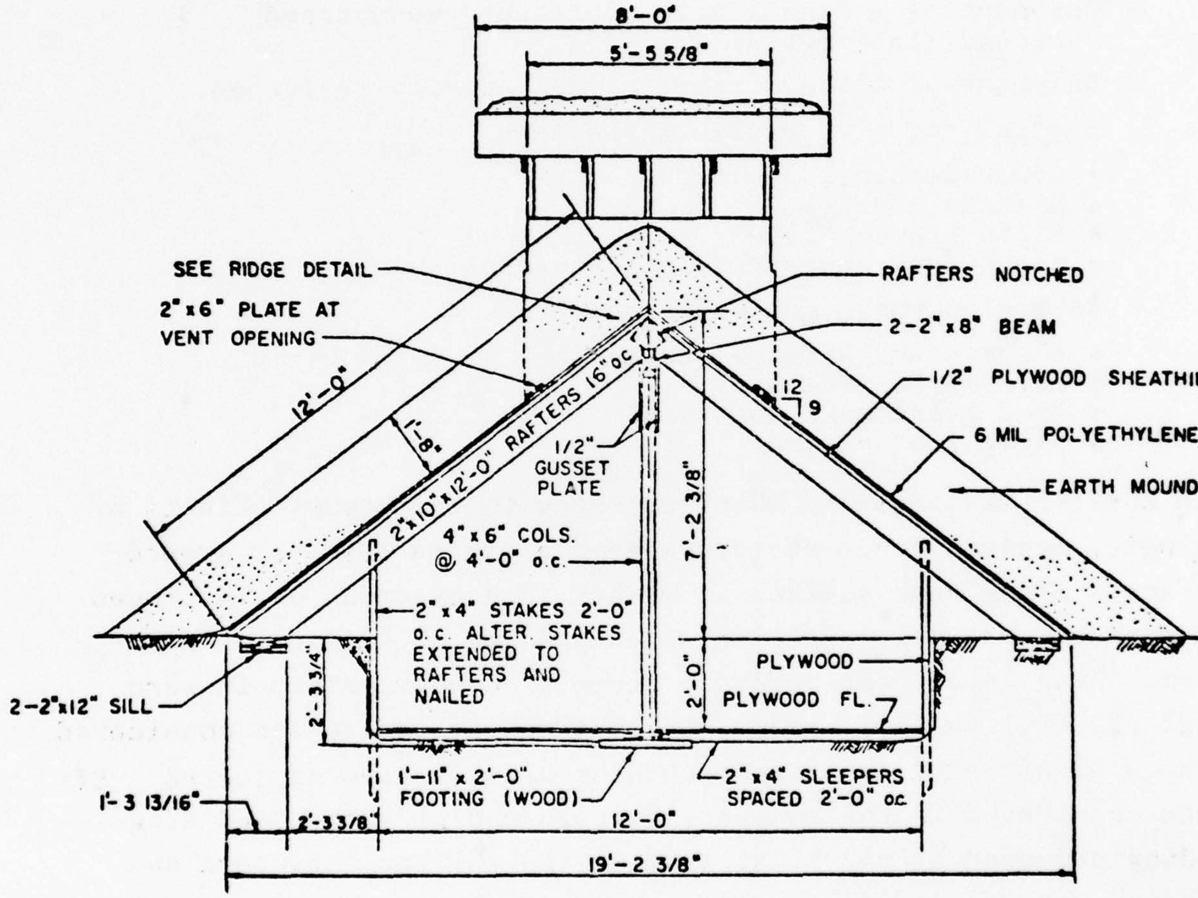


Figure 35. Shelter H, Gable Roof Shelter (Section B-B)

3. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The study reported was concerned with the development of survivability functions for people in regions with overpressure levels less than 2 psi (13.79 kPa) produced by the detonation of megaton range nuclear weapons. Shelters for which people survivability functions were developed include four types.

1. Basement of a school with a precast/prestressed overhead floor system.
2. Basement of a wood framed, single family residence.
3. Special purpose, basement shelters
   - Concrete block shelter
   - Lean-to shelter
   - Rigid frame shelter
   - Reinforced concrete block shelter
4. Special purpose, outside shelters
   - Aboveground A-frame shelter
   - Plywood box shelter
   - Wood grate roof shelter
   - Gable roof shelter

Each shelter was analyzed when subjected to the prompt effects of a single, megaton range nuclear weapon exploded near the ground surface. Since each shelter is either in a basement or is closed and mounded, thermal radiation was neglected as a casualty mechanism. Also, since the relevant range of overpressures is zero to 22 psi (151.68 kPa), prompt nuclear radiation was not considered to be a significant casualty mechanism and was also neglected. Effects considered in the evaluation include diffraction and drag loading produced by the blast wave on the shelter structure and dynamic pressure and debris (from the breakup of the structure) on the shelter occupants.

Table 8 contains a summary of results. In this table free field overpressures at which 90, 50 and 10 percent of the shelter occupants survive in each shelter are compared. For a more detailed comparison the reader is referred to Figures 6, 22, 24 and 25.

The school basement (item 1 in Table 8) is the least desirable of this set of shelters. Its overhead floor system consists of precast/prestressed concrete members. It was chosen for consideration because such structural elements are entering the building inventory at an increasing rate. Currently such elements comprise approximately 10 percent of the construction market. When service loads are concerned, such units are strong, durable, versatile and due to currently high labor costs, compete favorably with cast-in-place construction. They are found as part of the structural system, in retailing stores, schools, municipal buildings, office buildings and multistory and multifamily dwellings. When dynamic (blast) loads are concerned, the reserve strength of these units appears to be limited due to low shear strength.

Although these units may be difficult to upgrade for dynamic loads, it is believed that they can be effectively upgraded to carry additional soil cover safely. Several upgrading concepts can be used for this purpose. Two are described on page 30 of this report. Note that if only the corridor is used, without any upgrading of the classrooms or the corridor, the survivability function becomes a "cookie cutter" type and people survive to 3.89 psi (26.82 kPa). Obviously, further improvement is possible.

The framed house basement results included in Table 8 are average values between sugar pine and longleaf pine (see Figures 22 and 24). They are based on the assumption that people are uniformly distributed in all normally usable basement areas. Should people be located along peripheral walls only, then the 50 percent survivability is estimated at 10 psi (68.95 kPa) for the as-built structure and 16 psi (110.32 kPa) for the upgraded structure.

It is concluded that basements on the fringe of high risk areas can be effectively upgraded to provide the needed protection. Since the concern is with fallout protection and overpressures less than 2 psi (13.79 kPa), upgrading techniques need not be elaborate. In those areas where basements do not exist, a great deal can be accomplished by shoring and mounding of community centers, warehouses, residences, garages, etc.

TABLE 8. OVERPRESSURE FOR INDICATED PERCENT SURVIVORS

Shelter	As-Built						Upgraded					
	90%		50%		10%		90%		50%		10%	
	psi	(kPa)	psi	(kPa)	psi	(kPa)	psi	(kPa)	psi	(kPa)	psi	(kPa)
1. School Basement	0.85	(5.86)	0.85	(5.86)	3.89	(26.82)	2.29	(15.79)	2.29	(15.79)	3.89	(26.82)
2. Framed House Basement												
Type 1 Lumber*	2.21	(15.24)	2.60	(17.93)	4.40	(30.34	5.67	(39.09)	6.50	(44.82)	10.50	(72.40)
Type 2 Lumber	5.01	(34.54)	5.70	(39.30)	7.80	(53.78)	9.30	(64.12)	10.80	(74.46)	18.70	(128.93)
3. Basement shelters												
A. Concrete Block	3.30	(22.75)	4.20	(28.96)	6.30	(43.44)						
B. Lean-to	8.60	(59.29)	10.25	(70.67)	10.90	(75.15)						
C. Rigid frame	9.60	(66.19)	15.10	(104.11)	20.60	(142.03)						
D. Reinforced concrete block	9.40	(64.81)	14.10	(97.22)	18.80	(129.62)	NOT APPLICABLE					
4. Outside Shelters												
E. Aboveground A-frame	11.50	(79.29)	13.50	(93.08)	15.50	(106.87)						
F. Plywood box	8.70	(59.98)	11.10	(76.53)	12.60	(86.87)						
G. Wood grate	3.40	(23.44)	3.40	(23.44)	6.25	(43.09)						

* Type 1 Lumber is Sugar Pine

Type 2 Lumber is Longleaf Pine
(These two types of lumber were used to approximate bounds on the people survivability estimate for a resudence).

The analysis on which these results are based was hampered by lack of available data on the response of wood components from initial yielding to collapse under the action of static or dynamic loads. When such data become available results contained here should be revised. It is expected that the revision would be upward.

The special purpose family and community shelters (see Figure 26 through Figure 35) are considered to be very adequate for the fringe between the low and the high risk areas. Most of those considered in this report possess a fair degree of blast protection in addition to fallout radiation protection. Their designs may be easily modified to produce a higher level of protection. Their major drawback is that to implement them on a large scale would require a major preplanning and prestocking effort at the local civil defense level.

As with the wood frame house, this analysis effort was hampered by lack of available data on the behavior of structural lumber and structural plywood from yielding to collapse under the action of static and dynamic loads.

Several other structural systems and upgrading methods were considered but not carried to completion due to fiscal constraints.

<u>Open web steel joist floor system</u>: This effort was motivated by an experimental study conducted by Waterways Experiment Station (WES), Reference 30, in which several open web joist (OWJ) floor systems were tested to collapse using static load. In the study reported here, a floor system consisting of open web steel joists was designed using the local building code. The collapse load was determined by means of analytic procedures and was verified using experimental data both for the as-built and upgraded conditions. A people survivability function was not determined for the reason cited earlier.

<u>Upgrading of two-way slabs</u>: Two-way slabs will be found both in low and in high risk areas and therefore a set of effective upgrading measures is needed. In the previous study (Ref. 2) an upgrading concept consisting of a timber crib was developed and

analyzed. An alternative concept using substantially less timber is shown in Figure 36. Due to reasons cited it was not carried beyond the concept stage.

In the initial stage of this study a preliminary effort was devoted to determine if trussing (such as attaching a "king post" truss to a girder) or prestressing are effective upgrading measures. A design was performed which involved trussing the main girder in the floor system over a basement of a wood frame house. The design was very effective in carrying additional dead load such as the soil load. However, its implementation is considered to be beyond the capability of the average homeowner. This concept was abandoned as an expedient measure.

It was indicated earlier, this study was hampered by a general lack of mechanical data on structural timber and plywood up to collapse. It is recommended that studies be initiated to eliminate or minimize this difficulty. Response data should be produced for individual structural members such as beams and columns and for timber assemblages such as floor systems. It is also recommended that additional upgrading concepts be developed and rated in terms of people survivability. The two structural systems, i.e., OWJ and upgrading of two-way slabs which were not completed in the course of this study should be brought to a logical conclusion. Other concepts should consider community centers, warehouses and structures without basements such as residential parking garages for example.

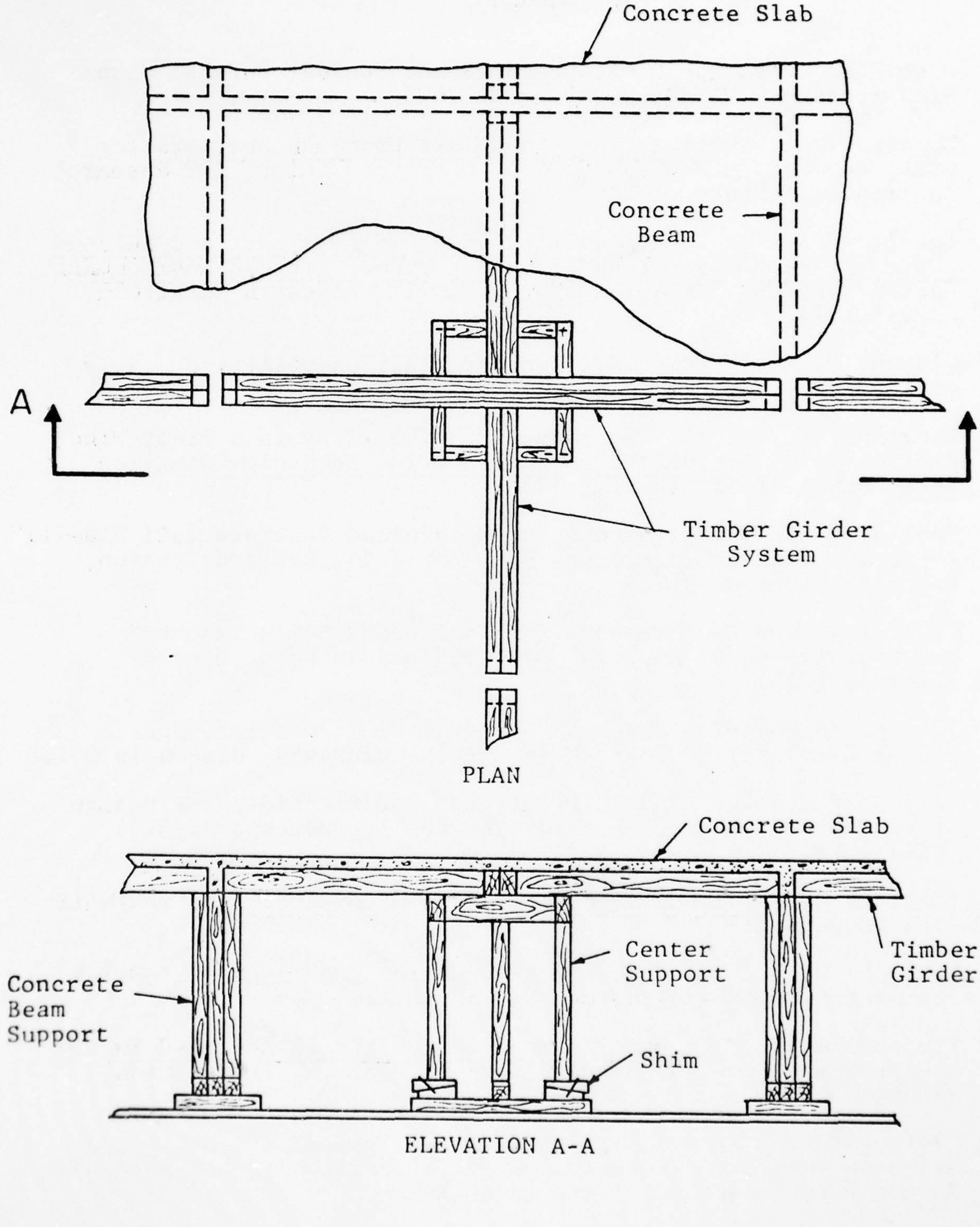


Figure 36. Two-Way Slab Upgrading System

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FINAL REPORT 82 pages
DCPA Contract DCPA01-76-C-0324 IIT Research Institute
Work Unit 1621G October 1978

ABSTRACT

This report contains the results of a study aimed at exploring the sheltering potential for people in low risk areas, i.e., in areas with overpressures less than 2 psi (13.79 kPa) caused by megaton range nuclear weapon detonations. Two basement shelters and eight special purpose shelters were evaluated and a "people survivability function" was developed for each. A survivability function relates percent survivors in a given shelter to a range of overpressures at the shelter site. Casualty mechanisms considered with this set of shelters include blast, i.e., dynamic pressures and debris from the breakup of the shelter structure. Results included in this report allow the civil defender to estimate the people survivability potential in and near the fringe of high risk areas when shelters of the type considered in this study are used to protect the population.

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