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Technical Report *May 1976*

Upgrading Basements for Combined Nuclear Weapons Effects: Expedient Options

For:

DEFENSE CIVIL PREPAREDNESS AGENCY
WASHINGTON, D.C. 20301

Contract No. DCPA01-75-C-0302
DCPA Work Unit 1155C

SRI Project 4270

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E R R A T A

The following pen-and-ink changes should be made:

On pages xiii and 99, insert "Garage" following "Underground" in title of Figure 22.

On page xiii, correct title of Figure 23 to agree with page 101.

On pages xiii and 108, delete "Building 239" from title of Figure 24.

In Figure 10, page 31, upper right note thereon, change "PLATES" to "CAPS."

On page 37: paragraph M, in first line change "communicators" to "communications"; paragraph 1, in sixth line, change initial word "or" to "on."

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20 ABSTRACT (Continued)

This The research ~~project~~ concerned expedient upgrading of existing basements to provide some degree of protection against combined nuclear weapons effects, especially air blast; such shelter would be for critical workers remaining in "risk areas" during a crisis period. As used ~~herein~~ in this report, "expedient" upgrading is that which can be accomplished within about a 24- to 48-hr warning period using indigenous labor and materials, and "basements" are those that have at most a small portion of any side exposed to blast or--in terms of a partially buried basement--perhaps no more than the upper 30% or so of any wall(s) exposed.

An extensive literature search* turned up no real expedient blast upgrading schemes, although some old (federal civil defense) fallout shelter ideas were found that had adaptation potential for blast upgrading, and thus parallel concepts/schemes shown herein for expedient structural strengthening and for "last resort" shelter within a shelter.

Research, ~~work~~, case studies, and this ~~A general~~ study ~~reported herein~~† clearly demonstrate a total lack of any correlation between floor design live load and mean blast collapse overpressure of a floor system. Important is the way the structure is put together and how the R/C designer chose to detail the reinforcing steel. Thus for expedient blast upgrading, availability of a set of "as-built" drawings is very important (for engineered upgrading, such a set is vital). With drawings available, some conclusions can be reached from engineering experience and existing structures evaluation techniques‡ and ideas developed, to exploit any inherent blast resistance or to enhance it. General schemes for such strengthening are presented (not to scale) and one scheme includes an example in terms of blast resistance.§

Collateral to this study, it developed that there is a real need for engineered upgrading techniques for existing EOCs, whether permanently built, or crisis installed.

For those interested particularly in EOCs, expedient upgrading (in a case study herein) improved the functional blast resistance from 2 psi to 17 psi,** i.e., the potential area of severe damage/collapse was reduced 91%.††

In the other three case studies herein,‡‡ expedient upgrading showed the following results:** 3.6 psi (open shelter) to 10.1 psi (closed); 3.5 psi (open) to 7.0 or 9.1 psi (open), the latter with a considerable increase in upgrading work; and 2 or 3 psi (open), representing the missile hazards of materiel, to 10.7 psi (closed). The first two require considerable upgrading work; the last requires only the addition of several expedient blast closures.

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* Appendix A describes the search; an annotated Bibliography provides some details.

† Chapter 5, Section I; case studies follow in Sections II through V.

‡ See numbers 11, 12, and 15 in References section.

§ See Figure 4 for upgrading scheme.

** Predicted mean collapse (incident or free-field) overpressure.

†† Chapter 5, Section II.

‡‡ Chapter 5, Section III to V.

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Technical Report *May 1976*

Summary

Upgrading Basements for Combined Nuclear Weapons Effects: Expedient Options

By: H. L. MURPHY
C. K. WIEHLE
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Facilities and Housing Research

For:

DEFENSE CIVIL PREPAREDNESS AGENCY
WASHINGTON, D.C. 20301

Contract No. DCPA01-75-C-0302
DCPA Work Unit 1155C

SRI Project 4270

DCPA Review Notice

SUMMARY

The research project concerned expedient upgrading of existing basements to provide some degree of protection against combined nuclear weapons effects, especially air blast; such shelter would be for critical workers remaining in "risk areas" during a crisis period. As used herein, "expedient" upgrading is that which can be accomplished within about a 24- to 48-hr warning period using indigenous labor and materials, and "basements" are those that have at most a small portion of any side exposed to blast or, in terms of a partially buried basement, perhaps no more than the upper 30% or so of any wall(s) exposed. Engineered upgrading was outside the project scope, but brief comments on it have been included with some of the case studies herein.

An extensive literature search* turned up no real expedient blast upgrading schemes, although some old (federal civil defense) fallout shelter ideas were found that had adaptation potential for blast upgrading, and thus parallel concepts/schemes shown herein for expedient structural strengthening and for "last resort" shelter within a shelter (basement). (The search turned up essentially nothing on engineered blast upgrading, either.)

Research work, case studies, and a general study reported herein† clearly demonstrate a total lack of any correlation between floor design live load and mean blast collapse overpressure of a floor system. More important is the way the structure is put together, and most importantly, how the R/C designer chose to detail the reinforcing steel. From the latter it follows that even for expedient blast upgrading, availability of a set of "as-built" drawings (structural and architectural, as a minimum) for the building is very important (for engineered upgrading, such a set is vital to the work). With drawings available, some conclusions can be reached (based on engineering experience, or better, using existing structures evaluation techniques‡) and ideas developed on what and where to strengthen, to exploit any inherent blast resistance or to enhance it. General schemes for such strengthening are presented herein, with the admonitions that they are not to scale and that any member dimensions shown are for illustration/approximation only, i.e., not engineered values suitable for all applications. One scheme includes an example in terms of blast resistance before and after application.§

Collateral to this study, it developed that there is a real need for engineered upgrading techniques for existing EOCs, whether permanently built, or crisis installed with materials prepared and stored close by. Future work on such techniques is planned.

In an upgraded basement intended for EOC or other operational use, minor holes should be plugged (e.g., stuffed with rags), because a small jet that is unimportant to shelteree survival can still knock small equipment around (e.g., a desk telephone) perhaps to the extent of knocking it out of use.

For those interested particularly in EOCs, expedient upgrading (in a case study herein) improved the functional blast resistance from 2 psi (used for any open shelter as an upper limit under which operations can be carried on) or 9 psi** (based on structural strength as an open shelter with all potential materiel missiles either removed or anchored) to a 17 psi** closed shelter.†† This improvement from 2 psi to 17 psi means that the potential area of severe damage/collapse has been reduced 91%.

In the other three case studies herein,‡‡ expedient upgrading showed the following results:** 3.6 psi (open shelter) to 10.1 psi (closed); 3.5 psi (open) to 7.0 or 9.1 psi (open), the latter with a considerable increase in upgrading work, probably too much to be worthwhile; and 2 or 3 psi (open), representing the missile hazards of materiel, particularly stud-and-plasterboard interior walls, to 10.7 psi (closed). The first two require considerable upgrading work; the last requires only the exploiting of inherent blast resistance by the addition of several expedient blast closures.

Acknowledgements

Through suggestions and guidance, the technical help of G. N. Sisson and M. A. Pachuta, U.S. Defense Civil Preparedness Agency, was freely given and is gratefully acknowledged.

Note

Every effort has been made to ensure the accuracy of all guidance and programs included herein. However, no warranty, expressed or implied, is made as to the recommended procedures or programs. The reader-user is expected to make the final evaluation as to the usefulness or all material contained herein. Recommendations made herein should not be substituted for the knowledge, experience, and judgment of the professional engineer or architect, but should be treated as guidance for consideration by the professional, regarding the best method of achieving specific design goals.

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* Appendix A describes the search; an annotated Bibliography provides some details.

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‡ See numbers 11, 12, and 15 in References section.

§ See Figure 4 for upgrading scheme; expedient upgrading of elevator shaft in basement, with blast resistance improved from about 1/4 psi to about 11 psi.

** Predicted mean collapse (incident or free-field) overpressure with zero rise time.

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TABLES

FIGURES

Chapter 1

INTRODUCTION

The work reported herein is in compliance with Contract No. DCPA01-75-C-0302, DCPA Work Unit No. 1155C, "Blast Upgrading Options for Existing Structures." The overall research work is divided into the following phases:

I. EXPEDIENT UPGRADING OPTIONS FOR COMBINED NUCLEAR WEAPONS EFFECTS

II. ENGINEERED UPGRADING OPTIONS

This project and report are confined to the first phase. A subsequent Institute project is expected to extend the first phase and include work on the second phase. The first phase, however, included a comprehensive review of protective literature, primarily for expedient, but keeping alert for engineered, upgrading concepts and opportunities. Comments on engineered upgrading potential resources are included in the Bibliography annotations.

Objective

The objective of this phase of the work was to develop a variety of expedient blast protection options for National Shelter Survey (NSS) and other building basements for implementation during a crisis period, giving consideration to both open and closed shelter modes.

Options that provide blast closures will increase initial and fallout radiation protection as an added benefit, although this is not a specific objective.

Work Performed

In furtherance of the objective, the following work was performed:

- A comprehensive search for literature pertaining to shelter and protective construction was carried out.*

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* Appendix A details the literature search made.

- The literature was reviewed to identify material of value to expedient and/or engineered protective upgrading.*
- The literature material and other expedient protective measures originating with the authors and other experienced persons were analyzed for practicality of implementation under crisis conditions.
- Specific "how-to-do-it" application instructions were developed for those measures deemed practical. The instructions include material sources and application sketches.
- Sources of required and alternate materials were evaluated and listed.
- Several specific existing building basements were examined in detail and expedient protective measures were designed and described, together with labor and material estimates. Details on these structures are in Chapter 5.

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* Any material found and used in this report is related by superscript numerals to sources in the References section. Other material found on upgrading is covered by annotations in the Bibliography section.

Chapter 2

BACKGROUND

Situation

The present government policy for protection of the civil population under the threat of nuclear attack is to take best available shelter in the event of a short-warning-period attack, or to relocate the population (from potential target areas to areas that are not expected to be targets) during a so-called "crisis buildup" period.

Such strategic relocation planning envisages the requirement for considerable numbers of the population to remain behind in the potential target area to perform essential functions, such as:

- Police and fire protection.
- Operation and maintenance of essential utilities.
- Operation and maintenance of essential industries.
- Operation and maintenance of communication systems (telephone, radio, and television).
- Civil defense planning and operations.
- Essential transportation functions.

It is expected that the staffs of organizations and firms engaged in these essential functions will be "thinned out" so that only those persons absolutely necessary will remain behind.* An appropriate level of combined nuclear effects protection must be provided these "stay behinds" if the crisis relocation policy is to be credible. Such protection is the fundamental problem to be approached in this study.

Alternative means of providing the necessary protection include:

1. Construction of special pre-designed and constructed shelters at the locations of the essential functions.

* Unessential persons who simply refuse to leave are on their own for purposes of this study.

2. Upgrading of potential shelter space in NSS and other buildings for nuclear blast and radiation protection through engineered designs and structural alteration, with the latter performed in advance, or during the warning period using stockpiled items, perhaps coupled with some early preparatory work. This is termed "engineered upgrading."

3. Upgrading of potential shelter space in NSS and other buildings for nuclear blast and radiation protection through use of readily available materials, tools, and equipment by the "stay behind" forces in a short period of time. This is termed "expedient upgrading."

Although alternatives 1 and 2 are not necessarily ruled out of crisis relocation planning, major opportunities for effective, economical protection accrue to alternative 3, which is the subject of this report.

Application

The main consideration in the selection of the shelter facility for each essential function would be the proximity to the function. The expected short warning times will not permit long travel times. Existing NSS buildings would be chosen if they were sufficiently close to the function. In many cases, however, a shelter facility will be required to be closer to the function. Preferably, the shelter should be located in the structure housing the function.

The shelter facility must also be located in basements* of buildings or other underground space in order to attain an acceptable degree of blast protection. Thus, this study concentrates on the upgrading of basement space.

Examples of building functional types that must be considered include:

- Existing Emergency Operating Centers (EOCs).
- Emergency Operating Centers and Evacuation Control Centers established during the crisis.
- Police stations.
- Fire stations.
- Manned telephone exchanges.

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* Aboveground locations in special hard-walled buildings without openings may also be suitable, but are unlikely to be available, and this study is therefore aimed at basements that are fully buried or nearly so.

- Radio and television stations.
- Electric power generation control rooms and dispatch centers.
- Gas transmission dispatch control centers.
- Water supply control facilities.
- Avaiation ground control facilities.

If the buildings housing these functions had suitable basements, such space would be selected for upgrading. Otherwise, the nearest NSS or other building having a suitable basement would be selected. For the purposes of this study, "suitable" implies that the basement possess the following desirable characteristics:

- Have a first floor (basement cover slab) that is at grade, or nearly so.*
- Have a minimum of window and door openings, window-wells, areaways, ventilation shafts, etc., through basement walls.
- Have a minimum of first floor penetrations for elevator shafts, stairwells, mezzanine openings, ventilation penetrations, etc.
- Have more than one means of ingress and egress.
- Have concrete floors, preferably at least 4 in. thick, with thicker floors increasing desirability.†
- Have basement reinforced concrete exterior and partition walls instead of frame, concrete block,‡ or similar light-weight or frangible walls.
- Be in a building of several stories so that the basement walls, columns, and footings have excess strength when the upper stories are removed or at least cleared of live loads by the blast forces.§

The general concept of application includes the following features:

1. The basic concept is to provide as much upgraded protection from combined nuclear effects as is possible in basement shelters within

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* Sub-basements are very desirable.

† Wood floors may qualify, depending on construction quality and details, plus the general situation on availability of basements.

‡ Unless reinforced, such as required in earthquake-resistant codes.

§ Underground parking garages normally having roof traffic loads are also desirable, especially multi-story.

the constraints of:

a. Existing structural configuration and strength of the selected shelter.

b. Limited manpower availability.

c. 48 to 72 hours warning time.

d. Materials available for use within the stated warning time.

2. Powered ventilation systems are not considered since the occupant density is expected to be low.

3. The upper stories are considered to be seriously damaged or removed during the passage of the blast wave, with at least part of the resulting debris falling on the floor above the basement. This debris will provide some added degree of fallout radiation protection.

4. The floor above the basement may receive damage to the extent of incipient collapse or actual collapse around one or more supports, forming lean-tos or tepees.

5. The expedient concept implies that a rigorous structural engineering evaluation of the upgrading options is not made. Instead, upgrading activities are to be applied in a progressive fashion in some order of priority using the principles, methods, and materials described or shown in this report.

Vulnerability Problems

Basement space provides the most desirable shelter from combined nuclear effects since the air blast wave will pass without creating amplified (through reflection) loads on the basement cover slab. The resulting "side-on" pressure is much less than that felt by the aboveground walls facing the path of the blast wave. In addition, the basement columns (if supporting aboveground columns) will have a strength much greater than that required to support the normal loads on the basement cover slab alone. Thus, the basement space in a multi-story building may have substantial air blast resistance.

A. Closed Shelter

Most basements, however, are characterized by numerous openings in both walls and ground floors, which will permit air blast wave entry. For any real degree of blast protection, these openings will require blocking to prevent the entry, or at least provide for substantial reduction, of the blast wave. Such blocking will also provide a measure of fallout radiation protection through shielding and exclusion of most or all fallout contaminant.

The blast loading on the floor above the shelter space may generally exceed the floor strength by values ranging from several times to an order of magnitude. Thus, structural strengthening of such floors will be generally the primary expedient upgrading need.

B. Open Shelter

Because of the lack of more suitable shelter in the proximity of the function, or the presence of large openings in the available shelter, an "open" shelter may have to be accepted. Blast protection, however, will be generally less than that offered by closed shelter. Vulnerability problems in open shelter are increased because of air blast flows within the shelter. These flows are unlikely to reduce significantly the net loading on the floor above the shelter space.

Chapter 3

EXPEDIENT UPGRADING PROTECTION PRINCIPLES

Introduction

The provision of effective, expedient shelter space by upgrading requires the application of some or all of the following protective measures:

- Prevention of air blast entry (closed shelter mode).
- Air blast loading reduction on basement exterior surfaces.
- Air blast structural strengthening.
- Provision of last resort protection from debris and failed floors and walls.
- Provision of additional radiation protection.

Although the crisis situation and expected warning time may permit some pre-planning, and perhaps assembly of materials and tools, it should generally be assumed that such prior activities are not available. Thus, expedient protective measures should be accomplished on a priority basis under the principle that an increasing degree of protection is attained with additional available time. For most structures, the proper priority of application of expedient protective measures will be in the order listed above. Principles applying to these expedient measures and others are briefly discussed in the following paragraphs and some are discussed in more detail in the next chapter.

Prevention of Air Blast Entry (Closed Shelter Mode)

Essentially all basement space will have openings closed by lightweight windows, doors, and the like. An effective degree of blast protection will require the upgrading of these openings to resist some degree of air blast. The basic principle to be employed in the expedient upgrading option is to block these openings (or at least all of them except one or two small ingress/egress openings) rather than attempting to provide structural alterations, which would be too time consuming. Openings to be considered include windows, window-wells, areaways, cargo and other doors, ventilation wells, and other openings in walls; and stairwells, elevator shafts, and ventilation and other building utility penetrations in the basement cover slab.

Air Blast Loading Reduction on Basement Exterior Surfaces

Although shelter space should be chosen in buildings whose basements are completely underground if possible, the choice will be restricted in some cases to buildings wherein some portion of the basement wall is aboveground. This aboveground portion is subject to reflective air blast loading amplification and should be upgraded in some manner to reduce the air blast load.* The basic expedient principle used entails adding exterior infill material against the exterior exposed wall, to reduce the reflective amplification loading and to transfer some portion of the air blast load into the ground. The infill should be as broad and at as flat a slope as possible consistent with the time and materials available.

Air Blast Structural Strengthening

Air blast strengthening opportunities will generally apply to the floor above the shelter space. An examination of many buildings indicates that the relative strengths of existing components of floor systems vary considerably depending on the type of construction, but can be taken to be in the following order (stronger to weaker) for concrete- and steel-framed buildings for the purposes of expedient upgrading in the absence of a specific structural evaluation.

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* For usual R/C basement walls, however, upgrading (infilling) may not be required. Assuming that basement and first floors are strong enough to take the wall reactions (and assuming 1 Mt, horizontal/vertical soil coefficient of 1/3, 2-story wall of 10 ft height each floor, and 2-way rebars at Code minimums), "design" strengths might be as follows (using 2/3 of incipient collapse mean overpressure estimated values):[1]

Exposed wall portion	Peak incident (free-field) overpressure capacity ("design") 8" thick R/C wall	12" thick	16" thick
0 %	12 psi	26 psi	45 psi
20	9	19	32
40	7	14	24

Infill may still be required to increase radiation protection, however, even if not needed for the blast loading on the R/C walls.

Concrete-framed Buildings

Columns* and footings†
Floor slabs
Joists
Beams/girders

Steel-framed Buildings

Columns and footings†
Floor slabs
Girders‡
Beams

In the light of the relative component strengths indicated, air blast upgrading measures should be applied in priority in reverse order to the listings.

The general principle to be employed in air blast upgrading is to reduce the spans of girders, beams, and slabs by the introduction of temporary support. This temporary support will also aid in reducing the possibility of massive collapse of the entire floor system.

In some types of building systems, particularly where precasting techniques have been applied, the supporting end connection means between floor system and walls may be a weak point, with a small building motion allowing the loss of support for the floor system. The principle to be applied in these cases is to provide additional temporary vertical support area at horizontal member ends, by means of expedient column/beam systems placed against the wall surface.

Debris Protection

Higher fundamental degrees of air blast protection may be obtained by providing some final protection within the shelter space, to reduce

* For flat plate floor systems, the weak point will be punching shear between the slab and column.

† Columns and footings may need strengthening or not, depending on how many floors are above the shelter and what normal-use floor and lateral loadings were designed for.

‡ The reason for the relatively higher strength in girders for steel-framed buildings is the requirement that they provide stiffness for rigid-frame connections between girders and columns, or at least moment resisting connections.

the possibility of danger from collapsing floors and debris from upper floors and other buildings. The principle to be applied is to install a secondary structure, objects, or materials within the shelter.

Additional Radiation Protection

Higher degrees of direct and fallout radiation protection may be attained by adding dense material to the floor above the shelter or by providing radiation shielding within the shelter.

Open Shelter Protection

Where time and other circumstances require the use of basement shelters with large openings that cannot be blocked,* some measure of air blast protection can be attained by one or both of the following methods:

1. Location of occupied space in areas remote from openings or places where air blast flows are expected to be strongest. Such location takes advantage of air blast attenuation or swirling within the structure. Occupied space near the center of the structure may generally be the most suitable.

2. Provision of expedient air blast shelter within the basement space with priority given to alcoves, offsets, corners, areas enclosed by reinforced concrete walls, and the like.

Other Protective Measures

Other expedient protective measures apply to the provision of emergency exit after the attack and to the prevention of flooding or fire secondary effects.

The most appropriate method of provision for emergency exit is to design and install opening blocking systems that can be removed by the shelter occupants with the tools available. In some cases the provision of special expedient emergency exit systems will be required or be desirable. In these cases all materials must also be removable by the occupants with the tools available.

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* Such as in underground parking garages.

Prevention of flooding and fire secondary effects entails the shut-off and draining of water supply lines, standpipes, gas lines, etc. It is to be assumed that the utility* and space conditioning systems of the building can be closed down during the 48 to 72-hour warning time.

* *Except for electrical service.*

Chapter 4

METHODS AND MATERIALS

Prevention of Air Blast Entry (Closed Shelter Mode)

Expedient methods of prevention of air blast entry into basement shelter space should provide for blockage of some form, which is supported by exterior elements of the structure or ground where possible. This will eliminate the requirement for massive structural support from inside the shelter space. Horizontal openings should be protected under the same principle where possible.

The more common openings into basement space include:

- Doors and windows, with or without window-wells and areaways.
- Truck cargo ramps with docks and loading doors.
- Ventilation openings, either through-the-wall aboveground or via a ventilation well.
- Elevator shafts.
- Stairwells.
- Floor penetrations for building utilities and air handling duct-work.

The following paragraphs discuss and illustrate expedient methods and materials for blocking the openings listed. The same methods and materials may be used for other types of openings.

This phase of the study (expedient options) does not consider the strength or air blast resistance requirements of blocking systems although such guidance is obviously necessary, even for expedient systems installed with very little preparatory time.* Guidance in this area will be included in the planned second phase (engineered upgrading) of the research effort.

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* An exception is the use of wood members, usually used flat-wise, which may be quickly sized (designed) through use of design charts, fully published in both References 2 and 3 (Chapter 6, Section G), and extracted herein as Appendix B.

A. Window and Door Openings

Where possible, selected basement shelters will ideally be totally below the exterior ground surface, thus having no openings to the exterior, or will be in a portion of the basement without exterior openings. Few such opportunities are expected to exist, however, and some degree of exterior opening blocking will ordinarily be required. The most common opening will be a window or door with or without a well or areaway.

Figure 1 represents a window situation that will commonly be encountered in many buildings with basements. The method of blocking consists of providing a closing structure held in place and buttressed by other materials. The buttressing not only holds the closure materials in place but also serves to reduce the blast loading on the closure through reducing reflection factors and transferring part of the loading onto horizontal surfaces. The buttressing material should extend well beyond the opening and be tapered at its ends to provide a streamlined structure when presented with a blast direction parallel to the building wall.

A large variety of closure and buttressing materials may be used in combination for closures at openings of this type. Alternative suitable materials are listed in Figure 1.

B. Ventilation Structures

Another form of basement wall opening will be found where sizeable air intake or exhaust structures lead to basement installed heating, ventilating, or air-conditioning (HVAC) equipment. A typical structure of this type is illustrated in Figure 2; an alternative form of blocking (as compared to Figure 1) is shown in which the blocking is installed within the ventilation well. This blocking system can be improved through filling the well with loose materials such as soil or sandbags.

Alternative materials listed in Figure 1 are also applicable in this case.

Another alternative blocking method would be to place closure materials over the grill and cover with loose materials such as soil or sandbags. The covering materials should be tapered in all directions in order to reduce reflection forces and to transfer a portion of the horizontal load into the ground around the opening.

Some ventilation openings will have a configuration similar to the window opening shown in Figure 1 and should be treated in the manner shown there.

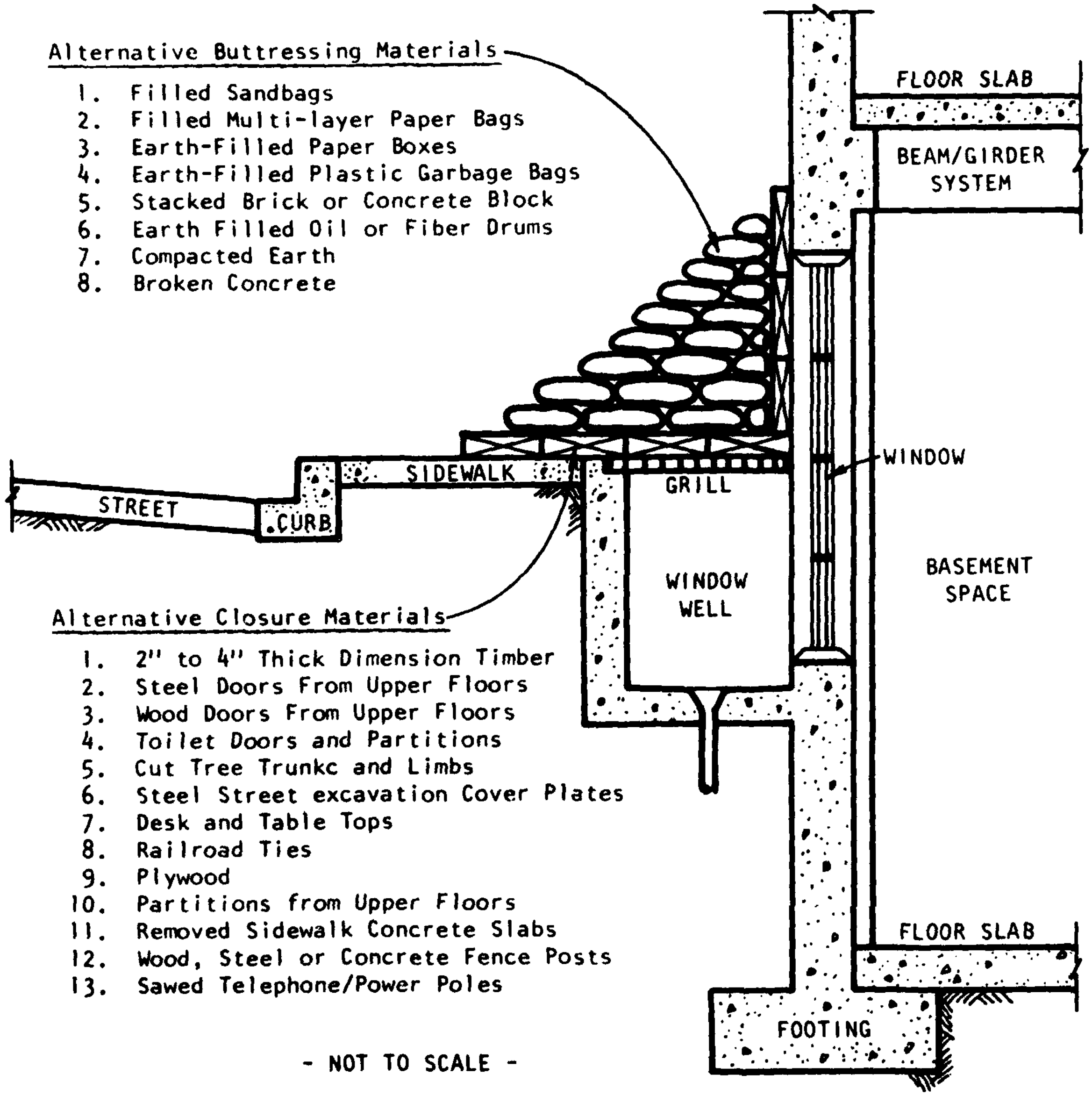
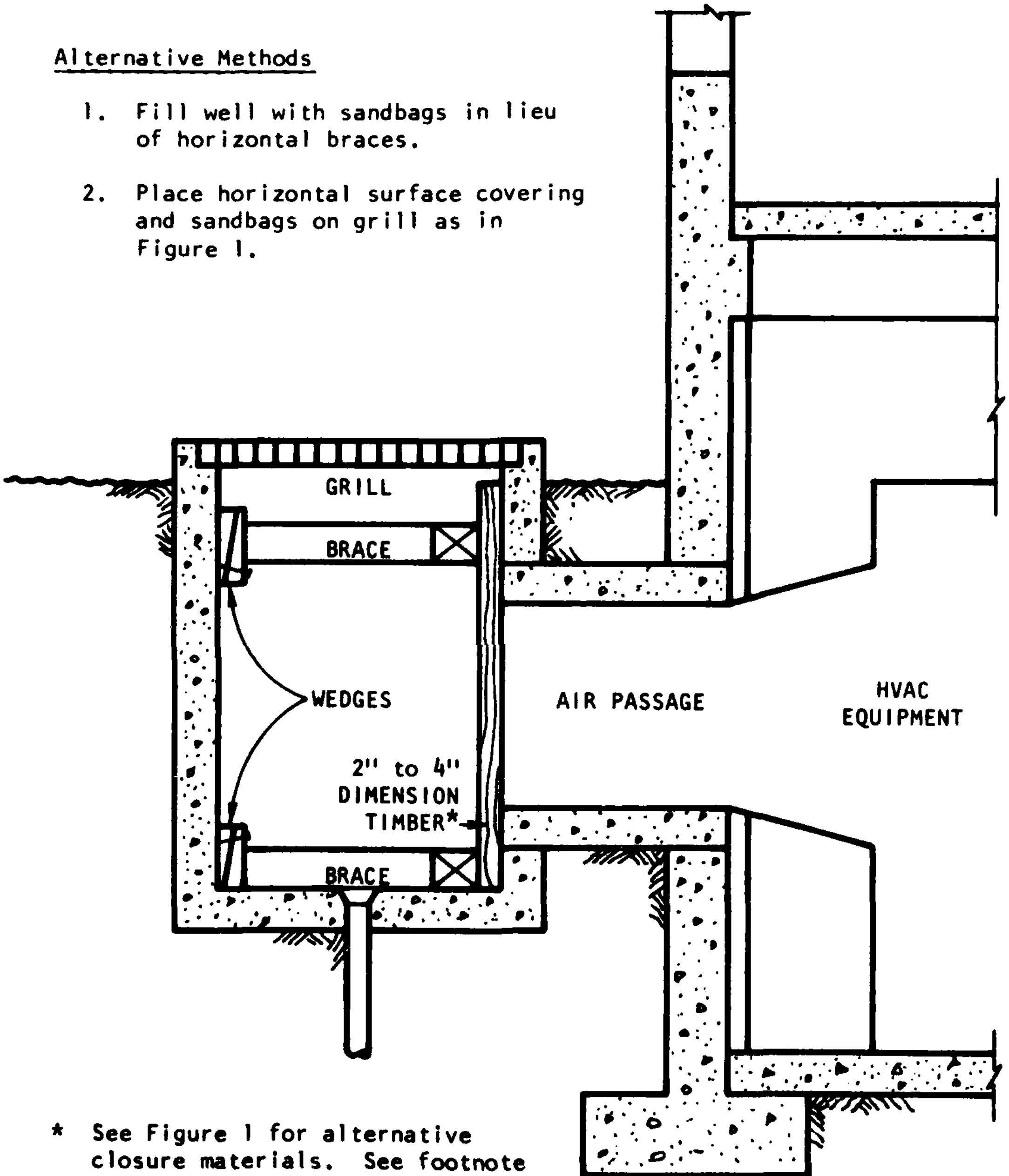


FIGURE 1 EXPEDIENT BLAST PROTECTION FOR BASEMENT WINDOWS WITH OR WITHOUT WINDOW WELLS

Alternative Methods

1. Fill well with sandbags in lieu of horizontal braces.

2. Place horizontal surface covering and sandbags on grill as in Figure 1.



* See Figure 1 for alternative closure materials. See footnote p. 15 regarding chart design of wood members.

\- NOT TO SCALE -

FIGURE 2 EXPEDIENT BLAST PROTECTION FOR VENTILATION STRUCTURES

C. Truck Loading Docks

Most commercial and other structures require accessibility for loading and unloading cargo from trucks. Where this is accomplished at the basement level, a ramp and loading dock facility will exist. Even where shipping-receiving is not a requirement, relatively large openings are still required to permit replacement of HVAC equipment, and other items. These large openings will present the most serious blocking problem for most buildings and should be given priority attention if the shelter is to be closed mode. A large variety of ramp, loading dock, and door opening sizes, as well as door types, will apply. Figure 3 shows a typical ramp dock and door, and one method of providing blocking.

Alternative blocking methods would include:

1. Spanning the truck ramp side walls (if present) with round or rectangular timber placed close together and covered with sandbags.

2. Placing large timbers from above the door to the pavement in a "lean-to" fashion with heavy timber plank covering.

3. Blocking the door opening with light materials and filling the entire ramp area with compacted earth.

The alternative materials listed in Figure 1 would also apply to the truck loading dock situation; however, the span requirements of this situation will demand the use of the largest available wood dimension materials.

The door used to close the cargo opening may be of a variety of types. A typical roll-up steel door is illustrated. This type of door will have very little blast resistance and should not be depended on for any degree of blast protection by itself.

D. Elevator Shafts

Buildings above 3 to 4 stories including basement will ordinarily be served with elevators, either passenger, freight, or both. Hydraulic elevators are ordinarily used below about 5 stories (including basement), and cable elevators for higher buildings. The openings in the floor for the elevator shafts will be one category of major horizontal openings requiring blast closures for buildings so equipped.

One method of providing closure for the elevator shaft opening for a cable-type elevator is illustrated in Figure 4. Alternate materials that would be applicable are listed in Figure 1.

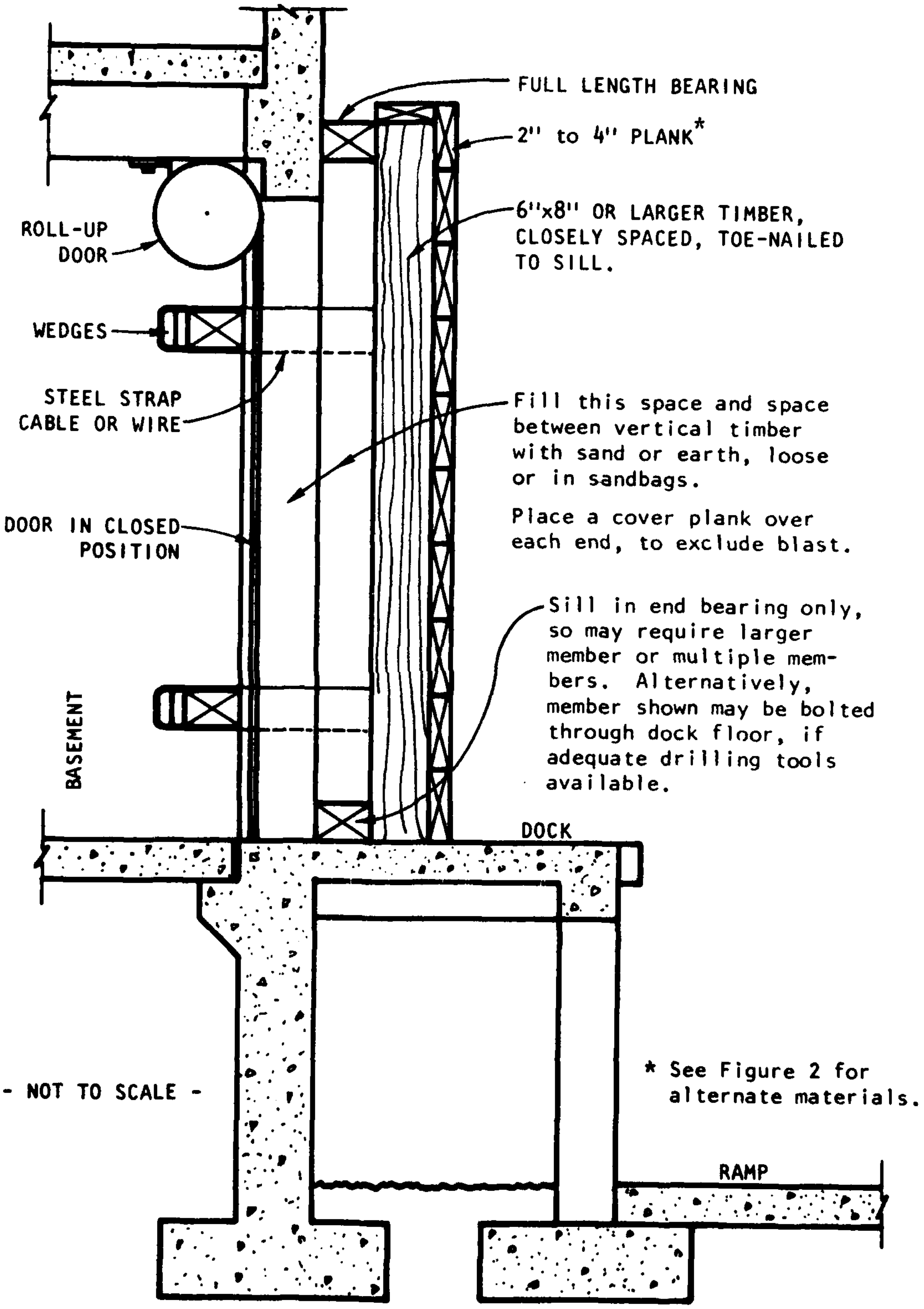
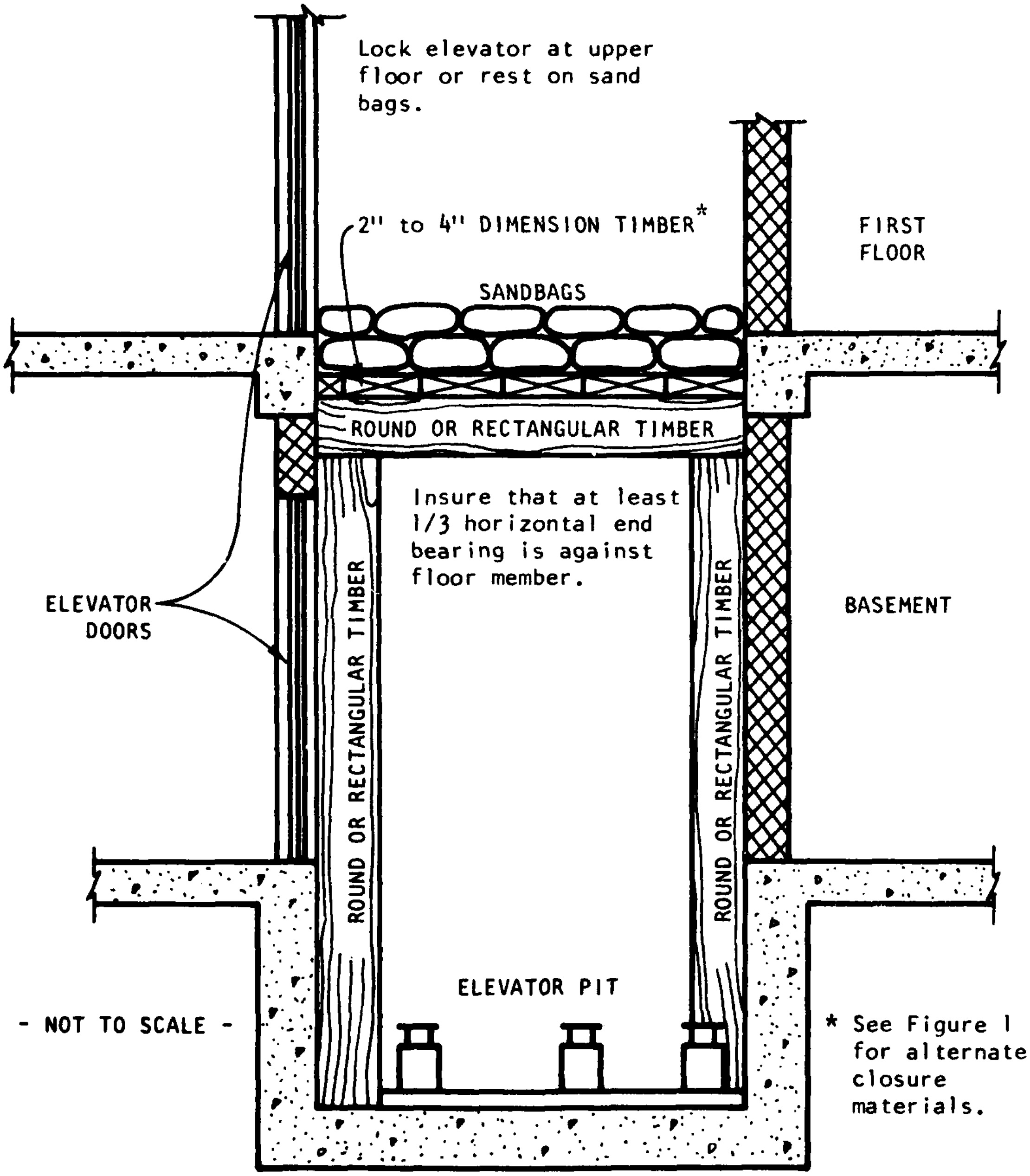


FIGURE 3 EXPEDIENT BLAST PROTECTION FOR A TRUCK LOADING DOCK



APPLICATION EXAMPLE:
Assuming elevator shaft opening 5'x6' and 6x6 structural grade timbers, reasonable working stresses, and short-way bents, plus 2x4s flat-wise for decking: 2 end and 1 intermediate bents, sandbagged closure offers about 7 psi; same but 2 intermediate bents, about 11 psi. For no closure, CMU wall offers only about 1/4 to 1/2 psi.

FIGURE 4 EXPEDIENT BLAST PROTECTION FOR ELEVATOR SHAFTS

An alternate method (less desirable than illustrated) would be to form holes at basement cover slab level (on top) in the elevator shaft walls,* place beams through these holes (supported on the floor), and cover the beams inside the shaft with plank or one of the alternative closure materials listed in Figure 1. The closure materials may also be held down with sandbags which should be placed in a streamlined manner to reduce air blast drag forces.

Hydraulic elevators will require careful sealing around the piston shaft. This can be best accomplished with sandbags. If the alternate method is used, the elevator can be placed at the basement level and the piston problem avoided.

E. Stairwells

Stairwells are one of the most difficult types of openings to close because of their size and complexity. Stairwells are of many different forms, and a variety of closure methods must therefore be considered. The basic principles to follow in closing a stairwell opening are:

1. A plane of closure that has the minimum possible shorter closure dimension should be selected. This plane may be horizontal, vertical, or at some other slope.

2. The strongest possible support must be selected for the closure system. Thus, closure systems supported by floor beams (horizontal systems), or reinforced concrete wall pilasters (vertical systems), would be favored. Systems supported by concrete block, other masonry, and frame walls should be avoided.

3. A system should be selected which provides side-on, rather than face-on, loading in order to reduce reflective loading forces.

4. Exposed elements should be streamlined in order to reduce drag loading forces.

It will be seldom possible to optimize all of these desirable conditions; however, as many as possible should be provided.

Figure 5 illustrates one method of blocking a stairwell opening. The type of stairwell construction illustrated in this case consists of a

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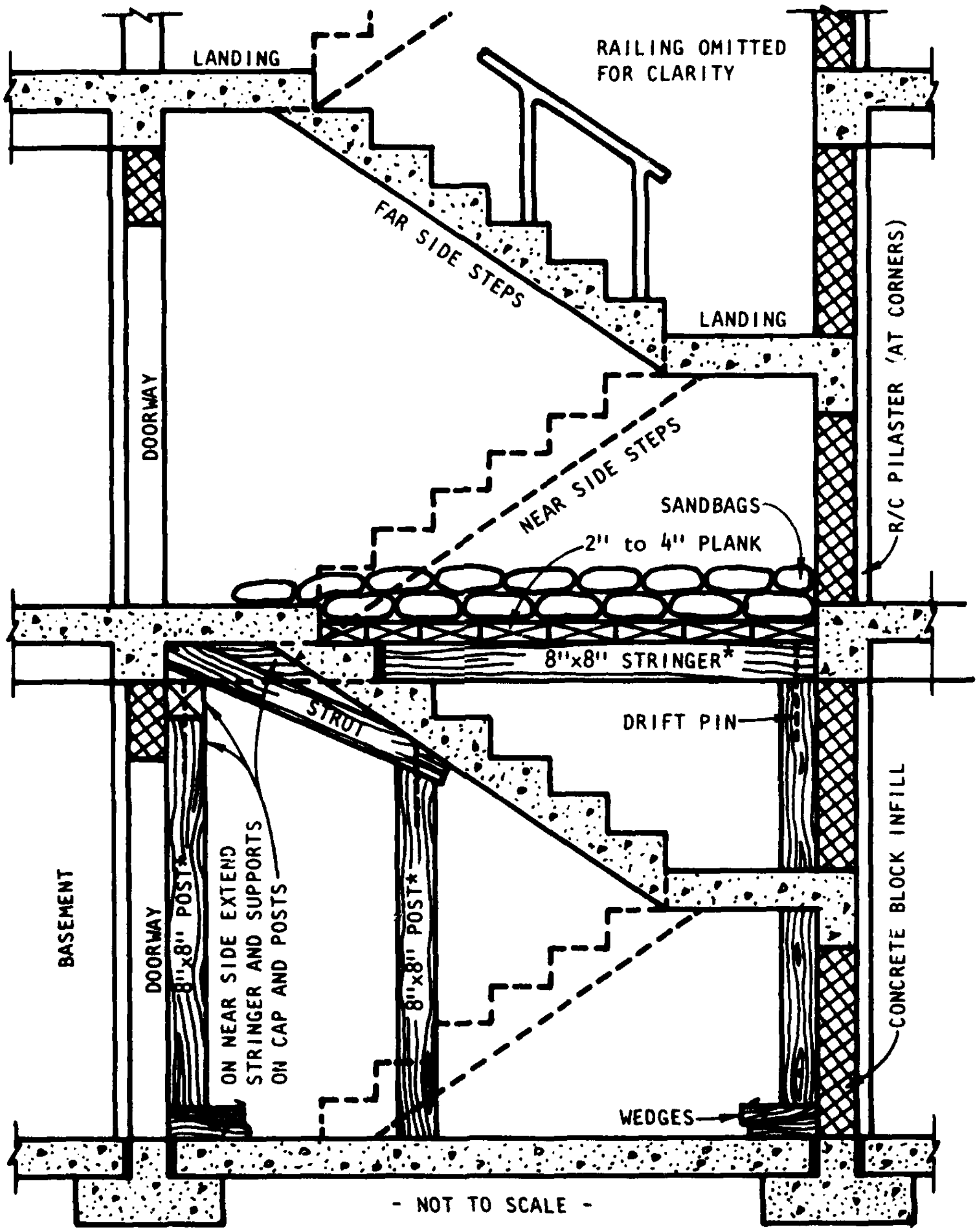
* Elevator shafts in all but reinforced concrete core buildings will have walls of concrete block, gypsum block, gypsum panel, or frame construction, and such holes may be readily formed.



Alternative Materials

1. Closure (See Figure 1)
2. Support
   a. Utility Poles (See Figure 8)
   b. Railroad Ties (See Figure 9)
   c. Tree Trunks

Note: Addition of vertical closure to door (See Figure 3) will provide "Last Resort" or open shelter protection.

* Dimensions shown are for illustrative purposes only.

FIGURE 5 EXPEDIENT BLAST PROTECTION FOR STAIRWELLS reinforced concrete floor slab-beam-girder-column system with concrete block masonry stairwell closure walls and cast-in-place reinforced concrete stairs without an interior dividing wall.

Alternative closure materials listed in Figure 1 apply to this case.

An alternative method of blocking would be to provide a vertical system similar to that shown in Figure 3 for the truck loading dock, with the system placed inside the stairwell. Another alternative is shown in Chapter 5 (Section II) under the Livermore EOC case study.

Another method would be a horizontal system covering the entire stairwell as illustrated and described for elevator shafts in the previous section. The system can be supported from below as illustrated in Figure 4 or by the floor system (where non-reinforced concrete walls are present).

In high buildings having reinforced concrete stairwells of sufficient thrust strength, the most suitable blocking system will be one in which the lower door is blocked by methods similar to that illustrated in Figure 3 with the planking on the inside of the stairwell. The lateral support provided by the reinforced concrete stairwell walls can be expected to be substantial in these cases, since they are supporting an axial load from higher floors.

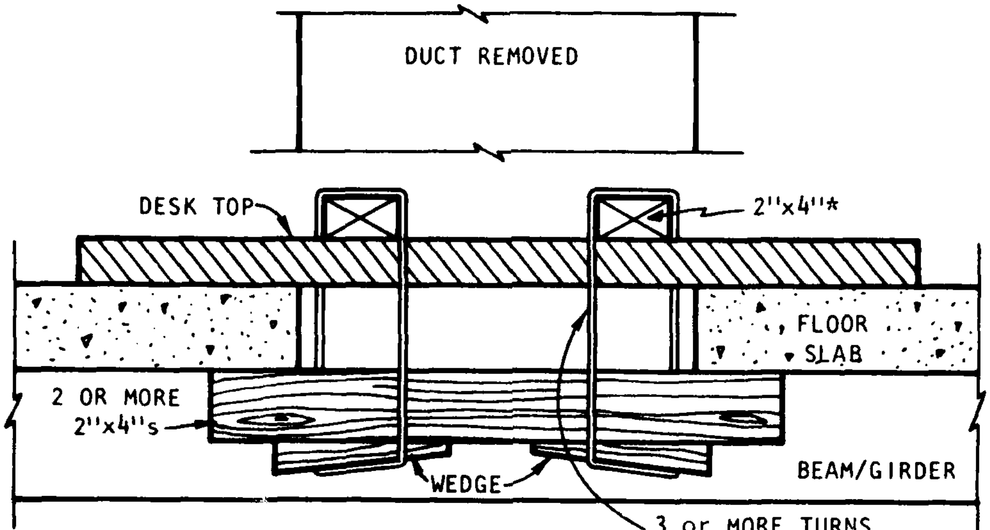
F. Utility Penetrations

A variety of relatively small penetrations will be present in most building floors over basements. To prevent air blast entry, these penetrations should be blocked in priority, with the largest opening covered first. The larger openings may be considered as means of emergency exit, and the blocking system, although on top, should be removable from the underneath side.

Figure 6 illustrates methods for blocking both large and small openings. For the larger openings, the alternative materials listed in Figure 1 will apply.

Air Blast Loading Reduction on Basement Exterior Surfaces

In some situations, higher degrees of protection from air blast may be attained by reducing the air blast loading on exposed portions of the basement shelter space. The critical condition affecting air blast loading arises where the basement wall is exposed to some degree above the

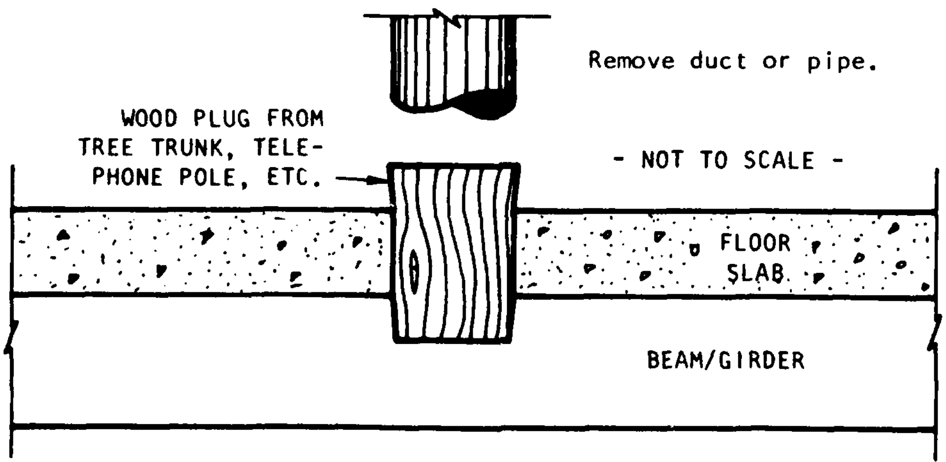


Alternative Cover Materials

1. Plywood (Several Layers for Larger Openings
2. Dimension Timber
3. Toilet Doors or Partitions

- NOT TO SCALE -

A. LARGER RECTANGULAR/CIRCULAR OPENINGS



B. SMALL CIRCULAR OPENINGS

* Dimensions are given for illustrative purposes only.

FIGURE 6 EXPEDIENT BLAST PROTECTION FOR FLOOR PENETRATIONS surrounding ground or street level. In the face-on blast direction, the exposed wall receives reflective amplification of the blast wave and substantially higher loadings than in the side-on case (e.g., 42 psi versus 15 psi, respectively). The opportunity therefore arises to reduce the reflective loads by providing a streamlined infill of some material.* This principle is illustrated in Figure 7. The streamlined infill not only reduces reflection factors but also transfers a substantial portion of the resulting loads into the ground exterior to the building. Additional radiation protection is provided as a bonus.

Other opportunities for reducing air blast loading lie in the provision of streamlined shapes for all air blast blocking systems, as described in the previous sections, in order to reduce air blast drag loading forces.

Air Blast Structural Strengthening

The previous measures of prevention of blast entry and reduction of air blast loading provide some measure of reduced vulnerability for the basement shelter space. Higher degrees of protection of the space from air blast will depend on strengthening of the basement structure itself. Expedient methods for providing such strengthening are discussed in this section. The structural portions considered are exposed wall areas and the floor system over the basement space.

G. Exposed Wall Areas

Although a building with non-exposed basement walls should be chosen for shelter wherever possible, use of a building with some degree of exposed basement wall will be dictated by circumstances in many cases. Passive measures, such as locating the space selected for shelter as far from the exposed wall as possible, are pertinent; however, it will be beneficial to strengthen the wall in many cases. Figure 8 illustrates the principles to be employed in providing such strengthening. Provision of strengthening will be most applicable in cases where the wall is constructed of brick or concrete masonry (as opposed to a reinforced concrete wall).

* See footnote to identically titled section of preceding chapter.

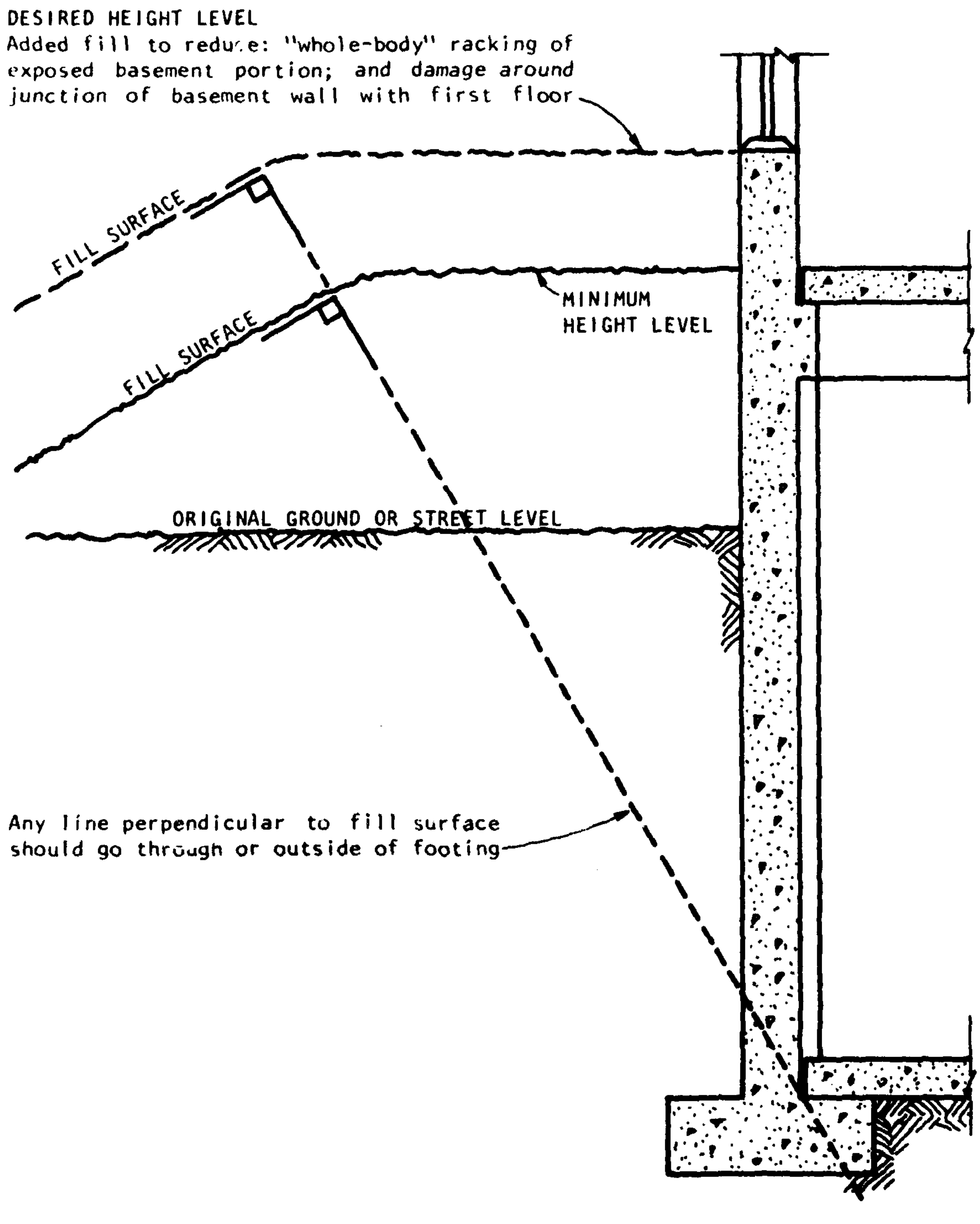


FIGURE 7 REDUCTION OF BLAST LOADING FOR EXPOSED BASEMENT WALLS

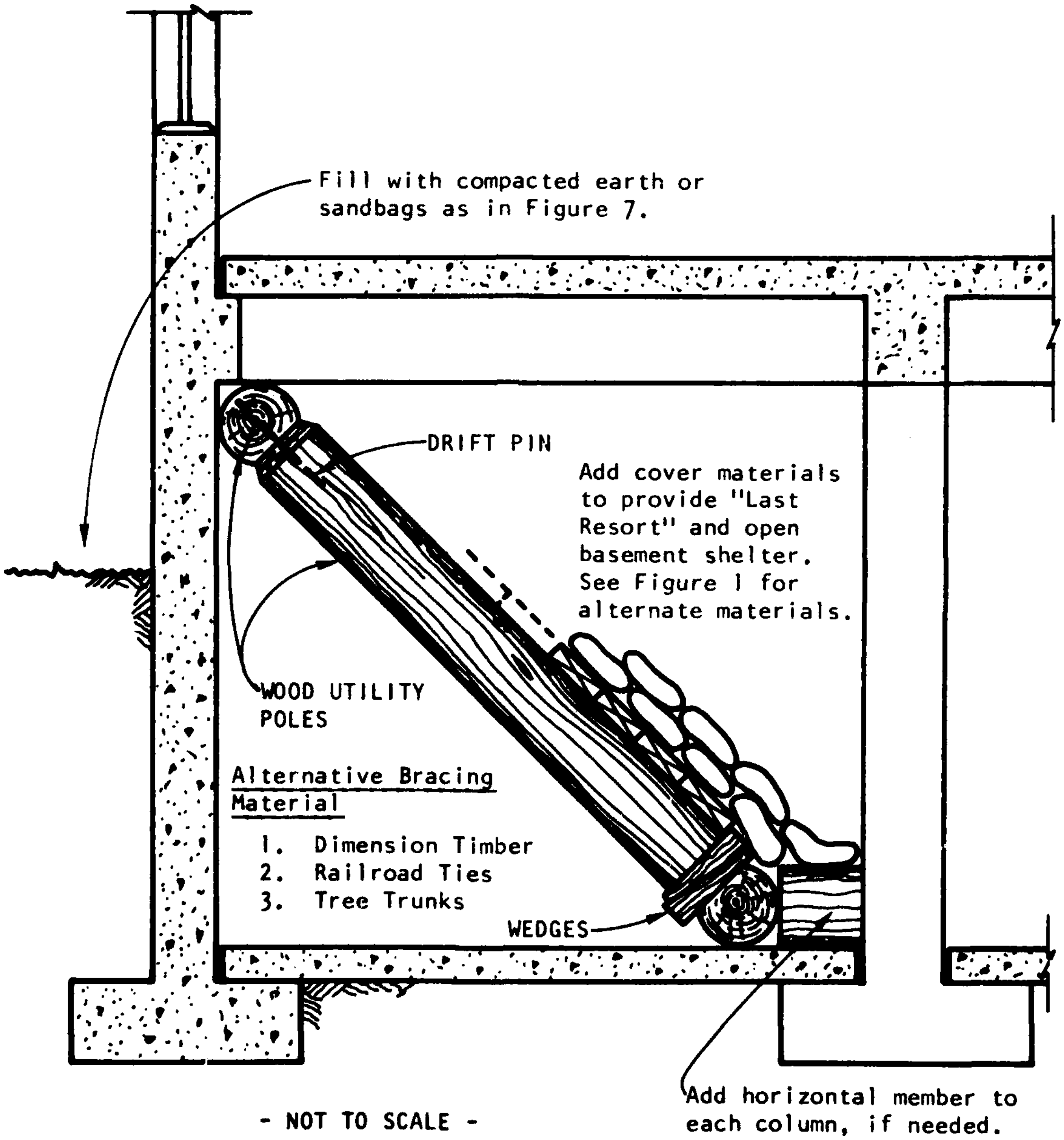


FIGURE 8 STRENGTHENING EXPOSED BASEMENT WALLS

H. Floor Systems

Expedient strengthening of floor systems provides opportunities for substantial upgrading of basement shelter space. Vulnerability problems associated with floor systems include:

1. Possibility of loss of support and collapse for beam and girder ends, particularly in precast construction.

2. Possibility of column collapse, particularly for low, lightly loaded buildings.

3. Possibility of shear failure between floor system and columns, particularly for flat plate floors and lift slab construction.

4. Possibility of failure and collapse of floor slab-beam-girder systems.

Floor system strengthening should progress, as time allows, from the area immediately over the selected shelter space to the entire floor system.

Expedient methods of reducing the air blast vulnerability problems are discussed in the following paragraphs.

Provision of End Support. Where reinforced concrete or steel floor girders are not well anchored to wall columns or pilasters, or where short ledge support is provided (as in precast concrete "T" beam floor construction - commonly used in parking garages), there is danger that whole building or basement wall motion will dislodge the support, permitting catastrophic collapse of the entire floor system. An expedient means of providing suitable assurance against such an event is illustrated in Figure 9.

Strengthening Columns. Where indications are that columns are the weak point, additional vertical support can be provided by adding expedient timber or steel columns, properly wedged to prevent dislodgement under total building motion. A method of installation is shown in Figure 10. This method also applies to cases, such as for flat plates and lift-slab construction, where column to slab shear strength may be the weak point.

Strengthening Slab-Beam-Girder Systems. Substantial additional strength may be added, by the provision of intermediate support, to slab-beam-girder systems. The intermediate support should be installed as illustrated in Figure 11. Such strengthening should be provided in

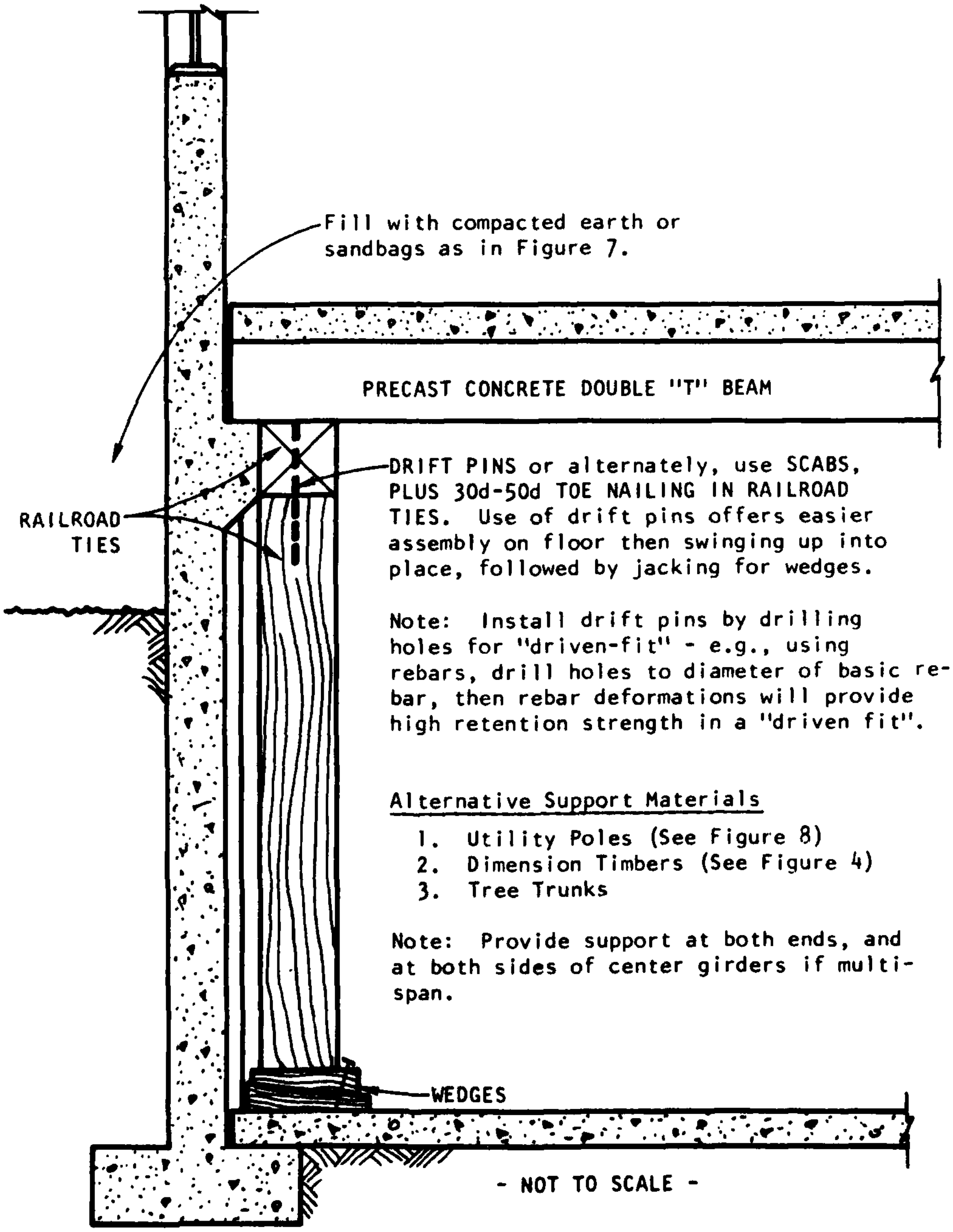
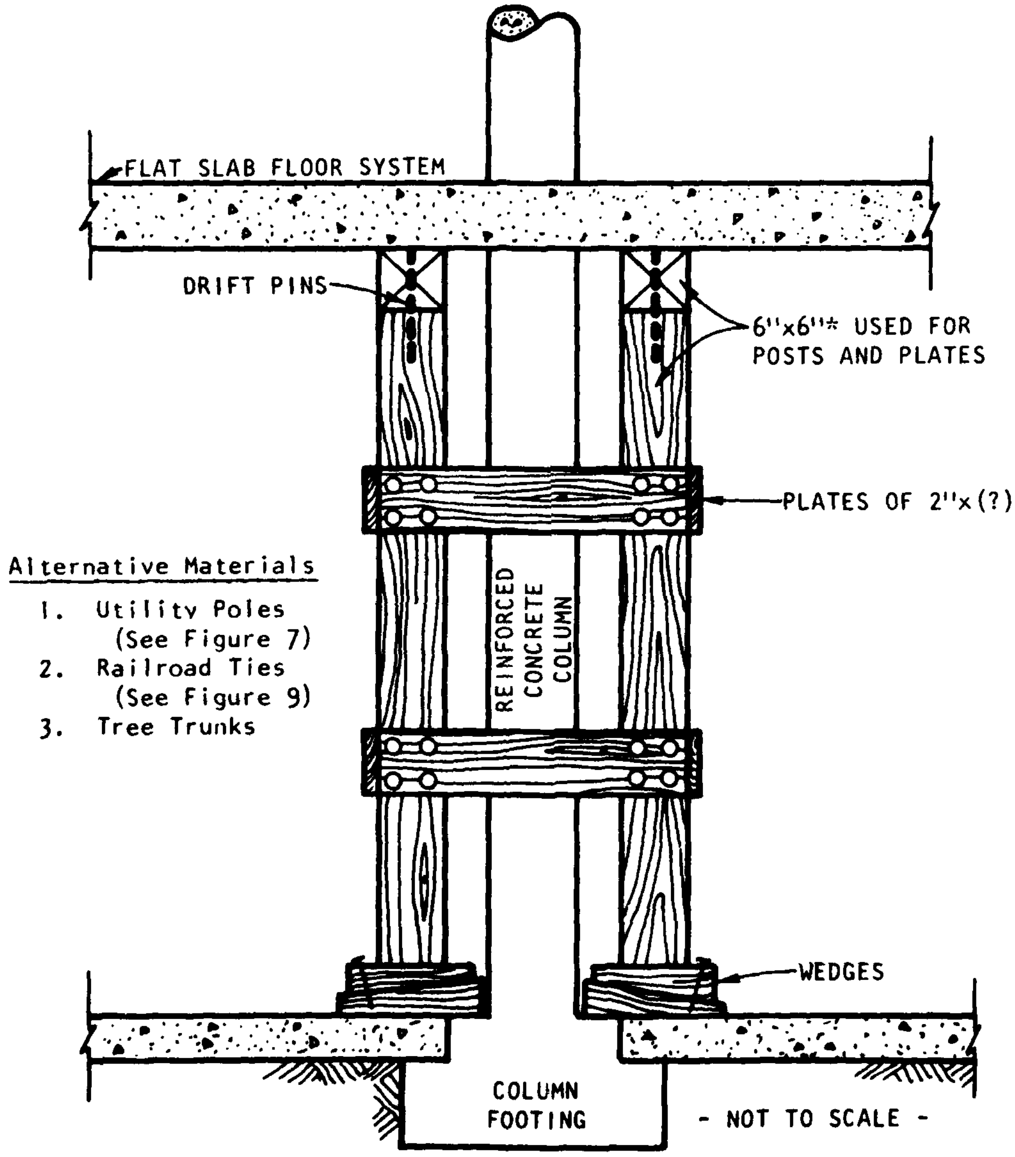


FIGURE 9 PROVISION OF END SUPPORT



* Dimensions given for illustrative purposes only.

FIGURE 10 ADDITIONAL COLUMN SUPPORT

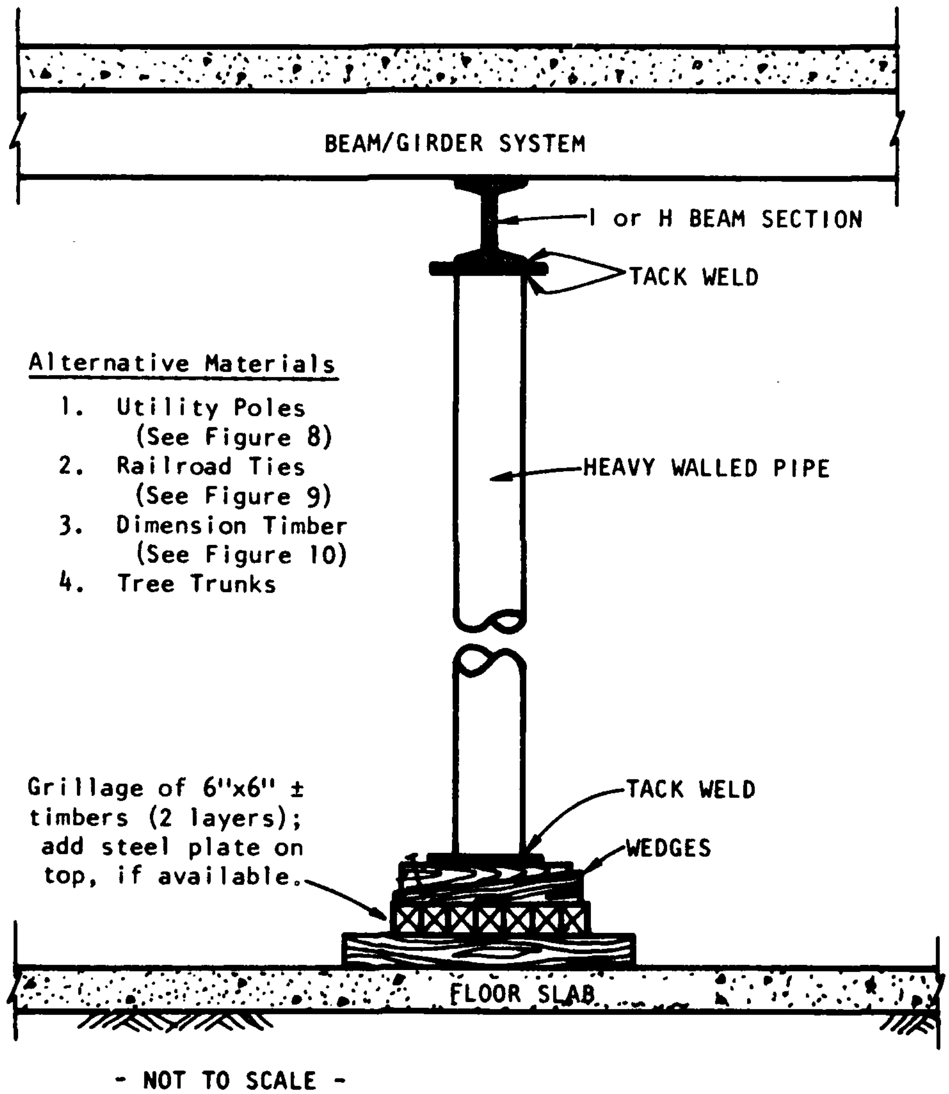


FIGURE 11 SLAB-BEAM-GIRDER STRENGTHENING

priority depending on the type of construction, as discussed earlier (Chapter 3). In addition, such strengthening should progress from the immediate area selected for shelter to the whole floor system as warning time allows.

Debris Protection

If time permits, some final degree of protection should be provided in addition to the previously discussed measures for blast entry prevention, blast loading reduction, and blast strengthening. This "last resort" protection is to provide some potential of survival where blast loading is such as to cause major portions or the entire floor system to collapse, or where falling debris is sufficient to collapse the floor system.* A variety of methods is available for provision of final protection, depending on availability of materials and the available time. The more effective measures are listed below.

1. Construction of a "lean-to" in a corner of the shelter with the open end(s) of the lean-to blocked by sandbags.† The lean-to should utilize the heaviest timbers available, with the lower ends secured as in Figure 8 to prevent dislodgement. Such a shelter may also be combined with the strengthening system illustrated in Figure 8 to provide both exposed wall blast strengthening and final (or last resort) protection.

2. Construction of a cubical shelter in a corner of the basement space from concrete block, brick, or similar available materials with a heavy timber roof. Again, the entryway should be blocked with sandbags.†

3. Construction of similar final shelter from desks, file cabinets, doors, or other materials available in the building.

4. As a last resort, some means of providing a stop for collapsing floor systems may be provided by the placement of any available large hard objects at intervals in the specific area to be occupied. These objects should be high enough to provide at least crawl space (say 24 in.) after the floor system has collapsed. Objects for this purpose (in the approximate order of desirability) include:

   a. Stacked concrete block or brick

   b. Stacked timber

* The latter situation is likely to occur only in load-bearing wall buildings.

† In open shelter mode, the sandbags (or earth-filled paper bags or similar means) should be in a sloping pile, having sufficient mass to resist sliding movement and withstand bursting or tumbling by the interior air blast.

c. Boxes of solid packed paper

d. File cabinets (on backs)

e. Desks (with legs removed)

f. Automobiles (in parking garages)(preferably with gas tanks drained and filled with water)

Open Shelter Protection

The presence of large vertical/horizontal openings in the only available buildings, or the limitations of time, or both, will require the use of an "open" shelter in some instances. Although providing a lesser degree of blast protection than "closed" shelter, open shelter situations can be upgraded considerably. Increasing degrees of protection should be attained through the accomplishment of the following measures, listed in order of priority:

1. Location of space within the basement to avoid blast concentration.

2. Provision of internal blast protection; if open to blast, removal of all inadequately anchored objects.

3. Blocking of such vertical and horizontal openings as time allows.

4. Air blast loading reduction on basement exterior surfaces.

5. Air blast structural strengthening.

6. Debris protection.

Measures 3, 4, 5, and 6 are similar to those discussed for closed shelter conditions in the previous paragraphs. The principles to be employed for measures 1 and 2 are discussed in the following paragraphs.

I. Avoidance of Blast Concentration

Depending on the size and location of the structural opening, the configuration of the shelter space, the direction and strength of the blast wave, and other factors, an air blast wave entering the open shelter may be reinforced or concentrated in the form of jets, vortices, and multiple reflections.[4] First priority must be given to location of occupants in an area(s) within the open shelter where these effects will be at a minimum. Since an infinite number of situations are possible, general principles only can be stated. These are:

1. Locate the space to be occupied as far away from, and as far as possible from a line normal to, the opening. Assume that a blast wave encountering a wall will turn and follow along the wall.

2. Avoid location in a closed end space where multiple end and side wall reflections may be severe.

3. Avoid location immediately behind large objects (such as elevator or stairwell shafts) where vortex effects may be severe.

4. Avoid location near walls, where a large opening is near a sidewall, in order to avoid the "swirl" effect of the wave movement under such conditions.

J. Provision of Internal Blast Protection

Open shelter situations may be improved considerably by the provision of internal blast resistant space. The opportunity for providing such space will be related to:

1. The presence of relatively "hard" enclosures within the basement, which can be upgraded by blocking and strengthening as described for closed shelter conditions in previous paragraphs. Potential enclosures that might be upgraded include: (a) sidewalk vaults, (b) reinforced concrete stairwells and elevator shafts, (c) photographic film and record storage fireproof vaults, (d) hard-walled electrical transformer vaults and communication terminal vaults, and (e) any other room or space with reinforced concrete or reinforced masonry walls.

2. The availability of materials, labor, and time for the erection of expedient blast shelter within the basement space. Such shelters would be located within the basement space to avoid the blast concentrations discussed above. Alternative types of shelters include the lean-tos, cubicle structures, and other types of "last resort" protective systems as described in the preceding section, Debris Protection.* In the open shelter case, special attention should be given to streamlining the internal structure in order to reduce air blast reflective and drag forces resulting from the movement of the blast wave within the shelter space.

* References 5 and 6 illustrate various types of shelters that, although designed primarily for fallout protection, could be suitable for some degree of open shelter or "last resort" blast and debris protection, and might be modified to provide higher degrees of protection.

Post-attack Considerations

The protective measures previously discussed will provide some probability of survival from the blast and radiation effects of the attack. The problem remains, however, of assuring survival during the immediate post-attack period when fallout radiation intensities and danger from fire will be very high.

Food, water, and sanitary measures are not considered in this project; however, there are several other critical post-attack considerations which are associated with the expedient combined effects measures previously discussed. These are:

- Ventilation
- Lighting
- Communications
- Emergency exit
- Additional fallout radiation protection
- Fire protection.

K. Ventilation

Ventilation considerations are not expected to be critical for the types of shelters and their uses considered herein, because of probable low density occupancy and the expected presence of small openings, especially if the structure has been damaged to some degree. There will be cases, however, where ventilation will be necessary, or at least desirable. Expedient systems of ventilation would therefore need to be provided for these cases and for use during the "buttoned-up" pre-attack phase. Three alternative types of expedient ventilation systems are:

1. Construction of manually operated "flap-valve" (Kearny Air Pump) type systems, prefabricated from plywood or hardboard plus other materials as described in Reference 7.

2. Removal of heating and air-conditioning blower systems from automobiles and connecting to auto batteries, also so removed.

3. Removal of HVAC fans and blowers from the building and powering them by small gasoline-powered generators, or an improvised bicycle powered system.[8] The rear wheel of a bicycle (on a stand and with tire removed) will serve as the drive pulley. Spliced ropes, electrical cable, or wire (pulled from the building conduit system) can serve as generator

power cables or as a bicycle system drive belt. Improvised duct can be fabricated from plywood or corrugated box material and tape.

L. Lighting

Expedient lighting may be provided in a variety of ways. Examples are:

1. Flashlights, lanterns, and candles brought in by shelterees.

2. Operation of bicycles equipped with small friction-powered lighting generators, available from bicycle supply sources.

3. Use of automobile batteries and lamps. Wiring can be stripped from automobiles or pulled from the building electrical conduit system.

4. Relocation of building lighting equipment where needed, powered by small gasoline-powered generators.

M. Communications

Listening-type communicators can be improvised by a variety of means:

1. Small battery operated radios.* When the contained battery for these runs down, a power supply can be improvised from 12-volt automobile batteries, correctly reducing line voltage to the radio by connecting a selected automotive lamp(s) in series or by using wire of sufficient length. It is also possible to power such radios from a light generator or a speedometer-equipped bicycle. If one generator proves of insufficient voltage, two connected in series may be suitable. A simple voltmeter would be useful in adjusting voltages.

2. Essentially every automotive vehicle has a 12-volt battery-powered radio receiver. These are quite easily removed (along with whip antenna) and may be powered by 12-volt automotive batteries, also so removed. Modern solid state automobile radios draw very little current and will operate for extended periods on a single battery charge.

Two-way communications using Citizens Band (CB) radios removed from vehicles may also be attained. Large numbers of these radios are installed in taxis, trucks, service vehicles, and contractors' vehicles, as well as in private automobiles. All-channel (23- to 69-channel) types are to be

* These are mostly powered by 9-volt dry cell batteries.

favored. Because of transmission power requirements, the current draw of CB radios is relatively high. As a consequence, several automotive batteries should be available for each CB radio.

N. Emergency Exit

Because of the potential for large amounts of debris, several means of exit from the shelter space should be planned. The most suitable exits will be those blocked against air blast entry, as previously discussed. Again, in placing these blocking systems, attention must be given to the possibility of their removal from inside the shelter with the tools available.

O. Additional Fallout Radiation Protection

Consideration should be given to adding fallout radiation shielding after the attack. The most suitable measures will be those that can be attained with very short personnel exposure times. Potential shielding materials will include debris from the damaged shelter building. Priority should be given to: (1) replacement of opening blocking that has been damaged or removed by air blast, or provision of blocking as needed, for fallout protection; and (2) provision of additional radiation shielding in the actual occupied space, particularly for "last resort" structures erected within the shelter space.

P. Fire Protection

Although the shelter occupants can do little or nothing if massive fire situations should occur, they should be prepared to combat local fires, particularly those occurring in the basement space and possibly in the story above. The primary means of combatting local fires will be through the use of portable fire extinguishers, which should be collected in large numbers from the upper floors of the building occupied and from elsewhere.

Materials and Sources

Alternative materials, useful for blocking of openings and structural strengthening for air blast upgrading, have been identified in previous figures and discussions. The following paragraphs suggest sources for these blocking and strengthening materials; desirable power tools and equipment that will accelerate expedient upgrading, together with suggested sources; desirable hand tools and their sources; materials and equipment useful in the post-attack phase; and finally, desirable general purpose materials that might be stockpiled in the selected shelter building.

Q. Blocking and Strengthening Materials

Except for doors, partitions, office furniture, and other materials that may be found within the building selected for the shelter space, suitable materials must be located from commercial sources or elsewhere. Primary sources are described below.

Heavy Dimension Timber. As illustrated in the previous figures, heavy dimension timber is the most effective material for expedient blocking of openings and for structural strengthening. Although all local lumber yards will carry large stocks of plywood and 2 in. thick lumber, the desired heavier (6 in. and thicker) sizes will be in limited stockage, and will be found only at the large yards and at yards and suppliers specializing in supply to waterfront construction and maintenance jobs.

Alternative Heavy Materials. Acceptable materials as alternates for heavy dimension timber include piling, utility poles, various structural steel shapes, railroad ties, and trees. These materials will be particularly useful as vertical supports for slab, beam, girder, and column blast strengthening. Potential sources of these materials are given in the following.

1. Piling and Utility Poles - These materials are ordinarily procured from regional suppliers who also engage in timber preservative treatment. Untreated materials should be obtained if possible, because of toxicity problems with some treatment processes, particularly creosote. Utility poles are also stockpiled at telephone and electric power utility service yards on a local basis.

2. Structural Steel Shapes - Steel supply and fabrication firms are located in the large centers of population. These firms maintain

stocks of the more popular shapes of structural steel. Stock lengths will be relatively long and should be cut to length at the steel yard if possible.

3. Railroad Ties and Rails - Railroad maintenance facilities stock sizeable quantities of replacement ties, rails, and heavy timbers for maintenance purposes. Large stocks may be present because of annual buying policies. These stocks also are usually well distributed along the right-of-way. Ties and other large timbers will be exceptionally useful for providing vertical support for slabs, beams, girders, and columns. Railroad rails (new and used) would be most useful in providing horizontal closure systems for elevator shafts, stairwells, etc. Their usual length, 33 ft, will require cutting under most conditions. This can best be accomplished by use of oxyacetylene cutting equipment or the special-handled chisels used by railroad maintenance workers.

4. Felled Trees - In some regions conifer or broadleaf tree stands may be sufficient to provide a source of heavy timber in natural round form. Such timbers will be most useful for vertical support of slabs, beams, girders, and columns.

5. Steel Cover Plates - A very useful item for covering horizontal and vertical openings is the steel street excavation cover plate used in underground utility construction and maintenance. Stocks of these plates are maintained by telephone and electric power utilities, municipal public works agencies, and contractors specializing in underground work.

6. Sandbags - As discussed and illustrated previously, filled sandbags may be considered as a basic protective material. Sources of these bags include stocks under the control of the U.S. Army Corps of Engineers, and other federal agencies. Substitutes are grain and feed bags available from animal feed mills, multiwall paper bags from cement mills, and the like.

7. Built-up Timber Sections - These may be assembled from thinner dimensioned stock than the heavy timber mentioned above; adhesives, simple nailing, and/or scabbing may be used.

R. Tools and Equipment

The rapid provision of expedient combined effects protection will obviously benefit if proper tools and equipment are available. Desirable tools and equipment are listed below.

1. Power Tools and Equipment

Bulldozer	Gasoline Chain Saws
Wheel or Track Loader	Gasoline Hand Saws
Dumptruck	Gasoline Electric Generator
Sandbag Loader	Electric Drill
Air Compressor w/Breaker	Electric Hand Saw
Oxyacetylene Cutting and Welding Equipment	Electric Reciprocating Saw
	Electric Welding Equipment

2. Hand Tools

Axes	Pike Poles
Hatchets	Peaveys/Cant Hooks
Carpenter's Hammers	8 to 12 lb Sledges
1 to 2 lb Hammers	Shovels
Wrecking Bars	36" to 48" Cross-cut Saws
Pinch Bars	Large Wood Chisels
Heavy Duty Bolt Cutters	Tow Chains
Set, Auto Mechanic's Tools	Load Binders
Set, Carpenter's Tools	Timber Carriers w/Tongs
Hack Saws	Set of Socket Wrenches
Ladders	Set of Open End Wrenches

3. Post-attack Aids

Fire Extinguishers	Automobile Batteries
Flashlights and Batteries	Bicycles w/Light Generators
Hand Powered Ventilators	Auto Radio Receivers
Small Electric Generator	Auto CB Radios
Manhole Ventilators	Electric Wire and Lamps
Battery Operated Radios	Selected Hand Tools

S. Desirable Stockpiled Materials

Since there will be a heavy demand for the more common materials required for expedient protection, it would be prudent to stockpile certain of them in the selected shelter buildings early in the crisis buildup period. Materials desirable for stockpiling will include:

Sandbags
Heavy dimension timber (4" and heavier)
2" to 3" thick plank
Plywood (3/4" and thicker)
Nails and spikes
Drift pins
Timber dogs
#16 and #10 Wire
1/2" Cable and clamps (Crosby clips)
1/2" to 1" Rope w/ 1-, 2- and 3-part pulleys
Miscellaneous bolts/nuts/lag screws.

Chapter 5

BUILDING BASEMENT UPGRADING EXAMPLES

This chapter consists of Sections I to V. Section I is a study, using existing structures evaluation techniques developed under other DCPA research projects by one of the senior engineers who also worked on this project, applying such techniques to a sampling of NSS buildings (basically reinforced concrete) to learn whether there exists a general hierarchy among floor structural member types by blast strength, and whether there is a consistent relationship between normal-use design loading and blast resistance. Sections II to V give several specific examples of expedient upgrading applied to basements in existing buildings.

I. BLAST STRENGTH LIKELY ORDER AMONG R/C FLOOR MEMBERS

Objective

The objective of this phase of the effort were: (1) to examine the SRI data on the collapse of floors over basement areas of NSS buildings to determine if a simple relationship exists between the design live load of the floor and its predicted collapse overpressure; and (2) to learn if there are trends in the data indicating the likely blast strength order among the significant structural members in floor systems over basement areas (slabs, beams, girders). The study was limited to the collapse data obtained in previous studies for DCPA using the SRI existing building evaluation procedure.

Background

As part of an integrated program to develop a survey procedure for all nuclear weapon effects, Research Triangle Institute (RTI) collected data for DCPA on a national sample of 219 NSS facilities.[9] The survey was conducted primarily to obtain a complete structural description of buildings that would be adequate for predicting building damage and casualties. The results of the field survey were recorded on forms and included sketches, photographs, and plans of the buildings. A complete copy of this information was furnished to SRI for analysis of the buildings for the effects of air blast from a 1 Mt nuclear surface burst.

Of the 219 NSS buildings making up the national sample, the SRI building evaluation procedure was used to predict the collapse overpressure of the exterior walls for 50 of the buildings and of floors over basement areas of 36 of the buildings.[1,10] The floors were analyzed using the mathematical models and computer programs developed by SRI for DCPA,[11] and the input information for each structure was obtained from the field survey data collected by RTI.

To adequately predict the collapse overpressure of the floors over basement areas for the 36 NSS buildings, it was necessary to analyze the dynamic response of 82 representative floor cases. Each floor case represents a detailed analysis of one or more structural elements, such as a reinforced concrete floor slab, a concrete joist, and a reinforced concrete or structural steel support beam(s).

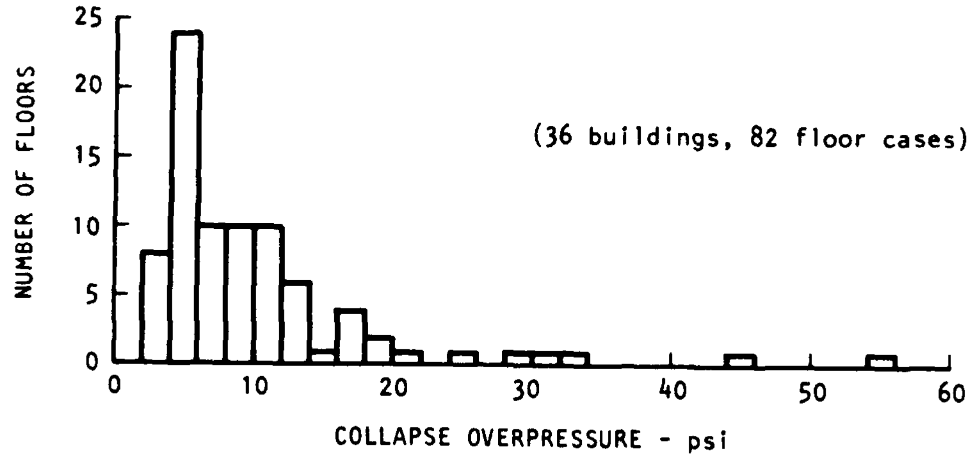
For completeness of this discussion on the relationship between the design live load for a floor system and its collapse strength under blast loading, the following pertinent information is extracted from Reference 1. The results of the dynamic analysis of the 82 floor cases for the 36 NSS buildings are summarized in the histogram and cumulative frequency distribution shown in Figure 12. As noted in the figure, the collapse overpressure for floors over basement areas had a relatively wide range from about 2 to 55 psi, with 50 percent of the floors predicted to collapse at 7 psi or less, and 90 percent predicted to collapse at 18 psi or less.

An examination of the floor collapse data showed that the data, in most instances (except as discussed in the next paragraph), were insufficient to establish the effect of various parameters on the collapse strength for the wide range of floor types included in the sample. For example, the results of the analysis of flat plate floor systems showed that collapse occurred at relatively low blast overpressure levels and resulted from a shear failure at the column head. On the other hand, it was noted in Reference 1 that flat slab construction is usually economical only for the heavier design live loads, such as encountered in warehouses, and therefore flat slabs would be expected to provide relatively high blast resistance. Although this factor was mentioned in Reference 1, no attempt was made to examine the data in detail, as will be done subsequently herein. However, another finding presented in the cited reference is of some interest to this discussion and is included in the next paragraph.

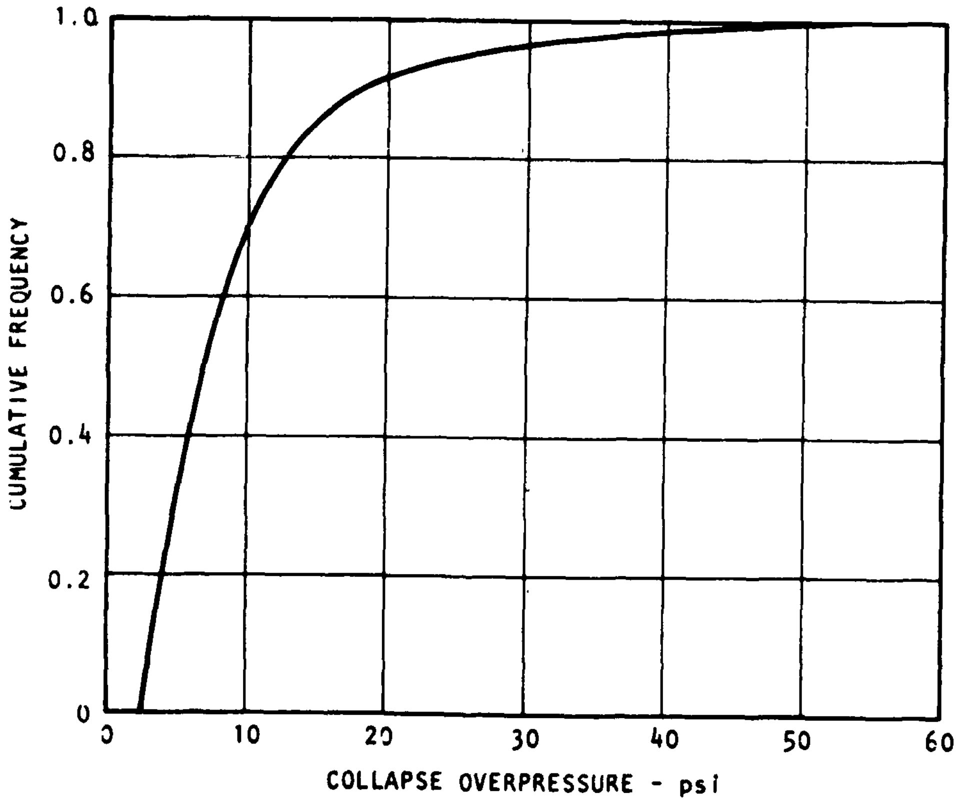
A. Effect of Type of Support Beam on Floor Strength

An examination of the results of the dynamic analyses of individual slabs and their support beams in the previous studies[1,10] revealed an interesting relationship between the relative blast strength of the basic elements of a floor system and the type of building frame. The results of the floor analyses for 20 of the structural steel and reinforced concrete frame buildings are summarized below.

Floor System Type	Total Number: Buildings	Total Number: Floor Cases	Collapse Controlled by: Slab	Collapse Controlled by: Beam
Reinforced concrete slab with steel support beams	12	32	20	12
Reinforced concrete slab with reinforced concrete support beams	8	15	2	13



(a) Histogram for all floor cases



(b) Cumulative frequency distribution for all floor cases

FIGURE 12 HISTOGRAM AND CUMULATIVE FREQUENCY DISTRIBUTION OF THE MEAN COLLAPSE OVERPRESSURE FOR THE FLOORS OVER BASEMENT AREAS OF 36 BUILDINGS

For the total number of floor systems supported by steel beams, the collapse of 63 percent was controlled by the failure of the slabs, and the collapse of only 37 percent was controlled by the failure of the steel beams. For the floor systems supported by reinforced concrete beams, the collapse of only 13 percent was controlled by the failure of the slabs, with the collapse of 87 percent controlled by the failure of the concrete beams. The effect of steel and reinforced concrete support beams on the collapse overpressure for the 47 floor cases is shown on the cumulative frequency distribution in Figure 13. Based on the above information, it was tentatively concluded in Reference 1 that, on the average, floor systems in steel frame buildings can be expected to be stronger in resisting nuclear air blast forces than floors in reinforced concrete frame buildings.

Discussion

The data available for study consisted of the analyses of floors over basement areas for the 36 NSS buildings treated in References 1 and 10, plus an additional NSS building that had been analyzed on special request.* The RTI field survey information was used to obtain the design live load for each floor, and although the total building population was 37, design live load information was available for only 34 buildings that were categorized by frame type as follows:

Frame Type	Number of Buildings
Steel frame	16
Reinforced concrete frame	7
Reinforced concrete flat slab	4
Reinforced concrete flat plate	4
Load-bearing wall	3

The analysis of the floor systems for the 34 NSS buildings required the dynamic analysis of 78 representative floor cases that were classified by slab and support type as follows:

* Bell Telephone Building, West Bloomfield, Michigan; RTI Building Number 164.

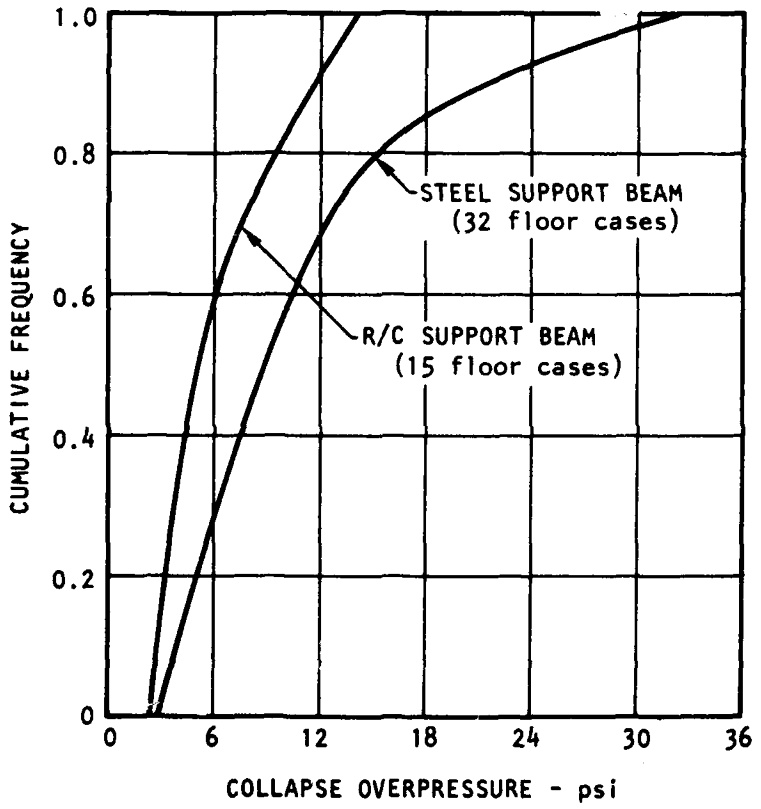


FIGURE 13 COMPARISON OF THE CUMULATIVE FREQUENCY DISTRIBUTIONS OF THE MEAN COLLAPSE OVERPRESSURE FOR FLOOR SYSTEMS BY THE TYPE OF SUPPORT BEAMS

Slab Type	Support Type	Number of Cases
Flat slab	-	6
Flat plate	-	4
R/C solid slab	R/C beam	16
R/C solid slab	Steel beam	27
R/C solid slab	Wall	3
R/C joist	R/C beam	7
R/C joist	Steel beam	9
R/C slab	Steel joist/steel beam	6

Table 1 lists pertinent information for the 78 floor cases, and includes the predicted collapse overpressure for both the floor slab and its supporting beam. A nomenclature listing appears at the end of the table. The data were examined for a possible correlation between the design live load of the floor and its collapse overpressure level. The findings are summarized in the following subsections.

B. All Floor Cases

The relationship between the design live load and the collapse overpressure for all 78 floor cases is illustrated in Figure 14. For each floor, only the lowest collapse overpressure for the slab/beam combination was plotted. That is, it was assumed that the floor system will collapse when the weakest structural element fails.

As can be seen from the figure, there is no simple correlation between the predicted collapse overpressure and the floor design live load. It is of interest, however, that no floor was predicted to collapse at a blast overpressure value less than 2.7 times the design live load. Furthermore, 89 percent of the floors were predicted to collapse at an overpressure level greater than five times the design live load.

The lack of a simple correlation between the collapse overpressure and the design live load is not surprising for a floor population consisting of the various types noted. Also, an examination of the analytical procedures in Reference 11 indicates that the predicted collapse overpressure of a floor system is dependent on a relatively large number

Table 1
FLOOR ELEMENT DATA

Building Number	Frame Type	Case	Design Live Load (psi)	Slab: L_S (in.)	Slab: L_L (in.)	Slab: hs (in.)	Slab: Reinforcement* Positive (%)	Slab: Reinforcement* Membrane (%)	Slab: Collapse pso (psi)	Support Beam+: Type	Support Beam+: L_b (in.)	Support Beam+: Collapse p_{so} (psi)
6	LBW	3F	75	90	-	4	0.67	0.33	5.6	RCB	222	3.8
15	ST	5F	250‡	Sidewalk - no data					-	SB	192	6
		6F	125	210	289	10.5	0.66	0.66	15.5	SB	289	7
		7F	125	201	210	8.5	0.56	0.56	18.2	SB	210	12.4
20	RCFP	3F	50	220	243	9	0.48	-	12.2	WS		
29	RCFP	3F	100	189	189	7.5	0.76	-	3.6	FP		
		4F	100	135	-	7.5	0.31	0.13	5.5	RCB - not analyzed		
35	ST	5F	100	96	290	4.5	0.42	0.42	8.6	SB	290	12.9
		6F	100	96	116	4.5	0.42	0.42	9	SB	116	15.1
		7F	100	84	252	4.5	0.42	0.42	10.3	SB	252	12.1
		8F	100	84	192	4.5	0.42	0.42	10.4	SB	192	11.4
36	ST	1F	100	83	-	5	0.38	0.38	11.3	SB	277	10.8
		2F	75	83	-	4	0.37	0.37	8	SB	277	4.4
37	ST	1F	100	113	-	8.5	1.08	1.08	41.4	SB	348	24.3
		2F	100	92	-	7	1.36	1.36	49.9	SB	348	32.9
41	ST	2F	100	65	-	4	0.35	0.35	6.6	SB	106	4.7
51	ST	2F	150	90	-	4	0.61	0.61	12.6	SB	312	15.8
		3F	150	66	-	4	0.61	0.61	18.3	-	-	-
		4F	300	72	127	6	0.30	0.30	28.5	SB	324	45.5
		5F	150	68	120	4	0.61	0.61	18.3	SB	120	5.3
55	ST	1F	100‡	84	240	4	0.48	0.48	10.7	SB	251	31.2
		2F	120‡	87	312	6	0.30	0.30	13.5	SB	312	25.4
		3F	50‡	96	240	4	0.48	0.48	9.1	SB	240	15.7
		4F	100‡	58	149	4	0.48	0.48	17.6	SB	149	19.9
56	RCFP	2F	40	192	-	8	0.31	-	7.1	FP		
63	ST	3F	100	R/C slab w/metal decking (not analyzed)						SB	344	5.9
		4F	100	R/C slab w/metal decking (not analyzed)						SB	404	5
		5F	100	R/C slab w/metal decking (not analyzed)						SB	288	11.6
66	LBW	3F	40	180	-	6	1.15	0.57	8.1	WS		
		4F	100	57	-	6	1.15	0.57	31.5	WS		
76	ST	4F	200	30	-	2.5	-	-	-	RCJ	228	4.5
										SB	228	11.1
		5F	200	20	-	2.5	-	-	-	RCJ	378	8
										SB	228	11.1
		6F	200	20	-	2.5	-	-	-	RCJ	294	6
										SB	232	12.8
		7F	200	20	-	2.5	-	-	-	RCJ	378	8
										SB	228	6.9

Table 1 (continued)

Building Number	Frame Type	Case	Design Live Load (psf)	Slab L_S (in.)	Slab L_L (in.)	Slab h_s (in.)	Slab Reinforcement* Positive (%)	Slab Reinforcement* Membrane (%)	Slab Collapse p_{so} (psi)	Support Beam[1] Type	Support Beam[1] L_b (in.)	Support Beam[1] Collapse p_{so} (psi)
81	RCF	3F	80	20	-	2.5	-	-	-	RCJ	264	7.6
										RCB	300	4.5
84	RCFS	5F	100	240	240	10	0.15	0.17	7.5	FS		
93	RCFS	3F	125	300	300	9.5	0.29	0.29	5.7	FS		
		4F	125	300	300	9.5	0.35	0.35	6.8	FS		
111	ST	1F	100	20	-	3	-	-	-	RCJ	228	5.7
										RCB	149	5.9
129	RCFP	1F	100‡	228	228	7.5	0.96	-	3.8	FP		
		5F	10‡	210	210	7.5	0.29	-	2.5	FP		
130	RCF	5F	50‡	30	-	3	-	-	-	RCJ	276	4
										RCB	288	3.5
		6F	50‡	30	-	3	-	-	-	RCJ	84	37
										RCB	288	3.5
136	RCF	6F	100‡	20	-	2	-	-	-	RCJ	168	9.3
										RCB	240	2.5
146	RCF	5F	40	306	354	8	1.28	1.28	13.9	RCB	314	6.6
		6F	40	276	732	7.5	1.20	1.20	13.6	RCB	135	15.7
		7F	40	234	666	7.5	1.20	1.20	16.7	RCB	222	9
		8F	40	234	666	7.5	1.20	1.20	16.7	RCB	135	12.1
		9F	10	276	732	7.5	1.20	1.20	13.6	RCB	227	5.3
		10F	10	264	-	7.5	1.48	1.48	17.6	RCB	379	4.8
		11F	40	270	-	8	1.37	1.37	17.3	RCB	201	10
		12F	40	270	-	8	1.37	1.37	17.3	RCB	228	6.1
147	LBW	5F	40	94	-	5	0.54	0.27	6.8	SB	308	5
152	ST	4F	100	42	-	4	0.65	0.65	23.0	RCB	360	3.2
		5F	250‡	91	-	6	0.67	0.67	17.2	SB	183	17.9
		6F	100	20.5	-	2.5	-	-	-	RCJ	182	3.6
		7F	100	21	-	2.5	-	-	-	RCJ	207	3.9
		8F	100	20.5	-	2.5	-	-	-	RCJ	151	5.1
161	ST	2F	50	216	-	8	0.56	0.56	9.4	SB	234	11.5
		3F	50	108	-	8	0.56	0.56	24.1	SB	312	13.3
		4F	50	92	-	8	0.48	0.48	26.4	SB	284	21.7
		5F	100	216	-	9.5	1.00	1.00	17.0	SB	288	19.6
		6F	50	108	-	8	0.56	0.56	24.1	SB	284	12.7
168	ST	2F	50	-	-	2.5	-	-	-	SB	400	9.8
179	ST	2F	125	24	-	4	-	-	-	RCJ	228	10.6
										SB	384	27.1
198	RCF	2F	80	30	-	3	-	-	-	RCJ	294	6.3
										RCB	288	5.8

Table 1 (concluded)

Building Number	Frame Type	Case	Design Live Load (psf)	Slab L_S (in.)	Slab L_L (in.)	Slab h_s (in.)	Slab Reinforcement* Positive (%)	Slab Reinforcement* Membrane (%)	Slab Collapse p_{so} (psi)	Support Beam† Type	Support Beam† L_b (in.)	Support Beam† Collapse p_{so} (psi)
204	RCFS	6F	250	204	204	8	1.61	1.61	15.8	FS		
		7F	250	207	207	8	1.93	1.93	51.1	FS		
212	RCFS	2F	150	250	250	8	0.38	0.18	1.2	FS		
223	ST	2F	100	54	-	1	-	-	-	RCB	310	5
		3F	125	11	-	2.5	-	-	-	RCB	311	5.3
		4F	125	26	-	2.5	-	-	-	RCJ	144	1
228	ST	1F	55	54	-	3	0.56	0.56	7.8	SB	324	4.5
		2F	55	54	-	3.5	0.18	0.18	7.8	SB	324	4.5
239	RCF	2F	10	251	-	12	0.50	0.59	10.7	WS		
164	RCF	3F	150	181	-	8	0.74	0.74	15.8	RCB	222	7.3
		4F	150	181	-	8	0.48	0.18	10.2	RCB	204	6.2
		5F	150	220	-	8	1.07	0.54	9.2	RCB	220	5

* Positive steel ratio in % at mid span (maximum only if 2-way action) membrane steel ratio in % continuous at support.

† If more than one beam analyzed for any one slab, only weaker beam noted.

‡ Code estimate.

NOTATION

FP	Flat plate
FS	Flat slab
h_s	Thickness of slab, in.
L_b	Length of beam, in.
LBW	Load-bearing wall
L_L	Length of slab in long direction, in.
L_S	Length of slab in short direction, in.
p	Steel ratio, tension steel
p_{so}	Peak incident overpressure, psi
RCB	Reinforced concrete beam
RCF	Reinforced concrete frame
RCFP	Reinforced concrete flat plate
RCFS	Reinforced concrete flat slab
RCJ	Reinforced concrete joist
SB	Steel beam
ST	Steel
WS	Wall support

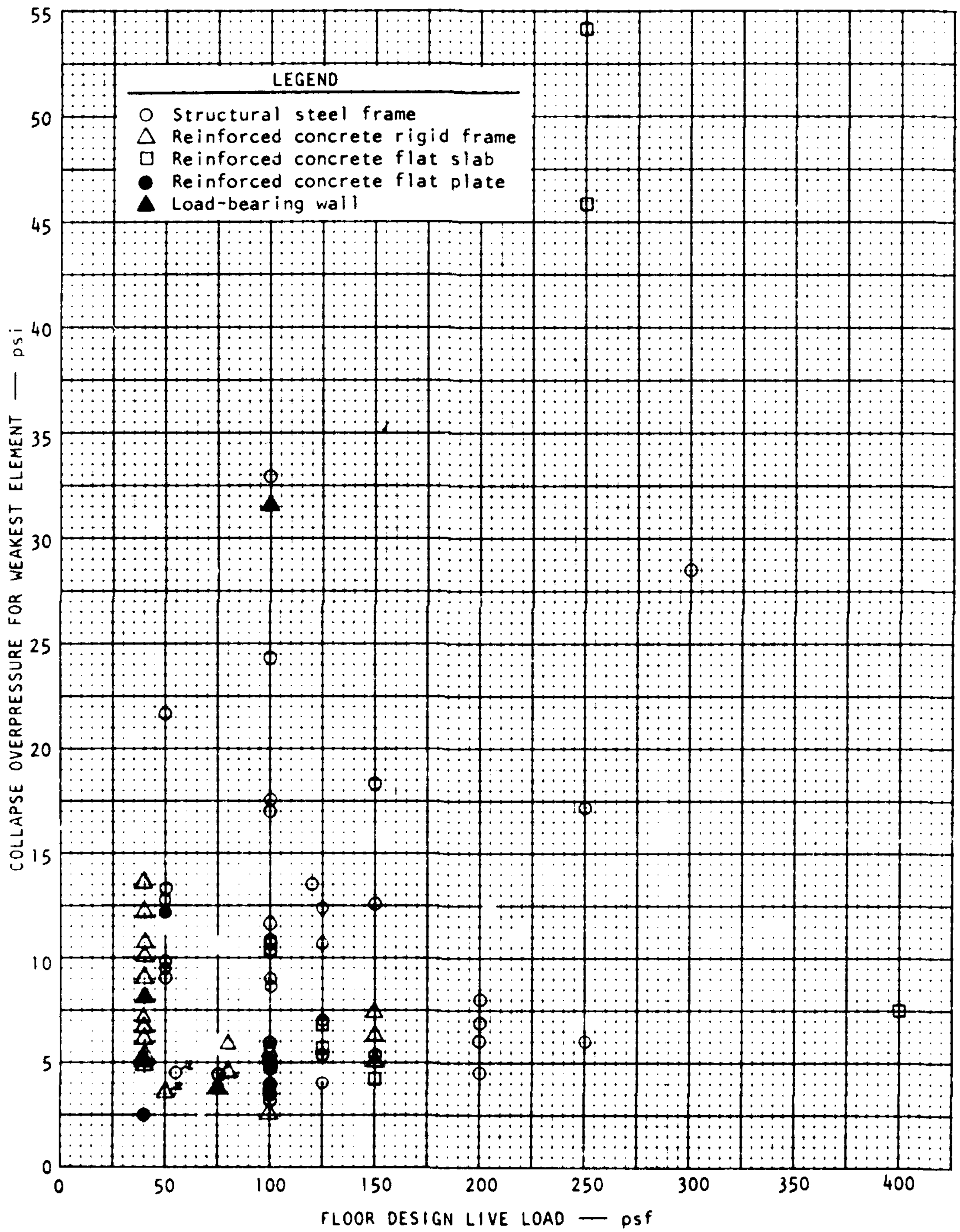


FIGURE 14 COLLAPSE OVERPRESSURE FOR FLOORS OVER BASEMENT AREAS VERSUS DESIGN LIVE LOAD

of variables, a few of which are noted in Table 1. For example, the dynamic analysis of a reinforced concrete slab for blast loading requires the following physical data for the slab:

Support type

One- or two-way structural action

Length, width, and thickness of slab

Moment of inertia

Dynamic yield strength of reinforcing steel

Area, location, continuity, and anchorage of reinforcing steel

Dynamic compressive strength of concrete

Modulus of elasticity of concrete and steel

In addition, of course, predicting the collapse of a floor system may also require the dynamic analysis of one to four support beams.

Since the above parameters may be a function of other variables, such as date of construction, building use class and size, design office procedure, and design codes, the variations shown in Figure 14 are not too surprising. The design live load as a measure of the relative blast strength of various floor systems may also be misleading because of less obvious factors such as:

- It is not unusual for a designer to specify only a few slab designs for an entire floor area of a building, especially for a large multistory building where the first story level may contain concourses, hallways, offices, stores, conference rooms, and elevator lobbies. Many different span lengths and several design live load requirements may be involved in such a building, but because of construction practices, it may be more economical to minimize the number of slab designs. The slab would, of course, be designed for the maximum span and load requirements, and would therefore be overdesigned for many areas. For example, consider slabs for floor cases 2F, 3F, and 5F of Building 51, or 1F, 3F, and 4F of Building 55 (Table 1).
- Many support beams, especially for large multistory steel frame buildings, are designed as part of the rigid frame, which requires a much larger beam section than would be necessary for the floor dead and live loads only. Such beams usually develop the full strength of the floor slab in resisting the blast forces. For example, the beams for floor cases 2F and 4F of Building 51 were predicted to be 25 and 60 percent stronger, respectively, than

the slabs they supported. In contrast, the beam for floor case 5F of the same building was an intermediate type beam, not part of the rigid frame, and it was predicted to have about a third of the blast strength of the slab.

- Reinforced concrete slabs and beams are usually designed for flexure/deflection and then checked to determine their adequacy to resist shear stresses; large deflections and inelastic behavior are not generally considered in static design. However, when evaluating the dynamic behavior of an existing structural member up to collapse, the internal resistance throughout the entire range of response including large deformations must be considered. It is possible that a member may respond in modes not important to, and therefore not considered directly in, the design, but such response modes may in fact control the collapse of the member. For example, the design of a reinforced concrete slab is controlled by its ultimate bending strength, yet a slab with continuous reinforcement at its supports has the potential for developing a tensile membrane resistance that is much larger in magnitude than its flexural resistance. On the other hand, a flat plate, which has no column capitals or drop panels, has no potential for developing a tensile membrane mode, and may even experience a shear failure prior to developing its full flexural resistance. For example, compare the reinforced concrete slabs in the following two floor cases from Table 1:

Building Number	Case	Design Live Load (psf)	L_s (in.)	h_s (in.)	p (%)	Collapse p_{so} (psi)
29	3F	100	189	7.5	0.76	3.6
161	2F	50	216	8	0.56	9.4

Floor case 3F, Building 29, is a flat plate with a floor design live load of 100 psf. Under blast loading, the slab did not develop a tensile membrane mode, but rather failed in shear at an overpressure level of 3.6 psi. Floor case 2F, Building 161, has a lower design live load, a greater span, and less steel, but since it developed a tensile membrane resistance as a result of the continuous reinforcement at the steel beam supports, its predicted collapse overpressure of 9.4 psi is more than 2.5 times that of a flat plate with similar dimensions.

- A reinforced concrete slab may be designed as a one-way slab spanning between intermediate support beams which frame into the building girders. In addition to the main slab steel reinforcement, the codes require a minimum transverse reinforcement, such as 0.20 percent of the concrete area, for shrinkage and temperature steel. If the ratio of the long span to the short span of the slab is less than about three, then the transverse steel will influence the collapse strength through two-way structural action, although it was not considered in the design as contributing to the load-carrying capacity.

Although the above factors illustrate some of the inconsistencies between the design and collapse analysis procedures for reinforced concrete floor systems that result in the poor correlation between the design live load and predicted collapse overpressure, no attempt was made, for this study, to exhaust all possibilities.

C. Flat Plate and Flat Slab Floor Systems

It is of interest for this discussion to re-examine the data from Reference 1 for the nine flat slab and flat plate floor systems.* The pertinent data from Table 1 are arranged in the following tabulation according to their predicted collapse overpressures:

Building Number	Case	Frame Type	Collapse p_{so} (psi)	Design Live Load (psf)	Ratio p_{so}/Live Load
129	5F	RCFP	2.5	40	9.0
29	3F	RCFP	3.6	100	5.2
129	4F	RCFP	3.8	100	5.5
212	2F	RCFS	4.2	150	4.0
93	3F	RCFS	5.7	125	6.6
93	4F	RCFS	6.8	125	7.8
84	5F	RCFS	7.5	400	2.7
204	6F	RCFS	45.8	250	26.4
204	7F	RCFS	54.1	250	31.2

* There are only nine floor cases included here, rather than the ten indicated in Table 1, because floor case 2F, Building 56, is supported by walls and therefore does not act as a flat plate.

Several observations can be made concerning the above data. First, although the sample is very small, all flat plates are predicted to collapse at a lower overpressure level than the flat slabs. However, the floor with the highest design live load (84-5F) does not have the greatest collapse overpressure level; it does have the smallest ratio of live load to collapse overpressure of any of the floor cases. Second, the floor with the lowest design live load, 40 psf, does have the lowest predicted collapse overpressure of 2.5 psi. Also, the floor with the next lowest design live load of 100 psf is the next weakest for blast load. However, the simple linear relationship between design live load and collapse overpressure noted for the first three floor cases is not supported by the data for the flat slab floors. This, of course, suggests that for some floor cases, other variables are more indicative of collapse under blast loading than the design live load.

D. Reinforced Concrete Slab Supported by Steel Beam Floor Systems

An examination of the data in Table 1 indicates that for selected cases, a simple linear relationship can be shown to exist between a single parameter and the predicted collapse overpressure. For example, all four floor cases analyzed for Building 35 had the same design live load, slab thickness, and steel ratio, and the slabs all collapsed at lower overpressure levels than their steel support beams; the only differences were in the slab spans. As shown in the table, when the slab spans decrease from 96 in. by 290 in. to 84 in. by 192 in., the predicted collapse overpressure increases systematically from 8.6 psi to 10.4 psi. On the other hand, no such simple relationship is apparent for other cases, such as the five floor cases for Building 161, where floor cases 2F and 6F have the same design live load, slab thickness, and steel ratios, but differ in span lengths. However, the predicted collapse overpressure for floor case 2F, with a span of 216 in., is 9.4 psi, whereas for case 6F, with a span of 108 in., the collapse overpressure level is 12.7 psi. The reason, of course, is that for case 2F, the steel support beams develop the full strength of the slab; the slab strength therefore controls the predicted collapse overpressure. The slab for case 6F is capable of developing a blast strength of 24.1 psi, but the prediction for the floor system is limited by the strength of the support beams to 12.7 psi. Neither the design live load for the two floor cases nor the large differences in slab span reflect the differences in predicted collapse overpressures.

To examine further a possible relationship between a single variable and the predicted collapse overpressure of a floor system, it was decided to perform limited statistical analyses of the data for a single type of floor system. As noted previously, the largest number of collapse analyses performed for any one category of floor system was 27 for reinforced concrete slabs supported by steel beams. For this study, there was only sufficient time to examine the relationship between the design live load and the predicted collapse overpressure of the 27 cases, using a simple linear regression analysis (one dependent and one independent variable). The regression analysis of the 27 cases showed a correlation coefficient of only 0.25 (1.0 is an exact fit) between the design live load and the predicted collapse overpressure. This, of course, supports the data spread for all floor cases shown in Figure 14.

Comments

The examination of the data for all available dynamic analyses of floor systems, as well as for several specific categories, showed that the predicted collapse overpressure of a floor system is dependent on a number of variables. The use of a single variable, especially the design live load, to judge the collapse strength under blast load of floor systems over basement areas of NSS buildings cannot be supported by the available analyses of floors of existing buildings.

II. EMERGENCY OPERATING CENTER (EOC), LIVERMORE, CALIFORNIA

Introduction

Early discussions and informal guidance, with and from the staff professionals at DCPA Headquarters, indicated that one or two of the planned five building basements to be considered in detailed applications under this project could be an existing EOC(s). Telephone inquires to DCPA Region 7 and (California) State Emergency Services Region II personnel were made, seeking a candidate basement EOC in the general area near the Institute but excluding the larger cities around the perimeter of the San Francisco Bay; EOCs of such cities would probably be atypical and have already been considered by another research firm looking at engineered blast upgrading, in contrast to this project's emphasis on expedient upgrading with indigenous materials and labor insofar as possible.

The EOC is integrated into the normal activities of Fire Station No. 2 of the City of Livermore, California. The basement, fully below grade and with a heavy cover slab further described below, contains: offices, a large general purpose room, toilets including showers, kitchen facilities, a dormitory room, a mechanical room, stairwells on each side of the building, and the secretary's office-plus-closet.

The general purpose room contains sets of building plans, wall maps, a blackboard, blank wall graphs and tables for fallout records, and is otherwise generally set up for alternate use as an EOC. The alternate entrance to the toilet spaces has a separate set of compartments constituting a decontamination shower and clothes-changing arrangement. The dormitory room has two wall lockers and a wall cabinet housing each bunk, for a total of ten such arrangements, but for emergency use there are, stowed elsewhere in the basement, pipes and canvas ready to convert each bunk into a tier of four bunks. This dormitory room has student armchairs stowed on the tops of wall lockers and bunk cabinets and has some study tables, so that conversion to a classroom, conference room or study/paperwork space is easy and quick. The secretary's office-plus-closet serves as both an administrative space and a communications center, the latter complete with hot lines (telephone) and radio facilities (police, fire, public works, and RACES) such that it is a full alternate to the centralized dispatcher services housed in Police Headquarters.

Fire Station No. 1 is the present* operational headquarters for the Fire Department and the EOC/Fire Station No. 2 is the present administrative headquarters; however, Fire Department operational and administrative functions will be combined as soon as a replacement building for Fire Station No. 1 is completed, expected about April 1976.* There are no changes planned for the EOC; obviously, more space may become available for it when the present administrative function is moved out of the building.*

The ground level of Fire Station No. 2 has space for four vehicles (two depart via doors in the west wall and two via doors in the east wall); two pumpers were in the station during our visit, one with foam tanks for aircraft fires. Their pumpers weigh about 15 tons, gross. A variety of small rooms, including an alarm room, extends along part of the north and south walls of the building.

Photographs were taken of all outside walls and the mechanical equipment in an outside fenced space (on the west wall at the south corner), as well as selected interior spots.

A full set of building drawings was available for study at the EOC; however, a set of prints of those drawings desired for further study was obtained, courtesy of the original designer, Earl E. Mason, P.E., of Associated Professions, Inc. (formerly Mason & Associates, Inc.), who was also kind enough to send a set of the structural calculations to the Institute in the next mail.

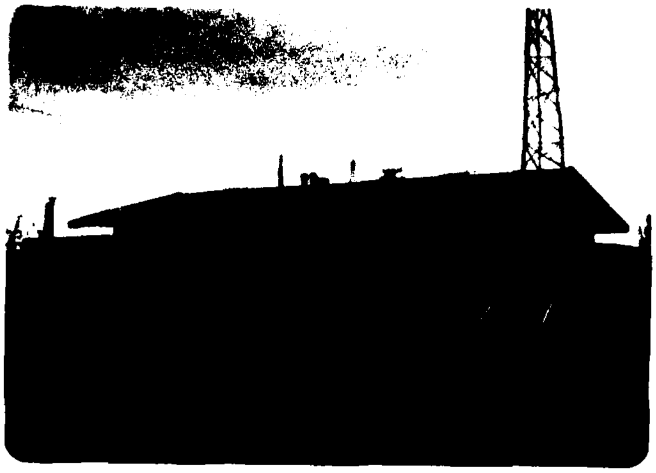
Description of Building

Livermore Fire Station No. 2, constructed in 1963, consists of an aboveground story and a fully buried basement. The overall height is about 20 ft, gross plan dimensions are about 60 by 60 ft, and floor areas are 2,880 sf on the basement level and 3,166 sf on the first story level. Figure 15 shows exterior views of the building.

The building has load-bearing reinforced concrete masonry exterior and interior walls. The walls on the first story level are 8 in. thick, and support the tapered timber laminated beam roof system. On the basement level, the exterior walls in contact with soil backfill are 12 in. thick, and all interior load-bearing walls are 8 in. thick. The vertical steel reinforcing in the 12-in. thick exterior basement walls consists of #6 reinforcing bars on 16-in. centers, and in the 8-in. thick walls of #4 bars on 24-in. centers; all walls also have horizontal reinforcing consisting of 2 #4 bars on 4-ft centers. The vertical steel in the basement

* As of January 1976.

A. EAST SIDE



B. SOUTH SIDE

C. NORTH SIDE

FIGURE 15 LIVERMORE EOC PICTURES

walls in extended into their footings and into the first floor slab, and all vertical wall intersections are dowelled.

The reinforced concrete slab over the basement area is 24 to 26 in. thick and was designed as a one-way slab continuous over two interior support walls. The most critical design load was the slab dead load plus a 200 psf live load. Figure 16 is a basement floor plan showing the location of all interior load-bearing wall partitions.

Analysis

The field survey indicated that the most critical structural element for determining the blast survival in the EOC area was the slab over the basement. Other structural elements that were identified as potential threats to EOC occupants under nuclear blast conditions were: (1) the mechanical room interior walls, which are subjected to room filling pressure as a result of two ventilating duct holes through the first floor slab; and (2) the stairwell interior walls. Although the collapse of these walls cannot be considered as catastrophic as collapse of the slab, if the walls collapse at an overpressure level less than that of the slab, then wall debris could produce casualties. Using the SRI building evaluation procedure,[11,12] collapse predictions were made for the slabs and walls subjected to the blast effects of a 1-Mt weapon yield. The results of the slab and wall analyses are presented in the following paragraphs.

A. Floor Slab Over Basement

Although the slab was designed as a one-way slab, it was analyzed as having two-way action because of the temperature steel placed in the transverse direction. A separate analysis was made of the slabs over the three portions of the basement area defined by the intermediate interior transverse support walls; see Slabs 1, 2, and 3 of Figure 16.

Slabs 1 and 3 are continuous over the interior support wall and simply supported along the other three edges. However, since this case has not yet been programmed for collapse analysis in our research work, the blast strength was estimated by analyzing the slabs, first, as two-way slabs simply supported on all four edges, and second, as one-way slabs with propped cantilever supports, i.e., simply supported at the exterior wall and continuous over the interior wall.

Slab 2 was analyzed as a two-way slab fixed along the long edges that are at the interior transverse support walls, and simply supported on the short edges by the interior stairwell walls.

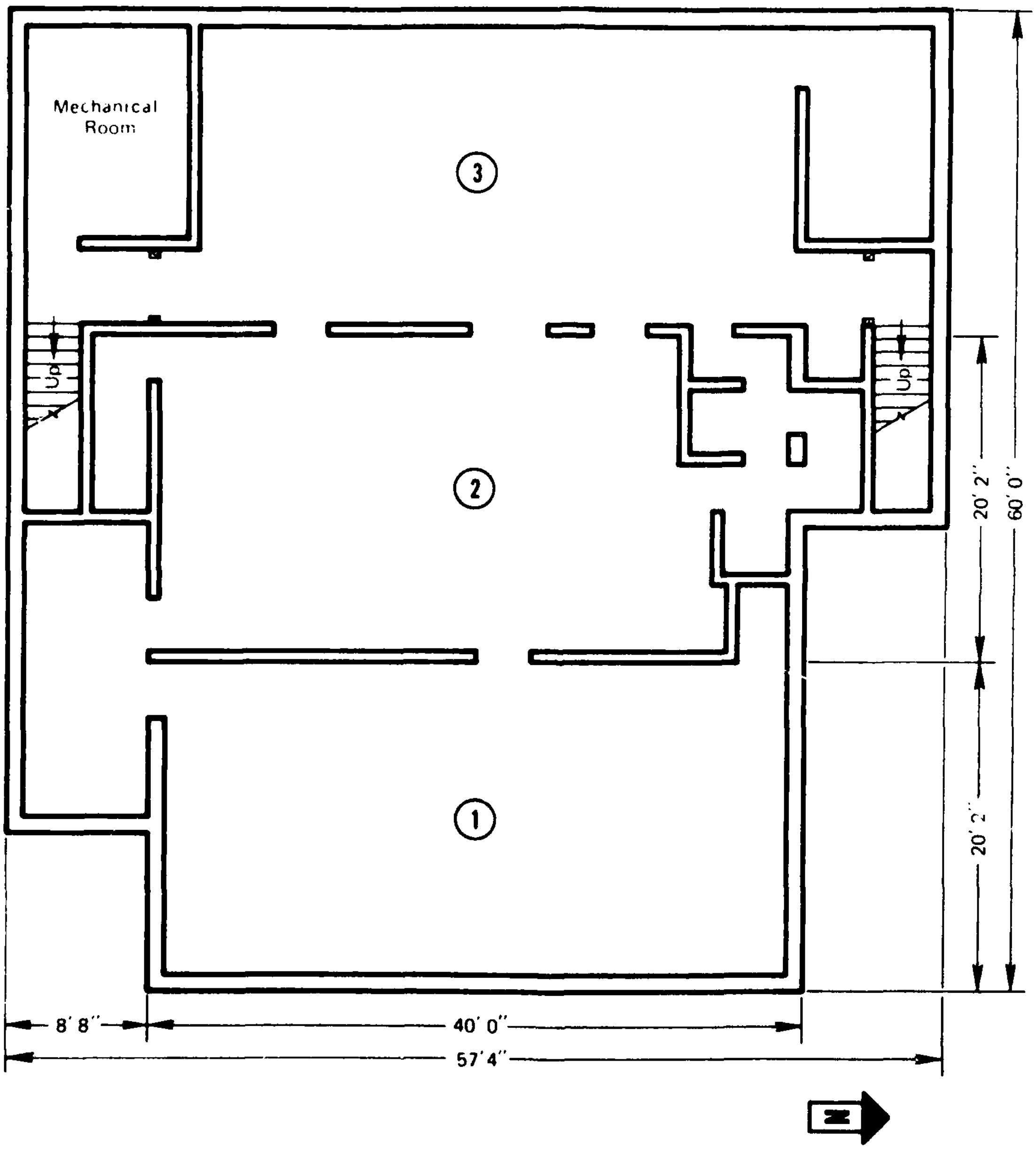


FIGURE 16 LIVERMORE EOC BASEMENT FLOOR PLAN

The results of the collapse analyses of the three reinforced concrete slabs over the basement area are summarized as follows:

Slab Identification	Predicted Mean Collapse Overpressure
1	16 psi
2	29
3	18

B. Mechanical Room Interior Wall

As a result of the blast entering the two ventilation duct openings through the floor slab, the 8-in. thick interior wall surrounding the mechanical room was investigated for its resistance to the room filling pressure. Since the mechanical room interior wall was well dowelled to all adjacent footings, slabs, and walls, it was analyzed as a two-way reinforced masonry wall with all four edges fixed. Also, since the wall supported the first floor slab dead load, as well as the slab blast load reactions, an axial force in the vertical plane of the wall was included in the analysis. The blast analysis indicated that the wall would collapse at a mean overpressure level of about 9 psi.

C. Stairwell Interior Wall

As can be seen on the basement floor plan in Figure 16, the 8-in. thick interior stairwell wall on the south side of the basement is common with the main EOC area, while on the north side there are reinforced concrete masonry walls between the main EOC areas and the stairwell. Therefore, only the south stairwell wall situation was investigated for its blast resistance; the drawings, however, show no structural difference between north and south interior stairwell walls.

The potential threat to occupants of the basement area by the blast entering the stairwell from the first story level is created by the section of the masonry wall spanning between the stairwell landing and the bottom of the first story floor slab. The wall is 8 in. thick and the vertical and horizontal reinforcement is similar to other walls. Although the load-bearing wall extends from the footing to the first floor slab, it has intermediate support at the wall-stair intersection, where the concrete stairs are inset in the wall and #4 dowels are placed on 16-in. centers. The wall analyzed for the direct blast forces was therefore a 46-in. high section of the wall that was common to both the EOC and the kitchen areas.

The predicted mean collapse overpressure for the interior stairwell wall was about 17 psi. The dowels at the wall-stairway intersection were found to be adequate to resist the inward reaction of the wall.

D. Summary

The results of the blast evaluation of the critical structural elements are summarized as follows:

Element Identification	Predicted Mean Collapse Overpressure*
Slab 1	16 psi
Slab 2	29
Slab 3	18
Mechanical Room Interior Wall	9
Stairwell Interior Wall	17

Basement exterior walls, under lateral (airslap and soil) and vertical loads, and footings were examined sufficiently to ensure that their resistance capacity considerably exceeds that of Slab 1 (16 psi) in the above tabulation.†

It is apparent from the predicted mean collapse overpressures for the various elements that the first level of blast upgrading of structural elements that should be considered involves the prevention of collapse of the mechanical room interior walls at an overpressure level of about one-half that of the floor slab over the basement. A second level of upgrading that could be considered would be to prevent the collapse of the stairwell interior wall in the area of Slab 2. The options for upgrading the structural elements to resist blast forces are considered in the next section on upgrading of the basement area for all nuclear effects.

* Free-field overpressure that is predicted to collapse the structural element.

† Interior walls and footings supporting the floor slab over the basement were examined, with the same results.

E. Blast Loadings

The preceding section has provided the free-field overpressure (that is, before interaction with any objects) resistance capability of certain key structural elements in Fire Station No. 2. The judgment of the engineer making the resistance analysis supplied the conversion from free-field overpressure to actual or incident overpressure felt by each element; such incident overpressure used for each element might well have a finite (not zero) rise time and a combination of overpressure and drag pressure (a combination known as stagnation pressure), perhaps even a reflected pressure. The effect of all such loading matters was considered by the engineer in arriving at a structural resistance or capacity for each member, which is stated in terms of free-field overpressure. For example, a finite rise time of even very short duration reduces the effect (of the same peak loading pressure) considerably over a pressure loading with zero rise time, which could have been considered to compensate in some manner for a tendency to build up some localized reflected pressures.

For the first floor slabs, the loading for collapse analysis was taken as the free-field overpressure applied with a rise time equal to the blast wave travel time over each slab, considered in its short direction. For the stairwell interior walls, the loading for analysis was also taken as the free-field overpressure applied simultaneously over the short wall height between the landing and the bottom of the first floor slab (this assumption covers many considerations: first floor wall/roof failure times; debris; minor reflections; etc.). For the mechanical room interior walls, the loading for analysis was taken as that due to room filling through the two 13-in. by 25-in. air duct openings in the overhead slab, initially ignoring the 7-in. diameter flue opening; a later check indicated that the latter action had negligible effect on the analysis results.

As a final analysis blast loadings item, the mean value (50% probability) of the analysis results should be considered for its relationship to design.

Because slanting and other protective design procedures[2,3] are predicated on 95-99% survival probability of each structural element against the assumed air blast loading, the analysis values are really needed in terms of 1 to 5% probability of collapse, rather than 50%. Since such values have not been calculated in the analysis (for computer and time cost reasons, not lack of capability), upgrading design to meet the analysis overpressures will be conservative (i.e., on the safe side), and the whole matter would merit further, detailed study *if* the proposed upgrading involved costs sufficient to make the detailed study worthwhile. Because such is not the case herein, as later paragraphs will indicate, the loadings used for the collapse analysis section above have been also used as the loadings for the upgrading design work.

F. Open Shelter Potential

There are two stairwells, two ventilation duct openings, and one flue opening through which air blast could enter the basement. The resulting V/A ratio (total basement volume to total area of apertures) is too small to offer more than a minor reduction in the air blast wave assumed to pass side-on over the basement slab.* With a basement containing glass (shatterable by about 1/2 psi blast) and lightweight partitions, as well as considerable quantities of loose materials offering a high missile hazard potential in a blast wave, use of this space for an open shelter is incompatible with its normal use as a fire station, fire department administrative headquarters, and EOC. Cleared of all the hazardous materials just mentioned, it could offer open shelter in most of its space; its air blast resistance would be limited by its interior walls constructed like those of the mechanical room, indicating a predicted mean (50% probability) collapse overpressure of about 9 psi, as shown in the table at the end of the analysis summary section above. This value is so far below the shelter potential of a closed shelter, which would require no change of interior arrangements or use, that no further consideration of open shelter appeared to be merited.

G. Closed Shelter Potential

From the analysis summary section above, it is obvious that blast closures at the stairwell landings and at the ventilation openings into the mechanical room will provide a substantial level of air blast protection to basement shelterees.

Fallout protection for basement shelterees was the obvious design criterion used when the EOC/fire station was constructed - an overhead slab of 24 in. to 26 in. of reinforced concrete, with entry paths having several turns - leaving only the stacking of some materials piles at strategic locations near the entry points to achieve a PF (protection factor) value in the hundreds.†

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* Reference 2 or 3; pp. 8-112 to 8-114, and Appendix E.

† Reference 3 or 13, Table 5.1; also, each additional 8 in. of concrete reduces fallout gamma radiation to one-tenth, 2.3 in. to one-half (Reference 14, Section 12.52 and Equation 8.74.1).

Initial nuclear radiation protection would be provided at a high level by the same cover slab plus the aboveground building. The protection should be adequate for any weapon yield higher than about 200 kt, and certainly for yields in the megaton range.*

Thermal radiation would offer no hazard to shelterees, nor would secondary fires from the surrounding area, which is separated by parking areas and streets around the EOC/fire station and offers only low-rise construction and almost no trees. Of course, such precautions should be taken as removing all fire trucks and other flammable liquid sources from and near the building at the time of first warning.

H. Sources of Indigenous Materials and Labor

Fire Station No. 2 is located in a residential neighborhood, bordering a large shopping center. Trees are small and rather scarce, unsuitable for cutting into columns, as indicated by a reconnaissance ride over an area within about five blocks of the fire station in all directions. The ride turned up no lumber or other building supplies yard or store; however, such supply sources are ample elsewhere in Livermore and in nearby communities.

In contrast, the duty and on-call firemen offer a ready pool of labor that is sure to include most if not all of the skills needed for expedient blast upgrading of the basement into a combined nuclear effects shelter.

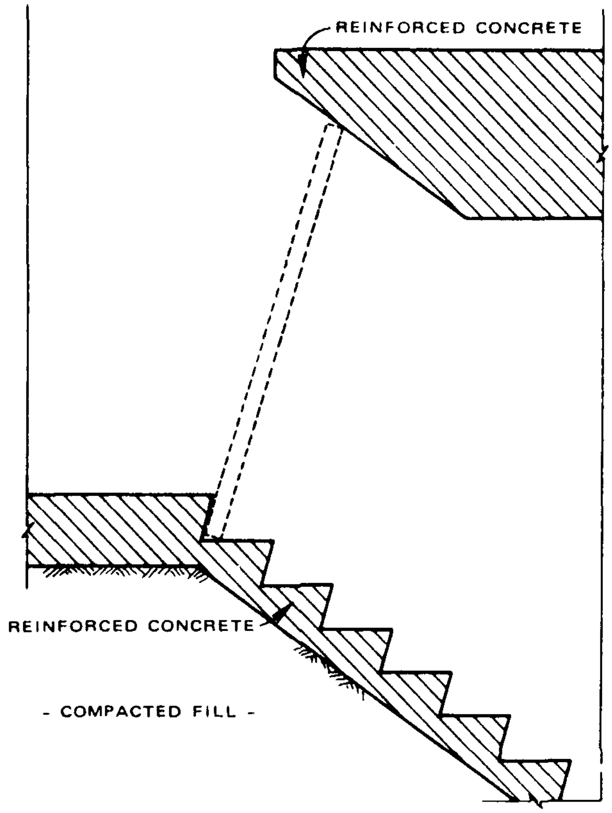
I. Design of Blast Closures

Stairwell Closures. Figure 17A shows the stairwell section used for the fire station construction. Also indicated in the section is the proposed location for a blast closure door, wedged between the first step down from the mid-story landing and the first floor slab edge above the step.†

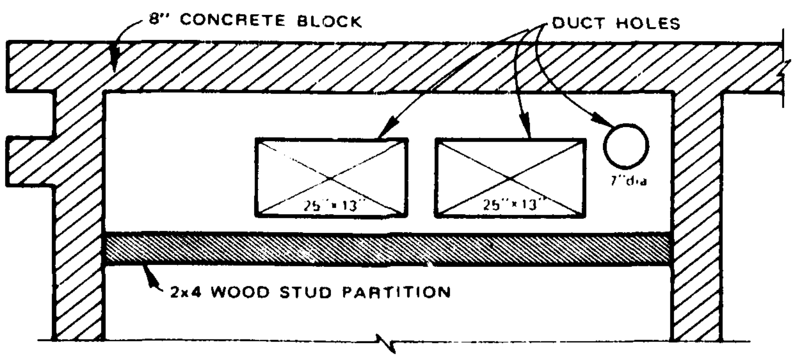
A wood door, spanning in the near vertical direction, is proposed, simply wedged in place, or with a tie cable(s) to some point in the basement. It is likely that debris from the destruction of the first story

* Reference 3 or 13, Figures 2-3 to 2-5; also, each 18-in. thickness of concrete reduces initial nuclear radiation to one-tenth, and 5.5 in. thickness to one-half (Reference 14, Section 12.45 and Equation 8.74.1).

† The step was checked and found adequate in pure shear and diagonal tension resistance.



A. SECTION — SOUTH STAIRWELL



B. SECTION — FIRST FLOOR — SW CORNER

FIGURE 17 BLAST CLOSURE LOCATIONS

would fall into the stairwell, blocking the proposed door in place. The door must be tailored to the stairwell width in each case, 42 in. to 42-1/4 in. Door span (height) of an even 6 ft is recommended. Each door can be held together with a typical Z-frame of boards on one side, or by plywood sheathing on one or both sides. Each door could be stored by mounting flat on a stairwell wall, either the one directly above the point of use or one of those around the landing.

Design of the wood door principal members, those spanning in the 6-ft direction just described, should follow the method described in available slanting guidance (Reference 2 or 3, p. 6-107)* and use the latest edition of applicable industry grading rules, also mentioned (including sources) in the slanting guidance.* Loading should be that described for stairwell interior walls, in paragraph E above, which translates into a design peak overpressure of 16 psi side-on with zero rise time. Positive phase duration would be 1.50 sec (Reference 3 or 13, Figure 2-1) for a weapon yield of 1 Mt. Design calculations may be made as follows: Using Equations 6-54 and 6-56,* Reference 2 or 3, solve for the extreme fiber stress in bending F_b (from grading rules) in terms of actual depth of member d; similarly, using Equations 6-55 and 6-56* of the same references, solve for the horizontal shear stress F_v (from grading rules) in terms of d; calculation of end bearing length is not needed for the contemplated use in this case. The following tabular values were readily calculated as described:

	$\mu = 3$		$\mu = 2$	
d in.	F_b psi	F_v psi	F_b psi	F_v psi
5.5	617	40	685	44
3.5	1524	67	1693	74
2.5	2986	97	3318	107

Because Equation 6-56 was used in the calculations, meaning that a step pulse loading was used, there is a little conservatism in the above values that could be eliminated by solving the equation of motion using the Newmark β Method or the Modified Newmark β Method (by J. E. Beck) (Reference 2 or 3). Use of the above tabular values, in selecting wood members from any available stocks of structural/stress-graded wood, simply requires identification of allowable F_b and F_v values from the appropriate grading rules, then selection of a member thickness such that the allowable stress values are <u>both</u> greater than those tabulated above for a particular thickness; use of $\mu = 3$ values is recommended, those shown for $\mu = 2$ being simply for comparison purposes.

* Extracted; see Appendix B herein.

Ventilation Ducts Closure. Figure 17B shows the location of two 13-in. by 25-in. openings through the first floor slab for ventilation ducts, 12 in. by 24 in., into the basement mechanical room. A wood stud partition on the first floor would have to be removed, or at least breeched, and the ventilation ducts removed in order to gain cleared access to the two openings.

A horizontal wood door, or two of them, spanning in the 13 in. direction is proposed, held in place by positioning blocks on the bottom surface of the door and by wire cable ties to a cross member under the slab or to equipment in the mechanical room. The door can be constructed as indicated above for the stairwell closures.

Design of the wood closure principal members, those spanning in the 13-in. direction, should follow the same guidance and procedures used above for the stairwell closures; loading and positive phase duration would be the same. The following tabular values were readily calculated as before:

	$\mu = 3$		$\mu = 2$	
d in.	F_b psi	F_v psi	F_b psi	F_v psi
1.5	270	24	301	27

The results indicate that any stress-graded 1.5-in. thick wood members woul offer more resistance than needed. End bearing should be checked using Equation 6-57,* Reference 2 or 3, and the allowable compression stress perpendicular to grain $F_{c\perp}$ (from grading rules); however, the recommended minimum bearing length of 1.5 to 2 in. at each end is certain to apply to this case.

Flue Closure. Figure 17B shows the location of an opening 7 in. in diameter, adjacent to the ventilation duct openings just discussed. This circular opening is for a 5 in. flue running from the basement mechanical room straight up through the roof. The flue could be readily cut through with a hatchet or fire axe, and the 7 in. opening blocked with a wood plug.

Air blast room filling calculations, using the slanting guidance and charts referenced in connection with the discussion above of open shelter potential, show probable increases in interior shelter pressure as follows, if the flue opening is the only one not closed: with mechanical room door

* Extracted; see Appendix B herein.

open, entire shelter used as a filling chamber, about 0.21 psi peak interior overpressure with a 2.7 sec rise time, assuming a free-field overpressure of 16 psi; with door closed, mechanical room only used as a filling chamber, about 4.5 psi with a 1.3 sec rise time; all calculations are based on a weapon yield of 5 Mt.

J. Materials/Labor Summary

This section concerns materials and labor requirements for the expedient upgrading options in converting the basement EOC into a closed shelter against all nuclear weapons effects.

Stairway blast closure doors: Any reasonably good wood materials, preferably the structural or stress-graded lumber described above, plus some boards or plywood sheathing to hold each door together and nails. The labor estimate is one man-day per door with hand tools, and one-half man-day per door if a power saw is available.

Ventilation ducts closure door: Materials can be pre-purchased, preordered, or scavenged from the wood partition that must be at least partially removed to gain access to the location for the proposed door. The labor estimate is one-half man-day with new lumber, and one man-day with reclaimed lum. er.

Flue closure (optional but recommended): Pre-prepared, tapered wood plug, long enough to accommodate variation in diameter of circular opening, shown in structural drawing as 7 in., would be the only materials item needed. Labor may be considered as included in that for the ventilation ducts closure door.

K. Blast Upgrading Engineered Options

Although blast upgrading engineered options are outside the scope of this research project, the following comments thereon are submitted.

Recommendations concerning the stairwell closure doors would be essentially unchanged from the blast upgrading expedient option described above. For the ventilation ducts closure, however, one large or two small properly detailed and anchored, flat steel, horizontal sliding doors are recommended for design, construction, and installation on the bottom side of the first floor slab. A similar recommendation is made for the flue closure, subject to such limitations from building and fire codes as may apply.

III. HAMILTON AFB (CALIFORNIA) BUILDING NO. 424

Introduction

Incident to the closure and disposal of Hamilton Air Force Base, Novato, California, many government agencies at all levels became interested in potential uses for various buildings and portions of this large base and its tremendous facilities. The Defense Civil Preparedness Agency asked that this research project specifically make an existing structures evaluation of one building, No. 424, which is one of four apparently identical buildings built in successive years in the early 1930s. Building 424, built in 1934, is a typical "Company" building of the day, including barracks, mess hall, office and recreation spaces, and storage spaces such as to support completely one Army company. Intended to house approximately 200 men, it is constructed of reinforced concrete, except for the roof, and consists of a full basement plus three stories fully aboveground. Details follow herein.* A considerable amount of effort was expended on the building, because the interest was at a very serious level from some time. Especially time-consuming had been the efforts made to locate structural drawings of the building and to attempt to learn location and bar sizes of reinforcing steel in principal structural members. (The instrument purchased for such purpose so failed to live up to its promises that we were allowed to return it for full refund after about one month's use; it did, however, substantiate that rebars in the cover slab over the basement had not been placed in a careful pattern at all.) The one architectural drawing about 17"x22" that was found (obviously a reduction, probably 1/2 size) shows a few structural details and was used to the fullest; portions of it are shown in Figure 18. Figure 19 shows appropriately labelled photographs.

Although included in the study (and in the Quarterly Progress Reports previously mentioned), reference to work on building elements above the basement and its cover slab has been eliminated in the following material insofar as possible.

* Many details, not needed herein, have been provided in unpublished Quarterly Progress Reports to DCPA (7/15 and 10/15/75, and 1/30/76), the latest of which indicated that DCPA's interest in the building had waned.

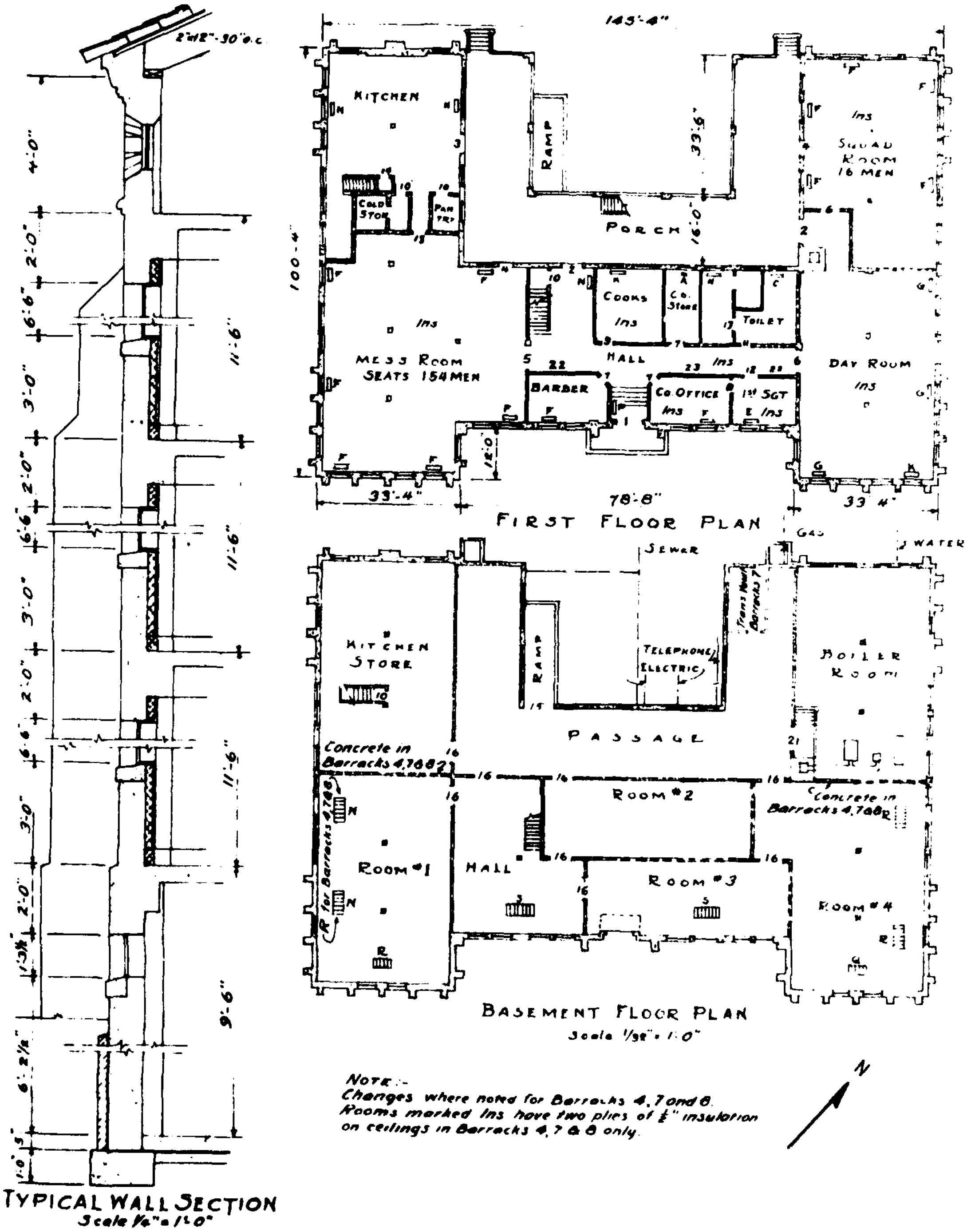


FIGURE 18 HAMILTON AFB BUILDING 424 FLOOR PLANS

A. FRONT (FACING FIFTH STREET)

B. REAR RAMP ENTRANCE

FIGURE 19 HAMILTON AFB BUILDING 424 PHOTOGRAPHS

Description of Building

Building No. 424 is located at Fifth Street and Hangar Avenue on Hamilton Air Force Reserve Base, California, and was constructed in 1934 as an Air Corps barracks for 200 men. The building consists of three stories, a basement, and an attic area. The building was visited by H. L. Murphy and C. K. Wiehle of SRI on May 21, 1975. A number of photographs were taken for study. Although building drawings were located at the Base Civil Engineer's Office and copies of some of these were obtained, unfortunately only one drawing showing a few structural details was ever located (Figure 18 shows a portion of that drawing); it shows floor plans and a typical wall section. The building was found to be excellent in both overall and structural conditions.

The building has a reinforced concrete frame, and the column spacing is generally 16 ft center-to-center in both directions. The columns in the basement area are 16-in. square. The floor system on all story levels, including the attic, consists of reinforced concrete solid slabs that span between the frame beams located along all column lines. There are no intermediate slab supports between column lines. The thickness of the first story slab over the basement area was measured at a number of existing slab penetrations, and the most probable slab thickness appeared to be 6 in.

The exterior walls are constructed of concrete throughout, and appear to be cast monolithically with the frame and floor slabs. The basement wall, up to the top of the first story floor slab, is 12 in. thick, and is about one-half exposed aboveground. In addition, many of the exterior walls have concrete buttresses that extend almost the full height of the building. The buttresses are probably reinforced and were apparently cast integrally with walls and frame; in any event, they would significantly increase the effective thickness of the walls for resisting the blast forces.

The location of the interior walls as originally constructed is shown on the floor plans in Figure 18. Although no typical interior wall details were found on the available drawings, the walls probably consist of tile masonry units.

Analysis

A dynamic analysis was performed to estimate the collapse overpressure level of the floor slab over the basement area, its supporting beams, and the exterior walls. In addition, a static analysis was made to estimate the collapse strength of the basement columns.

As stated, drawings showing structural details could not be located. Therefore, before a dynamic analysis could be performed, it was necessary to determine values of a number of physical parameters from other sources. The only definitive data available were the overall dimensions of typical elements of interest obtained by direct measurement in the field survey.

As one possible method of estimating the collapse overpressure of the various elements in Hamilton Building No. 424, an examination was made of all data previously collected by SRI for the analysis of NSS buildings for DCPA.[1,15,16] Although these data include information on a large number of wall and floor elements, none of the buildings was sufficiently similar to any others in type and date of construction to establish collapse overpressure levels for individual elements in Building No. 424.*

Therefore, for this case study, it was necessary to use a different approach. Since the exterior dimensions of typical elements were available, the primary problem remaining before realistic collapse predictions could be made was estimating the quantity and location of the reinforcing steel. An attempt was therefore made to duplicate the original design of each element by using the actual exterior dimensions, together with reinforced concrete design procedures in effect at the time of the original construction. Although the building code followed when designing the building was unknown, it was assumed that the 1928 ACI Joint Code, Building Regulations for Reinforced Concrete, was used. The candidate structural elements so designed were then analyzed using the computer codes developed for DCPA in the evaluation of existing structures program.

The primary elements of interest in establishing the blast shelter potential of the basement area were the first story floor system and the exterior basement walls. The design and analysis efforts were therefore limited to the floor system over the basement area, including an estimate of the basement column strength, and the basement exterior walls.

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* For comparative purposes, some of the previous analyses are used in a subsequent section.

A. Floor System

Slab. The first story reinforced concrete floor slabs are 6 in. thick and are supported on all four edges by beams or by a combination of beams and exterior basement walls. For this study, only the slab supported by beams was investigated, and it was assumed that the reinforcement was continuous at the supports. A preliminary analysis indicated that the design live load for the first story floor system was most probably 100 psi, which was then used in the study. The ultimate compressive strength of the concrete was assumed as 2,000 psi with a 1928 ACI Code design allowable unit stress of 800 psi. Since the area of steel is the most important parameter in determining the collapse strength of reinforced concrete members, three steel allowable unit stresses applicable at the time of the original design were selected for calculating the area of reinforcing steel for the candidate slabs. The design calculations resulted in selecting, for the dynamic analysis, three candidate slabs that had the following properties:

$L_L = L_S = 178$ in.*

$h_s = 6$ in.

$d = 4.25$ in.

$f'_c = 2{,}000$ psi

$f_c = 800$ psi

$f_s = 16{,}000$, $18{,}000$, and $20{,}000$ psi†

$f_{dyr} = 44{,}000$ psi†

Support case: two-way slab, fixed on four edges

$W = 1$ Mt

$S = 17$ ft (clearing distance)

The dynamic analysis resulted in the following predicted collapse overpressures for the three candidate slabs:

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* See Notation at end of this section.

† The three steel allowable unit stress values f_s represent those commonly in use when Building No. 424 was designed. However, the only reinforcing steel bars used during this period were A-15 billet steel, and therefore the dynamic yield strength f_{dyr} of 44,000 psi for structural grade was used for analyzing all members.

f_s (psi)	p_{so} (psi)
20,000	10.1
18,000	12.4
16,000	14.1

It should be noted that subsequent to the above slab analysis, structural drawings were obtained for a reinforced concrete barracks building constructed in 1938 at the Presidio of San Francisco. The Presidio barracks building, although not identical to Hamilton Building No. 424, is of the same type of construction. It is of interest that a first story slab in the Presidio building, with spans similar to the Building No. 424 slabs, has a thickness of 5-1/2 in. and an area of reinforcing steel equal to that calculated for the first candidate slab listed in the above table, i.e., using f_s = 20,000 psi.

Support Beam. The first story beams that support the floor slabs are 14 in. wide by 16 in. deep, including the slab thickness, and have a span between columns of 176 in. Although the beam candidates were designed in a manner similar to the slabs (i.e., by using the working stress method of the 1928 ACI Joint Code), there are several factors that result in less confidence in reproducing the beam designs for the Hamilton Building No. 424 than there was for the slab designs. First, the quantity of steel in a beam is dependent on both the floor loads and the frame forces, including lateral wind and earthquake forces. Since the method of frame analysis, if any, was unknown, there was no rational method to estimate the area of reinforcing steel included in the original design of the beams for resisting frame forces over that required for resisting floor dead and live loads only. Therefore, basement candidate beams were designed for vertical dead plus live loads only. Second, it was not known if the beams were designed originally as rectangular or T-beams. A preliminary analysis assuming balanced design, however indicated that they were probably designed as rectangular beams, and therefore the candidate beams in this study were so designed. The design calculations resulted in selecting, for dynamic analysis, three candidate support beams that had the following properties:

L_b = 176 in.

b_b = 14 in.

h_b = 16.5 in.

d = 14 in.

f'_c = 2,000 psi

f_s = 16,000, 18,000, and 20,000 psi

f_{dyr} = 44,000 psi

Support case: fixed, both ends

W = 1 Mt

S = 17 ft (clearing distance)

The dynamic analysis resulted in the following predicted collapse overpressures for the three candidate support beams:

f_s (psi)	p_{so} (psi)
20,000	3.6
18,000	4.0
16,000	4.5

Column. The reinforced concrete columns supporting the first story floor system over the basement area are 16 in. square, and there are 28-in. square pedestals at the column bases.* Since computer programs have not been developed here to predict the collapse of columns under dynamic loads, a static analysis was performed to provide an estimate of the strength of the columns relative to that of the floor elements. The 1928 ACI Code provides the following equation for calculating the permissible axial load for tied columns:

$$p = 0.225 f'_c A_g \left[1 + (n - 1)\ p\right]$$

with a steel ratio limitation of

$$0.005 A_g \le p \le 0.02 A_g$$

The vertical dead load acting on an interior column in the basement was calculated as 104 kips. If it is assumed that the floor design live load is 100 psf for the first story, 60 psf for the second and third

* From other studies and experience in existing structures evaluation for nuclear blast resistance, it was concluded that the footings in this building would be stronger in blast resistance than any other basement structural element.

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f_s (psi)	p_{so} (psi)
20,000	10.1
18,000	12.4
16,000	14.1

It should be noted that subsequent to the above slab analysis, structural drawings were obtained for a reinforced concrete barracks building constructed in 1938 at the Presidio of San Francisco. The Presidio barracks building, although not identical to Hamilton Building No. 424, is of the same type of construction. It is of interest that a first story slab in the Presidio building, with spans similar to the Building No. 424 slabs, has a thickness of 5-1/2 in. and an area of reinforcing steel equal to that calculated for the first candidate slab listed in the above table, i.e., using f_s = 20,000 psi.

Support Beam. The first story beams that support the floor slabs are 14 in. wide by 16 in. deep, including the slab thickness, and have a span between columns of 176 in. Although the beam candidates were designed in a manner similar to the slabs (i.e., by using the working stress method of the 1928 ACI Joint Code), there are several factors that result in less confidence in reproducing the beam designs for the Hamilton Building No. 424 than there was for the slab designs. First, the quantity of steel in a beam is dependent on both the floor loads and the frame forces, including lateral wind and earthquake forces. Since the method of frame analysis, if any, was unknown, there was no rational method to estimate the area of reinforcing steel included in the original design of the beams for resisting frame forces over that required for resisting floor dead and live loads only. Therefore, basement candidate beams were designed for vertical dead plus live loads only. Second, it was not known if the beams were designed originally as rectangular or T-beams. A preliminary analysis assuming balanced design, however indicated that they were probably designed as rectangular beams, and therefore the candidate beams in this study were so designed. The design calculations resulted in selecting, for dynamic analysis, three candidate support beams that had the following properties:

L_b = 176 in.

b_b = 14 in.

h_b = 16.5 in.

d = 14 in.

stories, and 40 psf for the attic, then the vertical live load on the column is 66 kips, for a total vertical axial column design load of 170 kips. Using the total vertical design load, the above equation provides a relationship between the steel ratio p and the concrete compressive strength f'_c that can be used to select candidate columns.* For the code-permitted range in p from 0.5 percent to 2 percent of the concrete area, f'_c varies over a rather narrow range from 2,800 psi to 2,400 psi, respectively.

To estimate the axial load capacity of a dynamically loaded column, the following equation from Reference 17 can be used:

$$P = (0.85f'_{dc} + pf_{dy})\ A_g$$

For the minimum strength column permissible under the 1928 ACI Code (i.e., $p = 0.005A_g$), the above equation yields a dynamic load capacity of 822 kips. Allowing for the dead load of 104 kips supported by the column, but not for the live load, and assuming a DLF (dynamic load factor) of 1.2, the column was estimated to resist a blast loading on the first floor of about 16 psi.

B. <u>Exterior Wall</u>

It was concluded that the collapse overpressure level of the exterior walls above the basement level in Building No. 424 would not directly affect the predicted collapse strength of the floor system over the basement.†

The reinforced concrete basement wall has a thickness of about 12 in., and is exposed aboveground for about one-half its height. It was not analyzed since the collapse overpressure level would be estimated to be greater than that obtained for the above-basement exterior two-way walls (10.1 psi). For example, for a 12-in. reinforced concrete wall, with minimum reinforcement and exposed aboveground for one-half of its 10-ft height, the predicted collapse overpressure level was about 17 psi (Reference 1). The basement wall in Building No. 424 is only 8 ft high, and

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* Where the modular ratio, $n = 30{,}000/f'_c$

† It should be noted that for extreme conditions, such may not be the case; e.g., for exceedingly strong first story exterior walls with very small openings, a greater free-field blast overpressure level would be required to produce a floor collapse pressure than would be the case for very large window openings or very weak walls.

since the minimum code steel requirement was greater than was used for the referenced wall, its collapse strength would also be expected to be greater.

C. Summary

Since structural drawings were not available for determining the quantity of reinforcing steel in the elements of the floor system of Hamilton Building No. 424, candidate slabs, support beams, and columns were designed with the working stress method according to the 1928 ACI Joint Code. The candidate slabs and beams were then analyzed dynamically using the SRI computer programs previously developed for DCPA for the evaluation of existing structures. The dynamic analysis of the first story floor system over the basement area indicated that the collapse strength of the floor slab is probably above the 10 psi blast overpressure level, while the collapse strength of the supporting beams is probably about 4 psi.

To determine if the column would develop the full strength of the floor system, a static analysis of candidate beams was performed. The results of the analysis indicated that the columns could be expected to resist a blast overpressure level of at least 15 psi, which is well above the strength of the floor system. The footings are more than adequate for the column capacity.

Design - Blast Upgrading Expedient Options

D. Blast Loadings

In addition to the foregoing evaluation of the basement and its cover slab, a blast resistance evaluation was made of upper floor members, as mentioned earlier. As a result of the work on the upper floors, it was concluded that the upper stories of this R/C building, less the roof structure, would remain standing (because of their many openings) under an air blast overpressure developed through room filling and sufficient to load the basement cover slab to collapse. Thus, in contrast to the free-field air blast situation applying to the first case study reported in Section II of this Chapter, the Building 424 basement cover slab blast loading used herein comes from air blast room filling into the first above-ground floor spaces. The basement cover slab evaluation was therefore based on a megaton-range weapon blast loading, but with a finite (non-zero) rise time; beam, column, and footing loadings follow, of course, from the loading on the basement cover slab.

The blast loading on the exposed portion of the exterior basement walls was taken as if each side of the building, in turn, was the side exposed to an advancing air blast front striking it normally - that is, a zero rise time fully reflected blast loading, decaying rapidly (as a function of clearance distances) to a stagnation pressure.

The blast evaluation work resulted in the finding that the basement exterior walls, columns, and footings were all blast resistant to such a degree that only the blast resistance of the basement cover slab and its supporting beams should be considered further in this section on blast upgrading design.

The evaluation work on the basement cover slab resulted in an estimate of 10.1 to 14.1 psi free-field overpressure to provide the room filling incident pressure on the slab sufficient to cause its collapse. The range of values is due to not knowing the designer's assumption as to the strength of rebars used: 16, 18, or 20 ksi. Evaluation of the support beams for the two-way slabs gave a similar range of values, in this case 3.6 to 4.5 psi, again depending on the rebar design strength assumed.

The final paragraph of Section A, Floor System, Slab (subsection) points to a parallel design situation at another old Army post in the area, wherein a rebar design strength of 20 ksi was used. The senior author also used the 1928 ACI code and an old ACI design handbook, plus memory (having taken his first concrete design course in the late 1930s), to conclude that the most likely design values used were 800 psi for concrete and 20 ksi for steel, with balanced design of a rectangular beam section, all of which led to a floor live loading of 100 psf, which was that prescribed at the time (dating from 1924) for the usage, as defined by the Department of Commerce. For possible interest, Figure 20 shows the references and the calculations. The same conclusions and design sources were used, as stated in the Analysis section, by the second author (Mr. Wiehle) in his evaluation work.

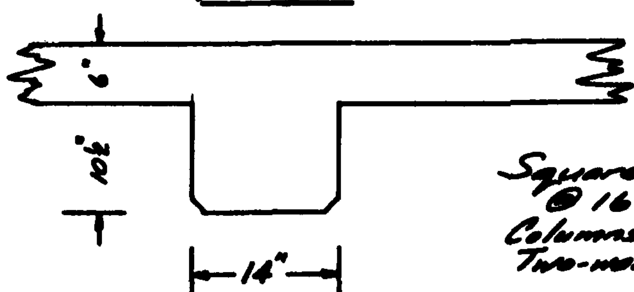
Pursuant to the foregoing: free-field overpressures of 10.1 and 3.6 psi were selected for use, based on the related incident overpressure resistance of the basement cover slab and support beams, respectively. (A lesson to be learned from this case study is to stress the need for structural design drawings, as-builts if possible.)

E. Open Shelter Potential

The building basement has the following openings:

Interior span, basement cover slab, support beam:

Analysis — *Refs:*



Square floor panels @ 16 ft. c-c of columns
Columns 16" square.
Two-way slab, on beams.

L = beam clear span = 176"	Assume f_c' = 2000 psi		
t = " th. = 16½"	f_c = 0.4 f_c' = 800 psi		1;306(a)
b = " width = 14"	(b) Assume f_s = 20000 psi		1;307(a)
	(a) or 18000 psi		"
	and f_v = 16000 psi		"
$K = f_c jk/2 = A_s f_s jd$ $= pbf_s jd^2$	Assume balanced design.		2;p.61
	(a)	(b)	
$A_s = pbd$ [2;p.45, Eq.1-4] K:	139	131	2;p.62
$M_r = Kbd^2$ [try d = 14"] M_r:	381+ in-k	359+ in-k	2;p.8
	31.8- ft-k	30.0- ft-k	

Contributory floor area
= [16']² × 0.5 = 128 ft²
= 128/(176÷12) = 8.73 ft²/ft beam

$M = wL^2/12$ $w = 12M/L^2$ = T.L.	w:	1774	1674	plf of beam — 1; Table(b); Eq.4
	w÷8.73	203	192	psf of fl. area
D.L. = $\left[\left(\frac{6''}{12} + \frac{10.5''\times14''\times12''}{1728}\times\frac{1}{8.73}\right)150\ \mathrm{pcf}\right]$;		92.5	92.5	psf " "
	LL:	110	100 [Most likely LL. value]	" " " — 2;p.114 (U.S.Dept.Comm.,'24)

1. Joint Code, Building Regulations for Reinforced Concrete, adopted as tentative standard by the American Concrete Institute and the Reinforcing Steel Institute, 1928.

2. Reinforced Concrete Design Handbook of the American Concrete Institute, Detroit, Michigan, First Edition (12/39 printing).

FIGURE 20 RECONSTRUCTION OF LIKELY DESIGN CALCULATIONS FOR HAMILTON AFB BUILDING 424

- Two stairways; kitchen about 3' wide, front entry about 4' wide
- Brick chimney, extending from the basement boiler room through the roof, measuring 4'7"x4'8" (o.d.) in the attic*
- Windows, about 37, each with about 16"x46" openings and with bottom edge about 6'0" above basement floor
- Loading ramp (small vehicle only) and doorway, about ' wide (Figure 19)

Hot water is used for heating as well as washrooms, and thus no ducts were found penetrating the basement cover slab.

The number of openings is such that there would be no significant reduction in the peak free-field overpressure due to room filling time (assuming a blast positive phase duration from a megaton-range weapon).

With a basement containing stud-and-plasterboard interior partitions in many places, as well as quantities of loose items (e.g., fluorescent light fixtures), the basement's open shelter potential is rather poor. If cleared of all loose materials, the blast resistance of the stud-and-plasterboard interior partitions should exceed (with all openings open) an equivalent free-field overpressure about that of the 3.6 psi estimated blast resistance of the basement cover slab support beams.

The real potential for this basement, however, is in a closed shelter configuration; it is posted as a National Fallout Shelter Survey shelter, meaning that it offers at least PF 40 fallout protection, and shows a capacity of 375 persons. For protection for that many people, the openings would be well worth closing to the blast wave, as is discussed below.

Whether used in the open or closed shelter mode, emergency power would be a necessity for keeping sumps clear (unless hand-operated bilge pumps could serve), because the basement is understood to be below the water table. The airstrip of this large AFB, which borders on San Francisco Bay, is understood to vary from plus to minus 4 ft relative to sea level.

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* The field party was unable to enter the boiler room conveniently.

F. Closed Shelter Potential

Because strengthening of the two-way R/C slabs over the basement would require a very large supply of materials and labor (say, for an additional line of support at mid-span in both directions of each two-way slab panel, about 16 ft square), the blast protection goal should be to exploit the blast resistance potential of these two-way slabs (about 10 psi free-field overpressure), by expediently closing all openings and by strengthening the slab support beams to at least the blast resistance of the slabs themselves.

For strengthening the slab support beams, two basic methods can be employed. The first method would add a steel beam or cable type support to increase the flexural or tensile membrane resistance of the existing beams. Such systems usually require some modifications to the building and therefore cannot be considered as expedient upgrading options available for implementation during times of crisis.

The second basic method of upgrading the floor system would provide direct support to existing beams such as by masonry walls for a continuous support, or columns that reduce the effective span of the beams. These approaches include those applicable to crisis periods, such as timbers useful as columns and easily installed when desired. Options also include crib type column supports that can be constructed during periods of crisis by unskilled labor using any available materials, such as 4"x4" timbers or RR ties.

Turning to the second of the two methods just described, the question arises as to how many added intermediate columns are needed to increase each slab support beam's strength by a factor of 10.1/3.6 (both psi), or about 2.8. By comparing positive and negative moments - first, those applicable to the mid-span of a bent of five equal spans (0.046 and -0.079 wL^2), and second, those for a propped cantilever simulating cutting the original span in half and correcting for the span change (0.0176 and -0.031 wL^2) - a strength increase ratio of about 2.55 is indicated, or from 3.6 psi to 9.2 psi. Assuming that the span is cut in half, however, implies a knife-edge mid-span added support; a finite width of support, say 1 ft, changes the calculations to give an increase from 3.6 psi to 10.5 psi, or more than enough for the need. Should a crib of 4"x4"s or of RR ties be used, then the reduced spans would be even smaller, but no quick conclusions on strength increase should be drawn, because the inflection point of the original steel would be farther away from its needed location for the reduced span lengths. In sum, one intermediate added support at mid-span should be sufficient for the expedient upgrading. Support beam loads would tend to redistribute, because of the strong likelihood of more "settlement" (by floor slab cracking) in the added than in the columns and footings.

Fallout protection for basement shelterees has been mentioned above in Section E, Open Shelter Potential. Closure of openings as described just above would increase the fallout PF, and berming (partially or fully) the exposed portions of the basement outer walls would increase the PF much more. Stacking of materials near the opening from the truck ramp might be appropriate, depending on the method chosen to close that opening (discussed below).

Initial nuclear radiation protection should be adequate for megaton-range weapon attack; it might require supplementary shielding from kiloton-range attack, especially from the low end of the range, assuming the same blast overpressure level hazard; such added shielding could come from the berming mentioned just above, plus the importation of a foot or more of earth on the basement cover slab.

Thermal radiation would offer no hazard to shelterees, provided that fuel sources have been cut off; the comments made in the preceding (Section II) case study also apply to this one.

G. Sources of Indigenous Materials and Labor

Such sources should be many and ample on a large AFB. (No guess will be made as to the situation that may prevail after disposition of this base.) It should be noted that Building 424 is one of four identical buildings; with a fallout shelter capacity of 375 each and an emergency blast shelter capacity of two to four times that, the potential for full effects shelter of just the case study building types is large indeed. Considering the four buildings alone, simply using the timbers and boards from the roof would provide more than enough wood materials for the work contemplated in expedient upgrading of the basement.

H. Design of Blast Closures and Beam Supports

Stairwell Closures. Spans are about 3 ft and 4 ft for the two stairwells, assuming a support ledge in the shorter span direction. The design procedure has been simplified (Appendix B) and would be applied just as it was for the Section II or first case study building herein (Section II, subsection I). Almost any good wood members 1-1/2 in. or 2-1/2 in. thick would be adequate for these closures at 3-ft and 4-ft spans, respectively; however, the design procedure is fast and accurate, with all dynamic correction factors already incorporated, and therefore its use is urged.

Chimney Closure. The question here is whether the blast wave can enter the chimney at its top (three stories above the basement) and get into the basement faster than the chimney can be broken through at the first floor level (above the basement). At any rate, remedial action would require some exploration as to the exact construction of the chimney, plus ingenuity, again because of the lack of structural drawings. A scheme that should be workable is as follows: working at the top of the basement cover slab, knock holes into the chimney on two opposite sides, such that needle beams may be inserted to cross the chimney opening into the basement (total closure should not be necessary, but near-total closure should be a goal). Bricks and debris from above and to the sides should, under blast effects, complete closing of the opening into the basement.

If the chimney flues are small enough, a situation may prevail similar to that in long tunnels; that is, the length to cross-sectional area ratio of the "tunnel(s)" may be such that greatly reduced blast pressure remains at the tunnel end. This too would require further consideration, based on detailed information about the chimney construction.

An alternate scheme may be possible, working from the boiler room end - that is, to simply fill the basement portion of the chimney with sand or other soil, knocking a hole in the chimney just above the basement cover slab to finish the filling job.

Basement Window Closures. These 16"x46" openings (about 37) could be easily closed with wood closures; any good wood of 1-1/2 in. thickness would be more than ample for the short direction span. However, the basement windows have heavy iron grillwork on each one, which would not have to be removed to place a wood closure "door." The simplest solution would be to simply construct the doors, then wire them to the grillwork (ordinary boards or plywood would then be adequate for the wood closures), with the bottom edge of each door resting on a window ledge.

Should a decision be made to "berm" the exposed basement walls, then sandbags, wood boards, or plywood could be placed against the grillwork of each window and the berm carried right up past the window.

Retaining the availability of the windows for ventilation has a strong attraction, however. Thus, a solution allowing quick closure of the windows against blast would be more desirable than any of those proposed above, except the one calling for removal of the grillwork and construction of wood blast closures (in which case, the wood closures would be simply wired to an interior wood strut to hold them in place; cleats on the closures, so positioned as to fit inside the window opening, would be of considerable help in holding each closure in the proper position).

Loading Ramp Closure. A closure constructed of leaning timbers* (or steel shapes), resting on the first floor slab edge at the top and on the ramp driveway at the bottom, running from ramp sidewall to sidewall, is recommended; the timbers or steel shapes should be amply sandbagged to hold them in place (or held by earth bulldozed against the leaning structural members). The longer the leaning members can be (i.e., the flatter the closure), the less will be any reflected blast pressure buildup.

Mid-span Supports for Basement Cover Slab Support Beams. The rationale for these supports has been discussed in the opening paragraphs of Section F above. Each column placed at mid-span must have its loading adequately spread on the existing floor slab to compensate for the lack of a footing; Figure 11 shows a scheme for these columns. If multiple members are used to make up each column, adequate fastenings (cleats, banding, sheathing) must be added to insure composite action of the cluster of members.

I. Materials/Labor Summary

This section concerns materials and labor requirements for the described expedient upgrading options in converting the basement of Building 424 into a closed shelter against all nuclear weapons effects.

Stairwell blast closures (2): Removal of stud-and-plasterboard partitions to clear the area around each stairwell opening on top of the basement cover slab would provide an ample stock of 2x4s (if not, more first floor partitions can be torn down) for constructing these two wood closures; any reasonably good wood used flat-wise should be adequate, but if a check using Appendix B does not confirm the adequacy, such wood

* Appendix B could be used to check adequacy of available timbers.

can be used on edge and should certainly prove adequate. Materials: about 100 sf of reasonably good wood, plus some pieces used as cleats to hold door in position within the stairwell opening (also wire and a cross-member(s) in the basement to anchor each closure door). Labor: assuming availability of a power saw, about 4 man-days including partition scavenging. (For timber-supported, sand-bagged closures, add 80 lf 4x6s or larger and 100 filled sandbags, delete partition demolition, and cut labor estimate in half.)

Chimney closure: Method depends considerably on further exploration of construction details, but assuming that the several-needle-beams approach is workable, materials/labor estimates follow. Materials: about 20 lf 4x4s or steel equivalent. Labor: 1 to 2 man-days, depending on brickwork to be broken through.

Basement window closures (37): Any reasonably good boards will be adequate, because the closures will be supported by imbedded iron grillwork in each window; boards should run in the long direction, because of a window ledge protruding from the wall at each opening, and cleats should be on the outside; closures can be wired to the grillwork on the inside; windows should be modified so that they can be fully opened, thus protecting shelter occupants against glass breakage should there be low blast leakage. Materials: 250 sf boards (or plywood say 1/2 in. or thicker), plus material for cleats, nails, wire, etc. Labor: assuming availability of a power saw, 4 man-days.

Loading ramp closure: Materials: timbers, 6x6s or larger, 10 to 14 ft, sufficient to cover 7-ft width (or 14 timbers if 6x6s); and 50 filled sandbags. Labor: 2 man-days.

Mid-span supports for basement cover slab support beams: Scheme would resemble Figure 11, whether made of timbers or heavy-walled pipe or other material. Materials: 600 lf 8x8s or larger timbers, plus material for wedges and grillages. Labor: assuming a large power saw is available, 7 man-days. And estimated 60 supports would be needed.

J. Blast Upgrading Engineered Options

Although blast upgrading engineered options are outside the scope of this research project, the following comments are submitted.

Stairway and window closures could be engineered, built, and stored on a wall space near the expected point of use if ever needed; wood will most likely be the material of choice, even in an engineered approach.

The chimney closure could be a sliding steel door, engineered for insertion at or just above the top side of the basement cover slab. All pipes, such as the hot water pipes used for heating and washrooms, could have cutoff valves installed at appropriate locations, probably near either the inner or the outer surface of the basement exterior walls and cover slab. The loading ramp closure could be engineered, with materials stored or pre-positioned. Concerning strengthening of the support beams for the basement cover slab, an engineered approach was briefly described above in the second paragraph of Section F.

NOTATION

A_g	Gross area of column, in.^{2}
b_b	Width of rectangular reinforced concrete beam, in.
d	Distance from compressive face of reinforced concrete member to centroid of tension steel, in.
f_c	Allowable (design) compressive stress in concrete, psi
f'_c	Static compressive strength of concrete, psi
f'_{dc}	Dynamic compressive strength of concrete, psi
f_{dy}	Dynamic yield strength of reinforcement, psi
f_{dyr}	Dynamic yield strength of reinforcing steel, psi
f_s	Allowable (design) tensile stress in reinforcement, psi
h_b	Depth of reinforced concrete beam or T-beam, in.
h_s	Thickness of slab, in.
L_b	Length of beam, in.
L_L	Length of slab in long direction, in.
L_S	Length of slab in short direction, in.
n	Modular ratio, E_s/E_c
P	Column allowable axial load, lb
p	steel ratio, tension steel
p_{so}	Peak incident overpressure, psi
S	Clearing distance, ft
W	Weapon yield

IV. MIDDLEFIELD PARKING GARAGE

Introduction

The third case study building, a single-level parking garage under a parking lot behind an office building at 200 Middlefield Road, Menlo Park, California, was visited by the two senior engineers working on this project, and a set of structural drawings for the building was obtained on loan. While the parking garage has had a considerable amount of floor space partitioned off for storage, this study considered its as-built (as-designed) state, because the partial conversion to storage purposes has been made by a tenant with a lease expiring next July.* The field visit indicated that only open, not closed, shelter would be an option worth considering, because of the large ramp opening; however, this conclusion was later changed.

Description of Building

The Institute's Middlefield Facility consists of a two-story wood-frame building with both street level and underground parking areas. The underground garage, which is of interest for this study, is fully buried, about 161 ft by 195 ft, and located primarily below the street level parking area, with a small portion of the garage under the building as shown in Figure 21. The openings into the garage consist of an elevator doorway, a vehicle entranceway that is about 8 ft high by 24 ft wide at the garage floor level, a pedestrian entranceway into a stairwell leading directly into the building, and two 28 in. high by 36 in. wide ventilation exhaust fan openings, both in the easterly wing.

The roof system over the garage consists primarily of one-way concrete joists supported by reinforced concrete girders spanning easterly-westerly between circular concrete columns. A typical juncture of joist, girder, and interior column is shown in Figure 22. For the small portion of the garage roof system that also supports part of the building, a combination of R/C beams and joists, as well as girders, is used. All concrete slabs are 4 in. thick. A structural calculations sheet shows 99 psf total D.L. on joists: for slab and joists 84, sprinklers 1, and asphalt wearing course and membrane 14.

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* 1977

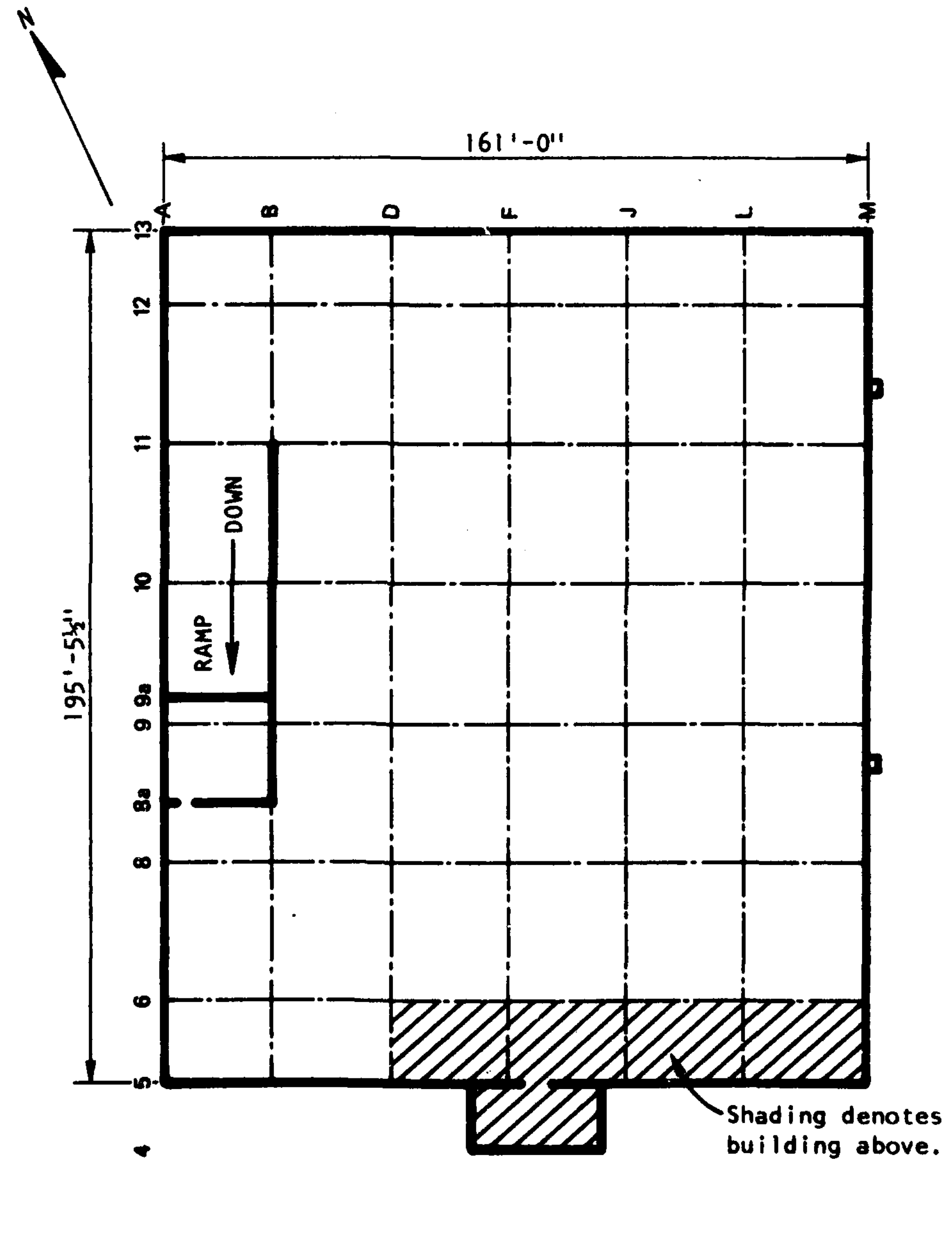


FIGURE 21 PLAN VIEW, MIDDLEFIELD UNDERGROUND GARAGE

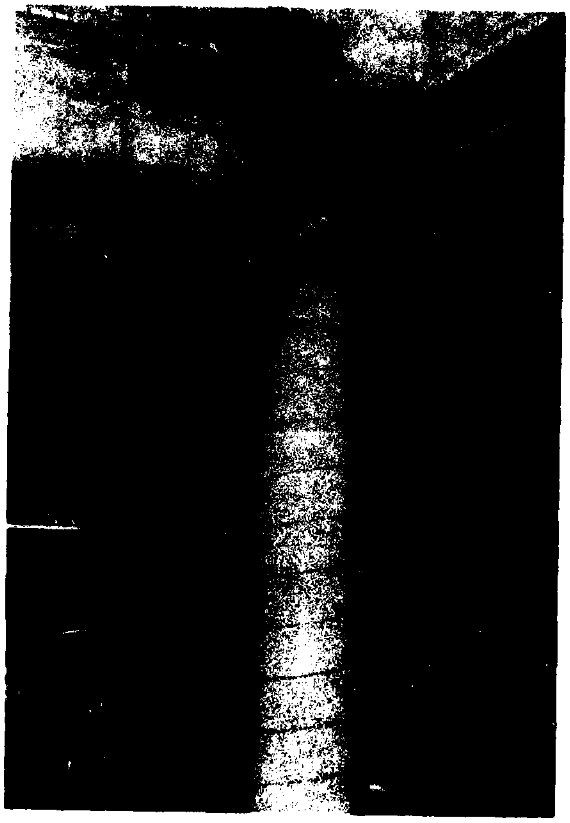


FIGURE 22   MIDDLEFIELD UNDERGROUND INTERIOR STRUCTURAL DETAIL PHOTOGRAPH

All exterior walls in the garage are 6-in. thick reinforced concrete, spanning vertically between the floor slabs and horizontally between exterior square columns. The wall reinforcement is typically vertical #5 bars on 9-in. centers, and horizontal #4 bars on 18-in. centers. The horizontal reinforcement is extended into the exterior columns.

Analysis

The field survey indicated that the critical elements for determining the blast strength of the garage area were the joists and beams, girders, and columns (Figure 22). The SRI building evaluation procedure[11] was therefore used to analyze the collapse strength of selected joists and girder elements, and a static analysis was made to check the adequacy of the columns.

An examination of the reinforcement schedule for the various structural elements indicated that the collapse strength of the garage roof system could be adequately represented by analyzing a few typical elements in each of the three areas shown on Figure 23. The results of the dynamic analyses of the various elements are summarized as follows:

	Predicted Mean Collapse Overpressure	
Area	Joist	Girder
1	3.6 psi	3.5 psi
2	5.9	4.3
3	5.4	7.7

The collapse overpressure of the slabs spanning between adjacent joists were far in excess (> 30 psi) of that of the main structural members.

The static analysis of the spirally reinforced columns and their footings showed that they would support the maximum dynamic reactions of of the girders.

Design - Blast Upgrading Expedient Options

A. Blast Loadings

As indicated in the preceding section on Analysis, and from discussion with the engineer who made the existing structures evaluation, the roof system slabs, columns, and footings, as well as all R/C walls (all

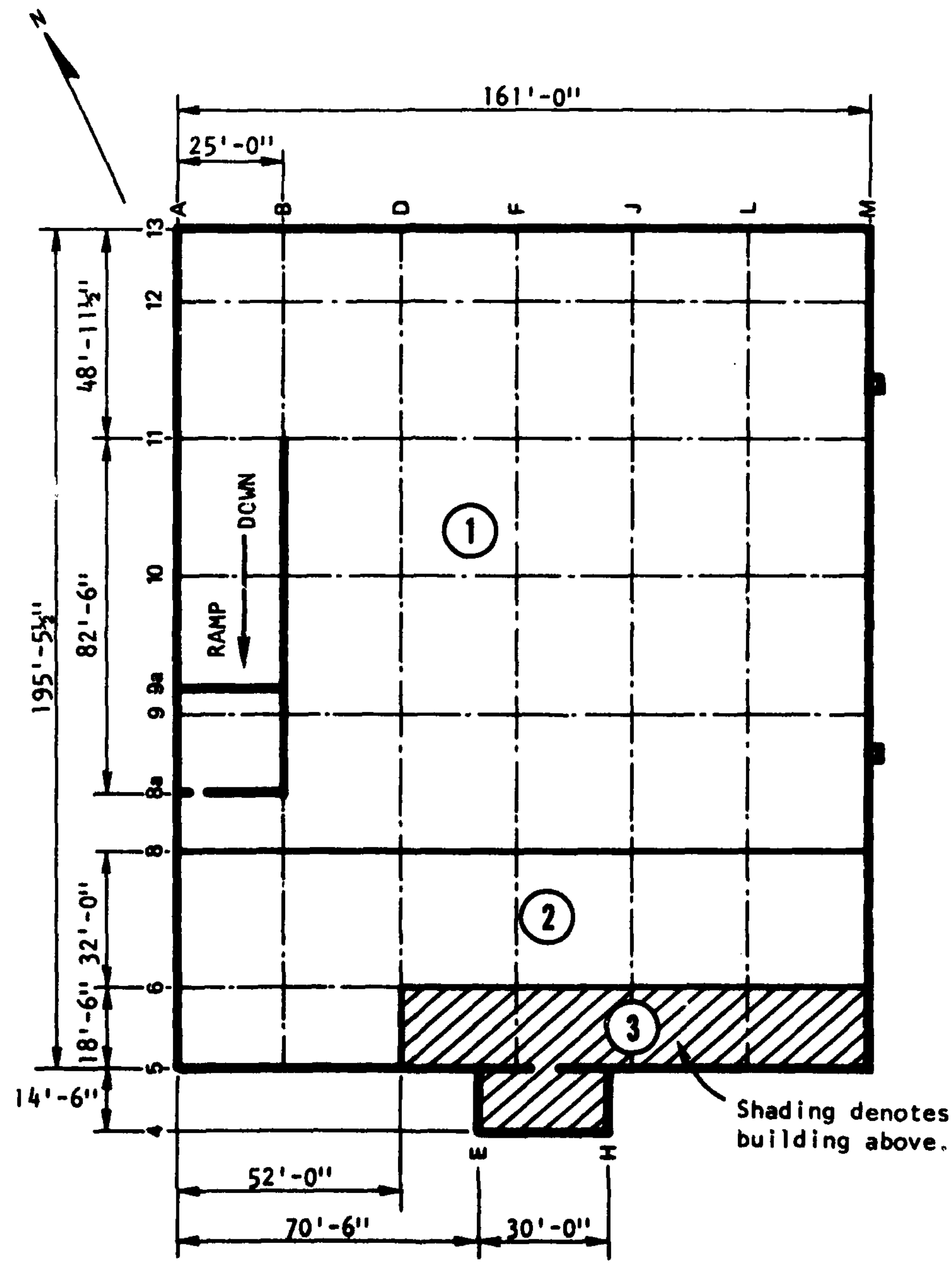


FIGURE 23 PLAN VIEW, MIDDLEFIELD PARKING GARAGE, SHOWING STRENGTH ZONES

solid lines of Figure 21), are considerably stronger than the roof joists and girders; for the latter two member types, their predicted mean collapse overpressures(by areas shown in Figure 23) are tabulated in the preceding section, where joists are resistant to 3.6-5.4 psi and girders to 3.5-7.7 psi. Paralleling the Section II case study, and contrasting to the Section III case study, these overpressure values are both the free-field and applied (side-on) pressures.

In an open shelter mode, the following openings would allow blast entry:

- Stair shaft opening, about 7'x8'6"
- Elevator shaft opening, about 5'x6'
- Ventilation shaft openings (2), about 2'4"x3' (with vertical wall openings, 2'4" high by 3' wide)
- Vehicle entranceway, about 8'x24'

All openings other than the one for vehicles could be readily closed with wood doors, all as described frequently in the two preceding case studies. The interior blast behavior would be such that the exterior garage wall opposite the vehicle entranceway would be heavily loaded, and cracked but unlikely to be damaged significantly under overpressures considered herein. The interior air blast effect on shelterees and materiel would be that of a very high wind traveling around the garage perimeter walls, decreasing somewhat in velocity with distance along the perimeter. A safe area for prone humans would probably be that bounded by the first column line in from each boundary wall.

B. Open Shelter Potential

One open shelter scheme would be as described just above; that is, close all openings listed in the preceding section except for the vehicle entranceway. Shelter capability in terms of blast resistance would then be as tabulated in the section on Analysis: area 1 (Figure 23), about 3.5 psi; area 2, about 4.3 psi; and area 3, about 5.4 psi. Resistance to other weapons effects, under this open shelter scheme, would be approximately as follows:

- Fallout PF, considering the average overhead mass (about 100 psf), should be only a little short of 100, higher nearer the outer walls as in all basements, but much lower for locations nearer to the vehicle entranceway (unless materials are placed to create local shielding, as needed, after the direct effects have passed) (Reference 2, Chapter 5).

- Initial nuclear radiation - comments are similar to those in the two preceding case studies.
- Thermal radiation would provide no hazard, except to anyone in a line of sight to the detonation; all flammable (e.g., gasoline) sources should be removed from the shelter vicinity.

A second and a third open shelter schemes would be as above, plus strengthening of roof joists only, or of both roof joists and girders, respectively, as described in the following section on closed shelter upgrading schemes. (To perform the work required, however, would be rather impractical to gain either scheme of open shelter, because for some further work - i.e., closing the vehicle entranceway - the result would be a closed shelter offering better protection from all effects and with a full shelter rather than partial shelter capacity.) For both additional open shelter schemes, only open-work (not wall) strengthening approaches should be considered, and protection available against other effects than blast would be as described for the first open shelter scheme.

C. Closed Shelter Potential

Two general schemes of blast upgrading in a closed shelter mode seem to offer a reasonable return for an admittedly substantial effort - stated another way, the candidate structure is more lightly built than the others considered earlier herein. Both schemes require closing all openings listed in Section A.

The first scheme would include strengthening all joists by providing a line of mid-span support, either additional girder lines (intermediate between all existing girder lines) with many columns so as to reduce required girder strength and amount of grillage to spread footing loads, or walls (such as of concrete block, stackable type or those using mortar, or unclad heavy stud "walls"). The result should be an increase in joist strength of 2.5 to 2.7 times, under the same approach presented in the preceding case study (Section III, subsection F and H). Girder strength would be doubled, because the contributory area for loading each existing girder would be halved. This scheme, then, should offer a shelter blast resistance capability of either, say 2.6, times the joist strengths, or twice the girder strengths, of the overpressures tabulated in the Analysis section above; thus, by areas of Figure 23: area 1, about 7.0 psi; area 2, about 8.6 psi; and, area 3, about 14.0 psi.*

* The reader should understand that values shown throughout this report do not imply, by number of significant figures or decimal places, a degree of accuracy, but are shown simply to assist the reader in following a line of thinking, reasoning, or calculation.

The second general scheme of blast upgrading in a closed shelter mode would include strengthening both the existing joists and girders. The joists would be strengthened as described in the preceding paragraph. The girders would be strengthened at their mid-span points, in a manner similar to that described for the cover slab support beams in the preceding case study (Section III, subsection F and H). Under this second scheme of upgrading, then, both joists and girders would have their blast resistance increased by, say 2.6, times the overpressure strengths tabulated in the Analysis section above; thus, again by areas of Figure 23: area 1 blast resistance, about 9.1 psi; area 2, about 11.2 psi; and area 3, 14.0 psi. From the resulting values for the first and second scheme, it is obvious that girder strengthening in area 3 need not be done because no increase in blast resistance results; however, it may noticed from Figure 23 that area 3 is very small, and thus the difference in work required would not be significant. Further, it may well be that the second scheme of strengthening does not offer sufficient increase in blast protection over the first, to warrant the work of strengthening the girders.

Protection against fallout, as well as initial nuclear and thermal radiation effects, would be as described in the preceding (open shelter) section, except that fallout protection would be much improved by the addition of the ramp closure.

D. Sources of Indigenous Labor and Material

Material sources near the structure are few, that is, within four or five blocks; large trees exist on nearby property, and the parent structure, a two-story woodframe office building, could be cannibalized. However, the building is located near the midlength of a stretch of cities continuous over some 60 miles, varying from a mile to several miles (and several cities) wide; no shortage of materials supply firms and equipment yards exists.

Labor supply would include many persons with construction skills to various degrees; thus no shortage is foreseen in terms of skills suitable for expedient blast upgrading.

E. Design of Blast Closures and Joist/Girder Supports

Stair Shaft Closure. With a horizontal opening about 7'x8'6", the construction of a wood closure presents no new problems over other closures discussed in the preceding case studies. Most any reasonably good wood members can be used (e.g., knocking the 2" x4" studs out of some interior building walls, for use either flat-wise or on edge), and their adequacy can be readily checked by use of Appendix B. It will be necessary to knock partition plasterboard lower edges out of the way, in order to gain a first floor level ledge (on top of the wall bottom plate) on which to rest the wood closure over the shaft. Closure should be anchored.

Elevator Shaft Closure. In this case the horizontal opening is about 5' x6', but otherwise the comments of the preceding paragraph apply fully to this closure. In its lowered position, the hydraulic-lift elevator will offer no obstruction. An alternate approach is shown by Figure 4.

Ventilation Shaft Closures. These two shafts have horizontal openings at their tops that are about 2'4" x3', as well as openings through the parking garage exterior easterly wall (Figure 21) with about the same dimensions. Construction and adequacy would follow the same approach as presented in the preceding paragraphs. For the "open" shelter modes described above, it would matter little whether the closures were located at the shaft tops or on the outside face of the exterior walls; for closed shelter, location at the shaft tops might be preferable, in order to keep the shaft(s) clear as possible emergency exits/ ventilation ducts, the latter for use both preceding and following the blast wave.

Vehicle Ramp Entranceway Closure. With an opening about 8' x24', the closure construction would parallel that described in the preceding case study (Section III, subection H).

Mid-Span Supports for Roof Joists. The rationale and two approaches for this scheme are discussed in Section F above. In sum, construction of an additional line of girders equal in strength to, and intermediate between, each line of existing girders, as well as related column/ footing structures (see Figure 11), is required; an alternate would be a heavy stud wall with wide top and bottom plates and studs well blocked (for example, 2x6s or heavier, closely spaced, say 2" to 3" depending on member size, with 6x6 top and 8x8 bottom plates).

Mid-Span Supports for Roof Girders. The rationale and approach for these supports are discussed in Section C above, which in turn refers to the preceding case study (Section III, subsection F and H); reference is is made there to Figure 11.

F. Materials/Labor Summary

This section provides materials and labor summaries for the expedient upgrading options described above, for both open and closed shelter modes (the only difference being the lack or use of a vehicle ramp closure).

Stair shaft closure: Materials: about 80 each 2x4s 8 ft long; obtain if necessary by demolishing cross walls of office partitioning (hallway and outside walls are almost all load-bearing). Labor: demolition, about 4 man-days; construction, about 2 man-days; total 6 man-days if demolition is necessary.

Elevator shaft closure: Materials: about 60 each 2x4s 6 ft long (min.). Labor: about 2 1/2 man-days demolition, about 1 1/2 man-days construction; total 4 man-days if demolition is necessary.

Ventilation shaft closures (2): Materials: assuming closing at top (or horizontal) openings, use parts of solid core doors, toilet partitions, or removed desk tops, all from within the office building; anchor with filled sandbags or earth mounds, or by wiring to a cross-member inside the garage at the exterior wall opening in each case. Labor: 1/2 to 1 man-day.

Vehicle ramp closure: Materials: about 10 vertical heavy timbers 6x8 minimum and at least 10 ft long (or equivalent in steel shapes, telephone poles, trees, etc.), covered by scavenged 2-in. thick (min.) timber, with small openings sandbagged. Labor: about 8 man-days.

Mid-span supports for roof joists (5 lines, 160 ft long each): Materials: about 2,300 lf of 8x8 timbers (about 80 for columns approximately 8 ft long, for spacing on about 10 ft centers; about 800 lf for girders at top of columns and similar amount for "footings" at bottoms); or equivalent in structural steel shapes, heavy steel pipe (for columns), telephone poles, RR ties, etc. Labor: about 20 man-days if materials are reasonably available; if not, upgrading of this structure may be a poor choice or impractical.

Mid-span supports for roof girders (33 columns): Estimates and scheme follow those for the similar section in the preceding (Section III) case study, wherein 60 column supports patterned after Figure 11 were needed. Materials: 350 lf of 8x8 or larger timbers, plus materials for wedges and grillages. Labor: assuming a large power saw is available, about 4 man-days.

V. WEST PAVILION, STANFORD UNIVERSITY HOSPITAL, STANFORD, CALIFORNIA

Introduction

Because of the unusually heavy research effort expended on the earlier case study of Section III, recourse was made to earlier research data wherein selected NSS buildings had been evaluated for blast resistance in their existing condition,[1] thereby allowing the effort under this project to be applied directly to blast upgrading options for the building selected. Three buildings were selected initially, from which this case study building was later selected; the first selection (from 50 buildings in the report) was aimed at variety in building types, considering those buildings already selected locally and reported herein in Sections II and IV, and the second selection (from three buildings to this one case study building) was a chance one.

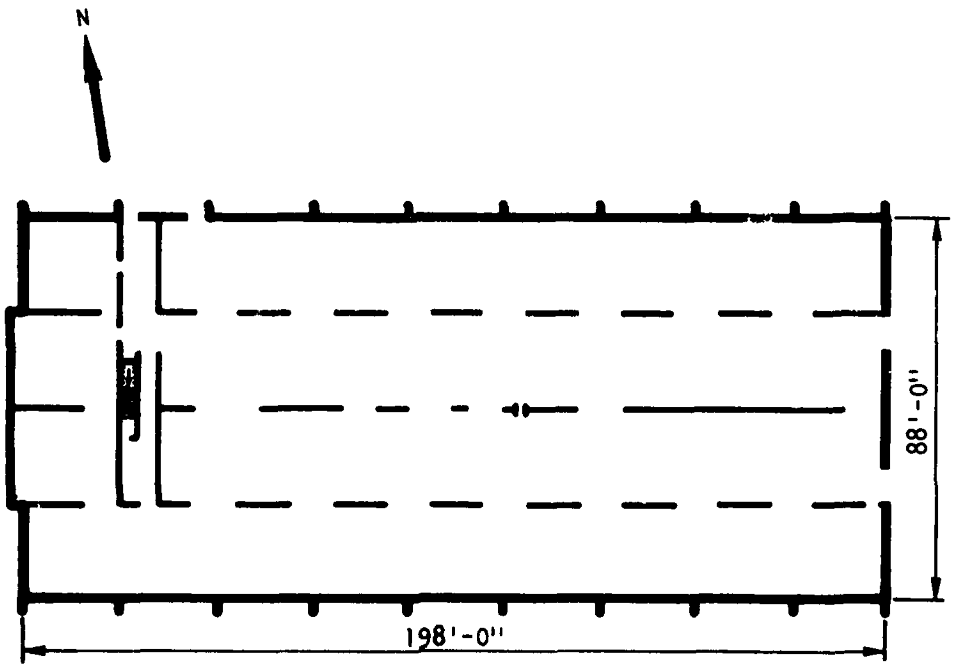
The latter selection proved to be a fortunate one, because it shows a building in considerable contrast to the one in the immediately preceding case study, wherein a considerable expedient construction effort is required to increase an inherent blast resistance of about 3.5 psi free-field blast overpressure to some 7 to 14 psi; a relatively minor expedient construction effort, as described in the remainder of this case study, serves to exploit an inherent mean collapse blast resistance of about 10.7 psi (free-field blast overpressure, again used as the incident or side-on overpressure with zero rise time).

Description of Building

The Stanford University Hospital West Pavilion, constructed in 1960, is located at 300 Pasteur Drive, Stanford, California. The West Pavilion, which was considered in this study, is one of several wings extending from the central core of the hospital. The building consists of three stories and a fully buried basement. The overall height of the building is 38 ft (aboveground), and the pavilion basement plan dimensions of 88'4" by 202'6" provide a gross area of about 17,900 sf, representative of each story level. Figure 24 shows the ground floor (basement) plan.

The building has a reinforced concrete frame with exterior columns and interior reinforced concrete load-bearing walls. The floor system consists generally of 12-in. thick, transverse R/C tube slabs, but with

SOUTHWEST CORNER



Note: All interior walls shown are 10" thick R/C.

FIGURE 24 PHOTOGRAPH AND GROUND FLOOR PLAN OF WEST PAVILION BUILDING OF STANFORD HOSPITAL BUILDING 239

12-in. thick by 24-in. wide solid slabs along transverse column lines (on 22-ft centers). The 7-in. diameter tubes are on 10-in. centers. The 12-in. thick tube slabs span between the exterior walls, and are continuous over three R/C interior walls (10 in. thick).

The basement interior partitions are constructed of 10-in. thick R/C, 6-in. thick concrete block, and 4-in. thick timber stud walls. All exterior basement walls are 12-in. thick R/C.

The openings into the basement consist of one exterior doorway leading into an areaway, one interior stairwell, and two corridor openings leading from the pavilion (or wing) into the building central core; the basement wing has no exterior windows.

Analysis

An examination of the structural drawings and reinforcement schedule indicated that the floor system over the basement area was sufficiently uniform to permit analyzing only one representative slab. The dynamic analysis of the slab indicated a predicted mean collapse overpressure of 10.7 psi.

A static analysis of the 10-in. thick interior R/C support walls and their continuous footings showed that they would satisfactorily support the maximum dynamic reactions of the floor slabs at the 10.7 psi overpressure level. Experience gained from other analysis/evaluation studies clearly shows that the 12-in. thick R/C exterior walls and their continuous footings have ample blast resistance for the maximum dynamic reactions of the floor slabs and any lateral soil loading including static and dynamic.

Design - Blast Upgrading Expedient Options

A. Blast Loadings

All blast loadings are taken as 10.7 psi side-on overpressure (or slightly more), for reasons stated in the Introduction and Analysis sections. This value is used herein for the stair shaft horizontal opening in the basement cover slab, and for the two corridor openings from the pavilion (wing) into a central core (there would be some room filling rise time working to counter any tendency toward reflection build up). For the remaining opening of the four described in the Description of Building section above, the doorway leading into an areaway, a fully reflected peak overpressure (as if applied to the front face of an aboveground rectangular structure) is reasonable, meaning about 30 psi peak value (Reference 14, p. 123).

B. Open Shelter Potential

Although the concrete bearing walls (10 in. thick) should present no hazard, concrete block walls, stud-plasterboard walls, and all kinds of unanchored materiel represent potential missile hazards to shelterees in an open shelter mode. The high protection level and relatively low upgrading costs of a closed shelter, as discussed in the following sections, dictate against further consideration of an open shelter mode.

C. Closed Shelter Potential

No structural member strengthening would be required for a closed shelter to exploit the blast resistance potential of the basement cover slabs, 10.7 psi mean collapse overpressure. To increase this blast resistance would require four additional support lines the full length of the basement wing, plus restudy of other existing structural members, clearly a non-cost-effective strategy. Accepting the inherent blast resistance of the basement structure, a closed shelter mode would require expedient upgrading work in closing the following basement wing openings:

- Exterior doorway into areaway: opening is about 4'3"x7'; drawings show a second such opening, blocked with a CMU wall and flush with the outer surface of the basement wall, but a site visit revealed a heavy supplementary external wall (set out about 3 ft), more than covering this second opening, and under local code requirements, this CMU wall would be reinforced. The conclusion was that the second "opening" need have no upgrading attention. The short remaining "as-built" wall and the closure of this exterior doorway into an areaway would be subjected to a reflected overpressure, as discussed under Section A above, and time did not permit a reevaluation of this short wall section (for itself and as a support for the doorway closure). For the purposes of this report, it is recommended that the area between this short wall section and the stairway it faces (the floor area is bounded by heavy interior walls on both sides) be kept clear of shelterees during arrival of the blast wave, because of the judgment that only localized damage would occur, if indeed any failure occurred at all.

- Stair shaft opening: there are 14 stair steps between the basement floor level and a landing, then 7 steps between the landing and the first floor level, meaning that the stairwell walls are short enough in height between the landing and the first floor level to resist air blast at the overpressure under consideration herein. Thus the horizontal opening in the stair shaft requiring a closure would be at the level of the landing and about 3'6"x15'; the type of closure would be that of Figure 5 (according to the drawings, the underside of the lowest flight of stairs is accessible; it is used as a storage space).

- Corridor openings (2) from wing into central core: openings are 8'x12' (high), in walls (at central core end of wing); walls are 10-in. R/C each for core and wing, with 6" gap between walls. An interference to expedient construction of closures is that CMU walls, flush with the opening vertical faces, extend toward the central core.

Protection against other weapons effect would be excellent. Fallout: the wing is a posted NFSS shelter, has been analyzed officially as worth at least PF 40, and considering all of the R/C and CMU construction plus a basement that is fully buried, is probably good for PF 100+. Initial nuclear and thermal radiation: protection would be of a high order in this basement shelter, again because of the entire building's heavy construction, all of which would be available for protection against these effects, occurring as they do before blast arrival.

D. Sources of Indigenous Materials and Labor

Building location is within the same 60-mile stretch of cities described for the preceding (Section IV) case study, meaning that there are many materials sources outside the immediate vicinity of the building; within the latter, there are separate facilities engineering/construction/maintenance departments for the University and the University Hospital, with their own stocks and construction yards, as well as a mixture of construction and engineering skills in their labor supply; large trees abound on University property. Within the Hospital building, many stud walls are non-bearing and thus are available for cannibalization.

As before, the labor supply would include many persons skilled and semi-skilled in construction; thus no shortage is foreseen in terms of skills suitable for expedient upgrading for blast.

E. Design of Blast Closures

Exterior Doorway-Areaway Closure. Any reasonably good wood, spanning the horizontal dimension (4'3"), could be used for this closure; 2x4s used flat-wise or on edge whould be adequate, but a check can be made (on the particular materials available) by use of Appendix B. A support edge 3-1/2 in. wide exists on the sidewall edge of the doorway opening, with a very wide edge available on the other side. Doorway opening height is about 7 ft (estimated, because complete drawings were unavailable).

Stair Shaft Closure. The scheme proposed is a horizontal wood closure at the level of the lowest stairway landing; as described in Section C, the scheme of Figure 5 appears to be the first choice. (An alternate, if one is needed, might be a scheme similar to that used in the Section II case study, Figure 17.) The horizontal opening to be closed is bounded by solid concrete walls and is about 3'6"x15'.

Corridor Closures (2), Wing-Central Core. Openings are 8 ft wide by 12 ft high. Several closure schemes could be used, perhaps; the one described has been used for preparing the labor/materials estimate: outside the double-wall (two 10-in. R/C walls with 6-in. gap, the latter closed for about the last 9 in. at any opening), knock holes in CMU walls and install about four horizontal needle beams, say 8x8 timber, heavy poles, or equivalent steel shapes, with perhaps 1 ft gap at top and bottom. Remove suspended ceiling as needed. Close openings with 2x4s (good wood) or 2x6s (poorer wood) on edge, running full 12-ft height, or splicing only near mid-height (to care for 1-ft cantilever ends). (Another scheme, for example, might be to drive one flange of a T-section into a sawed groove in the CMU walls just at the exterior face of the double-wall opening being closed; the other flange would then be used to support horizontal members installed side-by-side to close each opening, abutting the T-section web and perhaps sandbagged in place.)

F. Materials/Labor Summary

This section concerns materials and labor requirements for the upgrading expedient options described above, to make a closed combined nuclear effects shelter out of the hospital wing.

Exterior doorway-areaway closure: Materials: about 40 sf of 2-in. thick dimension lumber/timber. Labor: about 1/2 man-day.

Stair shaft closure: Materials: about 60 sf of 2-in. thick (min.) timber material, about 70 lf of 6x8 (min.) beam material, and 30 filled sandbags. Labor: about 2 man-days.

Corridor closures (2), wing to central core: Materials: 900 lf of 2x4s (for use on edge) and about 80 lf of 8x8 dimension timber. Labor: about 4 man-days.

Chapter 6

ADDITIONAL WORK NEEDED

Overall Objective and Status

The general objective of the present and needed further work in the area of blast upgrading options for existing structures, primarily building basements, is to develop a set of expedient and engineered techniques for upgrading potential shelter to be used by personnel remaining in target areas for the performance of essential functions. The techniques need also to be applicable to upgrading of shelter in the general population relocation areas (CRP). The result of the work will be a report that can be readily converted to an illustrated "how-to-do-it" manual.

The present work has resulted in the development of techniques for the expedient closure and strengthening of basement shelter space in a general way, and in evaluation of several specific building basements as examples. In the expedient portion, only general methodology and materials were considered without attention to the degree of blast protection provided. It remains necessary to provide degree of protection guidance as a function of materials and methods. The specific building examples include an evaluation of the existing blast resistance strength of basement walls, cover slabs, etc. Engineered upgrading of these walls and slabs was not generally considered, however, because such effort was beyond the work scope. Techniques for such upgrading must therefore be developed and recorded in a useful form.

Building Types

Although the drawings for many buildings* have been evaluated for air blast resistance, only a few have been examined in situ with the purpose of determining upgrading potential and techniques. Although additional NSS buildings should be examined (both drawings and in situ), additional attention should now be given to types of buildings and facilities in which the essential "stay-behind" personnel are likely to seek shelter, and which may not have been included in the NSS for various

* Mostly NSS buildings.

reasons. Because of essential functions, these building or facilities should include:

- Police Stations
- Fire Stations
- EOCs
- Manned Telephone Central Offices
- Manned Long Lines Telephone Switching Facilities
- Air Traffic Control Facilities
- Transportation Control Centers
- Television and Radio Stations
- Essential Government Facilities (Other than EOCs)
- Electric Power Generation and Dispatch Facilities
- Other Essential Utility Facilities

Buildings to be examined in situ in the future work should be drawn from the above list.

Expedient Upgrading

Additional work that should be accomplished under the heading of expedient options includes:

1. Development of additional cases and alternative upgrading techniques.

2. Development of hasty design processes involving charts, graphs, etc., for evaluation of the degree of blast protection afforded by various techniques and material combinations.*

3. Development of design, material, tool, and estimated labor (number and skill) packages for the more common upgrading problem areas such as stairwells, elevator shafts, and basement wall openings.

4. Development of design, material, tool, and estimated labor packages for several types of "last-resort" and open shelter protective systems.

5. Review of the special types of facilities listed above to detect any special problem areas, and development of techniques for their solution.

Engineered Upgrading

Additional work in this area should be based on the rather extensive work already performed on the evaluation of existing buildings. The

* This has been done for structural and stress-graded lumber.

evaluation processes previously developed should be refined and simplified, if necessary, and step-by-step guidance provided for application wherever needed.

Specific blast strengthening techniques should be developed for the more common wall, floor system, and column problem areas as found in past work. These techniques should be based on state-of-the-art structural engineering practice, but with necessary extension into blast-resistant stresses and short techniques. This work should be amplified by the inclusion of several examples illustrating the engineered blast upgrading of the more common building types.

Report

The report of this and the previous work should be arranged in such a form and with such content as to be readily convertible into a manual for direct use by those professionals likely to be engaged in both expedient and engineered upgrading of shelter space.

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BIBLIOGRAPHY

The following publications were selected for review as discussed in Appendix A. Each listing is followed by a short description of content and by a short paragraph as to its value to both expedient and engineered upgrading options for nuclear weapons effects, especially blast.

1. Zeitlin, E. A., The Blast Environment: Methodology and Instrumentation Techniques with Applications to New Facilities, NOTS TP 3870, NAVWEPS 8782, U.S. Naval Ordnance Test Station, China Lake, CA, August 1965. (AD-622 980)*

   Content: Presents air blast phenomena, loading and structural response from a theoretical point of view. Relates air blast to earthquake forces and discusses blast measurement techniques.

   Value:† None.

2. Surveys of Soviet-Bloc Scientific and Technical Literature: Nuclear Weapons Effects, Compilation of Abstracts, Aerospace Information Division, Library of Congress, 1 July 1964. (AD-602 358)

   Content: Abstracts of theoretical U.S.S.R. work in the strong shock field.

   Value: None.

3. Panufnik, Wladyslaw, "Chapter IV, Shelters and Personal Means of Protection," How to Protect Oneself from the Action of Atomic Weapons, Translation No. AEC-tr-3671, U.S. Atomic Energy Commission, September 1959.

   Content: An elementary discussion of personal protective measures ranging from group shelter to expedient individual measures, partially based on U.S. publication, Severud, F. N., and A. Merrill, "The Bomb, Survival and You," 1954.

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* Those references for which "AD-" numbers are shown are understood to be available for purchase from NTIS, Springfield, Virginia, 22151.

† Value is, of course, stated in terms of usefulness to this research study area and report.

Value: Suggestions for reinforced basement shelter, shelter in stairwells and building shells, and "final resort" shelter in such as corners of basements.

4. Civil Defense Systems: Preattack and Postattack (Nuclear Warfare), A DDC Bibliography, DDC-TAS-72-12-1, April 1972. (AD-740 950)

   Content: A compilation of abstracts of unclassified literature dealing mostly with non-shelter aspects of civil defense.

   Value: None.

5. Cristy, G. A., and C. H. Kearny, Expedient Shelter Handbook, Final Report, ORNL-4941, Oak Ridge National Laboratory, August 1974.

   Content: Presents designs, material and tool lists, and instructions for construction of 15 types of expedient radiation protection shelters, ranging from small family to large group types.

   Value: All designs are for locations exterior to buildings and are for radiation protection only. The details from some of the designs, together with the material and tool lists, are of some value, however, in the in-basement blast strengthening case.

6. Ostroukh, F. I., Construction of Quickly Erectable Blast and Radiation Shelters, JPRS 63455, Translation of a U.S.S.R. Monograph By the U.S. Joint Publications Research Service, 18 November 1974.

   Content: Presents data on U.S.S.R. concepts and materials for providing shelter from nuclear weapons effects. Concentrates primarily on prepared large group shelter using pre-cast concrete techniques and expedient timber materials.

   Value: Contains some material of value on strengthening basements of existing buildings and on expedient closure doors, emergency exits, and ventilation and lighting.

7. Civil Defense Systems: Shelters, A DDC Bibliography, DDC-TAS-72-14-1, April 1972. (AD-740 960)

   Content: A compilation of abstracts of unclassified literature on civil defense shelter.

   Value: Limited; no direct material related to blast upgrading of structures.

8. Protective Construction Concepts, The Ralph M. Parsons Co., for the Directorate of Civil Engineering, Headquarters, U.S. Air Force, 1 November 1968. (AD-850 286)

Content: Presents protection concepts and effects data for nuclear, HE and penetrating weapons. Concepts are qualitative in nature.

Value: Of no specific value for either expedient or engineered blast upgrading.

9. Longinow, A., and O. J. Stepanek, Civil Defense Shelter Options for Fallout and Blast Protection (Single-Purpose), Project J6115, IIT Research Institute, for U.S. Office of Civil Defense, June 1968. (AD-674 663)

Content: Presents cost estimates and designs for several permanent and expedient group shelters, together with blast resistance evaluations.

Value: Of no specific value other than general guidance in materials use, construction methods, and cost estimating.

10. Longinow, A., et al., Civil Defense Shelter Options: Deliberate Shelters, Vols. I and II, IIT Research Institute, for U.S. Office of Civil Defense, December 1971.

Content: Evaluates a number of single- and dual-purpose, combined effects, group deliberate shelters for survivability. Cost estimates are included.

Value: Of no specific value other than general guidance in blast resistant design, materials use, construction methods, and cost estimating.

11. Havers, J. A., and J. J. Lukes, Structural Cost Studies for Hardened Shelters, Project M6064(1), IIT Research Institute, for U.S. Office of Civil Defense, January 1965.

Content: Analyses optimum per occupant costs for a series of 100, 500 and 1000-man shelters at 10 and 200 psi overpressure levels.

Value: Very little except some cost guidance.

12. Cristy, G. A., Shelter for Critical Industry Workers, Final Report, ORNL-5022, Oak Ridge National Laboratory, for U.S. Energy Research and Development Administration, May 1975. (AD-A014 626/6GA)

Content: Reports the results of an investigation of an industrial area to determine the best methods of sheltering the "stay-behind" forces during a strategic evacuation.

Value: Nothing specific on blast strengthening, but of considerable value in CRP from a company point of view. Points out problems of available shelter space. Does examine expedient covered trench and culvert shelter.

13. Titon, M. N., et al., Civil Defense, Moscow 1974, edited by G. A. Christy, ORNL-TR-2845, Oak Ridge National Laboratory, July 1975.

Content: This publication represents the basic U.S.S.R. text book on civil defense policy and operations. Contains little specific shelter material.

Value: None for expedient and engineered blast upgrading.

14. Nordell, W. J., Active Systems for Blast-Resistant Structures, Technical Report R-611, U.S. Naval Civil Engineering Laboratory, February 1969. (AD-683 331)

Content: Presents theoretical concepts of providing blast strengthening.

Value: None.

15. Havers, J. A., Structural Materials for Hardened Personnel Shelters, Project M-254, IIT Research Institute, for U.S. Office of Civil Defense, December 1963.

Content: Presents analysis of conceptual buried group shelters in the 10 to 200 psi overpressure region. Costs are also derived.

Value: None.

16. Modification of Existing Buildings As Community Shelters, FINAL DRAFT, Ammann & Whitney, Consulting Engineers, for U.S. Office of Civil Defense, January 1965. (Prepared for OCD as Professional Guide Series, PG 80-3, but not published.)

Content: Presents principles and concepts for modification of existing structures as community shelters. Uses 3 protective levels: (1) Fallout only; (2) ≤ 5 psi blast; and (3) > 5 psi blast. Includes mechanical and electrical provisions.

Value: Very useful in concepts and methods of providing blast upgrading; however, the general approach is to provide permanent beforehand upgrading, rather than within the present concept of expedient or pre-engineered upgrading.

17. Longinow, A., et al., People Survivability in a Direct Effects Environment and Related Topics, Project J6144, IIT Research Institute, for U.S. Defense Civil Preparedness Agency, May 1973. (AD-764 114)

Content: Describes a computerized methodology for predicting survivability of people located in NSS buildings, and presents results for sample buildings. Also describes results and analyses of 8 types of special and expedient shelters, designed for use within basement spaces or exterior to buildings.

Value: Of value in solving vulnerability and people survivability problems; also for materials and methods of providing open shelter and "last resort" protection.

18. Cristy, G. A., Expedient Shelter Survey, ORNL-4860, Oak Ridge National Laboratory, for U.S. Atomic Energy Commission, July 1973.

    Content: Reviews the state-of-the-art of expedient shelter building, and develops a recommended list of shelter types to include in a proposed shelter manual for guidance during crises.

    Value: Very little since all shelters studied are exterior to buildings. Of some value as to material availability, uses and contruction methods.

19. Kearny, C. H., and C. V. Chester, Blast Tests of Expedient Shelters, ORNL-4905, Oak Ridge National Laboratory, for U.S. Atomic Energy Commission, January 1974.

    Content: Describes results of a 500-ton HE detonation test of 6 types of exterior buried shelters.

    Value: Very little since all shelters studied are exterior to buildings. Of some value as to use of materials. The very short durations of the 500-ton HE blast wave provides very small impulses and drag loads; the results of these tests are not therefore applicable to results expected under large (Mt) explosions.

20. Egorov, P. T., et al., Civil Defense, Moscow 1970, ORNL-TR-2793, Oak Ridge National Laboratory, December 1973.

    Content: Translation of basic U.S.S.R. civil defense document. Earlier version of #13 above, but contains more detail on protective methods.

    Value: Presents U.S.S.R. methods of providing external and internal shelter, as well as blast strengthening of buildings.

21. Lang, C., Blast-Resistant Characteristics of State and Local Emergency Operating Centers (EOC's), Final Report, Agbabian Associates, for U.S. Defense Civil Preparedness Agency, October 1975.

Content: Describes analysis of 64 EOCs surveyed in 1974 by DCPA, and of 7 EOCs in California examined by the author, as to their potential for blast upgrading to a 10 psi overpressure level. Discusses concepts for upgrading of both existing and new EOCs.

Value: Good guidance on concepts and principles of engineered blast upgrading.

22. Cristy, G. A., Blast Shelter Potential in New Government Buildings, ORNL-TM-3664, Oak Ridge National Laboratory, March 1972.

Content: Presents analysis of the potential for providing blast shelter in future federal building programs. Potential is to be realized by slanted design; 87 specific buildings under design at the time were analyzed.

Value: None.

23. York, S. B., et al., Alternative Ways of Providing Host Area Fallout Protection, Final Report, DCPA Work Unit 1621F, by Research Triangle Institute (Report 44U-988), for U.S. Defense Civil Preparedness Agency, December 1975.

Content: Analyses alternate means of providing fallout shelter for relocated populations under CRP. Both spaces in existing facilities (NSS) and expedient separate facilities were examined, and cost-effectiveness determined.

Value: Little except some guidance on cost estimating, as well as material and tool requirements.

24. Cox, F. B., A Study of the Feasibility of Methods for Increasing the Load-Carrying Capacities of Existing Concrete Beams, Technical Report C-70-3, U.S. Army Engineer Waterways Experiment Station, May 1970.

Content: Study examined feasibility of increasing the load-carrying capacity of existing R/C beams by adhesively bonding additional reinforcement to exterior surfaces.

Value: Research results are not applicable to expedient upgrading, but the techniques of bonding precast concrete panels to existing beams may be useful for engineered upgrading. Method may be of limited value, however, because of anticipated large deflections and the difficulty of developing tensile membrane strength in bonded members.

Appendix A

LITERATURE SEARCH

Appendix A

LITERATURE SEARCH

A literature search was completed, which sought ideas for blast upgrading options, solely for crisis implementation, especially those using nearby or indigenous materials and equipment. Details are as follows.

A search through the Government Reports Index (GRI) was made for the years 1955-1975. Selected key words were: blast; buildings; civil defencse systems; construction; fallout shelters; nuclear explosion damage; nuclear explosions, underground structures; shelters; structures; and subsurface structures. The key words were modified to reflect the changing usage of descriptors over time; e.g., since 1974 subsurface structures has almost totally supplanted its predecessor, underground structures.

A similar search in the GRI was made for 1974-75 through the Corporate Author Index for selected government agencies: DCPA, Oak Ridge National Laboratory, and the Navy's Civil Engineering Laboratory. Results indicated that no new references were recovered beyond those found in the key word search, and therefore, this aspect of the search was discontinued.

A search of Nuclear Science Abstracts (NSA) was also conducted for 1955-75 using both index (selected key words were: civil defense; shelter; and, structures) and table of contents (protective structures and equipment, health and safety, and civil defense headings).

A search was made of Applied Science and Technology Index (AS&TI), entitled Industrial Arts Index prior to 1960, for the same 1955-75 period. Selected key words were: air raid shelter; atomic blasting; atomic bomb shelter; atomic bombs and building; bracing; building; civilian defense; earthquakes and building; shelter; shoring and underpinning; structural engineering - design; underground construction; and underground structures.

Finally a search of the American Society of Civil Engineers (ASCE) Index was conducted for 1955-75 using these selected key words: atomic blast; building; construction; civilian protection; military engineering; national defense; structural engineering; substructures; and war and engineering.

In the case of the NSA, AS&TI, and ASCE Index searches, the key words list was modified from the basic GRI list to reflect the subject categories used by the individual indexes.

Two other possible sources of information, the American Concrete Institute and the Portland Cement Association publications, were checked in a previous search for similar information and yielded no references.

From the searches, some 294 references were selected (169 from GRI, 78 from NSA, 27 from AS&TI, and 20 from ASCE Index) for review by the Project Leader, who selected 61 for further consideration.

Following the basic search work, these references were considered, based on abstracts in nearly all cases, for detailed review. To this end, they were classified roughly as: reports about which nothing was known; those doubted to have contents of value to the project objective; those known to have no value to the project objective; and those definitely to be reviewed. Such rough classification work was done by drawing on the background knowledge of project personnel, with considerable assistance provided by Mr. G. N. Sisson of DCPA. Many publications were already in the files of project personnel or in Institute files; others were requested and all were obtained. Reading/scanning by one of three senior engineers working on the project was done. This literature search and selection process extended over several months, perhaps too many; however, the purpose was to use our group secretary, who has had library-literature search training and experience, for as much of the work as possible, thereby reducing such work by the project senior engineers and conserving funds for their work on other aspects of the research.

Appendix B

DESIGN OF WOOD BEAMS - SIMPLY SUPPORTED

Extract from

References 2 or 3 of basic report

Retaining

Original pagination and figure numbers

G. Wood Beams - Simply Supported

The wood contemplated for use under the design procedures described herein is structural or stress-graded lumber, which has been carefully graded in accordance with the standard grading rules for the appropriate trade association (e.g., References 52 and 53). A complete list of such associations is available.[54] It is urged that all lumber contemplated for shelter use - specifically, lumber in structural components or members whose stress-resisting capability is important to the survival of shelterees (in contrast to such things as a door cross-brace that simply holds together the structurally significant members) - be reinspected and regraded by particularly qualified personnel using the appropriate association's grading rules.

Other items for the designer's general consideration are:

- The lack of homogeneity in wood members dictates that every effort be made to design wood structural members so that they interact in such a manner as to transfer load from a weaker or below-standard member to the better members. Examples are: really good blocking between floor joists; and use of tongue-and-groove planking as members used flat in a blast door.

- Only very tight knots (preferably no knots) should be accepted in a situation such as that of an unclad wood shelter blast door where an air blast loading could make a missile or bullet out of a knot that is even slightly loose.

- Metal cladding may be indicated for some situations where wood is used, such as exposure to fires (or where required by local building code), but not necessarily when exposure is only to a nuclear thermal pulse (which may well char the door without setting it on fire, a difficult thing to do to a flat wood wall).

Because this guide is intended for use by engineers and architects with special training in DCPA-conducted courses or their equivalent (as has been stated earlier[50] in a Preface and Chapter 1), technical competence in the usual design of wood structural members is assumed,[54-57] and only those design considerations peculiar to nuclear blast effects loading will be treated in some detail in this section.

Design Procedure. Because wood beams are available in specific dimensions, the general design approach is to select a trial member depth (measured in the direction of the applied load) and width, then find the air blast peak overpressure it can resist; this overpressure is compared to the specified overpressure to be resisted. The resistance of the selected member is based on elasto-plastic behavior and associated stress resistances in flexure (bending), horizontal shear, and bearing on a support, which resistances are checked in that order. Specifically, the flexure and horizontal shear resistances are found, and then a new trial member is selected, repeating these steps until the lesser of the two resistances is found to be sufficient to meet the expected blast load. The required bearing area is then found directly.

The design steps are as follows:

1. A design air blast peak overpressure is specified, also whether its loading geometry will provide: a side-on overpressure (as in a wood door mounted flush with the earth's surface); a fully reflected overpressure (as in the front wall of a rectangular building); or a peak value of the average loading caused by a combination of side-on and drag pressure (as in the side-wall or roof of a rectangular building[1(§4.80-)]). Related variables, in the same order of loading geometries, look like this:

$$p_m = p_{so} \text{ or } p_r \text{ or } \left[(p_{so} + C_d q)\ L/2U\right] \tag{6-53}$$

where q is the dynamic (wind) blast pressure (unlike the q for structural resistance used in the remainder of this section).[1(p. 182-)]

2. A trial size of wood beam (actual depth d, measured in direction of load, and thickness or width b) and kind of structural or stress-graded lumber are selected, then the grading association's design stresses are determined from their publications. Need for the latter may be limited to F_b (extreme fiber stress in bending), F_v (horizontal shear stress), and $F_{c\perp}$ (compression stress perpendicular to grain, or bearing stress as used herein). For the short duration loadings furnished by nuclear air blast, dynamic values of the above three design stresses are recommended[23] as follows:

$$F_{db} = 4F_b \ ; \ F_{dv} = 4F_v \ ; \text{ and } F_{dc\perp} = F_{c\perp}$$

Some grading rules allow increases in design stress values for such things as: repetitive member design values (not recommended for use herein); and members used flatwise (probably appropriate for use herein).[52(p.130-1)]

3. A design ductility ratio μ is selected (see discussion in the earlier section herein, General Comments on Blast-Resistant Design . . .). A value of 3 is recommended,[23] certainly as an upper limit, and with 1.3 or 2 even better.[31]

4. A short design procedure[23] omits use of any loading decay (i.e., uses instead an instantaneously applied long duration load, or step pulse), load-mass factors, modulus of elasticity, elasto-plastic resistance function per se, etc., all in favor of the following approach: A step pulse is assumed, which is reasonable particularly when large yield weapons and short wood beams (therefore having very short periods of natural vibration) are considered.* The other things ignored have been found to have little effect on the structural member selected for most applications; and needed parameters then have the following relationship: $p_m/q = 1 - 1/(2\mu)$ where q is the ultimate resistance to blast loading of the wood beam. Using the recommended value of $\mu = 3$, the equation becomes: $p_m = (5/6)\ q$

5. Clear span L and support conditions are known or assumed. Formulas are included herein for three beam support conditions: simply supported (SS); propped cantilever (PC); and both ends fixed (FF).

6. Flexural or bending resistance q_b (in terms of load/unit area) is calculated for the trial member:

$$M = wL^2c = q_b bL^2c = F_{db}S = F_{db}bd^2/6$$

$$q_b = F_{db}(d/L)^2/(6c) = 2F_b(d/L)^2/(3c) \qquad (6\text{-}54)$$

where c = 1/8 (SS) and (PC), 1/12 (FF).

7. Horizontal shear resistance q_v (in terms of load/unit area) is also calculated for the trial member, with horizontal shear equal to vertical shear and taken at a distance d in from each end of the member: member:[23(p.161),54(p.4-12)]

$$V = w(L-2d)c' = q_v b(L-2d)c' = 2AF_{dv}/3 = 2bdF_{dv}/3$$

$$q_v = 2F_{dv}d/(3c'(L-2d)) = 8F_v d/(3c'(L-2d)) \qquad (6\text{-}55)$$

where c' = 1/2 (SS) and (FF), 5/8 (PC), the latter value being approximate but close enough for the purposes herein.

8. Wood beam resistance q is then equal to the lesser value between q_b and q_v and is converted to peak air blast pressure by using a formula given earlier:

$$p_m = (1 - 1/(2\mu))\ q \qquad (6\text{-}56)$$

or, when the recommended value of $\mu = 3$ is used, $p_m = (5/6)\ q$.

* Alternatives to this use of a step pulse are chart solutions and the Newmark β Method, described herein (page 6-12, third paragraph).

9. If p_m is less than the design air blast peak overpressure specified in the first step herein, a larger beam, or a different wood or grade having larger design stresses, must be tried. If p_m is larger than the design overpressure, than it may be desirable to try a smaller beam, or a different wood or grade, in an effort toward closer design. In either case, a new trial member requires that the designer return to the second step and repeat the procedure to this point.

10. Required bearing length L' at each end of the wood beam is calculated as follows:

$$V = qbLc' = F_{c\perp}bL'$$

$$L' = qLc' / F_{c\perp} \tag{6-57}$$

where the values of c' are the same as in step 7 above.[55(p.206-7)] It is recommended that L' be at least 1.5 to 2 inches.

Application to a Shelter Door Design. An application of wood beam design occurs when low-cost blast doors must be designed for shelters, in new designs or existing structures. For an application in existing structures, particularly, a pre-design or chart approach was needed as follows:

- An estimate, calculated or judgmental, is made of the blast resistance of the wall adjacent to an aperture (door or window opening) for which a wood blast door is needed. The only designed structural element will be a wood beam, or series of wood beams side-by-side and preferably tongue-and-groove, simply supported on the two sides of the door frame (that has been either strengthened or found adequate to take the load from the door onto the wall).

- Structural grades of various kinds of wood, in standard thicknesses (2, 3, 4, 6 inches, nominal; 1.5, 2.5, 3.5, 5.5 inches, actual) are checked for availability.[52]

The pre-design or chart approach developed for simplified handling of this problem was as follows:

- Obtain a copy of the industry association grading rules for each kind of wood contemplated for possible use; from this, make a tabulation (for each kind of wood and each thickness) of design stresses (psi) stated for use under normal loading for:

- Bending design stress (in extreme fiber), F_b
- Horizontal shear design stress, F_v
- Compression perpendicular to grain design stress, $F_{c\perp}$

- Conversion of design stresses to dynamic values (step 2 above) is unnecessary hereunder; the charts used include this conversion and are therefore entered directly with the design stresses for normal loading.

- For each wood and thickness, determine the blast resistance in terms of free-field overpressure:
  - For the specific thickness, use the pre-design chart, Figure 6-11, which consists of four charts, covering member thicknesses of 1.5, 2.5, 3.5 and 5.5 in. The charts are based on use of Equations 6-53 through 6-56, and assume $\mu = 3$, step pulse and simple supports; the charts are entered with design, not dynamic, stresses (per the preceding paragraph).
  - Enter the chart with the known clear span: first, use the left set of curves, moving up to the known allowable design stress F_b for the selected wood, interpolating as necessary and noting the related applied overpressure (psi) read on the ordinate scale; second, repeat the procedure with the right set of curves (for stress in horizontal shear F_v), again noting the related overpressure. Use only the lower of the two applied overpressures read!

- For each wood and thickness still of interest, determine the required bearing length at each end of the wood beam:
  - Use the last wood pre-design chart, Figure 6-12. The chart is based on use of Equation 6-57, and assumes $\mu = 3$, step pulse and simple supports. Thus $L' = (6/5)\, p_m L (1/2) / F_{c\perp}$ (from Eq. 6-57); or $p_m = (5/3)\, L' F_{c\perp} / L$ for which Figure 6-12 is a plot for several specific values of $F_{c\perp}$ and L as the independent variable, all with $L' = 1$ in.
  - Enter with the clear span, move up to the allowed design stress, and read the applied overpressure on the ordinate scale; this applied overpressure is for one inch of bearing length on each end of the wood beam.

FIG. 6-11A WOOD BEAM DESIGN, BENDING AND SHEAR

STRUCTURAL OR STRESS-GRADED LUMBER
Actual thickness 1.5 inches

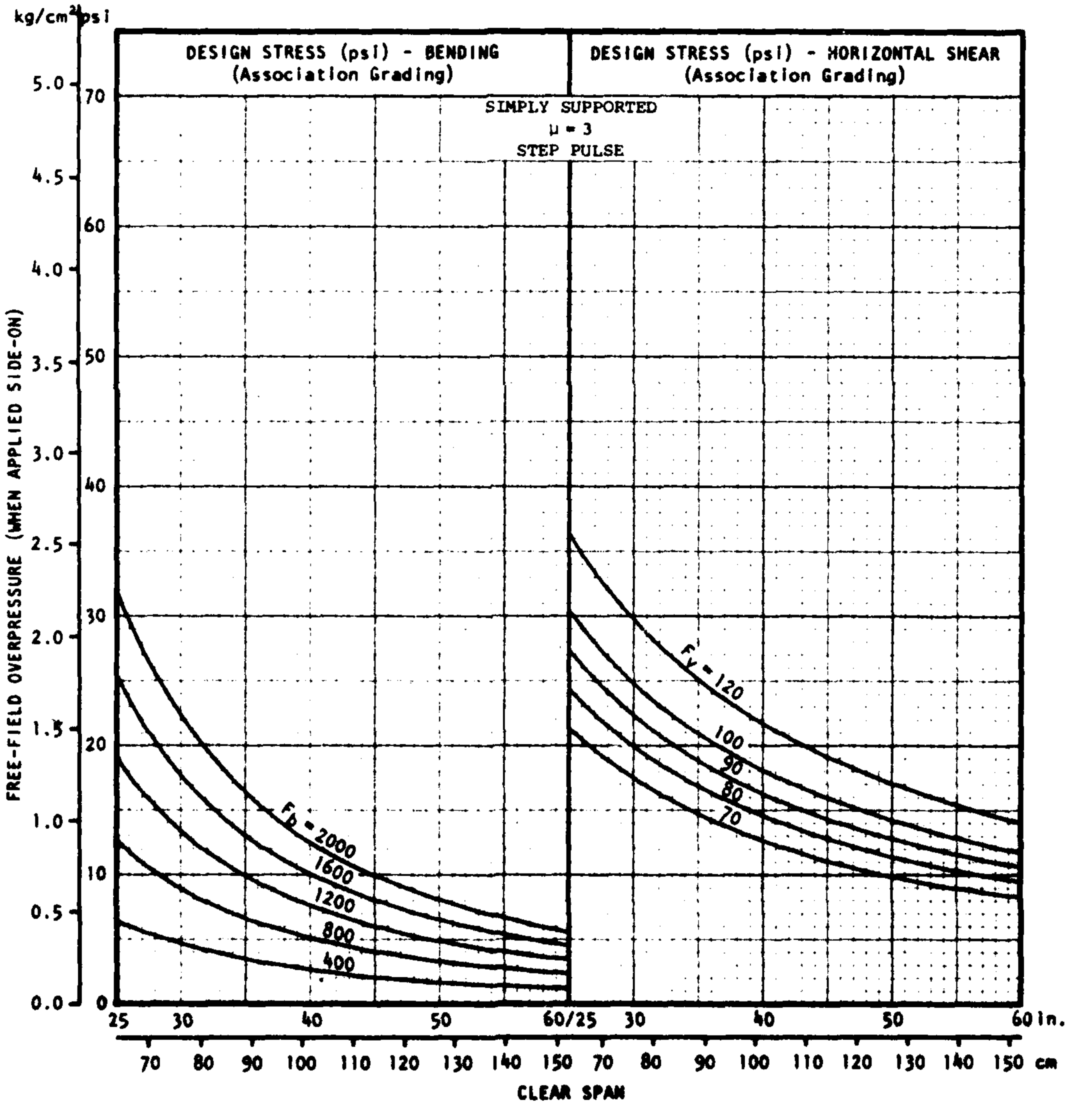


FIG. 6-11B WOOD BEAM DESIGN, BENDING AND SHEAR

STRUCTURAL OR STRESS-GRADED LUMBER
Actual thickness 2.5 inches

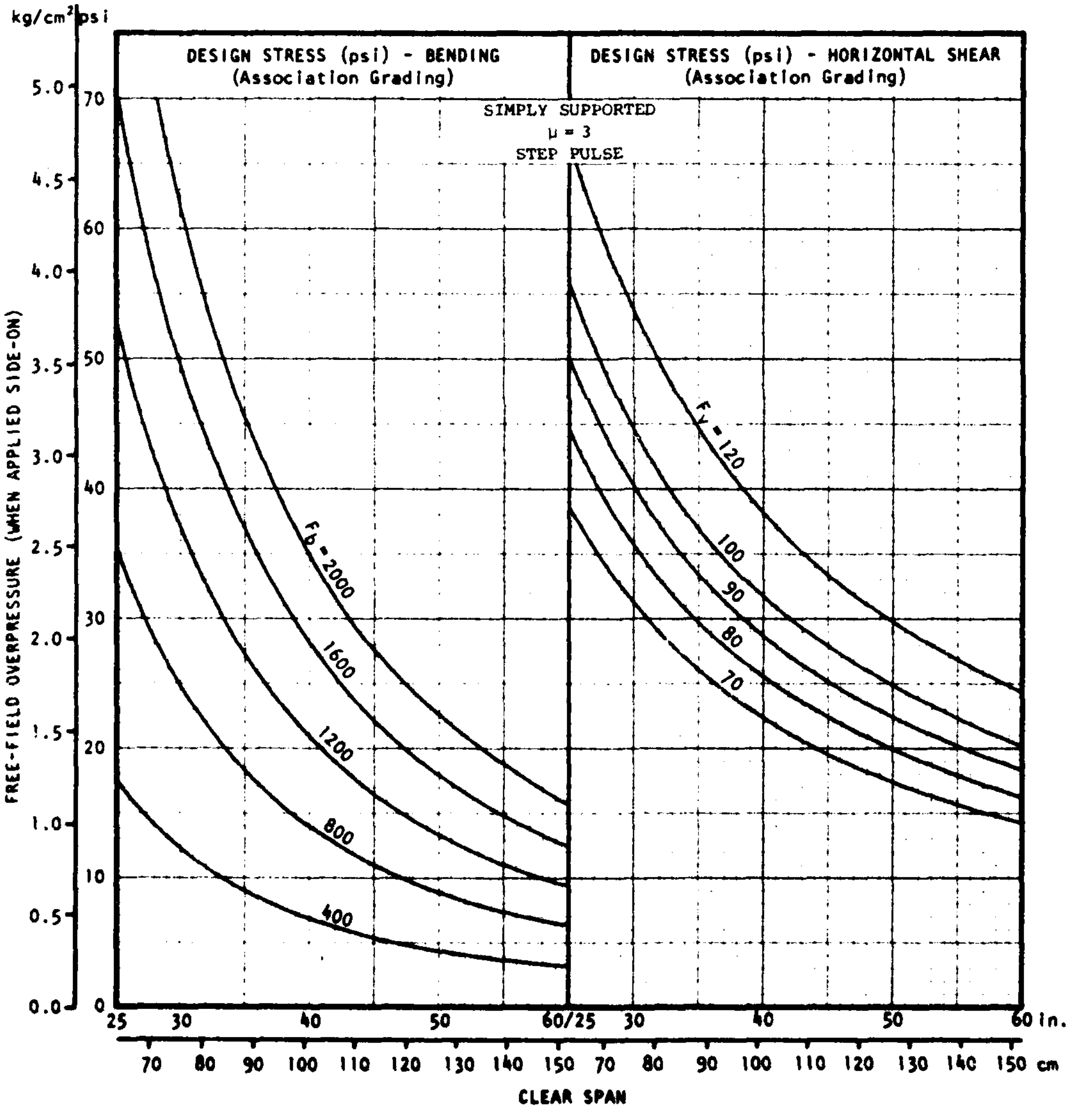


FIG. 6-11C WOOD BEAM DESIGN, BENDING AND SHEAR

STRUCTURAL OR STRESS-GRADED LUMBER
Actual thickness 3.5 inches

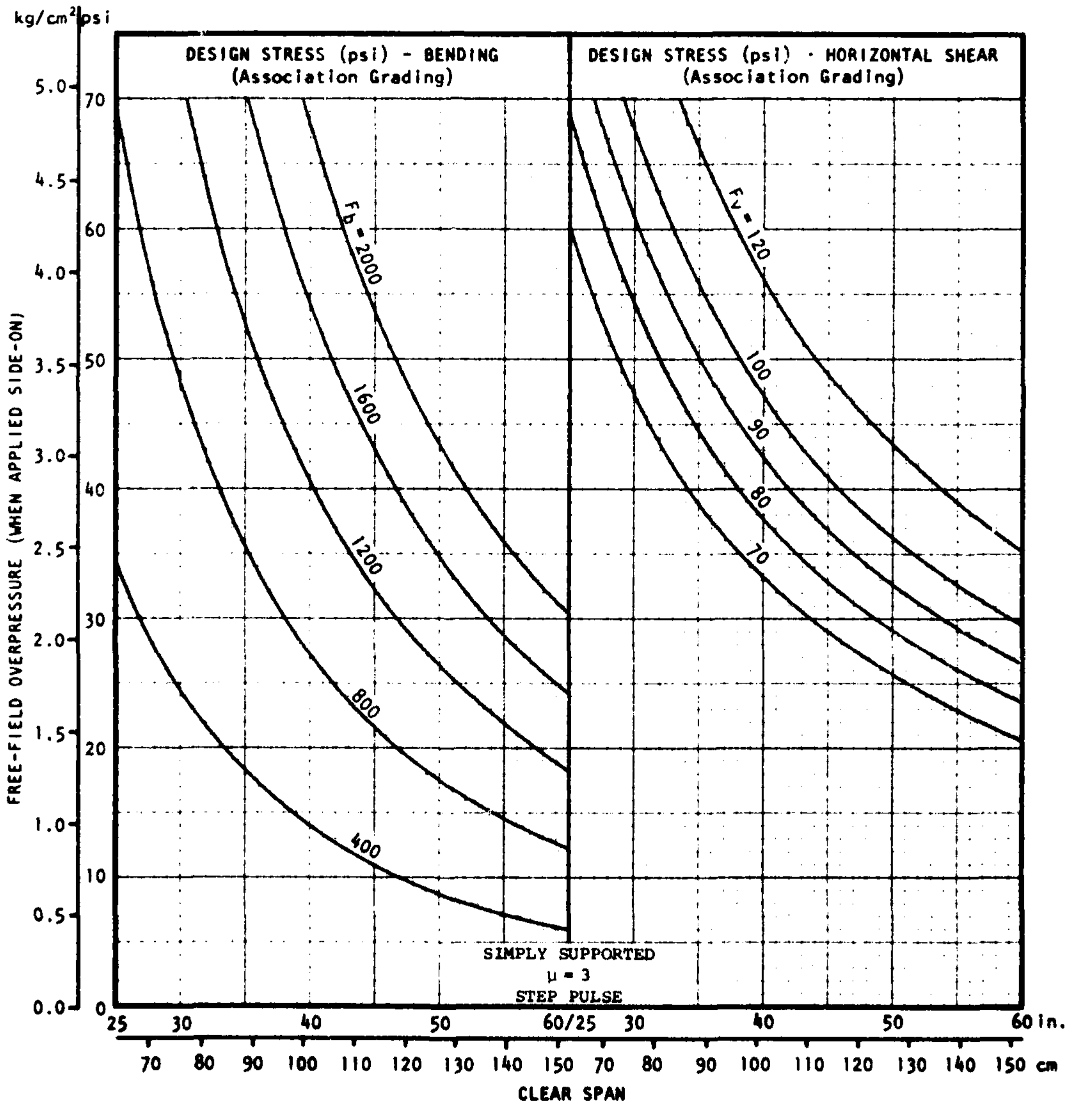


FIG. 6-11D WOOD BEAM DESIGN, BENDING AND SHEAR

STRUCTURAL OR STRESS-GRADED LUMBER
Actual thickness 5.5 inches

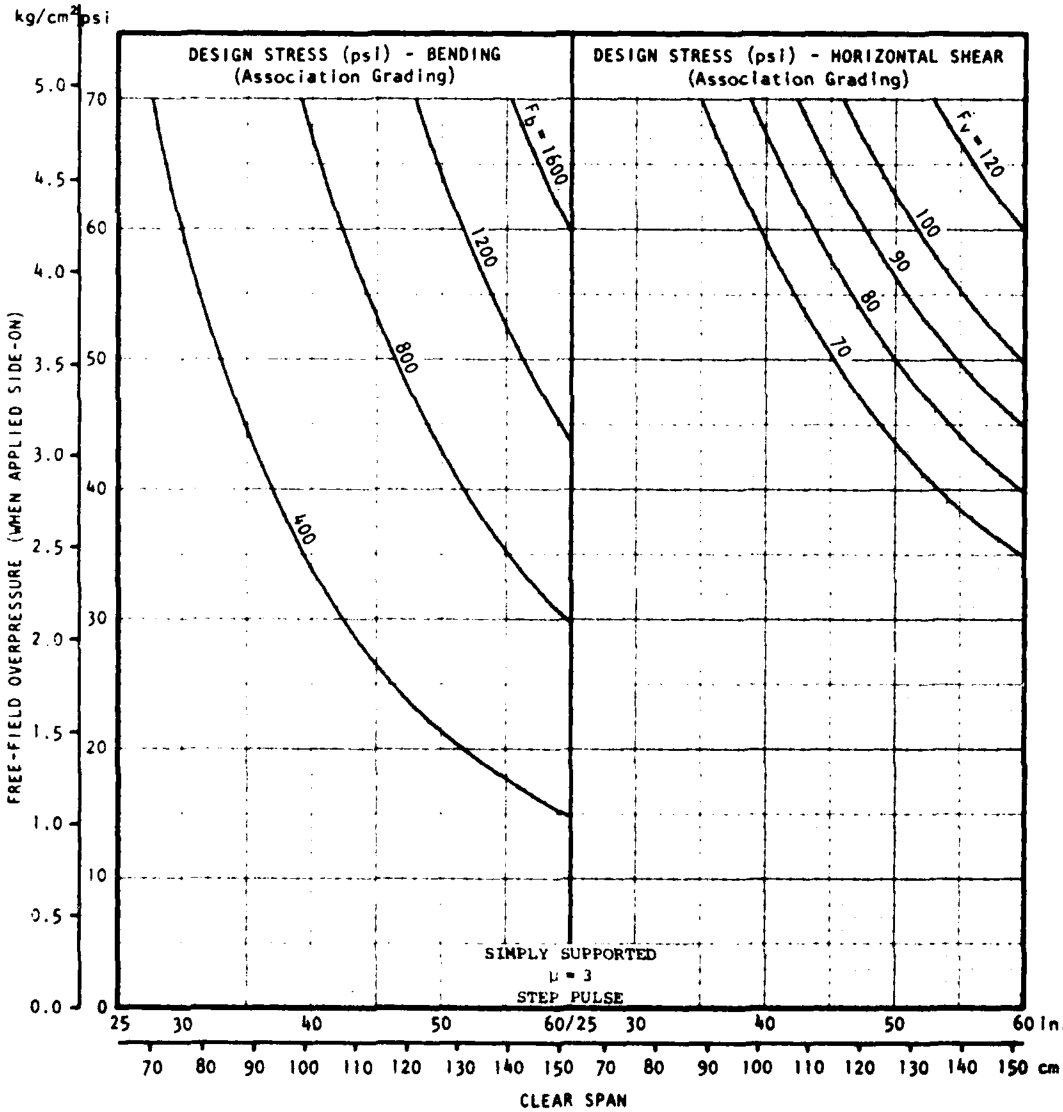
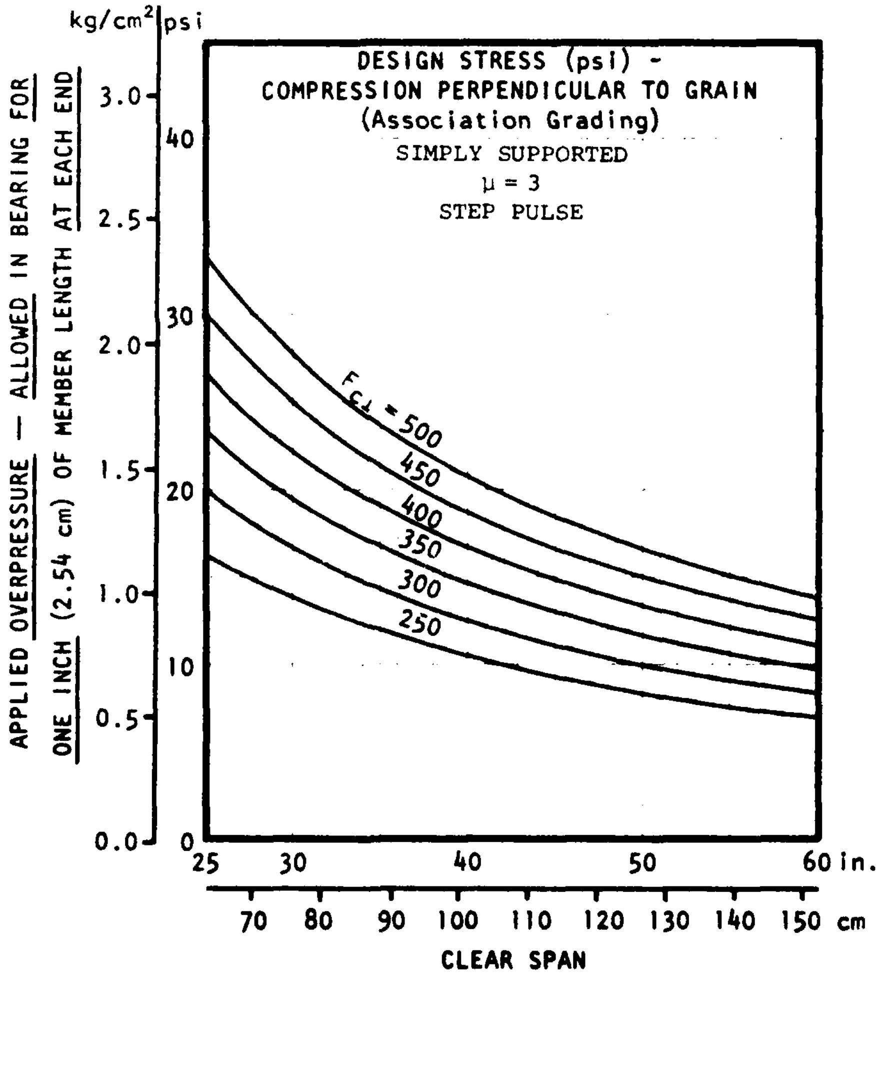


FIG. 6-12 WOOD BEAM DESIGN, END BEARING

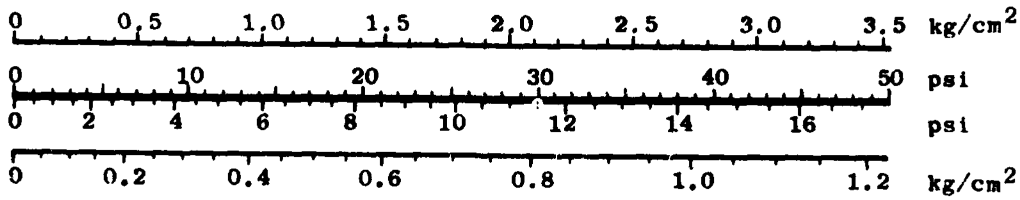
STRUCTURAL OR STRESS-GRADED LUMBER
Any thickness - END BEARING



- Divide this applied overpressure for bearing into the overpressure noted at the end of the preceding step (using Figure 6-11); the resulting quotient is the number of inches bearing length required at each end of the wood beam. It is recommended that a minimum length of, say, 1-1/2 or 2 inches be used.

- Applied overpressure, psi, determined above for the particular clear span and kind of wood (with its design stresses from the grading association), was the overpressure when applied side-on, such as if the wood beam were part of a cover or door, mounted flush with the ground and the blast wave passed over it flowing horizontally. If the member is to be used so that the blast wave strikes it head-on, as if the member were part of the front wall of a building struck by the blast wave, then the blast wave is fully reflected, making it equivalent in loading force to a much stronger wave applied only side-on. To relate these two situations by putting both in terms of free-field overpressure resistance (that is, out in the open, unaffected by structures), use the scales below:

FREE-FIELD OVERPRESSURE (WHEN APPLIED SIDE-ON)



FREE-FIELD OVERPRESSURE (WHEN APPLIED FULLY REFLECTED)

For example, a free-field overpressure of 45 psi hitting the member side-on gives the same peak loading to the member as a free-field overpressure of 16 psi hitting the member head-on, or fully reflected.

A numerical example of this procedure is as follows:

- Clear span 40 inches; bending design stress 1,250 psi; horizontal shear design stress 95 psi; compression perpendicular to grain design stress 385 psi. (These are the values for Douglas Fir, #2 Grade, under Structural Joists and Planks, Table 6.)[52] Assumed blast orientation is head-on, or fully reflected.

- First thickness of above wood to be checked for blast resistance is 3 in. nominal, 2.5 in. actual. Entering the chart, Figure 6-11B, for that thickness, clear span 40 in.: design stress in bending of 1,250 psi gives about 21 psi overpressure; design stress in horizontal shear of 95 psi gives about 30 psi overpressure.

- Required bearing length at each end of the wood beam is obtained by using the last chart, Figure 6-12: entering with a clear span of 40 in., and interpolating for a design stress of 385, gives an applied overpressure (per inch of bearing at each end) of about 16 psi. Dividing the 16 psi. Dividing the 16 psi into the 21 psi noted just above gives a member length at each end, for bearing, of $\frac{21}{16}$ or 1-5/16 in. for which the used length would be rounded (upward ALWAYS) to, say, 1.5 in. at each end (which is a minimum recommended above).

- Free-field overpressure applied head-on, i.e., fully reflected, is found by entering the scale above with the 21 psi side-on (free-field overpressure resistance) and finding this numerical example's answer of about 8.5 psi (free-field overpressure resistance for the wood member loaded by a fully reflected blast wave).

Appendix B

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UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: EXPEDIENT OPTIONS (UNCLASSIFIED)

By: H. L. Murphy, C. K. Wiehle and E. E. Pickering

Stanford Research Institute, Facilities and Housing Research Group, Menlo Park, CA 94025, May 1976, 174 pages
Contract No. DCPA01-75-C-0302, DCPA Work Unit 1155C

The research project concerned expedient upgrading of existing basements to provide some degree of protection against combined nuclear weapons effects, especially air blast; such shelter would be for critical workers remaining in "risk areas" during a crisis period. As used herein, "expedient" upgrading is that which can be accomplished within about a 24- to 48-hr warning period using indigenous labor and materials, and "basements" are those that have at most a small portion of any side exposed to blast or, in terms of a partially buried basement, perhaps no more than the upper 30% or so of any wall(s) exposed.

An extensive literature search* turned up no real expedient blast upgrading schemes, although some old (federal civil defense) fallout shelter ideas were found that had adaptation potential for blast upgrading, and thus parallel concepts/schemes shown herein for expedient structural strengthening and for "last resort" shelter within a shelter.

Research work, case studies, and a general study reported herein* clearly demonstrate a total lack of any correlation between floor design live load and mean blast collapse overpressure of a floor system. Important is the way the structure is put together and how the R/C designer chose to detail the reinforcing steel. Thus for expedient blast upgrading, availability of a set of "as-built" drawings is very important (for engineered upgrading, such a set is vital). With

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drawings available, some conclusions can be reached from engineering experience and existing structures evaluation techniques[‡] and ideas developed, to exploit any inherent blast resistance or to enhance it. General schemes for such strengthening are presented (not to scale) and one scheme includes an example in terms of blast resistance.[§]

Collateral to this study, it developed that there is a real need for engineered upgrading techniques for existing EOCs, whether permanently built, or crisis installed.

For those interested particularly in EOCs, expedient upgrading (in a case study herein) improved the functional blast resistance from 2 psi to 17 psi,[**] i.e., the potential area of severe damage/collapse was reduced 91%.[††]

In the other three case studies herein,[‡‡] expedient upgrading showed the following results:[**] 3.6 psi (open shelter) to 10.1 psi (closed); 3.5 psi (open) to 7.0 or 9.1 psi (open), the latter with a considerable increase in upgrading work; and 2 or 3 psi (open), representing the missile hazards of materiel, to 10.7 psi (closed). The first two require considerable upgrading work; the last requires only the addition of several expedient blast closures.

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* Appendix A describes the search; an annotated Bibliography provides some details.

\+ Chapter 5, Section I; case studies follow in Sections II through V.

‡ See numbers 11, 12, and 13 in References section.

§ See Figure 4 for upgrading scheme.

** Predicted mean collapse (incident or free-field) overpressure.

†† Chapter 5, Section II.

‡‡ Chapter 5, Section III to V.

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