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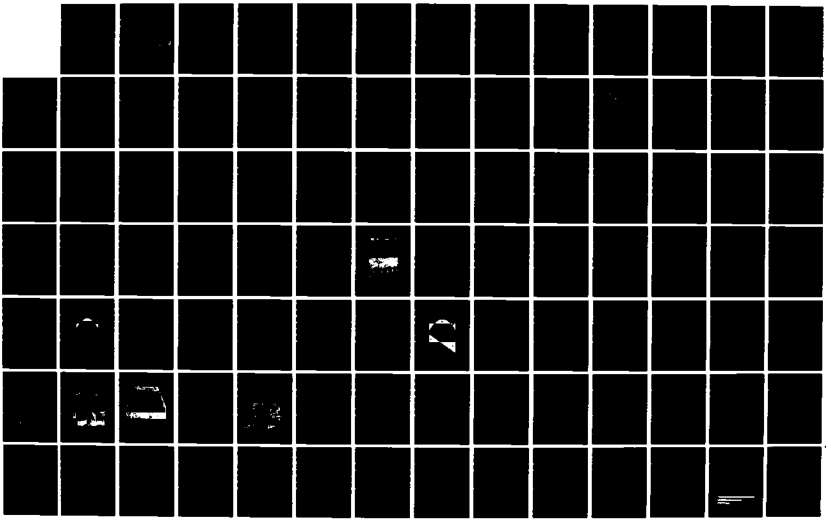

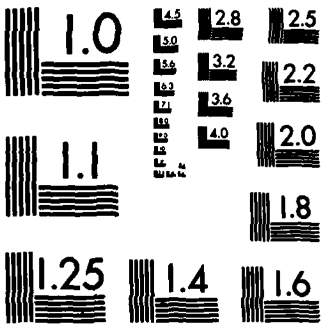

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SSI 8143-9

December 1982

DAMAGE FUNCTION RATING PROCEDURE FOR FLAT SLAB BASEMENT SHELTERS

FINAL REPORT

for

FEDERAL EMERGENCY MANAGEMENT AGENCY
WASHINGTON, D.C. 20472

Contract No. EMW-C-0678
Work Unit 1622D

SCIENTIFIC SERVICE, INC.

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20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

This report presents the results of the first year's work on the development of procedures for rating of damage functions and casualty functions for basement Civil Defense shelters. Suitable large basements, after ~~being~~ be upgraded during a crisis period, to withstand nuclear weapons effects including air blast, and nuclear radiation are expected to be utilized to provide protection for a large portion of the population in the event of a nuclear attack on the United States. Both risk area personnel shelters for essential workers and host area shelters

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for the general population are included. The objective of this planned five-year program is to provide FEMA with a sufficient range of damage and casualty functions to describe the survivability of people in various structural types of basement shelters. A procedure for rating shelter spaces will be produced by the program. By the end of the five-year program, this procedure will be in a form suitable for application by local civil defense planners.

The report includes: a descriptive listing of basement structural systems and other pertinent basement parameters; a description of the characteristics of typical flat slab basement designs; a review of applicable casualty data and prediction models for nuclear warfare casualties; a summary of previous research on development of casualty functions; a description of the current status of the damage and casualty function development procedure; casualty function predictions for representative flat slab basements; and conclusions and recommendations.

(DETACHABLE SUMMARY)

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by

R.E. Peterson, R.D. Bernard, R.S. Tansley, A.B. Willoughby,
C. Wilton, and T. Zsutty

for

Federal Emergency Management Agency
Washington, D.C. 20472

Contract No. EMW-C-0678, Work Unit 1622D

FEMA REVIEW NOTICE:

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

Scientific Service, Inc.
517 East Bayshore, Redwood City, CA 94063

(DETACHABLE SUMMARY)

DAMAGE FUNCTION RATING PROCEDURE FOR FLAT SLAB BASEMENT SHELTERS

This report presents damage functions and casualty functions for a range of flat slab basements. These functions were developed during the first year of a five-year program, which is intended to provide FEMA with a sufficient range of damage functions to describe the survivability of people in various basement shelters. Both as-built and upgraded basements are considered.

Casualty data and casualty prediction models were reviewed. The applicability of the principal nuclear weapons effects and casualty mechanisms to persons in as-built basements and upgraded basement shelters was analyzed.

The effort during the first year of the program concentrated on flat slab basements. The advantages and disadvantages of this type of construction, typical applications for it and typical ranges for live loads, span lengths, and other characteristics of this type of construction are described.

Failure properties of flat slabs under dynamic loading are described for flat slabs in general. Basements of representative actual buildings were analyzed to determine the damage functions and casualty functions applicable to them.

Since this program is expected to build upon previous shelter survivability research, a review of pertinent civil defense studies in this area is also included in the report of the first years's work.

A broad range of basement structural systems is described and their effect on the survival of a basement shelter in the event of nuclear attack is discussed. The

effects that the properties of the backfill around the basement, the upper stories of the building in which the basement is located, and other items in the vicinity of the basement might have on shelter survival were also considered.

Basement wall response and performance under dynamic loading conditions was another area studied. The results of the MILL RACE high explosive test basement wall experience, shock tube experiments with scale model walls, and analytical interpretations of the effect that air blast could have on basement shelter exterior walls were included in this part of the overall effort.

Based on the results of the first year's effort, conclusions are presented and recommendations offered concerning desirable areas for emphasis in future research in this program area.

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ACKNOWLEDGEMENTS

This report describes progress during the first year of a projected five-year program to provide FEMA with a range of damage functions to describe the survivability of people in various types of basement shelters. The authors wish to take this opportunity to thank all those who contributed to the previous research in this area and to the definition and furtherance of the current project. We particularly wish to thank Donald Bettge of the Federal Emergency Management Agency for his overall project direction; Dr. A. Longinow and his colleagues at the IIT Research Institute for their pioneering efforts in the study of People Survivability in Civil Defense Shelters; Dr. D. R. Richmond of the Lovelace Research Institute for his invaluable assistance in the area of Blast Biology; John Meehan of the California State Architect's Office and Albert G. Preske of Albert G. Preske and Associates for their assistance in obtaining the Mercy Hospital plans; the personnel of the San Mateo County General Services Office for their assistance in obtaining the plans for the San Mateo County Office Building; and the staff of the San Francisco Parking Authority for their help in obtaining the Civic Center Parking Garage plans.

The assistance provided by members of the SSI staff is also appreciated. Dr. B. L. Gabrielsen provided invaluable assistance in establishing the conceptual basis for the structural analysis portion of the program. James Gonzalez carried out the detailed calculations required for the structural analysis. The excellent work by members of the SSI support staff, Larue Wilton, Evelyn Kaplan, Maureen Ineich, and Michele Boyes, in the preparation and reproduction of the report is also gratefully acknowledged.

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Section 1
INTRODUCTION

BACKGROUND

Civil Defense programs are intended to provide effective protection for the population in the event of a nuclear attack on the United States. Because of the destructive nature of nuclear weapons and the capabilities of the Soviet strategic offensive forces, effective population protection is very difficult to achieve. Currently, civil defense programmatic emphasis is on crisis relocation planning, the intent being to evacuate the bulk of the population from risk areas that are likely to be targeted to rural host areas. In these host areas, away from likely targets, it is not considered necessary to provide high levels of protection against air blast or the other direct effects of nuclear weapons. Since by definition these areas are anticipated to be subjected to not more than 2 psi peak overpressures, direct effects protection at this level, coupled with adequate fallout and fire protection, should allow survival of a high proportion of the relocated population.

While the intent is to relocate the bulk of the population, it will be necessary for several million essential workers to remain behind in risk areas. These persons will require higher levels of protection against the effects of nuclear weapons. Current planning is to provide risk area personnel shelters capable of surviving peak overpressures of 40-50 psi and corresponding levels of other effects.

Shelter will be required for virtually the entire population. Except for a new program intended to provide for construction of risk area personnel shelters for essential workers, funding is not anticipated for construction of shelters for the population. Current plans are to utilize existing structures for shelters, with upgrading to be accomplished during a crisis period to allow these structures to survive the anticipated environments. Even in risk areas, this will be necessary until such time as sufficient specially designed shelters are available. (Also, since in many host and risk areas there are insufficient numbers of suitable structures for upgrading, these are expected to be supplemented where necessary by the construction of expedient shelters during a crisis period.)

In order for sheltering of the population during a crisis to be effective, a major prior planning effort by Federal, State, and local civil defense personnel is required during peacetime. Included in this effort is the location of existing buildings with areas suitable for upgrading to serve as shelters. In buildings with basements, the basement is ordinarily the most suitable portion of the building to upgrade. However, basements vary widely as to their types of construction and other relevant features. Some are much more suitable for upgrading to be used as shelters than others. In a crisis period, with severe limits on the available time, manpower, equipment, and materials, it would be very advantageous for the success of the effort to know the effects level to which structures would survive in their non-upgraded condition as well as which ones have the best potential for successful upgrading.

A national shelter survey program has been in existence since 1962. Currently some 300,000 structures have been identified as potential shelters. Research efforts, such as those discussed in References 1 through 18, have addressed survivability of people under various sheltering assumptions. However, existing techniques for selection of potential shelters, while they give some assurance that a level of fallout protection will be afforded, do not provide for differentiating between buildings as to the degree of blast or other nuclear effects protection that would be provided under either as-built or upgraded conditions. The general intent of the present project is to provide practical techniques to facilitate the comparative evaluation of basements being considered for use as either risk area or host area shelters.

These comparisons are to be made by developing damage functions and casualty functions for specific basement structural systems. Damage functions are primarily a means of relating structural damage to an overpressure. Casualty functions are a means of relating people casualties to overpressures. Figures 1-1 and 1-2, taken from Reference 17, illustrate a typical damage function and a typical casualty function. As will be discussed in detail later in this report, various nuclear weapon effects in addition to air blast may be included (or neglected) in constructing these functions. Numerous assumptions are required in their development, and we will also discuss these in detail.

During the first year of this project, damage functions and casualty functions have been determined for flat slab basements, both as-built and upgraded. During the next three years of this projected five-year program, casualty functions are to

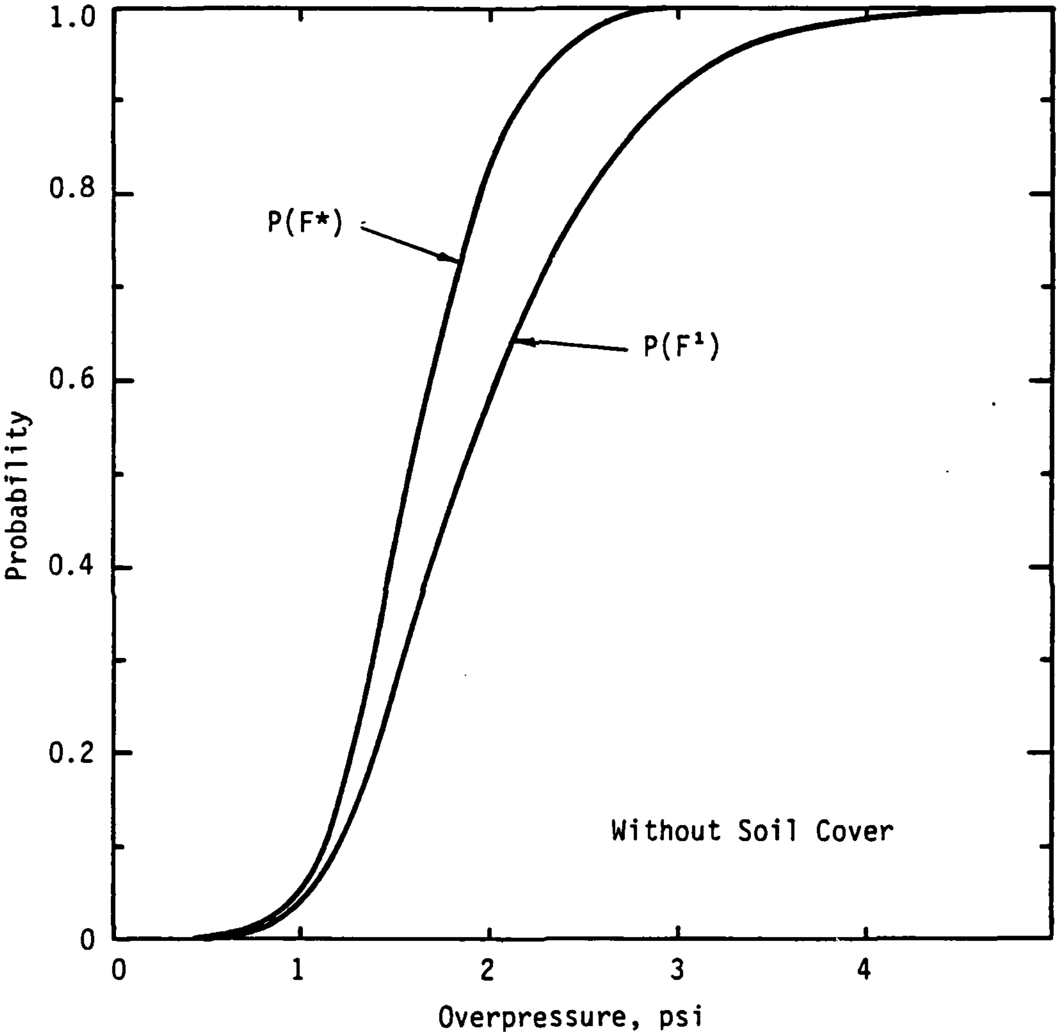

Fig. 1-1. Probability of Floor System Failure, Upper and Lower Bound. (Ref. 17)

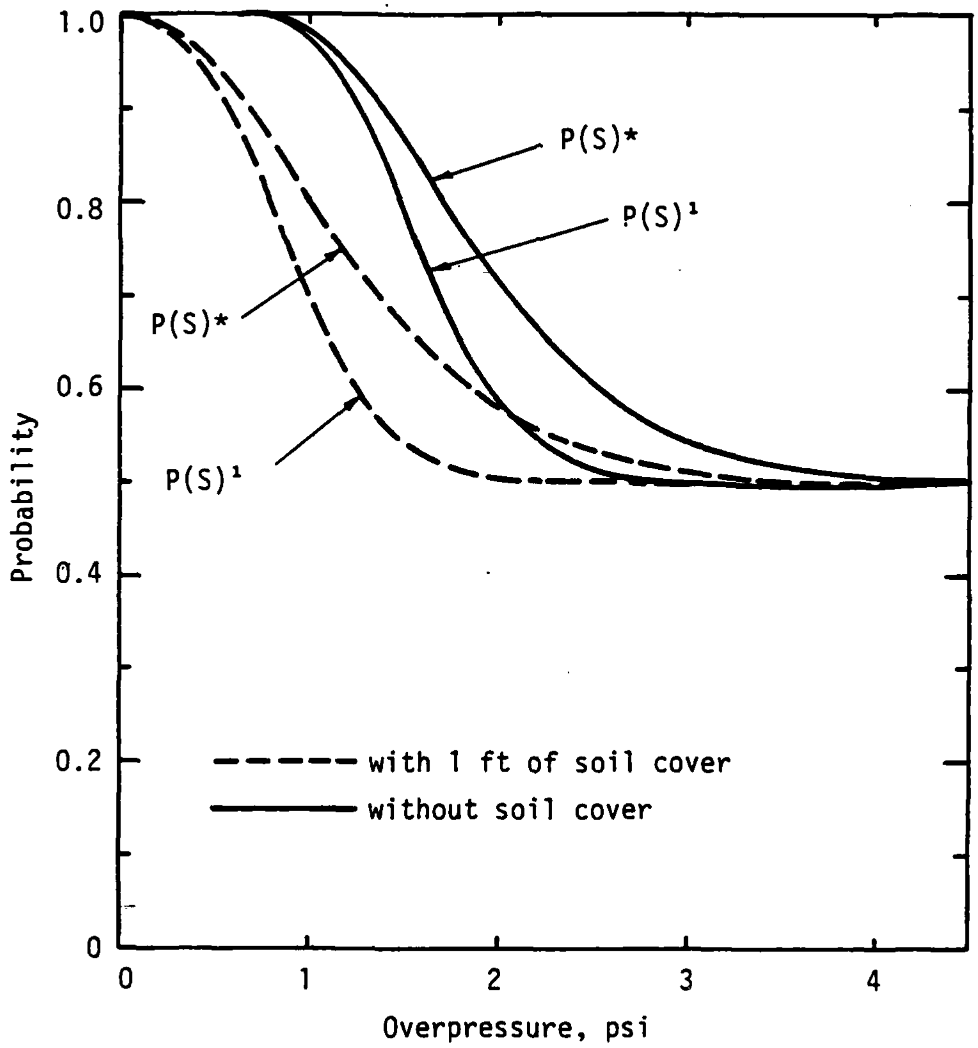

Fig. 1-2. Probability of People Survival, Upper and Lower Bound. (Ref. 17)

be developed for an additional seven basement structural types. In the final year of the program, a procedure is to be formulated and developed for rating similar basements, both as-built and upgraded. This procedure should require the use of data that can be obtained readily in the field by the local civil defense planner.

OBJECTIVE

The objective of the present research effort as given in the Statement of Work is to "Provide FEMA with a sufficient range of damage functions to describe the survivability of people in various shelters. Each shelter type has a unique protective capability and thus any assessment of national survivability of people can be altered by the shelter type mix and the protective capability assigned to them. Thus, national security policies, programs and survivability are dependent upon knowing shelter damage functions. A procedure for rating shelter spaces will be produced."

STRUCTURAL ELEMENTS TO BE CONSIDERED

As described in other sections of the present report, the intent is to consider not only the first floor slabs over the basements but all other structural elements and other factors that materially affect the survivability of the basement or the people in it.

TYPES OF SHELTERS TO BE CONSIDERED

The functional types of shelters that will be considered in the present study will be all those types of basement shelters expected to be required under present FEMA planning. Risk area personnel shelters, required to survive at peak free field overpressures of 40-50 psi, and host area shelters, required to survive at peak overpressures of 2 psi, will be emphasized. In view of the current FEMA sponsorship of slanting the designs for basements of two buildings so that they will survive at the 15 psi level and other FEMA-sponsored research addressing intermediate overpressures, a potential requirement for crisis period shelter upgrading to intermediate levels is also inferred, even though shelters specifically designed to withstand a defined level of nuclear effects are outside of the scope of the present program.

Thus, shelters capable of withstanding peak overpressures from 2 psi to 50 psi are of interest. Since damage functions are being studied, survival levels from higher than 50 psi to as high levels as any of the upgraded basements can survive and failure levels from 2 psi down to as low levels as cause failure in any as-built basements of the types to be considered are of interest.

ALL EFFECTS UPGRADING AND SURVIVABILITY TO BE CONSIDERED

When determining the upgrading for basements, it is desirable to specify hardening measures against blast, initial nuclear radiation, residual nuclear radiation, and thermal effects as appropriate, considering the structural characteristics and locations of the basements in question, and other factors, as discussed in more detail later in the report. Since this program is concerned with making reasonable estimates of people survivability in basement shelters, it is not desirable to determine their survival against certain nuclear weapons effects while neglecting other likely casualty producing effects.

YIELDS AND HEIGHTS OF BURSTS CONSIDERED

In determining damage functions and casualty functions for basement shelters, it is necessary to make certain assumptions as to the yields of weapons that are to produce the damage or casualties. As weapon yield increases, the characteristics of the air blast and other effects change (e.g., air blast positive phase duration increases with yield), and the relative intensity levels of the effects (e.g., blast vs initial nuclear radiation) also change. The character of the effects and the relative effects levels also are different for surface bursts as compared with air bursts of the same yield. Since this program deals with civil defense shelter effectiveness, it will be concerned primarily with the comparatively high yields associated with current Soviet strategic offensive weapons. The program will need to consider surface bursts as well as air bursts, since either type or some combination of both may be used for strategic attack. Previous researchers have frequently assumed the use of 1 Mt surface bursts. This program will also use this assumption, while retaining the option to use other yields and burst heights if this becomes desirable in the future.

SURVIVAL PERCENTAGE ASSUMPTIONS

The casualty functions to be developed for individual basement shelters will ordinarily cover the range from zero percent casualties to one hundred percent casualties. However, the question how survival levels, or casualty levels, are defined in terms of nominal overpressure or other effects specifications for shelters is still of interest. In the past, if a shelter was assumed to be capable of surviving say 2 psi, this was sometimes taken to mean that 50 percent of the people in the shelter would survive if the shelter were exposed to this level, or that 90 percent or some other percent would survive. Sometimes the approach has been to list survival overpressures for several different percentages of survivors, such as 10, 50, and 90 percent (see Reference 13). For the present purposes, it will be assumed that survival of 90 percent of the people in the shelter is the standard, rather than 50 percent. It is believed that this standard is a reasonable one from the point of view of civil defense planning, since it would be desirable to base planning on a specification that provides for a high percent of survivors.

CONCEPTUAL DESCRIPTION OF RATING SYSTEM

The overall objective of this program, at the end of five years, is the development of a procedure for rating existing basements, both as-built and upgraded, for use by Civil Defense planners using data readily available in the field. The intent is to provide a system usable in the field by people of many disciplines, not just the structural engineer. The work during the first year has established a portion of the basis for the eventual system, although many of the needed features remain to be developed. The system is expected to evolve as the work progresses, but the general concept for the fully developed system will probably resemble that described here. A somewhat similar prediction model, developed by SSI for the prediction of secondary fires produced by a nuclear weapon, is described in Reference 19. The proposed format for the damage and casualty function rating procedure is presented in Figure 1-3. This format contains six indices, which it is anticipated will be developed along the following lines:

Building Index

Based partially on the list of structural systems described in the present report, this research will develop a building index. This is expected to be more than a list

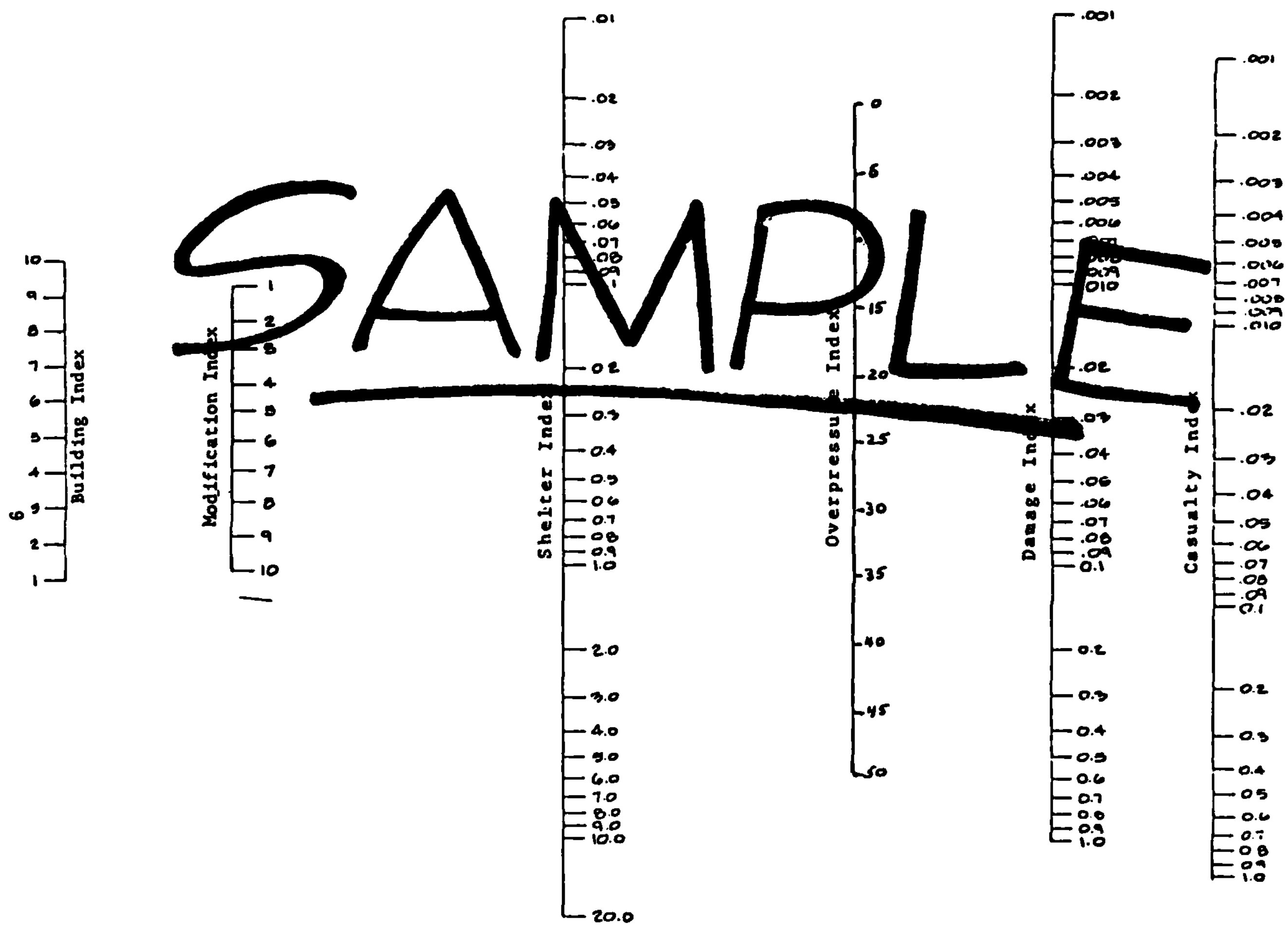

Fig. 1-3. Alignment Chart for Prediction of Damage and Casualty Functions.

of structural systems (flat slab, flat plate, etc.). In its final form this index will allow ranking structures using simple descriptions, sketches, photographs, etc. This building index will take into account the various structural elements that make up a basement shelter system (basement ceiling, basement floor, exterior walls, interior partitions, framing, connections, openings) and other items such as design live load (i.e., original intended use), soil conditions, and material properties. It will also take into account the aboveground characteristics of the building that have an effect on the integrity of the shelter.

In this procedure structures will be ranked as to strength or survivability taking into account each of the structural elements that make up the structure. This procedure will also take into account: Condition of the structure; age; soil conditions; size, location, and number of openings; location of structure and its relationship to other structures; and a range of other factors that could have an effect on the survivability of the basement shelter. Also to be taken into account will be consideration for initial radiation, which could significantly change the rating with regard to the prediction of casualties.

Modification Index

Field surveys indicate that very few actual basements are of the simple type usually considered in shelter survivability and upgrading studies; i.e., a wide open expanse with a basic basement floor, ceiling, and framing system taken directly from a standard design manual. In most cases, either in the original design or in subsequent remodeling, interior partitions are included, and holes are cut for utilities, doors, etc. Such modifications can either degrade the structure or, in the case of certain interior partitions, act as a form of upgrading. In the development of this index, these factors wil be taken into account. This is also the index that will account for the increase in survival rating accomplished by various types of upgrading. Both the effects of structural upgrading and of placing dirt on the floor above the basement for radiation protection will be included.

Shelter Index

This index is to be the product of the building and modification indices. It will serve to rank the basement shelters as to their survivability. Thus, it should be a valuable tool in the field, since it will provide a relatively easy way to rank shelters in a community and to quickly determine which of the available structures have the potential for being the best shelters and thus to establish priorities for upgrading.

Overpressure Index

This index will be used to allow data to be obtained from the remaining indices as a function of overpressure. Procedures will be developed to take into account the factors that might affect the actual overpressure loading on the basement structure, such as dirt cover, shadowing caused by nearby buildings, or the type of superstructure covering the basement.

Damage Indices

These indices will give a measure of the percent of damage to the basement space caused by a given overpressure. They will be developed from nuclear weapons and high explosives test data, and other experimental data obtained and analysis performed during the development of the Key Worker and Host Area Manuals (References 20 and 21) and in other FEMA research programs conducted by SSI. These include programs to obtain building collapse data, to determine the effects of frame response, and on upgrading structures for host and risk area shelters. Data from the recent MILL RACE test program, in which a number of representative basement ceilings, both as-built and upgraded, were exposed to approximately 40 psi, will be particularly useful (Ref 22). These tests also included a number of instrumented anthropomorphic dummies supplied by the Lovelace Foundation.

It is expected that a set of damage indices will be required, rather than a single damage index, to accommodate the range of basement types to be rated. Present planning is for the indices to be developed using probabilistic procedures. Using the fragility curves approach is anticipated, which has been used extensively to describe probability of failure of nuclear power plant components as a part of the assessment of nuclear seismic safety. These curves express, with a known degree of confidence, the probability of failure due to seismic loading of items that form nuclear safety systems. From these curves, probabilities of failure have been determined at various seismic levels and used as input to a "systems analysis" to calculate the failure consequences and overall system reliability. This type of methodology can be applied to the analysis of structural systems such as basement shelters in a manner similar to its application in the nuclear industry. The underlying distributions for this methodology are generally log normal, which make their manipulation convenient and easily combined with the casualty data described below.

Casualty Indices

These indices are to form the end product of the prediction procedure. They are to give the numbers or percentages of expected casualties as a function of the shelter characteristics and the expected overpressure. These indices will be developed from available casualty studies and related information, including nuclear test results, the results of high explosives tests such as MILL RACE, and from studies of other types of casualties such as those from vehicle accidents.

These indices will be developed using the same probabilistic procedures as are to be used for the damage indices. These also are expected to be based on a log-normal distribution function vs overpressure and are expected to comprise a family of curves similar to the structural survivability curves. The basis for the log-normal assumption is the extensive use of the log-normal distribution in analyzing injury and mortality data. It will probably also be desirable to have a family of casualty (and damage) indices that indicate different confidence levels; e.g., 50%, 90%, etc.

ANTICIPATED DATA REQUIREMENTS FOR DAMAGE FUNCTION RATING PROCEDURE

As has been indicated, the final procedure is to require the use of data that can be readily obtained in the field. The current expectation is that the user would require the information shown in Table 1-1 to utilize the completely developed Damage Function Rating Procedure for rating similar basements. Alternatively, if some of the information is unavailable, predictions could still be made, although accuracy might be degraded.

GENERAL CONTENTS OF THIS YEAR'S REPORT

Determination of the potential for survival of persons in basement shelters involves predicting when the basement structure may fail as a result of air blast effects. It also involves predicting the extent to which the people in the shelter will become casualties in the event that structural failure does take place. Casualties may also be produced through other casualty mechanisms and from other weapons effects and these must also be taken into account. In order to provide a current basis for this portion of the overall effort, a review of relevant casualty data

Table 1-1

LISTING OF DATA REQUIRED IN FIELD BY LOCAL CD PLANNER IN ORDER TO UTILIZE DAMAGE FUNCTION RATING PROCEDURE

Type of Information	Specific Data
Shelter Planning	Shelter category (key worker/host area/other)
Area Hazards	Collapse of other buildings, etc. Fire Hazardous material storage Flooding Likely targets for attack
Building History	Original intended use Approximate year constructed Building code used Current use(s)
Building Structure	
General	Basement usable floor area Basement ventilation capacity Basement water supply Basement sanitary facilities
Unusual Hazards	Types Quantities Locations
Openings	Doors to outside Windows to outside Stairways Elevator shafts Other openings into basement

Table 1-1 (continued)

Type of Information	Specific Data
First Floor Slab	Type (flat slab, etc.) Column spacing - bay sizes Thickness
Basement Exterior Walls	Type (cast in place, etc.) Thickness
Connections	Wall to floor Wall to wall (if precast) (vertical)
Basement Floor	Type (concrete, dirt, etc.)
Columns	Dimensions Capital and drop panel dimensions
Soil by Building	Backfill type Height on walls Availability for Upgrading
Basement Partitions	Type Locations Ease of removal (connections)
Upper Portion	Number of stories Spacing between floors Type of exterior walls

and prediction models was conducted. This information is presented in Section 2 of the report.

The emphasis during the first year's effort has been on flat slab basements. Section 3 of the report contains a general description of flat slab basement design practices, categories, and trends. This information is intended to provide background information on the specific class of basements analyzed this year.

Section 4 contains a description of relevant structural failure mechanisms for flat slab basements and of relevant casualty mechanisms for persons who may be in shelters of this type. It establishes the basis used in determining the damage functions and casualty functions for the specific as-built and upgraded flat slab basements analyzed this year. A discussion of current capabilities and limitations of the procedure is included. It is anticipated that this procedure will be completed during the remaining years of the projected five-year program. The slabs for the floor above the basement for the specific basements analyzed during the first year were of the flat slab type. However, the procedure being developed is expected to be capable of rating basements with a wide variety of types of floor slabs and other structural features. It is intended that basement ratings will take into consideration the main factors affecting survivability for basement shelters. The influence that basement walls may have on the survivability of the shelter has been investigated as part of the first year's work and is discussed in this section.

In order to illustrate the current capabilities of the casualty function development procedure, specific examples of the use of the interim procedure for both as-built and upgraded flat slab basements are provided in Section 5 of the report.

A summary, conclusions, and recommendations, including proposed modifications to the scope of work, are contained in Section 6 of the report. It is suggested that the present emphasis of the project on analysis of specified types of floor slabs be modified so as to place more effort on other parts of the overall program.

Four appendices are also included in the report. The Damage Function Rating Procedure work unit discussed in this report is expected to build on the previous research in the area funded by FEMA and its predecessor agencies. Since it is expected to be carried on for a five-year period, it is important that the present

program be based on a good understanding of previous work. Relevant research on developing damage function rating procedures is summarized and the capabilities and limitations of available survivability prediction techniques are described in Appendix A.

Appendix B contains a description of basement structural systems and the connections between them for various structural types of basements. One of the tasks of the first year's effort has been to prepare a descriptive framework of the elements expected to influence basement survival. This includes a listing and description of basement structural systems, and a discussion of relevant factors such as design live loads, ranges of span length (aspect ratios), soil conditions, and material properties.

In Appendix C, information on basement wall response and performance under dynamic loading conditions is presented. This information has been derived from field and laboratory testing and analysis. It was utilized to formulate an interim procedure for predicting the response of basement walls.

Appendix D contains a description of how the concept of "Potential for Upgrading" might be developed for future utilization. Although somewhat broader than the present program in concept, the techniques to be developed here would form the basis for the concept.

Section 2
CASUALTY PREDICTION MODELS FOR BASEMENT SHELTERS

INTRODUCTION

The survivability of people in basement shelters depends on many factors. These include: the structural characteristics built into the basement; the type of upgrading completed prior to the attack (if any); the extent to which structures, or other objects in the vicinity, may serve to shield or present added hazard to the basement; the yields, heights of burst, ranges, and other characteristics of the weapons that may be employed; the attack period winds, atmospheric pressure, visibility, cloud cover and other atmospheric variables; and numerous other factors.

In this section, some general considerations of casualty prediction models for shelters are discussed. Because there are significantly different casualty mechanisms for an upgraded shelter as compared with one that is in the as-built condition, both situations are covered.

GENERAL CONSIDERATIONS OF CASUALTY PREDICTION MODELS FOR SHELTERS

Up to the present time, methods have been developed for predicting personnel casualties in a variety of structures and shelters. Unfortunately, however, these methods are quite complex. To illustrate the complexities inherent in this problem Table 2-1 lists the various casualty producing mechanisms that need to be considered in developing comprehensive casualty functions.

Consider the important case of risk area personnel basement shelters that have been upgraded such that:

- o the roof slab will withstand 40 - 50 psi,
- o all entrances are sealed, also to a level of at least 40 - 50 psi, and
- o sufficient earth has been added to provide a PF (protection factor) of 1000 for fallout.

Table 2-1

CASUALTY PRODUCING MECHANISMS OF A NUCLEAR EXPLOSION FOR PERSONNEL IN SHELTERS

Primary blast	-	exposure to fast rising, long duration pulses
Secondary blast	-	exposure to impact of missiles accelerated by the dynamic pressure in the blast wave
	-	translation of the body by dynamic pressure and exposure to impact on the ground or other surfaces
Initial nuclear radiation	-	early time (less than 1 min) exposure to gamma and neutron radiation
Fallout	-	exposure to fallout radiation - primarily gamma in structures
Fire	-	exposure to the effects of fires caused by the thermal radiation and blast effects - flames - toxic gases - smoke - oxygen deprivation

Under this set of conditions calculations of casualty functions are considerably simplified since up to the point of initial roof collapse none of the casualty producing mechanisms in Table 2-1 is operative. The closed shelter eliminates thermal and fire effects, while the PF of 1000 eliminates fatalities from the initial nuclear radiation and fallout.* At the other limit when the slab completely collapses virtually all of the casualty mechanisms come into play, with the most serious being the collapse and initial nuclear radiation.

Considering slab collapse first, it seems reasonable to assume as in Ref. 14 that personnel located under the collapsed slab be considered as fatalities. However, as there may be some space clear near the walls, there would be expected to be some survivors from this mechanism.

Now with the roof collapsed, the initial nuclear radiation becomes much more serious because the PF of the shelter is greatly degraded.

From an examination of tables 8.72 and 9.120 of Ref. 23 it is concluded that for the intact shelter the initial nuclear PF is about 100, while for the collapsed structure it drops to less than 10.**

Table 2-2 gives the initial nuclear radiation dose in rems for both the intact and collapsed shelters at various pressure levels for a 1 Mt weapon and for various heights of burst (HOB) from a surface burst to the height that optimizes the various pressure levels. The next to last column gives the dose for the intact shelter and the last column that for the collapsed shelter.*** Table 2-3 gives calculations for the doses that appear in Table 2-2.

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* This is not quite true for the initial nuclear radiation. The higher energy gamma rays associated with initial nuclear radiation, as well as the neutrons, which are part of the initial nuclear radiation but not of the fallout, make it more difficult to shield against initial nuclear radiation than against fallout. Thus, additional earth for shielding against initial nuclear radiation may be needed.

** The reason for a smaller PF for the initial nuclear radiation is that its energy is considerably higher than that for fallout.

*** Note that for the Mt range weapons the roof collapses fast enough that essentially all the initial nuclear radiation dose is received after the collapse (see Fig. 8.47 of Ref. 23).

Table 2-2

INITIAL NUCLEAR DOSE FROM A 1 MT WEAPON FOR VARIOUS HOB'S, OVERPRESSURES, AND SHELTER CONDITIONS

HOB (ft)	OVERPRESSURE (psi)	DOSE**(rem)		
		FREE FIELD	INTACT SHELTER	COLLAPSED SHELTER
6000*	20	24	< 1	> 2
5300*	30	560	6	> 56
4700*	40	1800	18	> 180
3700*	50	12000	120	> 1200
2000	20	1100	11	>110
2000	30	7900	79	>790
2000	40	16000	160	>1600
2000	50	26000	260	>2600
0	20	2700	27	>270
0	30	19000	190	>1900
0	40	39000	390	>3900
0	50	95000	950	>9500

* Optimum HOB for given pressure level

** A protection factor of 100 was assumed for the intact shelter (Table 8.72, Ref. 23, shelter with 3 ft earth cover)

A protection factor of <10 was assumed for the collapsed shelter based on the same table as referenced above.

A relative biological effectiveness of 1.0 was assumed for neutrons.

TABLE 2-3: CALCULATIONS FOR INITIAL NUCLEAR DOSES

Y (MT)	P (psi)	HOB (ft)	GR (ft)	SR (ft)	SR (yds)	D_n (2) (rad)	$D_{\gamma s}$ (2) (rad)	$D_{\gamma f/We}$ $X10^{-3}$	W_e/W	$D_{\gamma f}$ (1) (rad)	D_T (rad)
1	20	6000**	9200	11000	3700	< 1	5	1	38	19	24
1	30	5300**	6400	8300	2800	22	90	35	26	450	560
1	40	4700**	5500	7200	2400	150	400	120	21	1300	1800
1	50	3700**	4900	6100	2000	1200	1800	1000	18	9000	12000
1	20	2000	7400	7700	2600	60	200	80	22	880	1100
1	30	2000	6000	6300	2100	600	1300	600	20	6000	7900
1	40	2000	5300	5700	1900	2000	2500	1500	15	11000	16000
1	50	2000	4700	5100	1700	5000	6000	3000	10	15000	26000
1	20	0	7100	7100	2400	75	200	120	40	2400	2700
1	30	0	5900	5900	2000	600	900	1000	35	17500	19000
1	40	0	5200	5200	1700	2500	3000	3000	22	33000	39000
1	50	0	4600	4600	1500	8000	6000	9000	18	81000	95000
	Fig. 3.73b Effects of Nuclear Weapons*					Fig. 8.123b*	Fig. 8.127b*	Fig. 8.130a*	Figs. 8.130b and 8.132*		

** Optimum HOB for given pressure level

(1) $D_{\gamma f} = D_{\gamma f/We} \times We \times 1/2$ for a weapon with 50% fission yield.

(2) Use factor of 1/2 for surface burst.

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* Ref. 23.

As a basis for evaluating the dose levels in Table 2-2 the probable effects for various dose levels (Ref. 23) are given below.

Dose (rems)	Probable Effect
0 - 100	No illness
100 - 200	No or slight illness
200 - 600	0 - 90% deaths
600 - 1000	90 - 100% deaths

From this it can be seen that for the intact shelter this casualty mechanism would not reach the threshold for fatalities until somewhere above 50 psi for the optimum HOB and slightly below 50 psi for the low air burst (2,000 ft HOB). For the surface burst condition, however, it starts at slightly above 30 psi and by 50 psi essentially 100 percent fatalities would occur.

For the collapsed shelter it shows the 100 percent fatalities would be obtained between 40 and 50 psi for the optimum HOB case, at about 30 psi for the low air burst, and between 20 and 30 psi for the surface burst case.

With the upgraded shelter designed to collapse at 40 psi or higher, only for the optimum HOB is there any chance of survival after shelter collapse, and that appears to be quite marginal. Futhermore, optimizing for 40 psi seems rather unlikely. Thus, to a good approximation the casualty function for the upgraded basement shelter is identical to the damage function of the roof slab. Survival is 100 percent up to the initiation of collapse and 0 percent after total roof collapse.

Note that the conditions set for the upgraded shelter -- roof slab strengthened to 40 psi, all entryways sealed to the same level, and sufficient earth cover added to give a PF for fallout of 1000 -- are not arbitrary, but are those used as a basis for the upgrading systems presented in Ref. 20 for a variety of roof slabs. In fact, it is suggested that when the details of the actual collapse process are worked out that the resulting casualty functions be included with the upgraded designs in Ref. 20.

Having shown how effects to be considered are limited in making casualty estimations for upgraded shelters, it is in order to comment about the additional factors that have to be included for shelters that have not been upgraded. These include some or all of the following:

1. Entry of the blast wave into the shelter through existing openings and/or as a result of collapse of part or all of the roof slab.

a. With blast entry there is the high probability of jet formation except for cases with a very small V/A ratio.* Very high dynamic pressures can be obtained within the jet, which can lead to secondary blast fatalities.

b. Primary blast would not cause a great many fatalities since its threshold is 40 (30-50) psi. Severe lung damage occurs at 25 (20-30) psi, however, and 50 percent eardrum rupture at 15 psi, Refs. 23 and 24.

c. Note that maximum internal pressure in the shelter is dependent primarily on the V/A ratio. For very small V/A ratios (on the order of 10) large internal pressures can be obtained by reflection and they may be significantly greater than incident, even by a factor of two.

CONSIDERATION OF PERSONNEL CASUALTIES IN OPEN BASEMENT SHELTERS

Blast Effects

There are four casualty causing mechanisms due to blast effects in open basement shelters. These are:

Direct blast	- exposure to fast rising, long duration pulses
Indirect blast	
translation/impact	- translation of the body by dynamic pressure and exposure to impact on the ground or other surfaces.
missiles	- exposure to impact of missiles accelerated by dynamic pressure
structural collapse	- exposure to collapse of the shelter roof or other structural elements

* V is volume of the shelter; A is the cross-sectional area open to the blast wave.

Direct Blast - The importance of direct blast depends primarily on the V/A ratio of the shelter where V is the shelter volume and A is the cross-sectional area through which the blast wave enters the shelter. For very small V/A ratios (say on the order of 10) direct blast casualties are expected to be equally as bad if not worse than in the free field at the same location. This is because the interior pressures are significantly greater than the incident pressure during the initial reflection and diffraction processes.

See for example Table 2-4 which gives experimental results for a shelter having a V/A ratio of 12.2. In this table Column 5 is the ratio of the peak interior overpressure to the peak free field value. It can be seen that the ratio varies from 1.9 to 2.5.

As the V/A ratio increases, the ratio of interior to free field pressure decreases. See for example Table 2-5, which gives experimental results for shelters having V/A ratios on the order of 50. It can be seen that the ratio of interior pressure to free field varies from about 0.5 to 0.8. Thus, for these cases the shelter is safer than free field.

It also should be noted that the maximum pressure in the interior of the shelter for the case of V/A ratios on the order of 50 is not generally achieved in a single step but rather in two or more steps. This is less damaging to the human body.

For large V/A ratios the interior pressure is a relatively small fraction of the free field value and is reached quite slowly, so that direct blast damage is of litttle importance. See, for example, Table 2-6, which gives values for V/A ratios from about 175 to 750. They range from about 0.24 to 0.10. Also note the long times necessary to achieve the maximum interior pressure (Column 6).

Table 2-4

EXAMPLES OF SHELTERS WITH SMALL V/A RATIOS

Event	V/A (ft)	P_o (psi)	P_i (psi)	P_i/P_o	Time to Peak (msec)
APPLE I	12.2*	17	41	2.4	20
APPLE I	24.2(12.2)**	17	43	2.5	29
APPLE II	12.2	40	86	2.2	4
APPLE II	12.2	25-30 (est)	53 (est)	1.9	-

Data from Ref. 24. APPLE I 14 kt, APPLE II 29 kt. Both tests had precursor wave forms. *Basement exit shelter 3 x 13 x 5 ft. **Probably opened by blast, P_o is peak incident overpressure, P_i is peak interior overpressure.

Table 2-5

EXAMPLES OF SHELTERS WITH MEDIUM V/A RATIOS

Event	V/A (ft)	P_o (psi)	P_i (psi)	P_i/P_o	Time to Peak (msec)
MILL RACE	44	27	14	0.52	-
APPLE I	65.8	47	34	0.72	69
APPLE II	65.8	92	67	0.73	99
KEPLER	65.8	42	26	0.62	53
GALILEO	65.8	39	30	0.77	65

Data from Ref. 24. APPLE I 14 kt, APPLE II 29 kt, KEPLER 10 kt, GALILEO 11 kt. MILL RACE (1 kt) and KEPLER had ideal wave forms, all others precursor. P_o is peak incident overpressure, P_i is peak interior overpressure.

Table 2-6
EXAMPLES OF SHELTERS WITH LARGE V/A RATIOS

Event	V/A (ft)	P_o (psi)	P_i (psi)	P_i/P_o	Time to Peak (msec)
APPLE I	586	47	6.7	0.14	206
APPLE II	172	92	22	0.24	126
KEPLER	174	42	9.5	0.23	98
GALILEO	757	39	4.1	0.10	378

See Footnotes for Tables 2-4 and 2-5.

Indirect Blast - Translation/Impact - The importance of the translation/impact casualty mechanism also depends primarily on the V/A ratio, however, in inverse order, i.e., the smaller the ratio the less important is this casualty mechanism. This is because the jet, which is the major source for translation in basement shelters, has a short duration for small V/A ratios. The reason for the short duration is that such shelters fill very rapidly (interior pressure equals exterior pressure). Ref. 25 gives the following as an approximation of the filling time of a shelter (t_f) for a constant pressure blast pulse:

$$t_f \simeq V/2A \text{ (msec)}$$

where V is in ft^3 and A is in ft^2

For a V/A ratio of 10 this equation gives a filling or jet duration time of only 5 msec which appears insufficient to cause problems.

For large V/A ratios (greater than 100) translation/impact under jet action can be very serious because the duration of the flow is relatively long and the dynamic pressures so high. As an illustration of the magnitude of the jet flow, a 5 psi overpressure loading on the wall of a building will produce a dynamic pressure in the jet flow equal to that produced by a 14 psi overpressure shock wave in the free field. Note that the 5 psi loading can be produced by as little as a 2.2 psi overpressure

shock wave incident normally on the wall. The relationships for other overpressures are given below:

Table 2-7
OVERPRESSURES FOR EQUAL DYNAMIC PRESSURE

Free Field Shock Flow (psi)	Jet Flow From Side-on Shock (psi)	Jet Flow From Normal Incidence Shock (psi)
10	2.5	1.2
14	5	2.2
19	10	4.3
26	15	6.1

Casualties from jet flow can be avoided by restricting regions in the shelters that will be occupied by personnel to those outside of the jet region. Ref. 11 gives the area of the shelter that poses a potential impact hazard as a function of overpressure for various geometries.* At 15 psi, depending on shelter geometry and size, between about 10 and 40 percent of the shelter source would be unusable. Calculations were not given for higher overpressures, however, it is readily apparent that the percentages will increase with increasing overpressure. Note also that this space could not be used for storing supplies or equipment, as these might become dangerous missiles.

Ref. 11 also showed that certain shelter geometries are worse than others for jet flow and that, as the overall size increases, the problem becomes more serious. However, even for quite small shelters the jet problem can be very serious. Consider for example the personnel shelter treated in Ref. 26, which had a V/A of 160 and a total volume of 1274 ft^3. It was calculated (with experimental backup) that a person in line with the entryway tunnel to this shelter had a 5 percent chance of impact injury at 8 psi incident and 50 percent chance at 15 psi. This was for a 1 kt weapon.

* Impact velocity greater than 10 ft/sec. Note that Ref. 24 gives a 5 percent chance of serious injury for an impact velocity of 8.4 ft/sec for normal incidence against a nonyielding flat surface.

For intermediate V/A ratios (say of the order of 50) it is believed that the jet can still be a serious problem for personnel located in its region despite its short duration. Consider, for example, the rather small impulsive loading necessary to accelerate a man to a hazardous velocity. Assuming pure impulsive loading (from Ref. 25)

$$v = I_D/M$$

where

v = change in velocity of a man
I_D = drag impulse
M = mass of a man

Further

$$I_D = C_D A I_q$$

where

C_D = drag coefficient
A = area of man presented to flow
I_q = dynamic pressure impulse

Thus

$$v = (C_D A/M) I_q$$

The factor $(C_D A/M) = C_A$ is called acceleration coefficient. Values of C_A for a 168 lb man vary from 1.67 ($ft^3/lb\ sec^2$) for a man standing facing the wind to 0.2 prone and aligned with the wind. (Ref. 25)

For I_q in psi-sec this equation becomes

$$v = 144 C_A I_q$$

Now consider the impulses necessary to achieve a velocity of 13.4 ft/sec, which according to Ref. 24 is the velocity necessary to achieve a 50 percent probability of serious injury. These are given in the following table for several body orientations.

Table 2-8

BODY ORIENTATIONS vs DYNAMIC PRESSURE IMPULSES REQUIRED FOR 50 PERCENT PROBABILITY OF SERIOUS INJURY

Body Orientation	Dynamic Pressure Impulse Necessary for 13.4 ft/sec
Standing/facing	0.056
Standing/sidewise	0.13
Prone/perpendicular	0.13
Prone/parallel	0.47

As a first approximation the dynamic pressure impulse from a jet with a peak dynamic pressure of q and a duration of t due to shelter filling is 0.5qt. Thus, for example, with a V/A ratio of 50 or an estimated jet duration of 25 msec, it would require q values of 4.5, 10.4, and 38 psi for the three dynamic pressure impulse values given above. The first two of these can be obtained with peak overpressure loadings of 5 and 13 psi respectively (side-on) while the third condition cannot be met.

At first glance the above calculations appear to be inconsistent with the experimental results obtained from the Swedish Field Fortification on Event MILL RACE (Ref. 24). It is estimated that the V/A ratio was 44. The measured free field incident pressure was 27 psi. By use of instrumented dummies it was concluded that one would not expect any personnel injury from blast displacement in the field fortification. The dummies used were mostly seated, with one lying prone. The seated dummies are estimated to have the same C_A as the side-on man or the intermediate case discussed above. The above calculation showed that only 13 psi was necessary to create severe jet acceleration problems. Why then did this not show up in the experimental field test? One of the major factors may well be that the jet does not form immediately. An approximate equation of the formation time is (Ref. 25):

$$t_s \approx 4B_o/C_o$$

where

t_s is the formation time of the jet in msec

B_o is the diameter of a circle with the same area as the entrance area to the shelter in ft

C_o is the velocity of sound in ft/msec

Taking B_o as 4.1 ft and C_o as 1.13 ft/msec gives a jet formation time of 15 msec. For the free field pressure pulse the overpressure had dropped to about 18 psi 15 msec after the start of the pulse. Also, during the filling time of the shelter, estimated to be 22 msec, the pressure drops from 18 psi to about 10 psi. In the earlier calculations it was assumed that the pressure remained constant.

Other factors for the difference in results could be that the dummies were not fully in the jet flow and the possibility that the assumed C_A value was too high.

Indirect Blast - Missiles - Missiles can be a problem whenever jet formation occurs if there is loose material in the path of the jet or if an interior partition fails under the loading of the jet. By elimination of interior partitions and good housekeeping this problem can be eliminated.

Indirect Blast - Roof Collapse - For large V/A ratio shelters the effects are the same for open or closed shelters, since interior pressure builds up too slowly to reduce exterior loads. For other cases it would seem to depend on the actual geometry.

Brief Comment on Other Weapons Effects - An open shelter permits the entrance of scattered prompt nuclear radiation, which could significantly increase the dose in the vicinity of the entrance. Fallout is somewhat similar, and in fact it could even enter an open shelter directly, however, there is time after the blast to close up the shelter for fallout. These problems are clearly more severe for small V/A ratio shelters.

An open shelter also permits the entrance of noxious or toxic smoke or gases from fires. This is a somewhat more serious problem for small V/A shelters.

Summary

From the foregoing it would appear that any use of open basement shelter should be considered very carefully because of the potential hazards. For shelters with very small V/A ratios (say, of the order of 10) there appears to be no

displacement/impact problem but the direct blast problem is more severe than in the free field since the interior pressures may be several times the incident value. Thus the 50 percent eardrum rupture* level, which is approximately 15 psi, may be caused by as low as 6 or 7 psi incident. Further, the threshold for lethality from direct blast, which is reported to be about 40 psi, may occur at incident levels as low as 16 to 20 psi.

For intermediate V/A ratios (say of the order of 50) the direct blast problem decreases since the interior pressures are some 50 to 80 percent of the free field values. Thus, the 50 percent eardrum rupture pressure would be some 20 to 30 psi incident, and the lethality threshold is moved up to 50 to 80 psi incident.

The jet flow translation problem, however, is starting to become important. Simple dynamic pressure impulse calculations suggest that there is sufficient impulse to give damaging velocities to personnel in shelters; however, at Event MILL RACE in a shelter with a V/A of about 44 there was no translation/impact damage for dummies at an incident overpressure level of 28 psi.

For larger V/A ratios (say greater than 100) the direct blast problem no longer is important, however, significant jet formation seems assured. Thus the jet region of the shelter has to be avoided if translation/impact casualties are to be eliminated. The unusable floor area of the shelter is reported to be from about 10 to 40 percent of the total depending on geometry at a pressure level of 15 psi incident (side-on) with the percentage increasing with increasing overpressure level.

Any open shelter would permit the entrance of scattered prompt nuclear radiation and possibly create a dose problem in the vicinity of the entrance, particularly for the smaller V/A ratio shelters. Similar fallout problems can be eliminated by closing the shelter after the blast.

Any open shelter also would permit the entrance of noxious or toxic smoke and gas, which could create problems throughout the shelter. Again, this is likely to be a somewhat more severe problem for small V/A shelters.

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* It is noted in Ref. 24 that eardrum rupture can easily be avoided by holding or placing hands or fingers over the opening into the external ear canal.

Section 3
REINFORCED CONCRETE FLAT SLAB CONSTRUCTION

INTRODUCTION

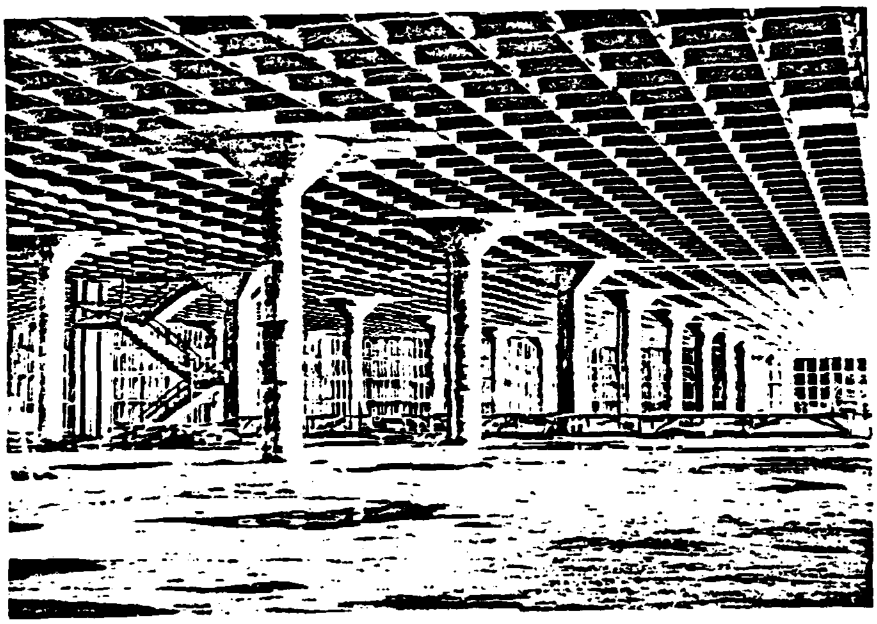
The principal direction of this year's effort is to develop the casualty functions for one specific type of construction, reinforced concrete flat slabs. This section of the report will describe this type of construction, discuss the basic methodology used in the design of these slabs, outline the parameters, both economic and structural, that are presented to the design professional when considering using flat slab construction, and to the extent possible, discuss this system on a comparative basis with other similar systems. It is the intent of this section to present an overview of this type of construction, using a minimum of technical terminology, that would complement and assist in a better understanding of the other sections of this report. Additional background material on basement structural systems in general can be found in Appendix B.

DESCRIPTION

A flat slab is defined as a concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to the supporting members, usually columns. To assist in transferring the loads on the flat slabs to the supporting columns, the upper portion of each column may be enlarged to form a column capital. To strengthen the portion of the slab immediately surrounding each column or column capital so that it may better resist the intensified stresses in this region, the slab may be thickened in this area by adding to its bottom side to form a drop panel. This drop panel is cast monolithically with the remainder of the slab. A photograph of this type construction is shown in Figure 3-1 (a).

In some slabs, to save concrete and reduce the weight of the slab, much of the concrete on the lower side of the slab is eliminated, leaving only ribs or joists at right angles to each other with solid heads at the columns. These types of slabs are

a. Typical Flat Slab Construction



b. Typical Waffle Flat Slab Construction

Fig. 3-1. Typical Flat Slab Construction.

called ribbed, or "waffle", slabs and for design purposes are considered flat slabs with the solid heads at the columns performing the same function as the drop panels. Waffle flat slab construction is shown in Figure 3-1 (b).

DESIGN

Historical - Flat slabs were constructed for many years before an adequate analysis was available. Although controversy exists as to the originator, an American, C. A. P. Turner, is considered by many to deserve that honor. By 1913, over 1,000 flab slab buildings had been built, most all from intuition as to how they would function. Many required that proof loads be applied to satisfy the owners, and numerous slabs were load-tested during the period from 1910 to 1920, with most performing well. Unfortunately, there remained a difficulty in closely correlating these test results with the actual moments present, as indicated by a straightforward application of static analysis. These early tests were not conducted to failure and, as a result, led to overly optimistic evaluations of the reserve strength, particularly in that they reflected less tension cracking than would develop at ultimate (Ref. 27).

In 1914, J. R. Nichols developed equations based on statics for the total positive and negative moments (Ref. 28); however, this solution did not provide any information on how these moments were distributed between positive and negative, nor did it indicate how either type of moment varied along the slab width. In 1921, H. M. Westergaard and W. A. Slater published their work on the analysis and design of slabs (Ref. 29). This paper established a theoretical slab analysis for the distribution of positive and negative moments that would exist throughout a flat slab. Their work, together with analyses of load tests, formed the basis for the subdivision of the total moment into positive and negative, and for futher subdivision of these moments applied to two design strips. This concept, obviously with some modifications, is similar to that used in current design practice. Extensive analytical work and testing at the University of Illinois (Refs. 30, 31 and 32) and by the Portland Cement Association (Ref. 33) resulted in the distribution of moments actually used in the 1971 Code (Refs. 34 and 35).

Current - The 1977 ACI Building Code (Ref. 36) specifies in detail two methods for determining the design moments for two-way (flat) slabs. These are the **direct design method** and the **equivalent frame method.**

The **direct design method** provides a procedure with which a set of moment coefficients can be determined based on the estimated flexural and torsional stiffness of the slabs and columns. This method has a number of limitations, including the following:

1. A minimum of three spans each way, directly supported on columns.
2. Rectangular panels with the long span not more than twice the short span.
3. Successive spans not differing by more than 1/3 of the longer span.
4. Live load not more than 3 times dead load.
5. Columns may not be offset by more than 10% of the span length in the direction of the offset from either axis between center lines of successive columns.

This is an approximate and relatively simple method. Since the great majority of flat slab designs fall within these limitations, this method is used wholly or partially for most designs.

The **equivalent frame method** is much more exacting and time consuming, and involves analyzing a portion of the structure, taken out by itself, much like a building frame, in order to determine the moments. This method is required when the limitations of the direct design method, as listed above, are not met, and for slabs with unsymmetrical dimensions and loading patterns.

Flat slabs, as is the case with any two-way slab, bend under load into dished shape surfaces so that there is bending in both principal directions. As a result, they must be reinforced in both directions by layers of steel reinforcing bars that are perpendicular to each other (Ref. 37). As indicated above, a theoretical elastic analysis for such slabs is a very complex problem because of their indeterminate nature. Techniques such as finite difference and finite elements are required, but such methods are not really practical for routine design.

As a result, the design of flat slabs is generally based on empirical moment coefficients, which, although they might not accurately predict stress variations, result in slabs with satisfactory overall safety factors. The fact that a great deal of stress redistribution can occur in such slabs at high loads makes designs based on theoretical analysis unnecessary. If as the result of using approximate design methods, too much steel is placed in one part of the slab and too little somewhere

else, the resulting slab behavior will probably still be satisfactory. The total amount of reinforcement in a slab seems more important than its exact placement.

It has been design practice for many years to use approximate analysis for flat slab design and to use average moments rather than maximum ones. Slabs designed in this manner have proved to be very satisfactory under service loads, and have proved to have appreciable overload capacity.

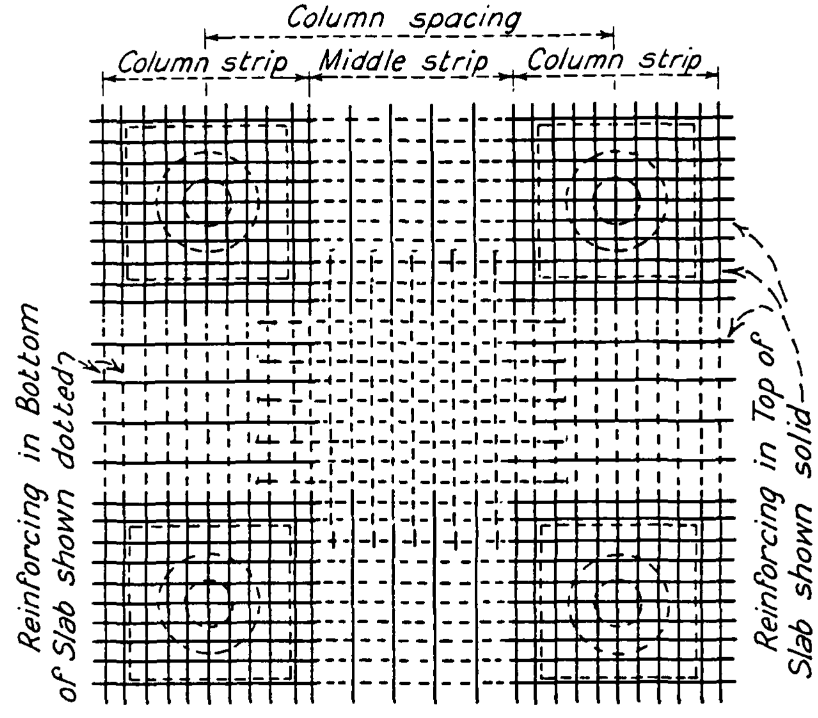
After the design moments have been determined by either method, they are distributed across each panel. The panels are divided into column and middle strips as shown on Figure 3-2 (Ref. 38), and the positive and negative moments are estimated in each strip. The column strip is considered as a slab with a width on each side of the column center line equal to 1/4th the panel dimension (1/4th the lesser dimension if rectangular). The middle strip is the part of the slab between the two column strips.

For the shear analysis, two types of shear are considered: beam shear and punching shear. For the beam shear, the analysis considers the slab to act as a wide beam running between supports, and for punching shear, the strength is as in a footing with the column punching through the slab and drop panel. In flat slabs shear may be the critical factor in design, as in many tests of such structures, failures have occurred as a result of shear, or a combination of shear and torsion.

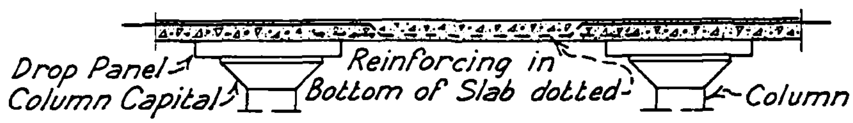
For most flat slabs the theoretical calculation of deflections is too complex for practical use, and as a result, the usual practice is to follow certain rules from the ACI Code (Ref. 36) for the determination of minimum slab thickness, thus negating the requirement for deflection calculations.

ADVANTAGES AND DISADVANTAGES OF A FLAT SLAB

For heavy live loads, over 100 psf, flat slabs have long been recognized as economical construction. However, as forming costs have increased, designers have been encouraged, where possible, to eliminate the projections at the slab soffit and the column tops; i.e., the drop panels and column capitals. This type of construction, called "flat plate", lends itself to the installation of partitions, and is now used more extensively than flat slab, particularly for light loading and short



(a) Arrangement of reinforcement



(b) Section through middle strip

Fig. 3-2. Two-Way Flat Slab Construction (Ref. 38).

spans. As a result, the majority of flab slab structures are more than fifteen years old, and are primarily found in warehouses, parking structures and industrial buildings, and similar structures where exposed drop panels or column capitals are acceptable.

Although the disadvantages of drop panels and column capitals are considerable in certain type structures, flat slabs have a distinct advantage in many types of structures. For heavier loads and longer span combinations, the flat slab with adjustment of drop panel depth will require less concrete and reinforcement, and can utilize smaller columns than the flat plate for the slight added cost of forming drop panels. For longer spans, the waffle flat slab will provide increased stiffness with less dead weight for overall economy.

REPRESENTATIVE USES OF THE FLAT SLAB

One of the principal reference handbooks used by engineers in the design of flat slabs and waffle flat slabs is the CRSI Handbook (Ref. 39). This handbook contains load tables and design information for many types of reinforced concrete one and two-way slabs, including flat and waffle slabs, and includes what CRSI considers to be the "practicable" range of load and span combinations for each type of slab. The ranges tabulated in this publication are wide, and although they certainly are "possible", one doubts that they are "practicable". For example, the tables for flat slabs span from 15 to 34 ft with loading from 100 to 700 psf; those for waffle flat slabs span up 54 ft with loads of from 50 to 500 psf. It would be very difficult, if not impossible, to find flat slab structures near either end of either of these ranges. Our experience in the selection of floor systems for structures strongly suggests that alternative systems, either of concrete, possibly precast or post-tensioned, or structural steel, would have distinct economic and structural advantages at the extreme loading ranges and spans listed. A more useful approach to a discussion of what would be considered representative construction for flat slabs might more accurately be based on the type of structures in which they are most frequently used.

As mentioned previously, flat slab construction is primarily used in structures where exposed drop panels and column capitals are acceptable, such as parking structures, industrial buildings and warehouses, and with respect to this investigation,

our interest lies only with the basements of such structures. Each building has occupancy characteristics, which, along with the loading requirements, usually dictate the bay sizes; i.e., the slab spans and column spacing. Some of the more representative types of buildings that utilize flat slab construction are described below.

Parking Structures - The layout for parking structures requires that the columns be located to accommodate traffic flow as well as the parking stalls. The basic dimensions for these areas are normally specified by local ordinances and codes, but do not vary widely. Parking stalls are generally 10 by 18 ft, and driveways vary from 12 ft wide for one-way traffic to 24 ft wide for two-way. The required loading for parking structures is relatively light, 50 psf. Therefore, using these parameters, in one direction the columns would be required to be at 10, 20, 30 or 40 ft spacing so as not to fall in a parking stall, and in the other direction, assuming two-way driveway traffic, no closer that 24 ft but no farther apart than 60 ft (24+18+18). Since in flat slab design, an attempt is made to keep the bays approximately square, and as in this case, with a light loading, it would appear that 30 by 30 ft. bays would be a good choice. A 60 ft span is too long from a structural standpoint, and 50 ft would not permit an economically advantageous symmetrical layout. Accordingly, a representative flat slab system for this example structure would be 12 in. columns spaced 30 ft in both directions, a 10 in. thick slab, and drop panels projecting 7 in. below the slab soffit.

In general, for underground parking structures with no other applied loads, such as 3 or 4 ft of earth or a building structure on the top level, a 24 to 34 ft span with slab thickness of 8 to 12 in. would be typical for flat slab construction. If waffle flat slab construction is used, the spans would typically be longer, up to 40 ft, with a total joist depth of 17 in. If, however, as is often the case in multi-story underground parking garages, the top floor is designed to carry heavy earth or building loads, these loads must be carried down through all levels to the foundation. This would result in reduced span and column spacing, probably to 20 ft. Of course, it would then be possible to reduce the thickness of all of the slabs below the top level by 2 or 3 in.

Warehouses - The bay spacing in warehouses is for the most part controlled by the specified loading, which may be 125 psf for light storage or 250 psf for heavy storage, and may also be subjected to additional special loading requirements for

forklifts and overhead crane rails. Flat slab and waffle flat slab construction lends itself quite well to these type structures since the column location is usually not one of the critical criteria. Typical bays in warehouses would be from 20 to 30 ft, and the slab thickness from 7 to 12 in. The columns are usually quite large, 20 to 24 in.

Industrial Buildings - The floors over the basements of industrial buildings generally have no fixed design criteria since they are specifically designed for a particular operation or type of machinery or equipment. They are, however, usually of quite heavy construction since it is normal practice, based on economics, to locate the largest and most massive machinery on this lower floor, thereby not requiring the upper levels to be of heavy construction. Typical bay sizes, slab thicknesses, and column sizes would be similar to warehouse construction.

Miscellaneous Buildings - In the north central and eastern part of the United States, particularly in the large industrial cities, flat slab construction in basements is more prevalent than in other areas. This construction is found in all types of older buildings, is generally very massive, and in most cases, overdesigned by today's codes. The bays are generally 20 ft or less, and the slabs 10 to 12 in. thick. Structures of this type in such locations as Chicago, New York, Detroit, and Cleveland include basements of large department stores, office buildings, theaters, and railroad and bus stations. Flat slab construction, when used in this manner, is no longer economical, and the majority of these structures were built prior to World War II.

This section has described typical flat slab design methods, advantages and disadvantages of flat slabs, and representative uses for flat slabs. A further discussion of flat slab construction, as well as other principal types, is found in Appendix B. In the next section, general resistance characteristics, failure modes, and upgrading of flat slab basements are discussed. Specific flat slab basement casualty function predictions are contained in Section 5 of the report.

Section 4

UPGRADING OF CASUALTY FUNCTION DEVELOPMENT PROCEDURE

INTRODUCTION

This section describes the current basis for the casualty function development procedure. In it is presented what is known about the resistance and failure characteristics of reinforced concrete flat slab structures. Methods by which these slab structures may be strengthened are also discussed. In addition, a discussion is included of the resistance and failure characteristics of basement walls as well as the current capability to predict their response. Other basement structural elements are discussed briefly. In addition, casualty mechanisms other than structural failure are described, and their contributions to the survivability of people in basements are put in perspective. Since the requirement is to analyze upgraded basements in addition to as-built basements, the basis for upgrading is also discussed. The information discussed in general terms in this section of the report will be applied to specific as-built and upgraded basements in Section 5 of the report.

RESISTANCE AND FAILURE CHARACTERISTICS OF FLAT SLABS

This section will treat slab strength characteristics according to: first, slab behavior, given that the supporting elements and slab boundary at these supports do not fail; second, support element and slab boundary behavior.

Slab Behavior Under Conditions of Adequate Boundary Support

Figure 4-1 shows a typical slab element between drop panels, where the element is bordered by the column lines. The sequence of behavior under increasing levels of vertical load is as given in this figure:

(a) A definite arch action of the uncracked slab against its support elements (drop panel edges)

(b) Development of enough flexural bending and twisting to a state of plate bending action

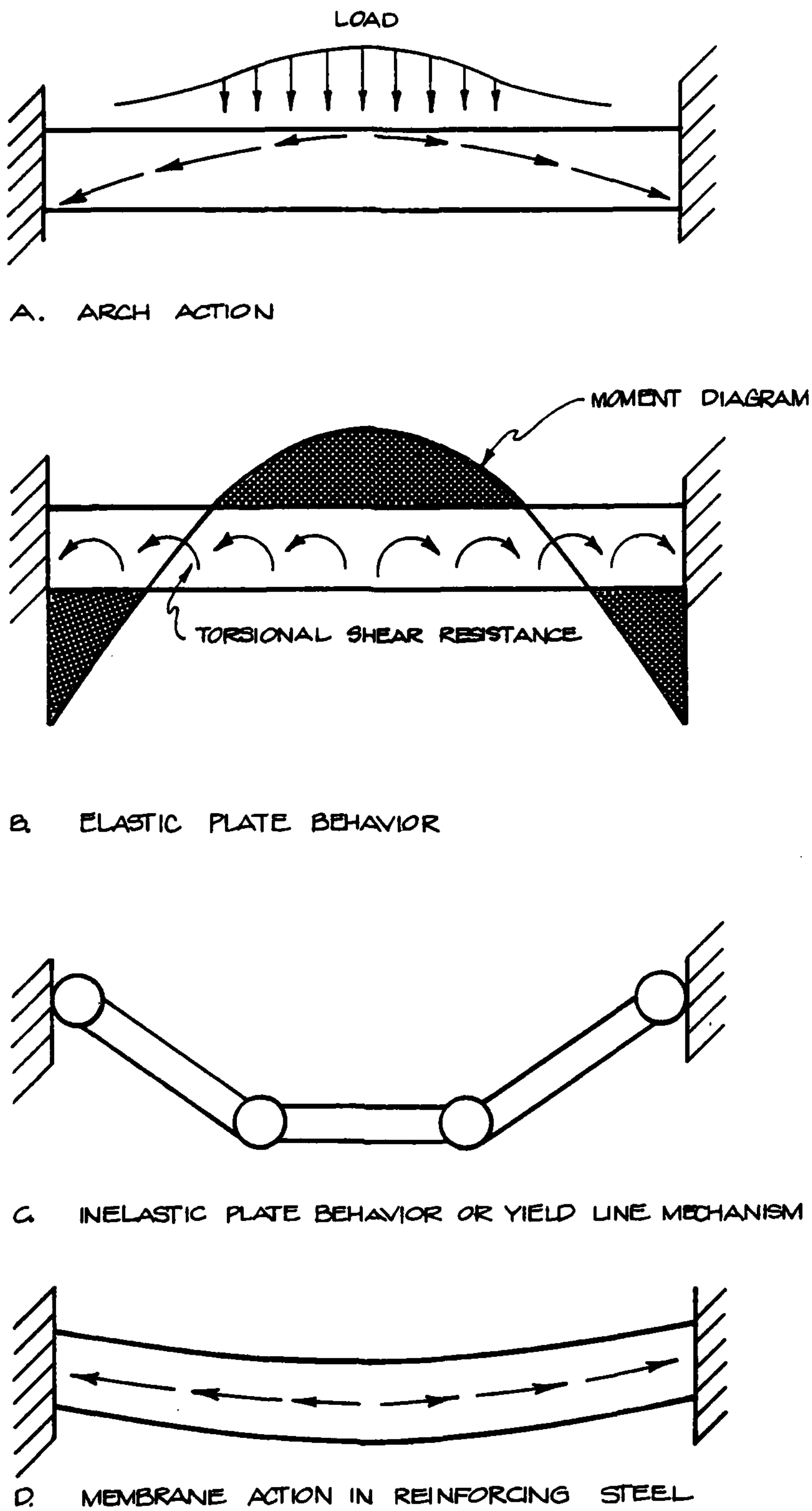


Fig. 4-1. Flat Slab Behavior Under Increasing Levels of Vertical Load.

(c) Cracking and yielding at maximum moment regions to form a yield line mechanism

(d) Further yielding and deformation such that the reinforcement develops a tensile membrane or net.

It has been reported in tests and in actual building demolition works that the slab, itself, can take almost any load and that failure occurs when the slab tears away from the support or the supports fail. Thus, aside from the difficulties required for repair of highly "dished" slabs, the main avenues of failure are concerned with the slab support boundaries (in shear), the support elements themselves, and interior column-drop panel shear.

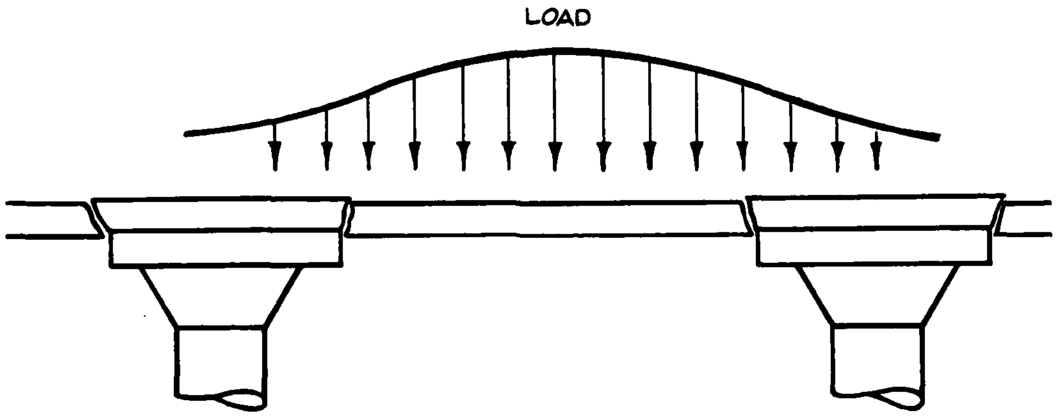
Supporting Elements and Slab Boundaries

Figures 4-2a and 4-2b show the failure modes for the flat slab system. Assuming that flexural yield line behavior is included in the slab element failure modes, then the critical failure mode is either a symmetrical punching shear around the drop panel boundary as shown in Figure 4-2a, or an eccentric or flexural punching shear due to non-uniform loading or non-symmetrical conditions. This eccentric mode may occur at an exterior span location or be due to a moment redistribution caused by support failure or punching failure at an adjacent span or by lateral loading of the entire frame. The flexural punching shear failure is shown in Figure 4-2b. This failure mode may occur whenever there is a moment transfer requirement at the column, and it therefore represents the primary weakness of the flat-slab system, see Figure 4-3.

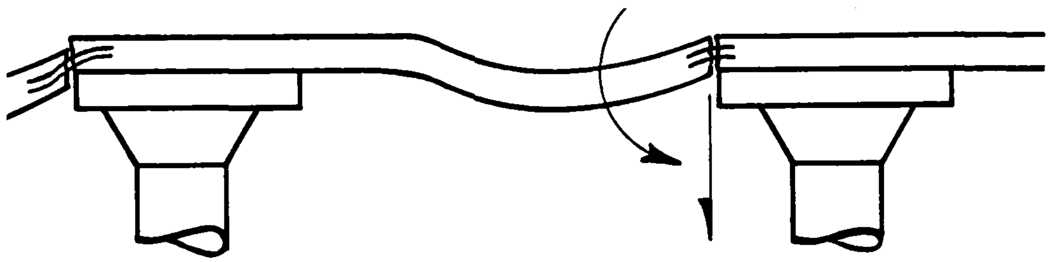
Flat-slab structures as designed by stated code procedures have a safety factor of about 1.7 to 2 against failure (primarily punching shear around drop panels) for symmetrical and uniform (all panels loaded) vertical loads, see Figure 4-4a. These structures, however, may have a much lower margin of safety if moment transfer is required at a column joint. Flexural punching shear can occur because of this moment transfer condition, at vertical load values significantly smaller than the failure loads with symmetrical conditions.

This flexural shear weakness affects the stability of the entire structure since the moment transfer condition can occur because of the following events:

o See Figure 4-4b: Non-symmetrical vertical load pattern or loss of flexural capacity at one local column joint. In these cases, the



a. Punching Shear Due to Uniform Load Condition.



b. Eccentric Moment and Shear.

Fig. 4-2. Failure Modes for the Flat Slab System.

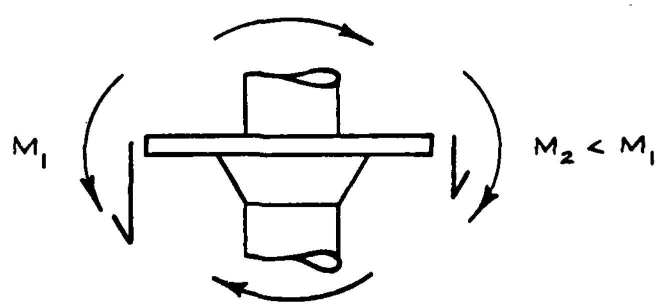


Fig. 4-3. Eccentric Moment Case.

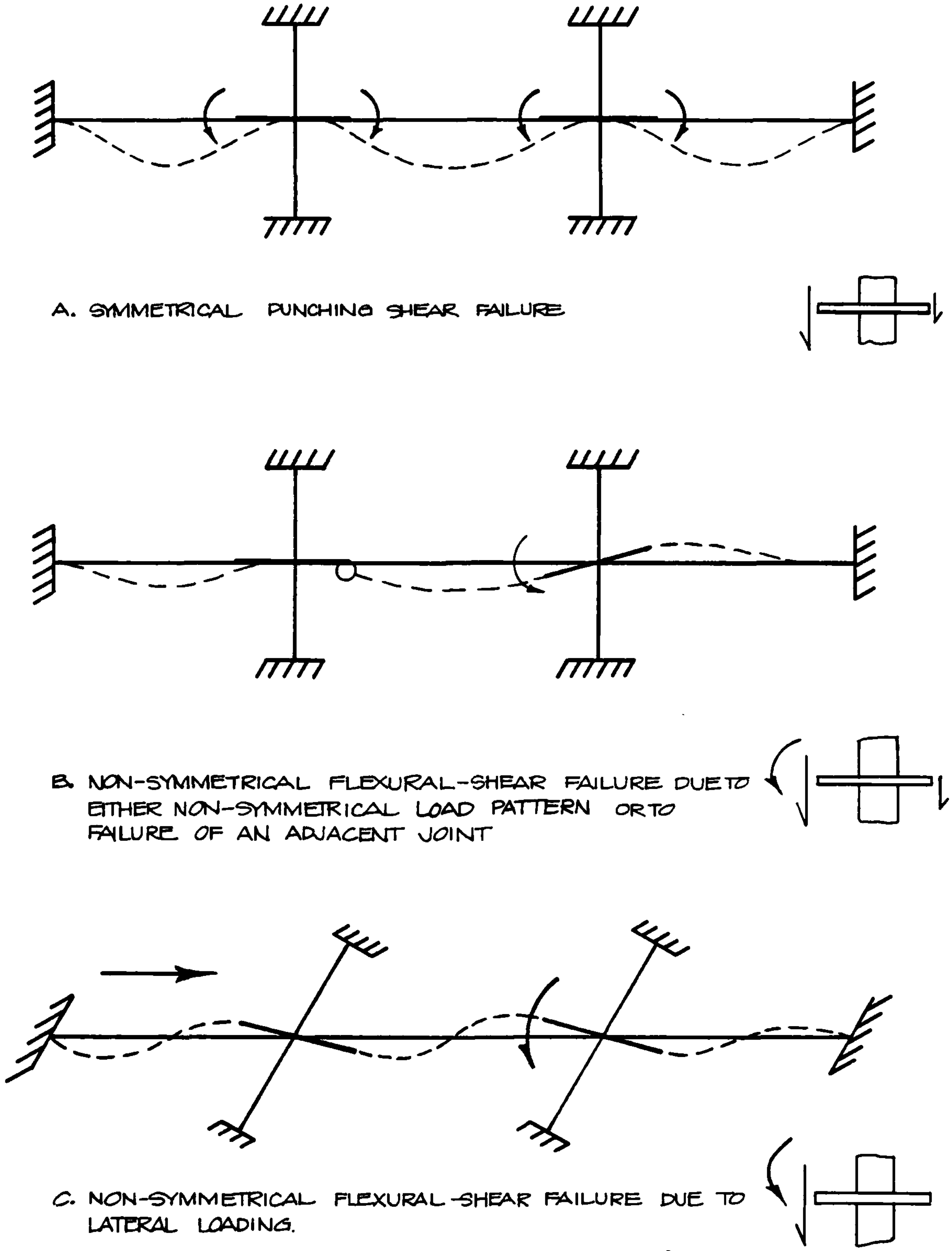


Fig. 4-4. Modes of Slab-Frame Failure.

redistributed moments create the moment transfer requirements at adjacent column joints and a progressive collapse condition may be propagated throughout the entire structural frame.

o See Figure 4-4c: Lateral loads or deformations create moment transfer requirements at all joints and progressive collapse can occur if one or more joints fail. Note that this lateral load condition would not occur for the basement shelter structure with perimeter wall support.

Strengthening Principles

From this discussion, it can be realized that the primary function of a strengthening scheme for a flat-slab system is to provide support at the slab drop panel and perimeter boundaries where potential flexural shear or slab tearout can occur. The scheme must also provide lateral support so as to protect the basic equivalent frame from moment transfer due to lateral deformation. As a first step, it will be assumed that the shelter structures (such as basements) are braced by perimeter walls against lateral deformation.

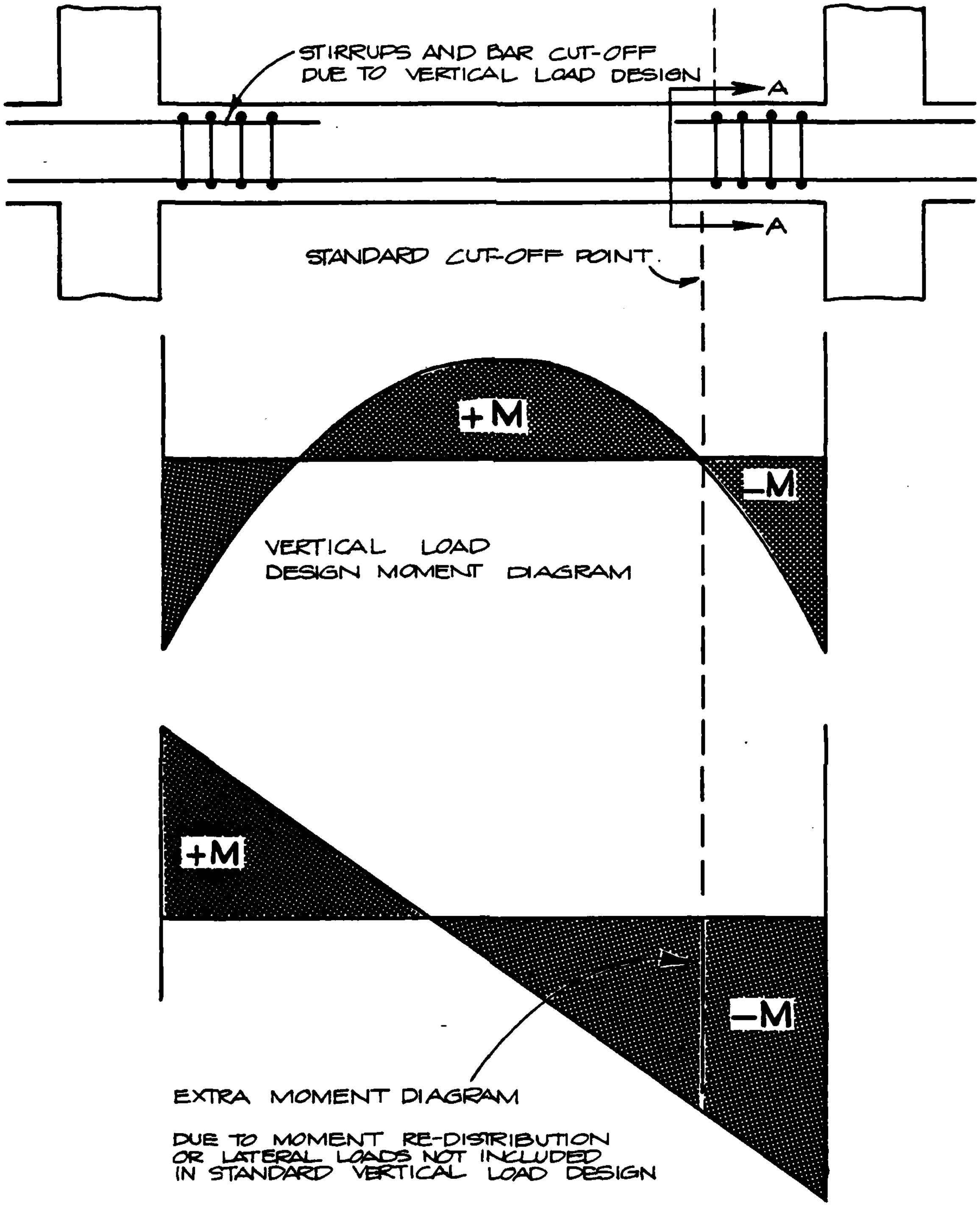
With respect to flat slabs, supported at their perimeter by frame beams, the failure mechanisms for these elements are: flexural-shear failure of the support beams, torsional shear failure of exterior spandrel support beams, or perhaps column failure due to combined axial load, shear, and flexure. Therefore, a primary function of a strengthening scheme for slab boundaries is to provide support at beam sections subject to shear failure and lateral bracing for control of lateral load deformations on the frame. These beam types of boundary support may be basement entrances or infilled (masonry) frame systems of perimeter walls.

The principal weakness of the beam-column support frame is caused by the "economical" code provisions for reinforcing steel. These provisions, such as the allowed cutoff of all negative moment steel at the quarter-span length, or the "non-requirement" of stirrups when calculated shear stress is below a given limit, are justified by the moment and shear diagrams for the design vertical load conditions on the all-elastic structure. No safety margin is available if these load diagrams change significantly because of moment redistribution caused by inelastic behavior or failure at one or more locations; and certainly no provision is made (except in seismic zones) for lateral load effects. These conditions (not included in the standard design) may cause radical changes in shear and moment requirements. Thus, in the

classical, very common economically designed support system, there are potential weaknesses due to lack of some continuous negative steel and stirrups at possible failure sections, such as section A-A in Figure 4-5. It is important to recognize that economics or the understandable quest for minimum cost and maximum profit can generate two sources of weakness in the slab capacity of existing structures. **First,** engineering design office economics dictate that standard (or empirical coefficient) design for vertical loads be used whenever possible. Therefore, there will be very few cases where any extra analysis (and reinforcement) will be added beyond "code" requirements. This means that resistance to non-calculated lateral loads or moment redistribution will not be present. **Second,** the economics of construction related to reinforcing steel fabrication (cutting and bending) dictate that the use of bent or truss bars be avoided whenever possible (see Figure 4-6). These bent bars are both hard to fabricate and difficult to place in comparison with multiple straight bars. Therefore, the lines of continuous reinforcement as furnished by the bent truss bars will become more rare as time goes on (and costs go up). This will very much affect (prohibit) the final development of slab membrane action (which is so well evident as a strength advantage in slab blast tests).

Strength Performance

In the literature, there are a reasonable number of reports on the strength performance of flat slab and flat plate systems (Refs. 27, 32, 40, 41). While this report is concerned only with flat slab systems, the flat plate behavior is useful to illustrate why flat slabs have superior resistance qualities in comparison with flat plates (see Figures 4-7 and 4-8). Specifically (Ref. 41) the flat plate system is weak in eccentric shear or punching shear at the columns. Any non-symmetry of loading or lateral loading can create substantial reduction in the vertical load capacity of the slab. The flat slab, however, with its enlarged column capitals and drop panels is able to resist punching failure by these strengthening elements. The slab portions of all systems (flat slab, flat plate, two-way slab) are exceptionally strong elements under vertical load. The compression arch action, plate flexure, and final tensile membrane action (see Figures 4-9, 4-10) can provide a total load-deformation behavior that is well adapted to blast load resistance. The short duration, peak shock pressure may overcome the compression arch and plate strength, but the very ductile yield line mechanism and tensile membrane remain to resist drag pressure and absorb its effects through large deformation. This successive progression of failure mechanisms produces a load deformation behavior that absorbs large amounts of blast force energy. Weakness can occur, however, at the edges and



Fi.g 4-5. Possible Failure Section for Flat Slab.

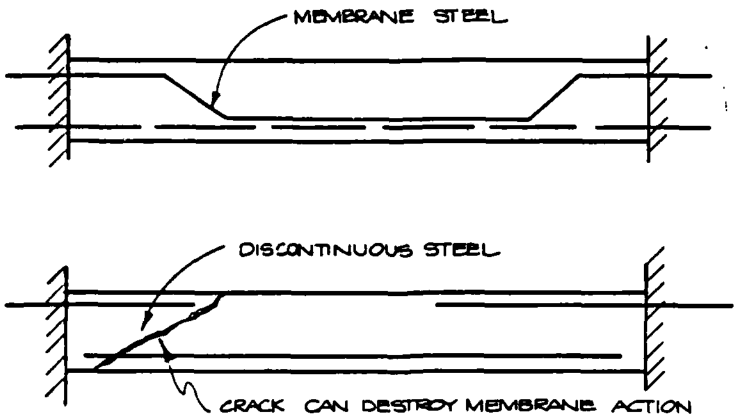


Fig. 4-6. Continuous vs Discontinuous Reinforcement.

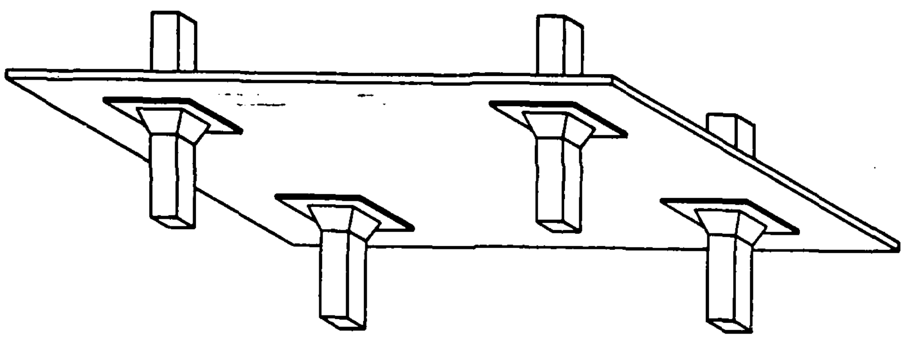


Fig. 4-7. The Flat Slab.

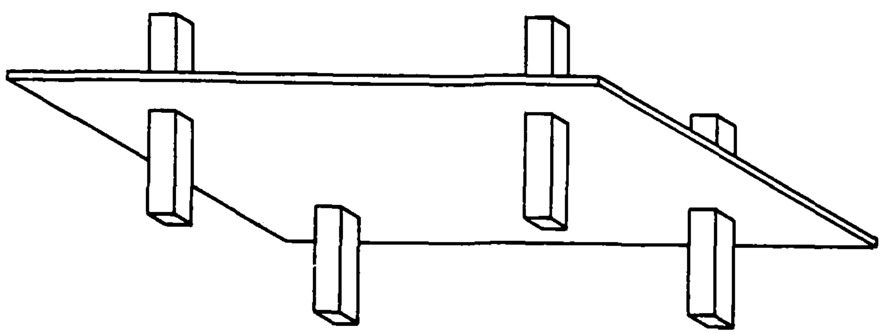


Fig. 4-8. The Flat Plate.

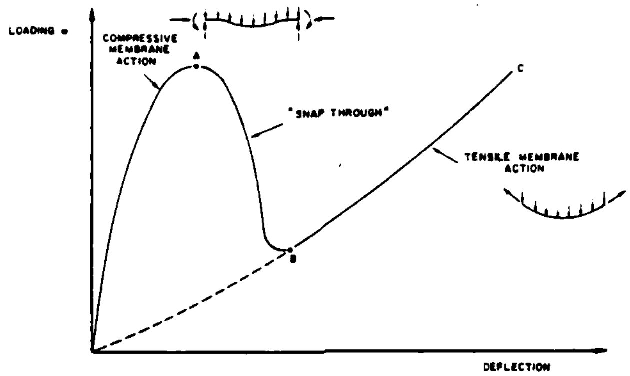


Fig. 4-9. Development of Tensile Membrane Action.

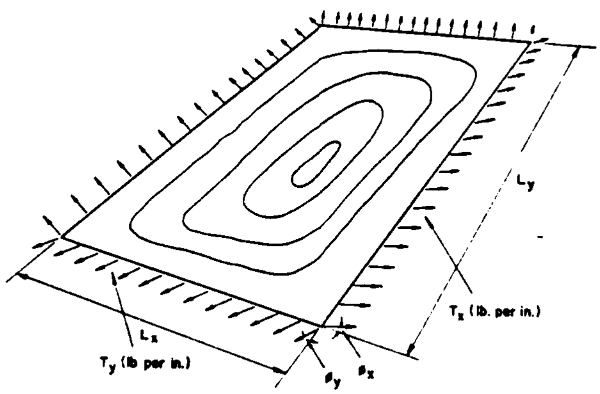


Fig. 4-10. Equilibrium of Rectangular Tensile Membrane.

at drop panel boundaries. Thus, while the actual slab has never shown total failure in tests, the edge and column boundary failure commonly occurs at two to three times design load. Also, if there are discontinuities in reinforcing steel due to straight bar cutoffs, then the membrane behavior may be weakened, and upgrading systems (to be discussed later) should reinforce or support the regions of rebar discontinuity.

Debris Behavior

Predictions of large chunks of debris resulting from shattered drop panels or column capitals, Figure 4-11 (Ref. 9), are not substantiated by tests. The drop panel and capital portions are actually the more resistive areas of the flat slab and severe cracking or shear rupture is most likely to occur in the slab outside of these thickened concrete areas. Ref. 9 also neglects to recognize that there is orthogonal reinforcing steel passing through the drop panel and slab (see Figures 4-12, 4-13 from Ref. 42) forming a grid or net that can support most of the cracked concrete. It is true that debris will form (see Figures 4-14 to 4-18 from Refs. 43 and 44), but this is most likely to be from compression spalling of the negative moment region of the slab at the junction with the drop panel. The debris chunks will be relatively small, but could have high velocity due to blast shock loads (see Ref. 44).

Column punching shear, or even capital or drop panel punching shear at the slab interface does not appear, in test behavior, to be a principal source of debris. The punching provides a rather clean failure crack perimeter on the bottom (shelter ceiling) side of the slab, see Figure 4-19, with a possible spalling at the top (shelter roof surface) side, see Figure 4-20, from Ref. 45. This top-side spalling would not endanger shelter occupants.

Summary of Non-Shored Flat Slab Strength Behavior

The various phases of the slab load deformation history: (1) compression arching, (2) elastic plate action, (3) flexural yield line formation, and finally (4) tensile membrane action provide a tough, energy-absorbing mechanism that is ideal for blast resistance. However, the standard reinforcing details and configurations at the interior column supports and the perimeter beam or wall supports at the slab boundaries are initial sources of failure: where these may cause non-symmetrical load conditions to propagate through the structure and create additional progressive failure. The initiating failures may be columns (including capitals and drop panels) punching through the slab, shear or torsion failure of slab perimeter support beams (spandrels), or the tearing away of slabs from perimeter walls due to reinforcing

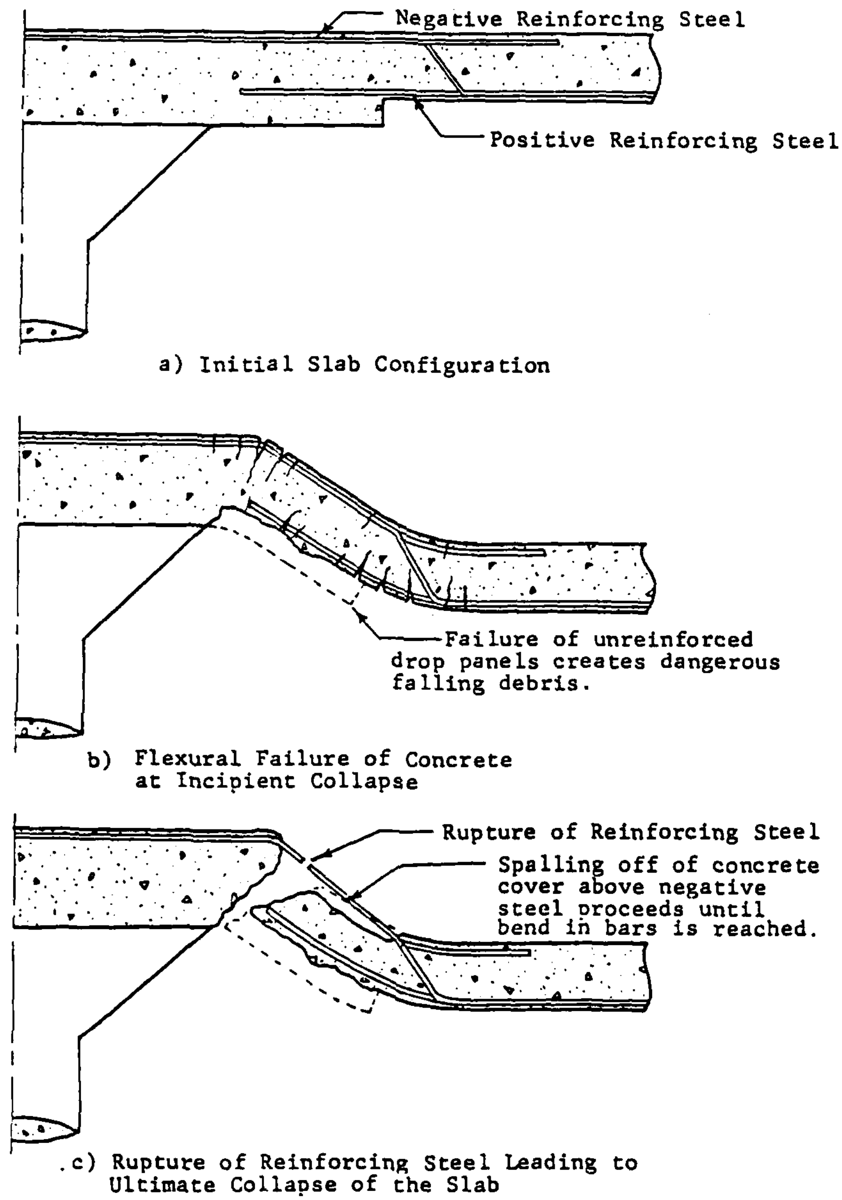


Fig. 4-11. Postulated Flexural Failure Mechanism. (From Ref. 9)

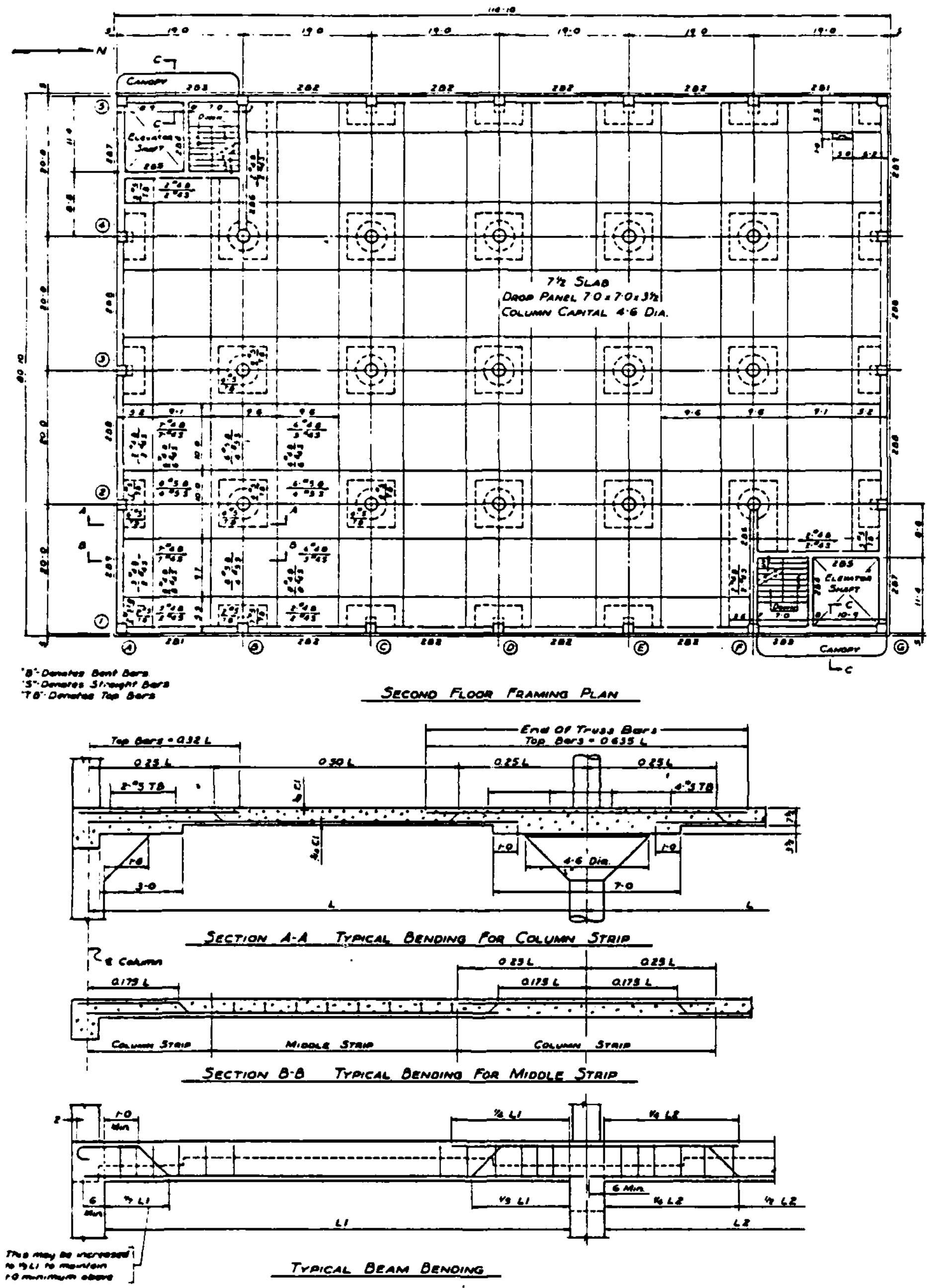
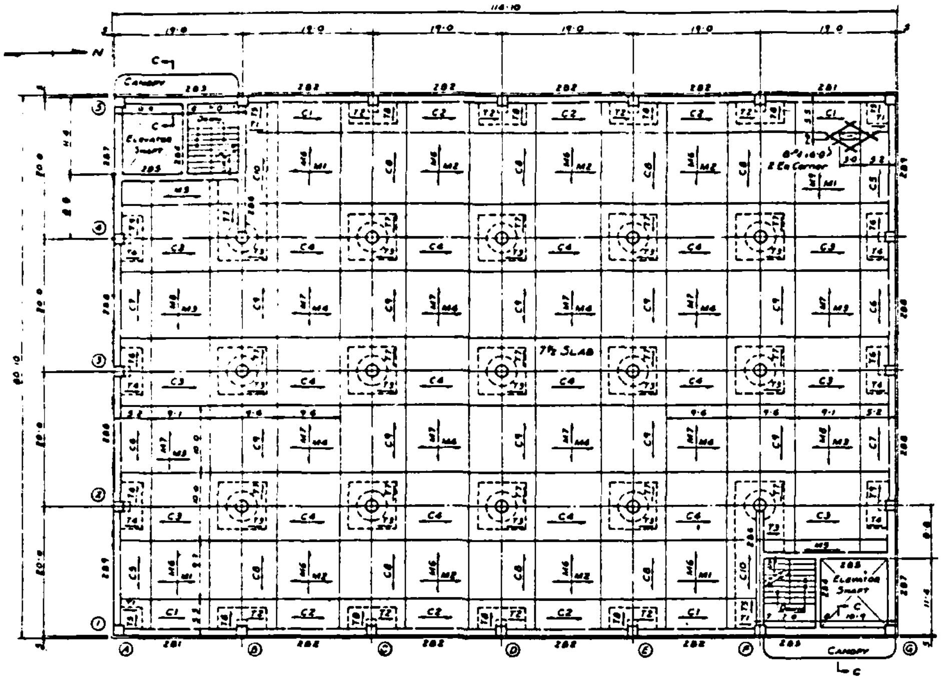


Fig. 4-12. Standard Slab Details. (Ref. 42)



SECOND FLOOR FRAMING PLAN

SECOND FLOOR BEAM SCHEDULE

No	Mark	Size W	Size D	Placing Notes	Reinforcing No	Size	Length	Mark	Sketch	Type	A	B	C	D	E	F	G	H	J	O	2" Chairs No	Type
2	281	12	20	Bot	2	#6	17-3														4	BB
				Bot	2	#7	25-1	700		3		2-8	1-10½	10-8	1-10½	8-0		1-4				
				Str 3@7 3@12 Ea End	12	#3	4-5	300		38	5	1-5	9	1-5			5					
6	282	12	20	Bot	1	#7	18-3														4	BB
				Bot	2	#7	30-8	701		3		9-2	1-10½	8-7	1-10½	9-2		1-4				
				Str 3@6 2@12 Ea End	10	#3	4-5	300		38	5	1-3	9	1-3			5					
2	283	12	24	Bot	2	#6	17-3														4	BB
				Bot	1	#7	25-4	702		3		7-8	2-4	10-4	2-4	2-8		1-8				
				Str 3@7 Ea End	6	#3	5-1	301		39	5	1-9	9	1-9			5					
2	284	6	14	Bot	2	#5	11-4														3	BB
				Bot	1	#5	13-10	500		3	7	1-10	1-2	6-8	1-2	1-10	7	10		12-0		
				Str 3@7 1@12 Ea End	8	#3	3-1	302		39	5	11	5	11			5					
2	285	12	20	Bot	2	#7	18-3														4	BB
				Bot	2	#8	22-5	800		3	1-1	2-1	[illegible]	12-4	[illegible]	2-1	1-1	1-4		19-2		
				Str 3@7 3@12 Ea End	12	#3	4-5	300		38	5	1-5	9	1-5			5					
2	286	14	30	Bot	3	#8	17-9														4	BB
				Bot	2	#10	25-4	1000		3	1-5	2-10	3-1	10-6	3-1	2-10	1-5	2-8		20-6		
				Str 3@6 2@6 2@12 Ea End	14	#3	6-3	303		38	5	2-3	11	2-3			5					
2	287	14	24	Bot	2	#5	18-3														4	BB
				Bot	3	#9	27-9	801		3	1-1	2-10	2-4	10-11	2-4	8-3		1-8				
				Str 3@6 3@12 Ea End	12	#3	5-3	304		39	5	1-9	11	1-9			5					
4	288	12	20	Bot	1	#7	19-0														4	BB
				Bot	2	#7	33-1	703		3		9-8	1-10½	9-8	1-10½	9-8		1-4				
				Str 3@6 2@12 Ea End	10	#3	4-5	300		38	5	1-5	9	1-5			5					
2	289	12	20	Bot	2	#6	18-3														4	BB
				Bot	2	#7	26-3	704		3		2-9	1-10½	12-2	1-10½	8-7		1-4				
				Str 3@7 3@12 Ea End	12	#3	4-5	300		38	5	1-3	9	1-3			5					

Fig. 4-13. Standard Slab Reinforcing. (Ref. 42)

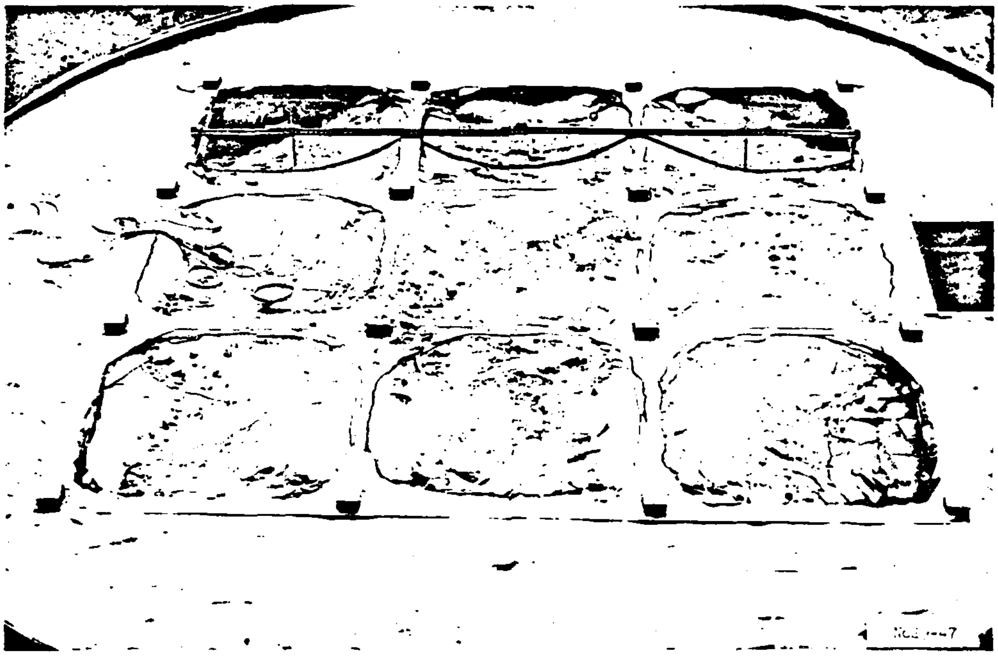


Fig. 4-14. Posttest View of Dynamic Model Prior to Removal of Debris. (Ref. 43)

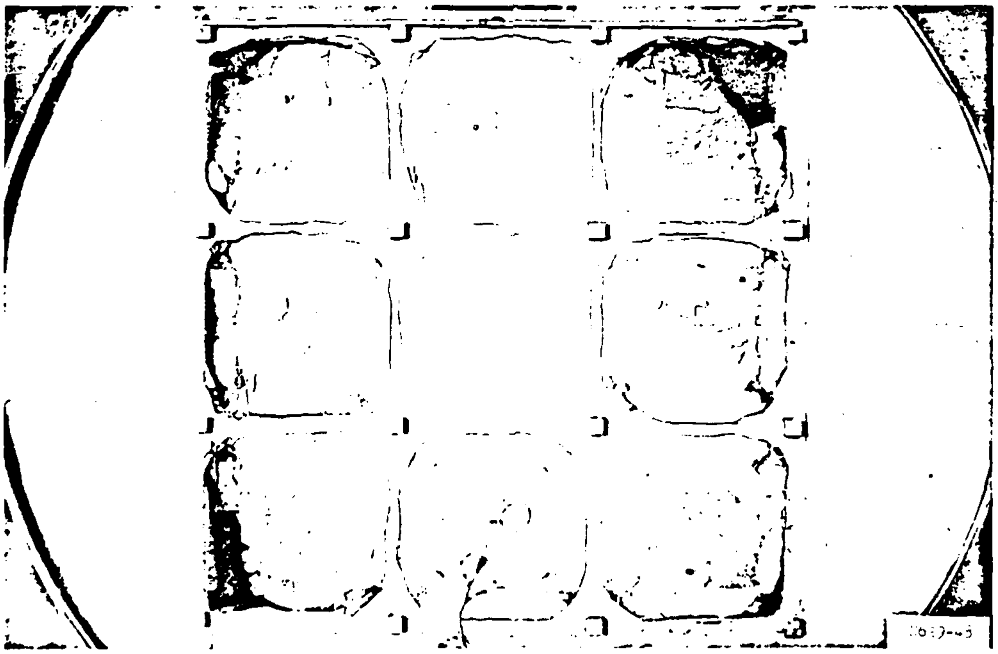


Fig. 4-15. Posttest Overhead View of Dynamic Model After Removal of Debris. (Ref. 43)

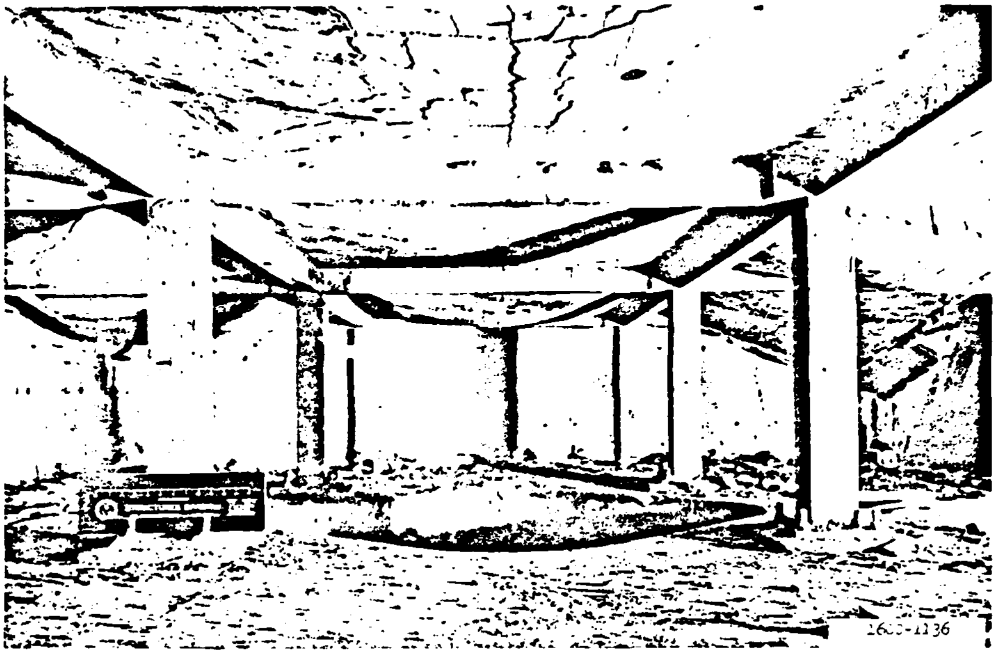


Fig. 4-16. Interior View of Dynamically Tested Model, Looking Through Entranceway. (Ref. 43)

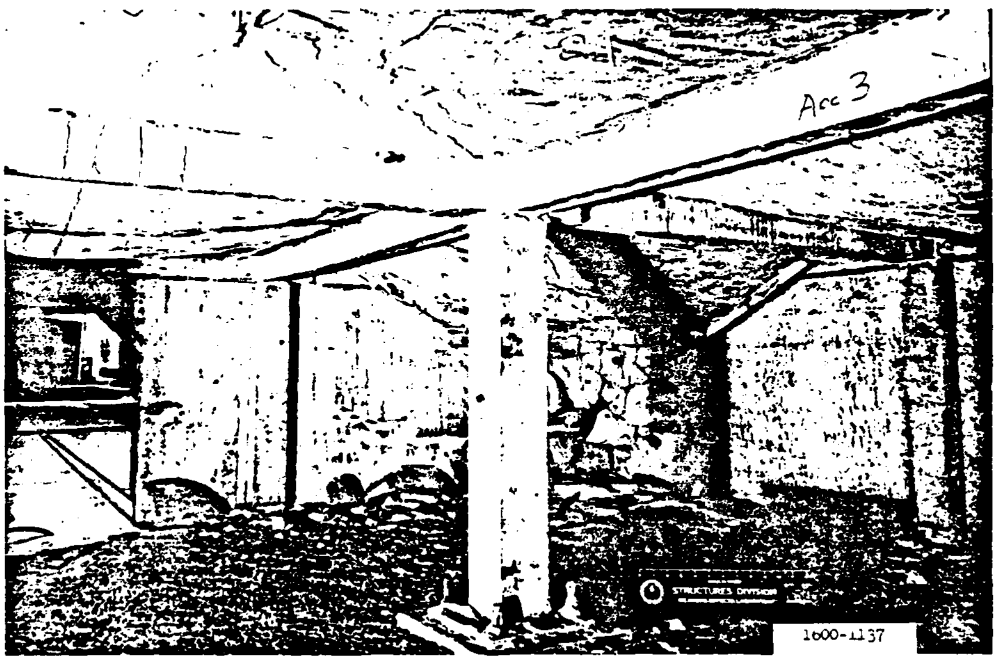


Fig. 4-17. Posttest View of Southwest Column and Framing Beams; Dynamic Model. (Ref. 43)

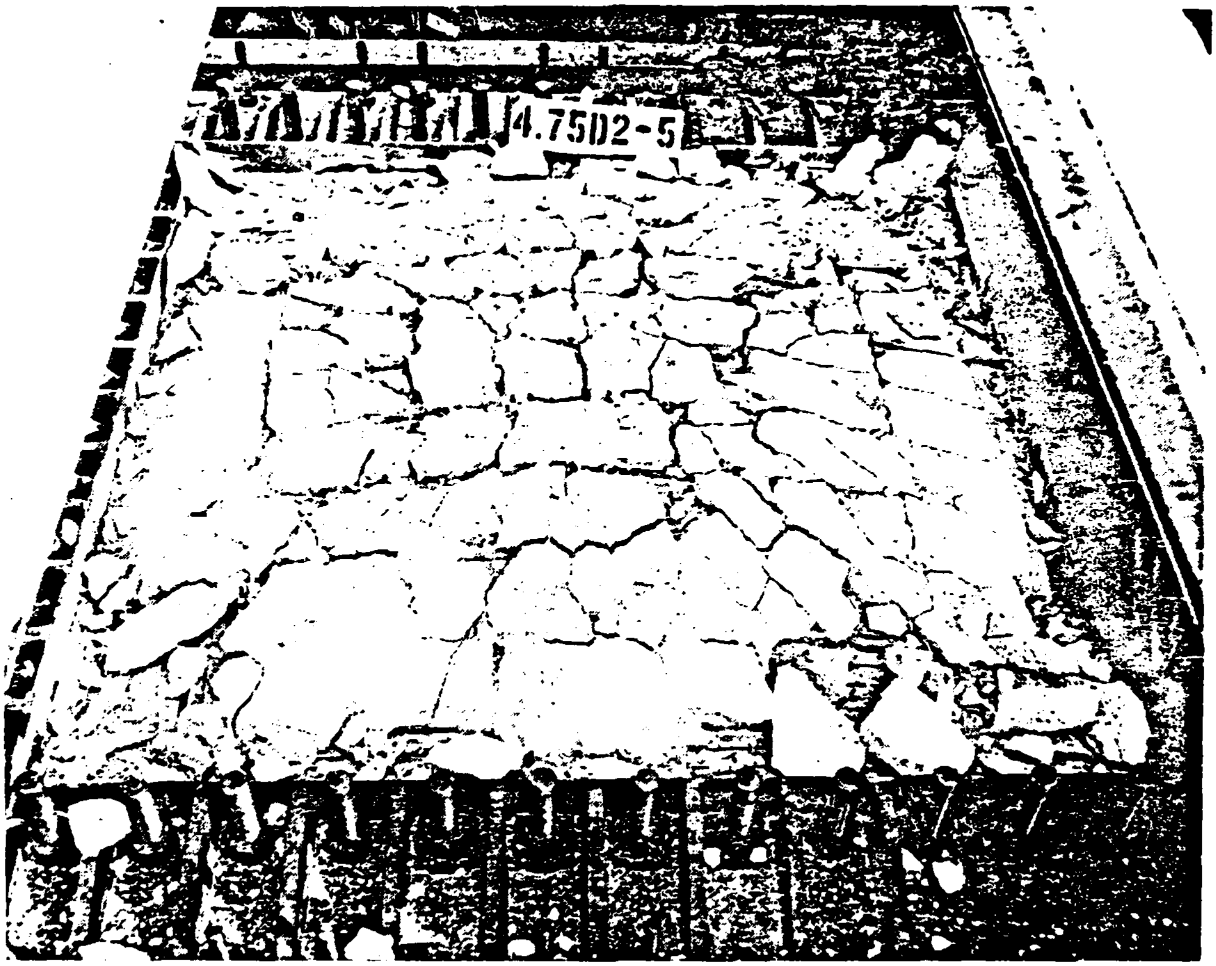
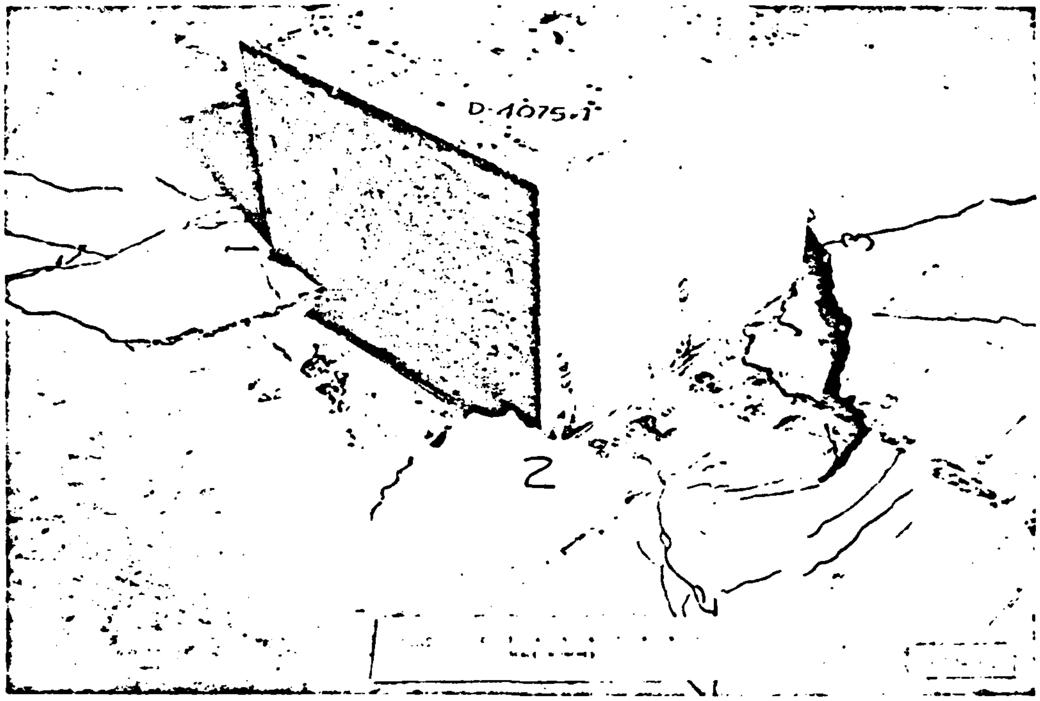
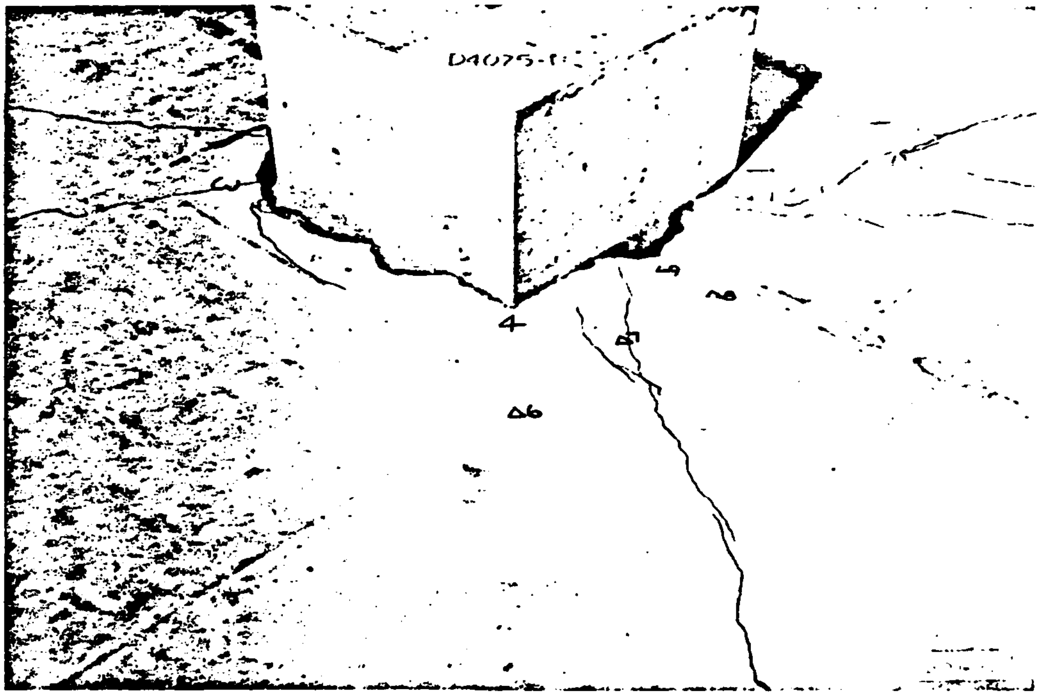


Fig. 4-18. Slab With Edges Fixed and Laterally Restrained; Dynamic Pressure Load of 101 psi. (Ref. 44)

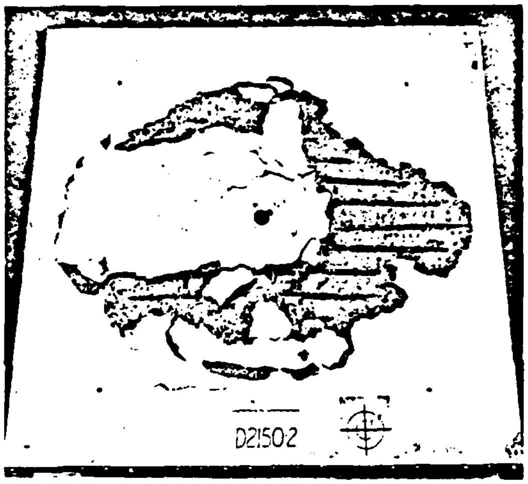


a. Column Side of Slab, Cracks Marked

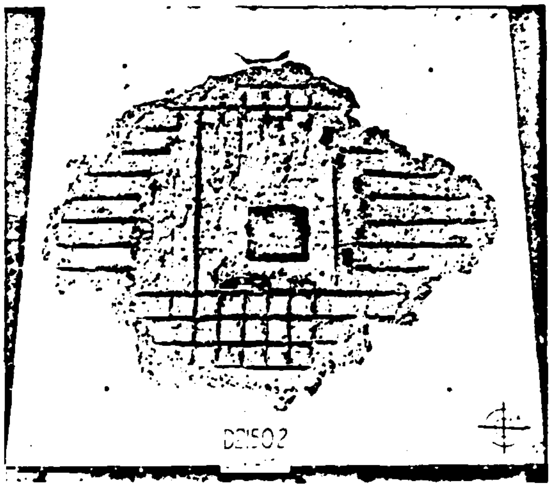


b. Column Side of Slab, Cracks Marked

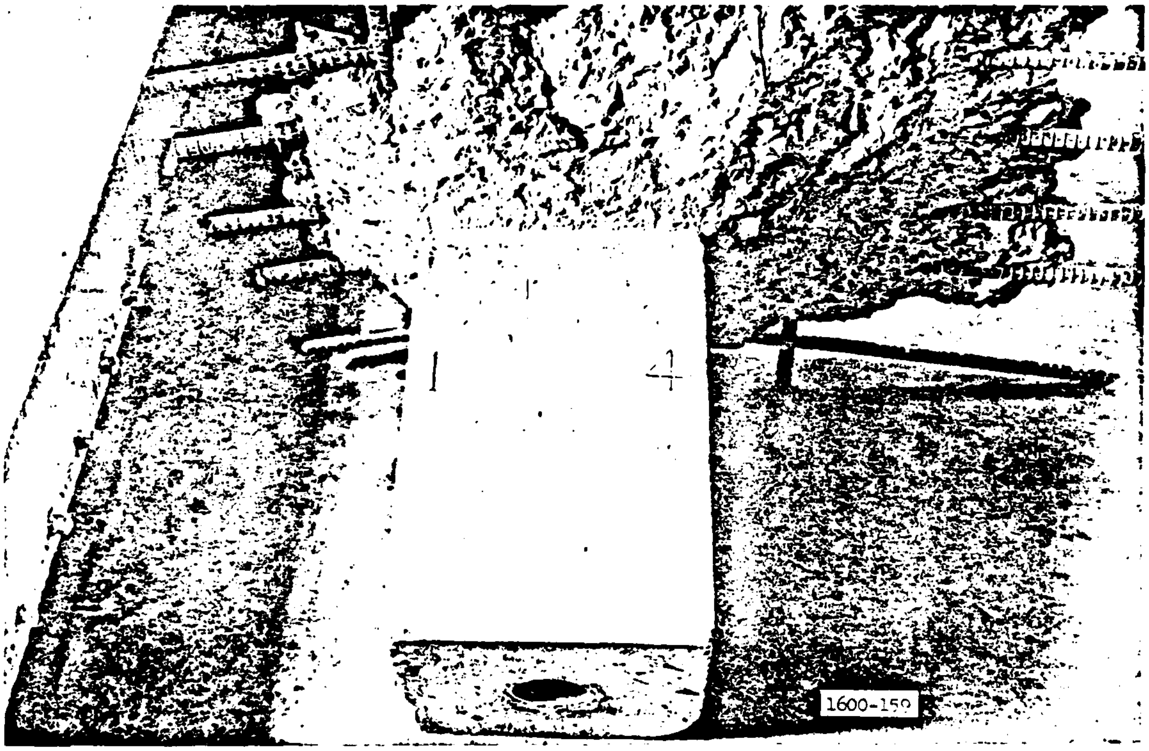
Fig. 4-19. Posttest Photographs of Specimen D4075-1. (Ref. 45)



a. Crack Pattern



b. Crack Pattern, Column Removed, Cracks Marked



c. Column and Failure Cone

Fig. 4-20. Posttest Photographs of Specimen D2150-2. (Ref. 45)

anchorage failure. A general rule from tests is that these local initiating failures can occur at about the code load-factored (or strength capacity) design load, or roughly at about twice the design working load (dead plus specified live load) capacity.

Strength Behavior of Shored Slabs

While the "early" failures of column punching shear and slab boundary member failure are the "Achilles' heel" of nonshored flat slabs, these weak points can be very effectively strengthened by an adequate system of shoring. Specifically, the drop panel areas around the columns are ideal strong points for shoring strut locations; similarly, the exterior boundary members can also be supported by shores to prevent slab tearout or spandrel beam shear and torsional failure, see Figure 4-21. These special reinforcing shores are, of course, in addition to the regular system of interior support shores that are required to upgrade the blast resistance of the slab itself. Note that the special shores do not encroach significantly into the shelter space. A detailed discussion of shoring or upgrading systems is given in the next portion of this section.

SLAB UPGRADING SYSTEMS

In addition to the required shores positioned around the columns at the drop panels, and at the exterior boundary members, as discussed above, shoring would be required at the interior of the slab in order to complete the upgrading of the entire floor system. A significant amount of work has been done by SSI in this area, and this portion of this section of the report will be specifically directed toward presenting these data as they apply to flat slabs.

Selection

The paramount objective in the selection of a shoring system is to obtain the maximum vertical load resistance possible within certain practical parameters. These parameters include taking into account the available material and labor resources, and determining the minimum shore spacing acceptable within the designated shelter area. Clearly, the maximum resistance would be obtained by completely filling the entire shelter area with shores, thus negating its intended use as a shelter. On the other hand, some minimal resistance would be obtained by locating a single shore at the slab mid-point, but as will be discussed later, for heavy

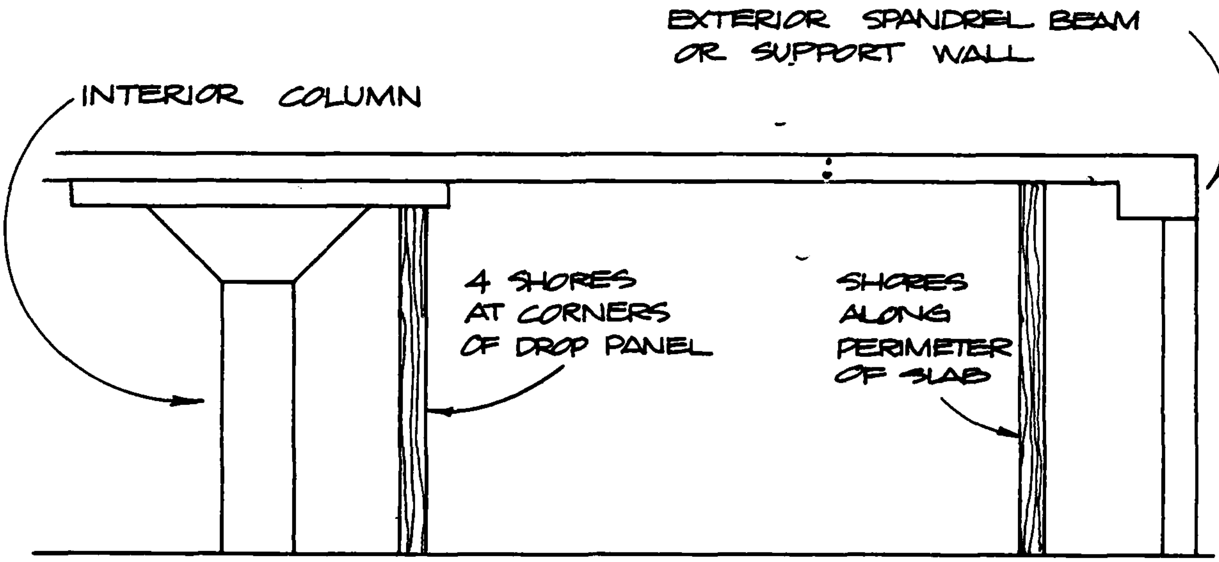


Fig. 4-21. Shoring System for Prevention of Localized Early Failure.

blast loading, this one shore configuration would be in an area of low negative moment resistance, and in addition, might be vulnerable to failure by punching shear. A reasonable approach to the selection of an upgrading system that would be germane to this investigation might best be based on the practical assumption that, within the above mentioned parameters, the candidate shelter area is to be upgraded to the maximum feasible level of blast resistance.

Since flat slab systems, as is true with any two-way slab system, require shoring in both directions, post shores are normally the most practical. Based on resource availability, these shores might be dimension timber, telephone or power poles, structural steel sections or tubes, or some type of commercial shoring. Manuals indicating the size and spacing of shoring have been developed by SSI for both host area (Ref. 21) and key worker (Ref. 20) shelter upgrading. At the present time, these two manuals are limited to the presentation of upgrading guidance for overpressures up to 2 psi (host area) and at 40 psi (key worker), but the general theory used in their development may be used to determine upgrading for other levels of resistance.

The evaluation process is based on an "intended use" code-based design criterion. With few exceptions, most buildings constructed during the past 50 years were designed using some type of building code, either national or local, and these codes specify the recommended minimum floor live loads as a function of the original intended use of the structure. A list of these minimum loads as they would appear in a building code is shown in Table 4-1. Once the original intended use of the structure and the type of structural system and materials are known, it is possible to establish the collapse loads of the system. In general, this is accomplished by using the design load and increasing it to the "as built" collapse load by factoring out the built in safety factors for the particular system that are required by the codes. As shores are added, theoretical calculations may be made, using the "as built" collapse load as a base, to determine the collapse loads under various shoring configurations and spacings. This prediction methodology has been validated in a considerable number of laboratory tests, particularly with concrete floors, as reported in (Refs. 46, 47, 48, and 49). Additionally, this methodology was used to determine the shore size and spacing in a flat slab subjected to a blast loading of 40 psi at the MILL RACE event in September, 1981 (Ref. 22), with satisfactory and predictable results.

TABLE 4-1

DESIGN INFORMATION: RECOMMENDED MINIMUM FLOOR LIVE LOADS

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Apartments (see Residential)	
Armories and drill rooms	150
Assembly halls and other places of assembly:	
Fixed seats	60
Movable seats	100
Platforms (assembly)	100
Bowling alleys, poolrooms, and similar recreational areas	75
Corridors:	
First floor	100
Other floors, same as occupancy served except as indicated	
Dance halls and ballrooms	100
Dining rooms and restaurants	100
Dwellings (see Residential)	
Garages (passenger cars only)	50
Grandstands (see Reviewing stands)	
Gymnasiums, main floors and balconies	100
Hospitals:	
Operating rooms, laboratories	60
Private rooms	40
Wards	40
Corridors, above first floor	80
Hotels (see Residential)	
Libraries:	
Reading rooms	60
Stack rooms (books & shelving at 65 pcf)	
but not less than	150
Corridors, above first floor	80
Manufacturing:	
Light	125
Heavy	250

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Office buildings:	
Offices	50
Lobbies	100
Corridors, above first floor	80
File and computer rooms require heavier loads based upon anticipated occupancy	
Penal institutions:	
Cell blocks	40
Corridors	100
Residential:	
Multifamily houses:	
Private apartments	40
Public rooms	100
Corridors	80
Dwellings:	
First floor	40
Second floor and habitable attics	30
Uninhabitable attics	20
Hotels:	
Guest rooms	40
Public rooms	100
Corridors serving public rooms	100
Corridors	80
Reviewing stands and bleachers	100
Schools:	
Classrooms	40
Corridors	80
Sidewalks, vehicular driveways, and yards, subject to trucking	250
Skating rinks	100
Stairs and exitways	100
Storage warehouse:	
Light	125
Heavy	250

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Stores:	
Retail:	
First floor, rooms	100
Upper floors	75
Wholesale	125
Theaters:	
Aisles, corridors, and lobbies	100
Orchestra floors	60
Balconies	60
Stage floors	150
Yards and terraces, pedestrians	100

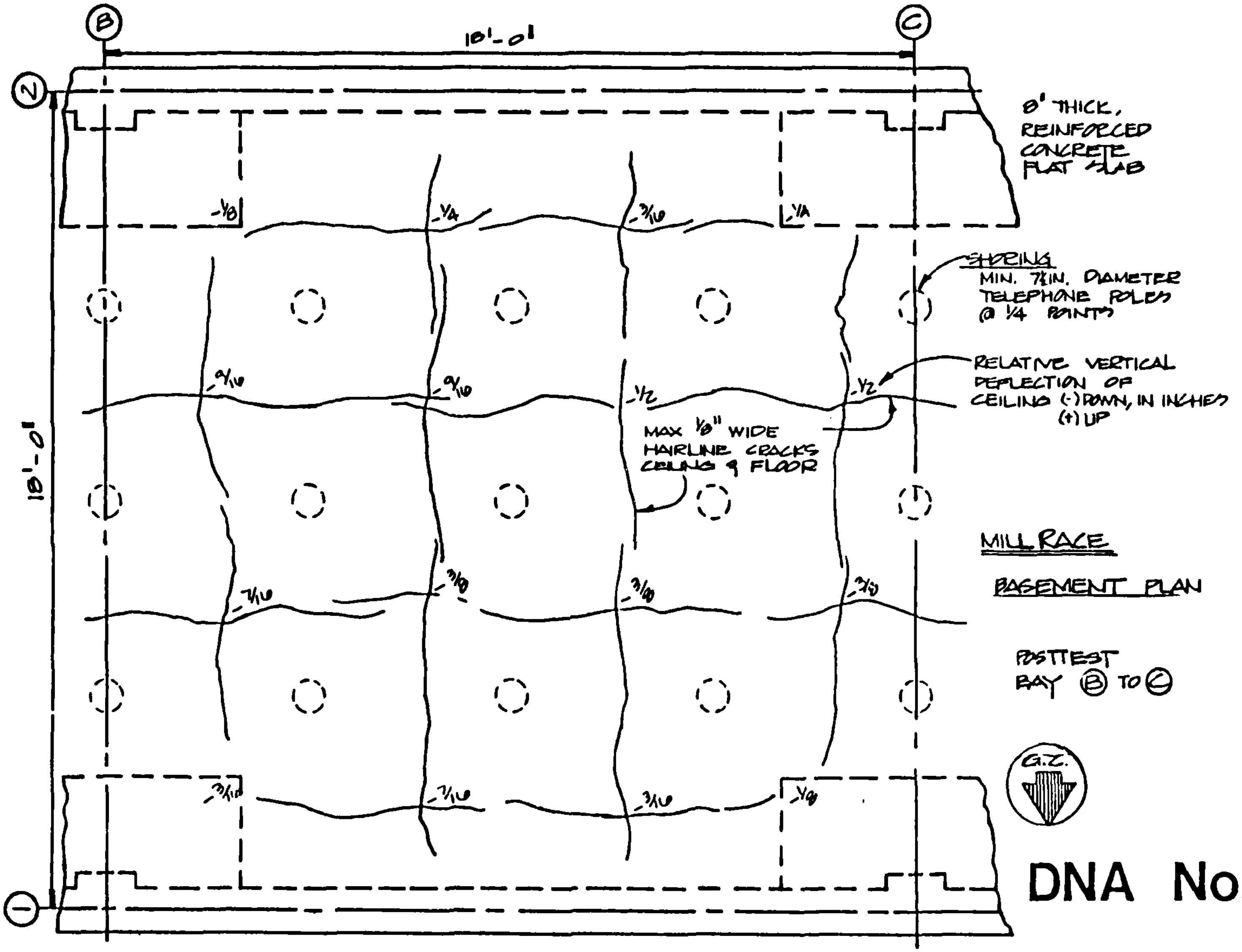
Performance Evaluation

Along with the consideration of upgrading for moment failure in the interior of flat slabs, another area of critical concern is the punching strength of these slabs, as well as the basement slabs on grade. This area was addressed in Ref. 49 in some detail, and although it was primarily directed at analyzing the enhanced resistance of constrained slabs to the punching of shores, it is believed that a short summary of that investigation is appropriate in this discussion, particularly as it applies to the arching capability of these slabs when shored.

The main purpose of this investigation was to develop an analytical approach to the punching of reinforced concrete slabs by timber or steel shoring. This effort was prompted by the results of both field and laboratory test programs that have indicated an increased punching capacity in continuous concrete slab systems, well beyond the established code values. As an example, the flat slab tested at MILL RACE was subjected to an overpressure of 40 psi (Ref. 22), which was sufficient in intensity to develop flexural cracking, but did not evidence any indication of punching shear cracking at any of the shore locations, as shown in Figure 4-22. Additionally, previously conducted laboratory tests (Ref. 48) included a series of isolated (non-continuous) slab elements containing either top or bottom reinforcing steel, loaded to failure with various types of shores, and all failing at loads 50% to 70% greater than predicted by existing building code procedures.

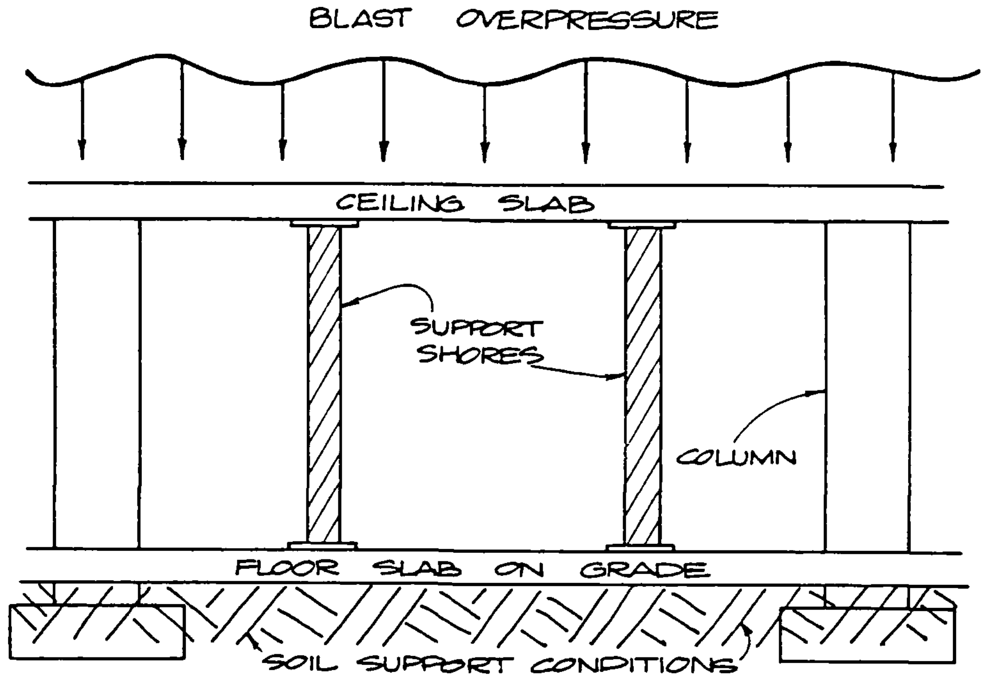
The investigation included shoring support configurations as shown in Figures 4-23a and 4-23b, and considered the more critical modes of failure such as shore compression failure, flexural yield collapse, slab-to-frame connection failure, as well as slab punching. Using an extensive literature review as a base, this analysis presented an evaluation of this enhanced punching shear resistance based on the theory of arching action. This arching action was analyzed in detail in the normal-to-plane blast loading of constrained wall panels (Ref. 50) and was directly related to floor slabs. See Figures 4-24, 4-25, and 4-26.

With respect to the evaluation of punching shear capacity, Figure 4-27 shows a typical shoring layout in a slab, and section A-A shows a shore in the negative moment region where top reinforcing steel is present. At this location it could be possible that compressive arch action may be weakened by the frame and/or beam movement, or by non-symmetrical loading, however, the top steel assists in providing good punching shear resistance even where arch action is small. Section B-B shows

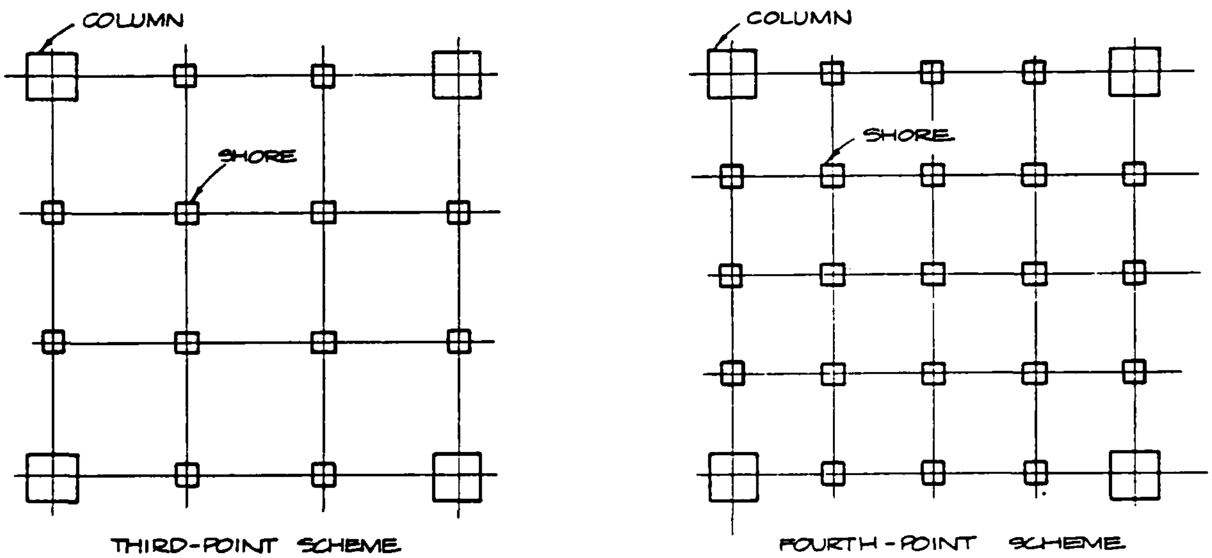


DNA No. 5201

Fig. 4-22 MILL RACE Slab Cracking Pattern.



a. Shored Slab System.



b. Third- and Fourth-point Shoring.

Fig. 4-23. Shoring Support Configurations.

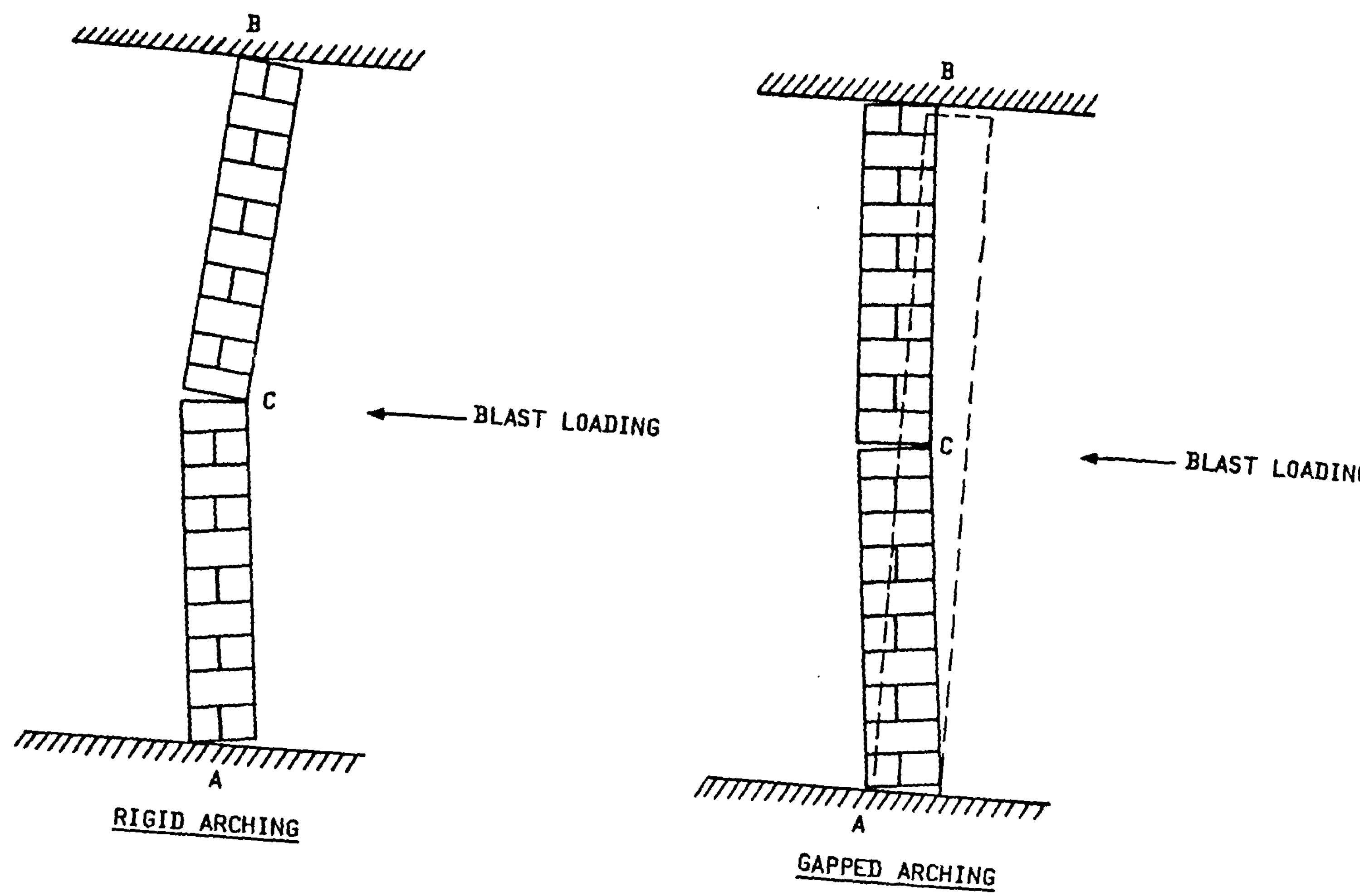


Fig. 4-24. Arching Mechanisms in Walls.

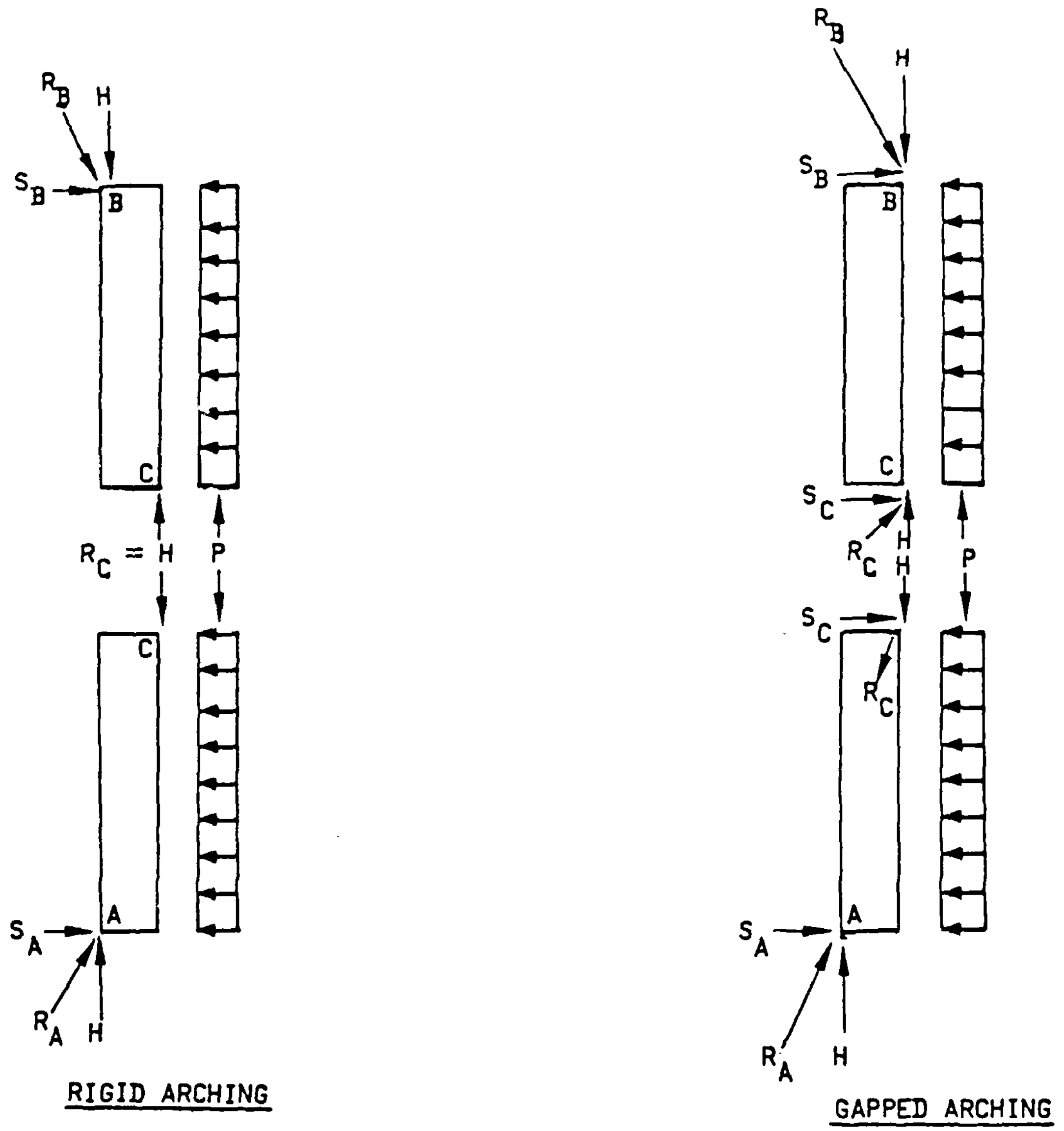


Fig. 4-25. In-Plane Compression Conditions.

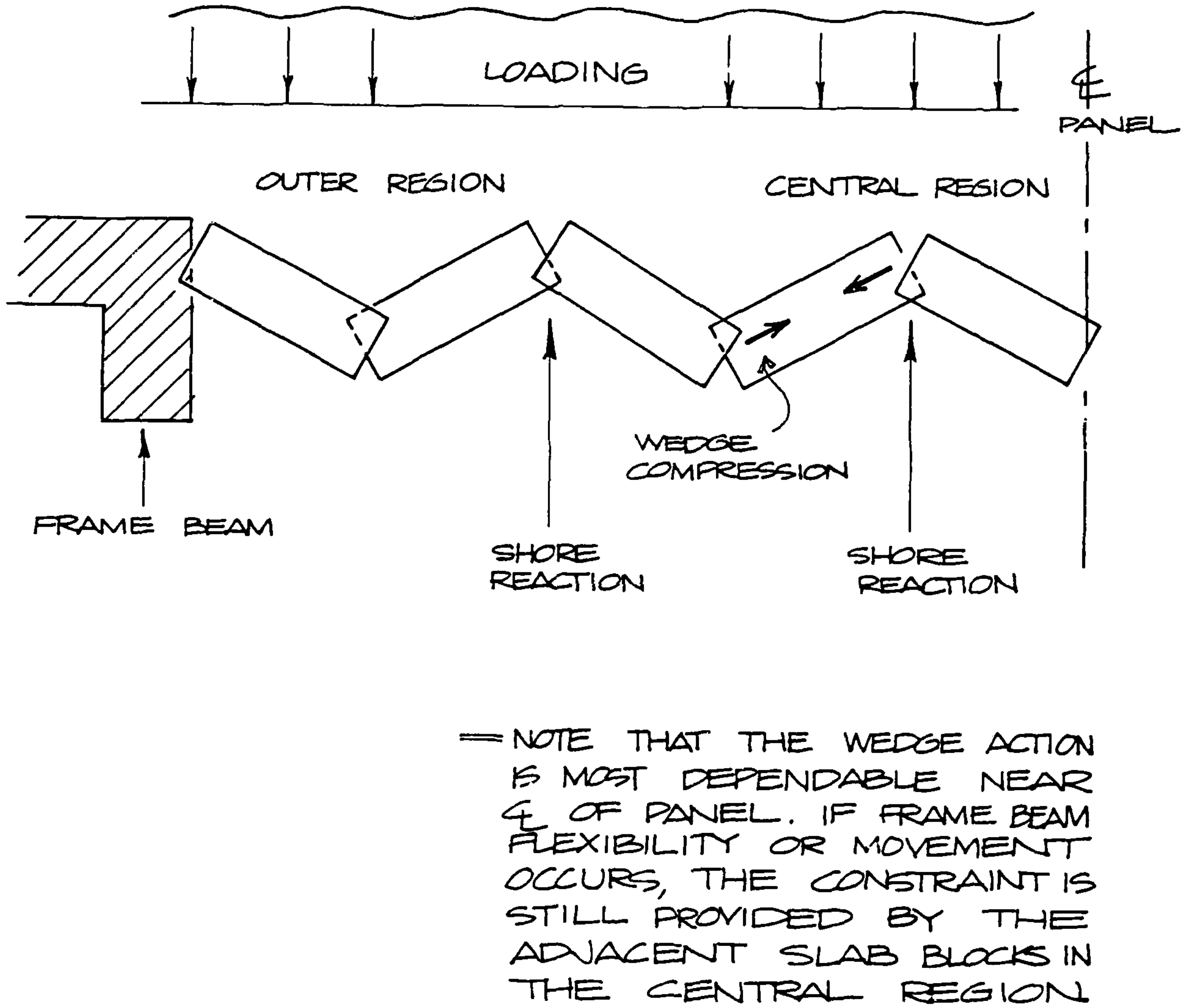


Fig.4-26. Arching or Wedge Mechanism in Shored Slabs.

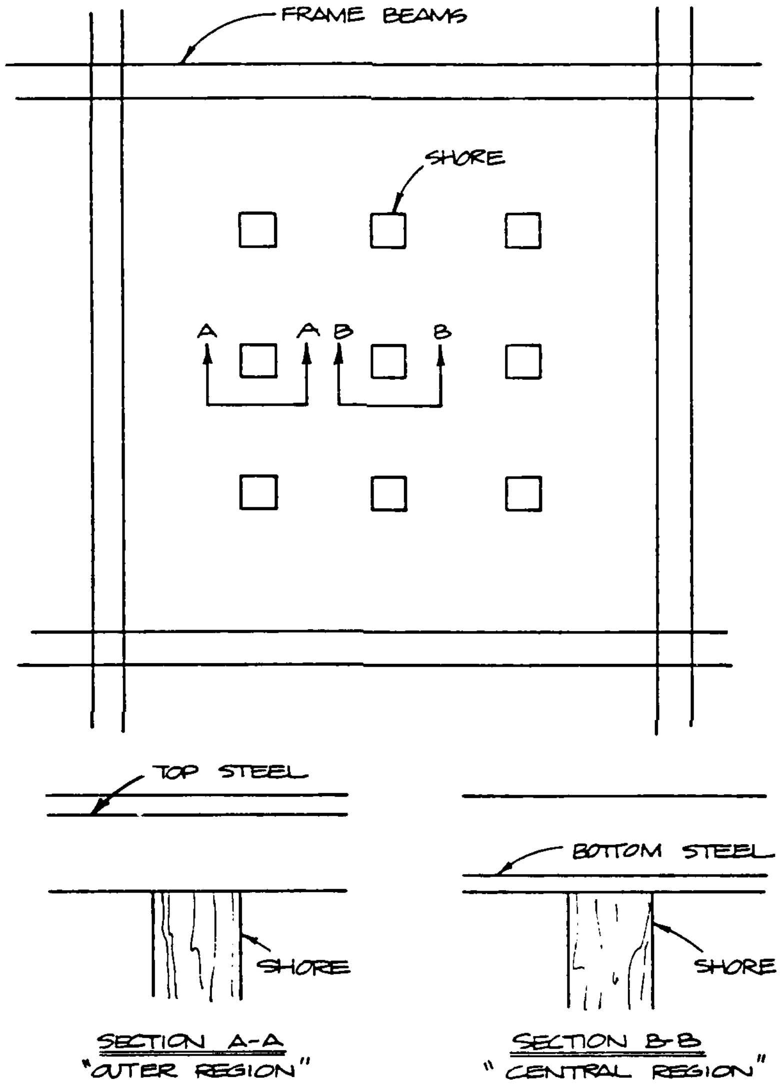


Fig. 4-27. Shored Slab Reinforcing Steel Conditions.

a central location where top steel may not be present, but since the central portion of the slab is strongly confined, there exists a dependable amount of arch action, and the punching strength is enhanced.

While the arching theories presented in this investigation have been primarily concerned with structural floor slabs, such as the first floor above a basement, they are also equally applicable to the punching resistance of floor slabs on grade.

As a result of this investigation, as well as data developed from past analyses and testing, it was concluded that in most cases punching shear would not be the critical mode of failure. It is more probable that failure would occur as a result of connection fracture, shore failure, or slab flexure — all areas currently, or planned to be, under investigation. The relationships developed as a result of this investigation, when presented in a usable format, will be included in the revised and updated shelter upgrading manuals. They will also be utilized in the determination of damage functions.

WALL RESPONSE

General

A portion of this year's effort has been expended on developing a simplified capability to predict the response of basement walls. This is based on the premise that basement walls may be the "weak link" structurally in some basement shelters. In previous shelter survivability studies, the question of the response of basement walls has customarily been dealt with indirectly. That is, the assumption has usually been made that it was not necessary to predict the response of basement walls because they were not the weakest element. However, the stated assumptions have usually been that the floor slab for the floor above the basement is the weakest structural element, and that suitable closures are installed for openings into the basement. This indirectly eliminates from consideration not only walls but all other structural elements except the overhead slab. See, for example, Refs. 13, 14, 15, and 17, as well as other portions of the present report, which contain statements of this kind. The analyses have then gone on to predict the failure mechanisms and overpressures for the slabs.

Basement Wall Blast Tests

Unfortunately, there is only limited experimental information on the response of basement walls to airblast overpressures. Blast shelters, including fully buried, basement, and aboveground types, have been tested during atmospheric nuclear and high explosive tests (Refs. 51 and 22). However, for the most part, these structures have not resembled the types of as-built or upgraded basements and blast environments important to the present project. In most cases, the walls have survived undamaged, so only limited information on basement wall failure mechanisms has been provided by these tests.

MILL RACE Wall Experience

The basement walls tested in the key worker shelter portion of the September 1981 MILL RACE test (Ref. 22) were of applicable types. However, the shelter consisted of only an earth-covered basement without any building above it, and thus was not fully representative of the type of structures being considered here. That is, it could model neither the response due to the frame of the building transmitting loads to the basement structure, nor any effect that aboveground walls might have on the basement walls. At MILL RACE, test walls were constructed using three types of wall construction: unreinforced hollow unit masonry, reinforced hollow unit masonry, and precast concrete wall panels. Two wall sections of each type were tested, with one being upgraded by the application of 2-in. thick sheets of expanded polystyrene to the exterior, and the other remaining as-built. The walls were backfilled to grade with granular material and the entire structure was covered with loose native soils (See Figure C-1, Appendix C). The peak air blast overpressure at the key worker shelter was approximately 40 psi. A soil pressure gauge, located in the backfill at the mid-point of one of the unreinforced masonry test walls and approximately 12 inches away from it, recorded a peak pressure of about 9 psi. Only the two precast concrete walls exhibited any damage as a result of the test. They each had a single crack, approximately 1/8 inch wide for their full width, at about mid-height.

Follow-on Wall Studies

The survival without damage of the masonry walls, especially the unreinforced masonry walls that were expected to fail, is indicative that the walls did not receive any significant lateral loading. This unexpected result is indicative that further study is required of the soil/structure interaction for basement walls under blast loading. A limited program using a 12-in. shock tube and 1/20th scale models of the

MILL RACE basement walls has been undertaken at SSI. This is expected to lead to better understanding of the MILL RACE wall test results (See Appendix C.)

Current Wall Failure Predictive Capabilities

The capability to predict the static loads on basement walls is well developed. There is also a large amount of experimental information on the failure of above grade walls due to air blast (Ref. 52). However, based on the MILL RACE experience, and with the follow-on shock tube experiments still in progress, the present capability to predict the response of basement walls to air blast effects of nuclear weapons is less than satisfactory. Thus, without an adequate capability using sophisticated analytical techniques to predict the response of basement walls, it becomes very difficult to construct a valid simplified capability.

Interim Predictive Techniques for Walls

This does not mean that there is nothing that can be done about predicting failure levels of basement walls. If one assumes that the walls in a prospective basement shelter can be categorized as to type, and that the characteristics of the soil or other material surrounding the walls, and of the structure above the basement, can also be determined or estimated satisfactorily, it may be possible to determine the strength characteristics of these walls and the static loading to which they are subjected. From this process, the magnitude of the additional (dynamic) loading that the wall can sustain prior to failure should be obtainable. However, in view of the MILL RACE results, a question then arises as to how to relate the free field air blast peak overpressure to the pressure and impulse transferred to the below-grade wall. There is experimental data from atmospheric nuclear testing on air blast effects on underground structures that may be relevant. Pressure measurements in a 60 psi free field environment were made at the Nevada Test Site during Operation Upshot-Knothole, at the surface and at depths of 4, 8, and 12 feet. It was concluded that "In well-compacted silty subsoil of the type at the test site, there is no effective attenuation of a pressure pulse applied at the surface with depth through the subsoil when the pressure is transmitted to a structure in the soil." (Ref. 51).

For the present purposes, as an upper bound, we can assume that the pressure pulse is not attenuated as it passes from the air into the ground. Then, the failure loading can be obtained directly in terms of peak overpressure acting on the entire face of the wall. This neglects the possibility, currently being investigated experimentally, that reflected pressure, from the incident blast wave striking the

aboveground wall of the building, may cause the pressure at the ground surface just above the basement wall to be significantly higher than the free field value.

Possible Wall Hardening Technique

Another option would be to assume that the walls wouldn't fail, but on a slightly different basis than used in the past. That is, one would take some steps either through upgrading or during construction of the building to help ensure that they did not fail prior to the failure of the first floor slab. As explained in Appendix C, there is reason to believe that the application of in-plane loading to the walls increases their rupture strength and thus enables them to survive at higher overpressures. This can be accomplished either structurally at the time of the construction or by the addition of steel plates, timbers, concrete slabs, etc., when upgrading is accomplished.

RESPONSE OF OTHER BASEMENT STRUCTURAL ELEMENTS

As described in some detail in Appendix B, there are seven primary structural elements that are important to the integrity of a basement structure. These are:

- Ceiling (basement)
- Floor (basement)
- Exterior walls (both backfilled and above grade)
- Interior walls (bearing and non-bearing)
- Framing system (beams, girders, columns)
- Connections (between system elements)
- Openings (doors, windows, stairwells, ducts, etc.)

This section of the report has considered the flat slabs that form the ceiling over the basement, the exterior walls of the basement, the columns that provide the framing system for this type of basement, and the connections between these elements. The possible failure of basement floors, or interior walls, or openings has not been discussed. Failure of basement floors and interior walls, though possible, is considered less significant than failure of the elements that have been considered. In as-built basements, openings can make an important contribution to the production of casualties. However, in upgraded basements, it is generally assumed that closures can be provided that will survive at least as well as the rest of the basement

structure. Further work will be done in future years on all of the basement structural elements.

DAMAGE FUNCTIONS FOR AS-BUILT BASEMENTS

The damage functions for the as-built basements will be based primarily on the failure characteristics of the overhead slab and its associated connections and framing in response to the air blast loading imposed on it. Although an interim procedure for considering exterior wall failure has been devised, it has not been applied to the specific cases considered elsewhere in the report.

NON-STRUCTURAL CASUALTY MECHANISMS

So far, the current basis for the portions of the overall casualty function resulting from the structual collapse of the first floor slab into the basement, and from the collapse of the basement walls, have been described. However, if the overall casualty function is to be realistic, the casualty increments attributable to the other nuclear weapons effects and other casualty mechanisms that might be expected to accompany the phenomena being considered should also be included. If other effects or mechanisms that could be expected to contribute to decreasing survivability are not included, erroneous conclusions may be reached. Unfortunately, there are some serious obstacles to an all-inclusive approach.

Even if one knows all of the structural details of the basement being considered, and assumes specific overpressure, yield, and height of burst parameters, one will still not have defined the intensities as a function of time or other measures of the capabilities of other weapons effects to cause casualties. This problem will be illustrated by going through the relevant nuclear effects and casualty mechanisms one by one and commenting on some of the difficulties that are encountered in trying to include them. Where possible, plausible assumptions and procedures that may provide at least partial solutions to the problem will be suggested. In this regard, the addition of upgrading can effectively eliminate casualties from some of the effects and can significantly reduce casualties from other effects.

Before the individual effects are discussed, however, it may be useful to divide the difficulties into two different categories. The first of these is difficulty in specifying the "free field" values at the location of the shelter for the effect being considered. For example, in the case of fallout radiation, the extreme dependence on winds makes it impossible before the fact to predict what the free field fallout intensities will be outside a shelter, in order to combine fallout effects with the air blast effects specified. The second type of difficulty involves effects where one can predict with reasonable accuracy what the free field intensities would be associated with specified air blast levels, but where going from this information to predicting the resulting casualties presents major difficulties. For example, for initial nuclear radiation, the intensities inside the shelter are highly dependent on the shielding, which could vary significantly for identical bursts at a specified range, if they occur in different directions from the shelter. In addition, people in different locations in the shelter could receive significantly different doses from the same burst.

Direct Blast

For cases where it is possible for direct blast to enter the shelter, this can be an important cause of casualties. Especially for situations where the ratio of the volume of the shelter to the cross-sectional area through which the blast wave enters the shelter is small, the interior pressures may be higher than free field values. It is estimated that the threshold for primary blast fatalities is 40 (30-50) psi, but severe lung damage occurs at 25 (20-30 psi) and 50 percent eardrum rupture at 15 psi (Refs. 23 and 24.) Section 2 of the report contains a fuller discussion of direct blast effects. For the purpose of determining the contribution that direct blast can make to the overall casualty function, if the V/A ratio can be calculated, the ratio of interior pressure to free field can be estimated, and the casualties resulting from direct blast can be included in the overall casualty function. If upgrading is accomplished that will preclude any blast entry until the entire structure fails, this effect will not need to be considered.

Blast Translation/Impact

For a basement with existing openings to the outside, or where openings are created by the blast effects on the structure, people can be blown around by the blast winds inside the shelter and injured or killed. Assuming that closures are provided in upgrading the basement and that they are strong enough to survive as long as the structure itself survives is one way around the problem. For a structure

having openings to permit entry of the blast winds, methods have been developed to calculate for simple geometries, and with other simplifying assumptions, what casualties might result from this effect. (See Ref. 14). Experiments have also been done using anthropomorphic dummies, and relationships have been developed to help predict casualties, including the effect that the ratio of the shelter volume to the cross-sectional area through which the blast wave enters the shelter has on the casualties to be expected. However, these methods are not developed to where they can be applied routinely to determine what this effect might contribute to casualty functions for realistic basements. However, thumb rules can be utilized to specify situations where this effect would not be expected to be a significant cause of casualties. See Section 2 for a more complete discussion of this topic.

Initial Nuclear Radiation

As was already mentioned, it is possible to make reasonable estimates of initial nuclear radiation free field intensities as a function of ground range, having specified yield and height of burst. Although variations in weapon design and in the density of the air through which the initial nuclear radiation travels can introduce differences, these would not preclude making usable free field estimates. However, "it is impractical to calculate the shielding effectiveness of even simple structures without resort to complex computer codes." (Ref. 23, para 871). Thus, to correctly estimate the contribution that initial nuclear radiation would make to an overall casualty function for a specific basement may be very difficult. What one may be able to do is to rule out casualties from INR for certain situations. For example, for 1 Mt weapons with HOB optimized for 10 psi, radiation protection is required only for 25 psi or stronger shelters (Ref. 53). Also, provision of sufficient shielding would preclude casualties at higher overpressures. See Section 2 for a more detailed discussion of this topic.

Thermal Radiation

Free field values of thermal radiation from a specific yield and height of burst of weapon can be predicted with reasonable accuracy for specified visibility-atmospheric condition situations. However, when the casualty function is being determined for assumed conditions prior to an attack, the contribution that thermal radiation might make could vary between wide limits. For as-built basements, where the possibility exists of a direct line of sight between persons in the basement and the fireball, thermal radiation could cause skin burns or eye injuries, or it could cause fires to start inside the shelter. The usual assumption is that all openings are

closed as part of the shelter upgrading, so one does not have to be concerned with thermal radiation effects inside the shelter. For as-built basements with windows or other openings, one might assume that no persons or combustible materials are in the direct line of sight, although this would be less justified than the previous assumption.

Thermal radiation can also start fires outside the shelter, as can blast (so-called secondary fires). These fires can cause casualties inside the shelter, for either as-built or upgraded shelters. In principle, it might be possible to preclude fire casualties if one were to provide an integral supply of oxygen, to keep toxic gases from entering, and to keep the temperatures in the shelter within acceptable limits, but this would go well beyond the type of upgrading that has been assumed elsewhere in this report. Unfortunately, because of the unpredictable nature of fire starts and fire spread from a postulated nuclear attack, it is practical neither to predict fire casualties nor to preclude fire casualties to persons in shelters. Some past studies have been limited to "prompt effects", thus eliminating consideration of fire effects (and fallout), or have sought to rule out fires by putting the shelters off by themselves and taking everything that might burn out of the area. A recent study has formulated an approach to blast/fire/people interaction in an attack environment, but further work remains to be done to achieve a usable capability for combining blast and fire effects realistically (Ref. 18). None of the available alternatives is particularly satisfying. Another technique is to estimate the fire hazard in the area around the potential shelter and to disqualify a basement from further consideration if the assumed fire hazard is too severe.

Residual Nuclear Radiation

As was brought out earlier, it is not practical to make predictions of the amount of fallout that would accompany specified air blast levels. On the other hand, having assumed that protection might be required from a specified fallout deposition, it is practical to provide earth cover over the basement or other shielding that could provide the required protection to prevent casualties due to fallout. This does not help us to arrive at a reasonable casualty function for as-built basements, but it can mean that fallout casualties will not need to be considered for adequately upgraded basement shelters. On the other hand, the addition of as much as three feet of earth will materially change the load, and thus reduce the survival levels of the slab over the basement. In some casualty functions, since only prompt effects

are considered, adding fallout protection serves to increase the percentage of casualties for a given overpressure. (Ref. 16 and 17).

CASUALTY FUNCTIONS FOR AS-BUILT BASEMENTS

For the specific as-built flat slab basements considered this year, casualties are assumed to result primarily from the structural failure of the flat slab. Initial nuclear radiation was considered to the extent of determining free field doses corresponding to the overpressures causing structural failure, but detailed calculations were not attempted. The ratio of the volume of the shelter to the area open to the blast wave was determined where possible to estimate the potential for blast translation of people in the basement, or for primary blast casualties. Casualties from thermal radiation and residual nuclear radiation were not included.

SHELTER UPGRADING GROUND RULES USED

General Considerations

The shelter upgrading techniques used in this report are based on those for flat slab basements currently contained in Refs. 20 and 21, augmented as described earlier in this section. For heavily constructed basements, it has been assumed that the sheltering requirement is for key worker shelters, so designs from Ref. 20 are appropriate. For basements of lighter construction, it has been assumed that the sheltering requirement is for host area shelters, so designs from Ref. 21 would be appropriate.

Nuclear Radiation Upgrading

As a starting point, the shielding provided by the slab itself is determined. This is augmented by the addition of shielding material, normally soil, where a higher protection factor is desired. For key worker shelters, a protection factor of 1000 against fallout radiation is specified in the upgrading manual (Ref. 20). Addition of 2 feet of earth is required to achieve this level of protection. For host area shelters, protection factors of 40, 100, or 1000 may be specified, requiring the addition of no soil, 0.5 ft, or 2 ft, respectively.

Flat Slab Structural Upgrading

The heavily constructed flat slab basements that would be suitable for use as key worker shelters are upgraded by post shores at intervals not exceeding one-quarter of the span, according to Figure 4-28, taken from Ref. 20. The table of shoring possibilities is reproduced from the same reference as Table 4-2. This has been augmented by the additional shoring illustrated in Figure 4-21, where appropriate. For host area shelters, post shores at midspan are specified for light design and medium design flat slabs. These would be selected from the chart reproduced here as Figure 4-29. Heavy flat slab designs do not require upgrading for host area use.

Exterior Wall Upgrading

As mentioned earlier in the present section, exterior walls can be upgraded by various techniques in order to increase their in-plane loading, and thus their resistance to failure under blast loading. This kind of upgrading has been assumed where appropriate.

Closure Upgrading

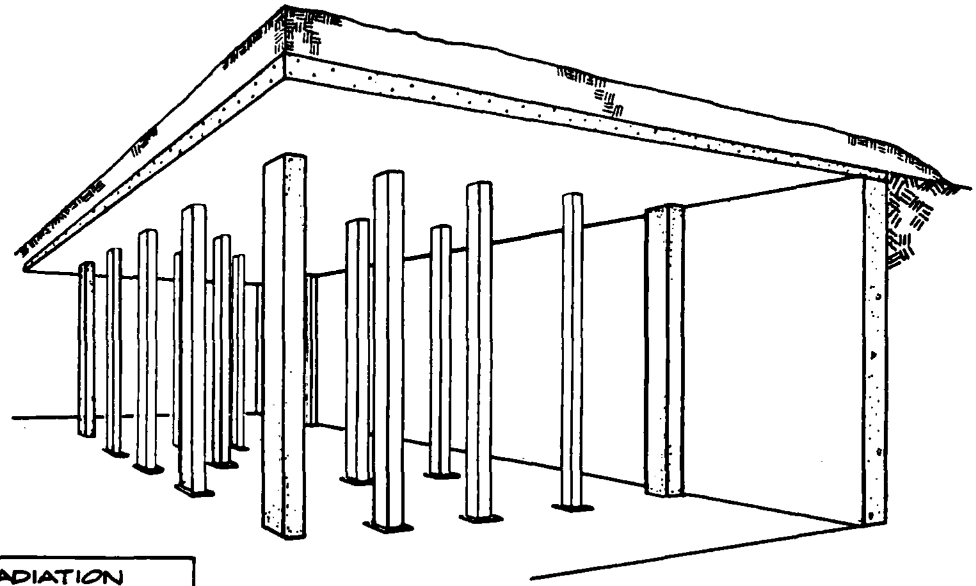
In addition to the blast protection provided by upgrading the resistance of the flat slab to failure, it is necessary to provide closures in order to protect the people in the shelter from primary blast effects and from blast winds. This is accomplished by the installation of closures such as those illustrated in Refs. 20 and 21.

Specific Applications

As is apparent from the material just presented, the upgrading applied to any flat slab (or other) basement can be varied to provide for different degrees of radiation shielding and for different levels of predicted slab and closure survival against blast loading. Even for a given fallout protection factor and designated survival overpressure, different upgrading materials, with varying dimensions and spacings may be used. For this reason, specific details of the upgrading that was assumed are required for each upgraded basement considered.

DAMAGE FUNCTIONS FOR UPGRADED BASEMENTS

The structural failure of the upgraded basement floor slab provides the basis for the damage functions for the upgraded basements considered in this report.



RADIATION	
P_F	DEPTH
1000	2.0

Shoring

The recommended method for shoring flat plate and flat slabs is to use post shores, as shown in the sketch above. For shore classification, see spacing chart below, and for types of shores refer to pages A-3 to A-5. Maximum unshored distance should not exceed one-quarter of the span.

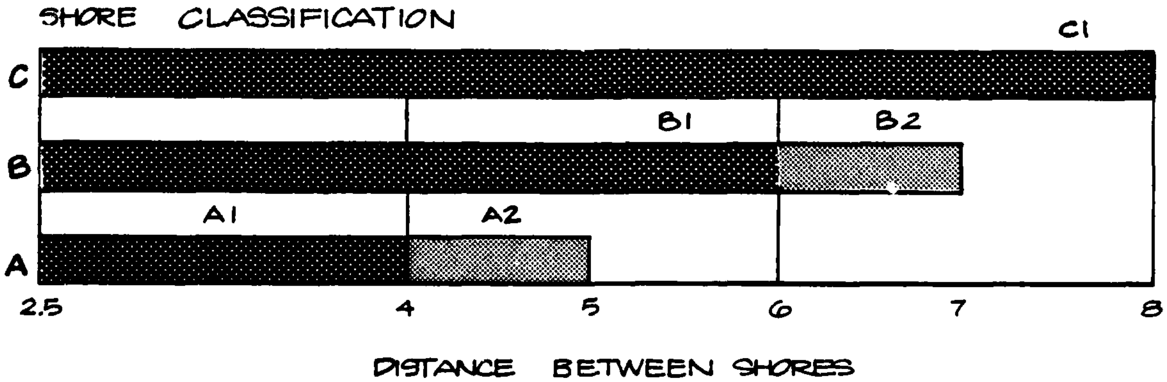


Fig. 4-28. Flat Plate and Flat Slab Upgrading. (Ref. 20)

TABLE 4-2: SHORE DESIGNATION

STRUCTURAL TYPE AND DIMENSIONS	MAXIMUM SHORE LENGTH - FEET				
	TYPE A		TYPE B		TYPE C
	A1	A2	B1	B2	C1
SHORE SPACING	* to 4' x4'	4'x4' + to 5' x 5'	5' x 5'+ to 6' x 6'	6' x 6'+ to 7' x 7'	7' x 7'+ to 8' x 8'
WOOD POST (NOM.)					
6" x 6"	8'				
6" x 8"	9'	7'			
8" x 8"	14'	11'			
8" x 10"		12'	10'		
10" x 10"				12'	
12" x 12"					12'
STEEL PIPE					
STANDARD STRENGTH					
4" x 0.237"	8'				
5" x 0.258"	12'				
6" x 0.280"		12'			
8" x 0.322"			12'	8'	
EXTRA STRONG					
3½" x 0.318"	10'				
4" x 0.337"	12'				
5" x 0.375"		12'			
6" x 0.432"			12'		
DOUBLE EXTRA STRONG					
3" x 0.600"	10'				
4" x 0.674"		12'	10'		
5" x 0.750"				12'	
STRUCTURAL STEEL TUBE					
4" x 4" x 3/16"	10'				
4" x 4" x 1/4"	12'				
4" x 4" x 5/16"	12'	8'			
4" x 4" x 3/8"	12'	10'			
4" x 4" x 1/2"		12'	8'		
5" x 5" x 3/16"	12'				
5" x 5" x 1/4"		12'			
5" x 5" x 5/16"		12'	8'		
5" x 5" x 3/8"			12'		
5" x 5" x 1/2"			12'	10'	

*minimum shore spacing should not be less than 30" (2.5') on centers under most austere [illegible]

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SHELTERS(U) SCIENTIFIC SERVICE INC REDWOOD CITY CA
R E PETERSON ET AL. DEC 82 SSI-8143-9 EMW-C-0678

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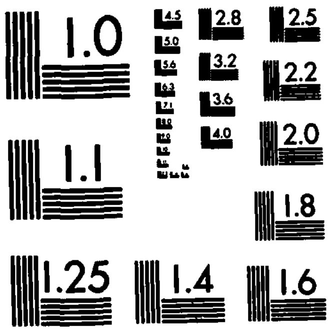


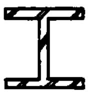
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

TABLE 4-2: SHORE DESIGNATION (contd)

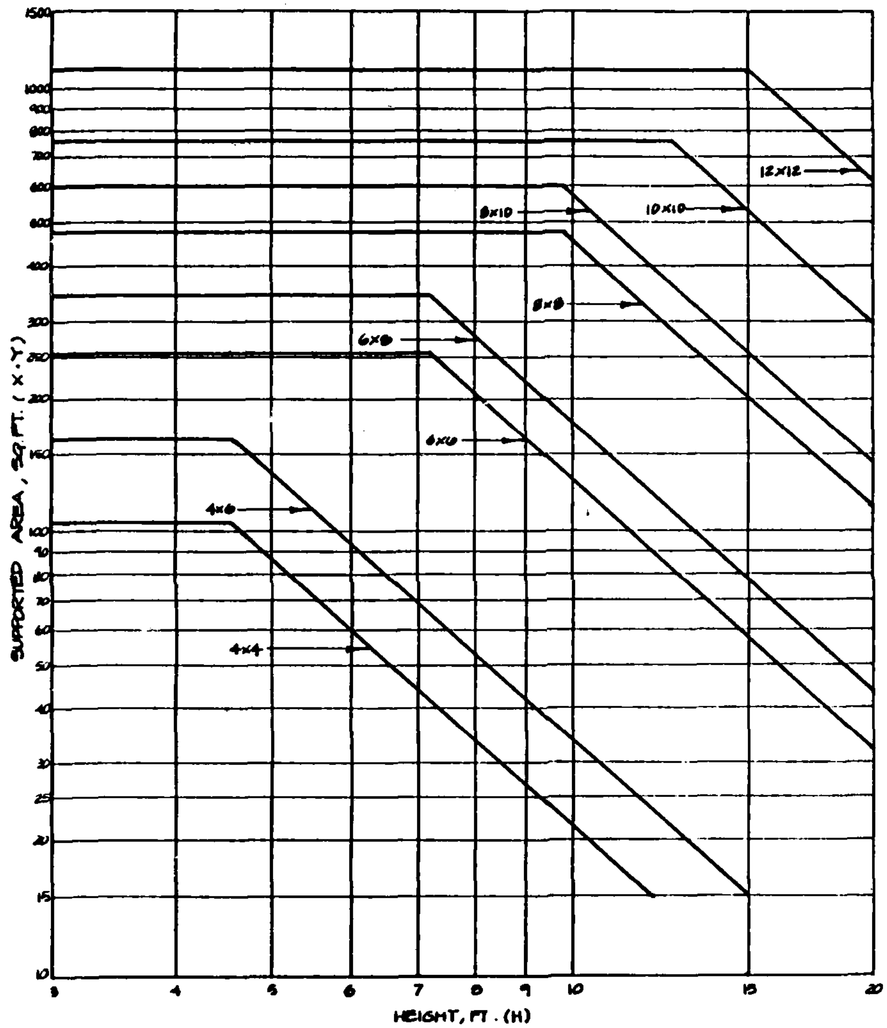
STRUCTURAL TYPE AND DIMENSIONS	MAXIMUM SHORE LENGTH – FEET				
	TYPE A		TYPE B		TYPE C
	A1	A2	B1	B2	C1
SHORE SPACING	* to 4' x 4'	4' x 4'+ to 5' x 5'	5' x 5'+ to 6' x 6'	6' x 6'+ to 7' x 7'	7' x 7'+ to 8' x 8'
STRUCTURAL STEEL TUBE					
(cont.)					
6" x 6" x 3/16"		12'			
6" x 6" x 1/4"		12'	10'		
6" x 6" x 5/16"			12'		
6" x 6" x 3/8"				12'	
6" x 6" x 1/2"					10'
7" x 7" x 3/16"		12'	8'		
7" x 7" x 1/4"			12'		
7" x 7" x 5/16"				12'	
8" x 8" x 1/4"				12'	
4" x 3" x 5/16" 	8'				
5" x 3" x 3/16"	8'				
5" x 3" x 1/4"	10'				
5" x 3" x 5/16"	10'				
5" x 3" x 3/8"	10'	8'			
5" x 3" x 1/2"	12'	8'			
6" x 3" x 3/16"	8'				
6" x 3" x 1/4"	10'				
6" x 3" x 5/16"		8'			
6" x 3" x 3/8"	12'	10'			
6" x 4" x 3/16"	12'				
6" x 4" x 1/4"	12'	10'			
6" x 4" x 5/16"		12'	8'		
6" x 4" x 3/8"			10'		
6" x 4" x 1/2"			12'	8'	

*minimum shore spacing should not be less than 30" (2.5') on center under most aushore conditions.

TABLE 4-2: SHORE DESIGNATION (contd)

STRUCTURAL TYPE AND DIMENSIONS	MAXIMUM SHORE LENGTH - FEET				
	TYPE A		TYPE B		TYPE C
	A1	A2	B1	B2	C1
SHORE SPACING	* to 4' x 4'	4' x 4'+ to 5' x 5'	5' x 5'+ to 6' x 6'	6' x 6'+ to 7' x 7'	7' x 7'+ to 8' x 8'
STRUCTURAL STEEL TUBE					
(cont.)					
7" x 5" x 3/16"		12'			
7" x 5" x 1/4"		12'	10'		
7" x 5" x 5/16"			12'		
7" x 5" x 3/8"			12'	10'	
8" x 4" x 1/4"		12'	8'		
8" x 4" x 5/16"			12'		
8" x 4" x 3/8"			12'	8'	
8" x 6" x 1/4"			12'		
8" x 6" x 5/16"				12'	
STEEL WIDE FLANGE BEAMS					
M5-18.9 5" wide by 5" deep 	12'	8'			
M6-20 6" wide by 6" deep	12'	10'			
W5-16 5" wide by 5" deep	12'				
W5-19 5" wide by 5 1/8" deep	12'	8'			
W6-16 4" wide by 6 1/4" deep	8'				
W6-15 6" wide by 6" deep	12'				
W6-20 6" wide by 6 1/4" deep	12'	10'			
W6-25 6 1/8" wide by 6 3/8" deep		12'	8'		
W8-24 6 1/2" wide by 7 7/8" deep		12'	8'		
W8-28 6 1/2" wide by 8" deep			12'		

* minimum shore spacing should not be less than 30" (2.5') on centers under most austere conditions.



NOTE:

USE THIS CHART WHEN SUPPORTING BEAM <u>OTHER</u> THAN TIMBER (STEEL, CONCRETE, ETC.)

TIMBER POST DATA

Fig. 4-29. Timber Post Data. (Ref. 21)

Structural failure of the basement exterior walls or other basement structural elements, while possible, is not assumed to contribute to the damage functions presented for specific basements.

CASUALTY FUNCTIONS FOR UPGRADED BASEMENTS

The casualty functions presented for specific upgraded basements are based on the structural failure of the flat slab. Casualties due to initial and residual nuclear radiation are assumed to be prevented by the shielding provided. Closures are assumed to survive well enough to prevent casualties due to primary blast, blast translation, and from thermal radiation entering the shelter. Fires in the vicinity of the shelter could cause those inside to become casualties, but this possibility is not included in the casualty functions shown.

Section 5
CASUALTY FUNCTION PREDICTIONS FOR REPRESENTATIVE BASEMENTS

INTRODUCTION

In this section the casualty function development procedure, as it has evolved during the first year of the program, is utilized for the evaluation of three representative flat slab basement structures. The application of this system to a range of flat slab structures is then described. The basements evaluated in this section are located in:

- San Mateo County Office Building, Redwood City, CA (SMCO)
- Civic Center Parking Garage, San Francisco, CA (CCPG)
- Mercy General Hospital, Sacramento, CA (MGHS)

GENERAL DESCRIPTION OF BUILDINGS

San Mateo County Office Building

The San Mateo County Office Building is located in downtown Redwood City and is in close proximity to other City and County government buildings. It was designed in 1960 and is 200 ft by 174 ft in plan and has five stories above the ground and a single basement. The two-way reinforced concrete flat slab examined in this analysis is $7\frac{1}{2}$ in. thick with square drop panels 7 ft 6 in. on a side and $3\frac{1}{2}$ in. thick. The concrete used in the slab had a 28-day compressive strength of 3000 psi. The slab is supported by columns, which are 2 feet square and are 20 feet on center in both the north-south and east-west directions. The reinforcement used was assumed to have a tensile yield strength of 40,000 psi.

Civic Center Parking Garage

The San Francisco Civic Center Parking Plaza is located in downtown San Francisco and is in close proximity to a number of City office buildings, which

include the City Hall Building. In addition, it is close to several theaters and is connected by a tunnel to the Brooks Exhibition Hall. This three level parking structure was designed by a private firm for the city in 1958. All three levels are underground and the garage is approximately 370 ft by 330 ft in plan. The area over the garage is a city park. The two-way reinforced concrete slab chosen for this analysis separates the second sub-level from the third sub-level. It is 12 in. thick with drop panels, which are 10 ft by 10 ft by 6 in. The concrete used in the slab had a 28-day compressive strength of 3,250 psi and the reinforcement was assumed to have a tensile yield strength of 40,000 psi. Typical columns used to support this slab are elliptical in shape and have major and minor dimensions of 54 in. by 18 in. These columns are typically spaced about 27 ft in one direction and about 30 ft in the other direction.

Mercy General Hospital

Mercy General Hospital is located in a residential neighborhood of Sacramento, California, less than ten miles from the State Capitol. It is noteworthy that the structural engineer who designed this building in 1978 used a finite element method, which treats the entire slab as a unit and analyzes different combinations of slab and drop panel thicknesses, giving the deflections for each slab system. The product of this design process is a slab that is 12 in. thick around the perimeter of the building and 10½ in. thick in the slab's interior section. Typical drop panels are 12 ft 6 in. by 12 ft 6 in. by 6 in. thick and typical columns are 2 ft square. The columns are spaced 31 ft 6 in. on centers throughout a major part of the building. The concrete specified for the slab had a 28-day compressive strength of 3000 psi and the reinforcement has a tensile yield strength of 60,000 psi. The major part of the hospital is two stories high and has a single basement beneath it. The design includes landscaped, outside stairs, which lead into the building through the basement. The building is approximately 220 ft by 156 ft in plan.

STRUCTURAL ANALYSIS

Structural drawings were obtained for the three structures listed above and were utilized to obtain the information needed to perform the analysis. See Figures 5-1 through 5-3, which show sections of the ground floor structural framing plans for the three structures. It is still anticipated that the final rating procedure, which is scheduled to be developed by the end of the five-year program, will enable local

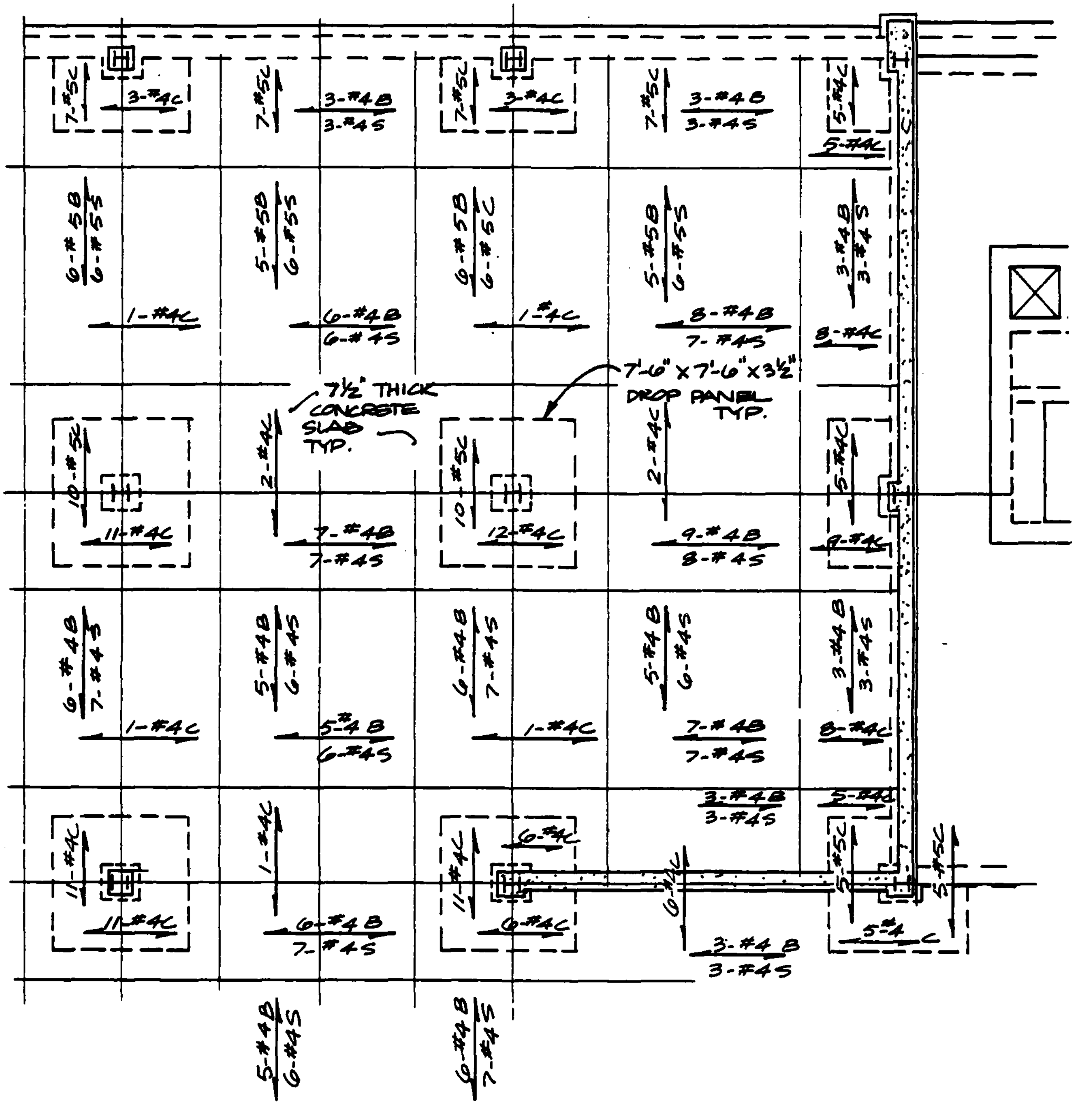


Fig. 5-1. First Floor Framing, San Mateo County Office Building.

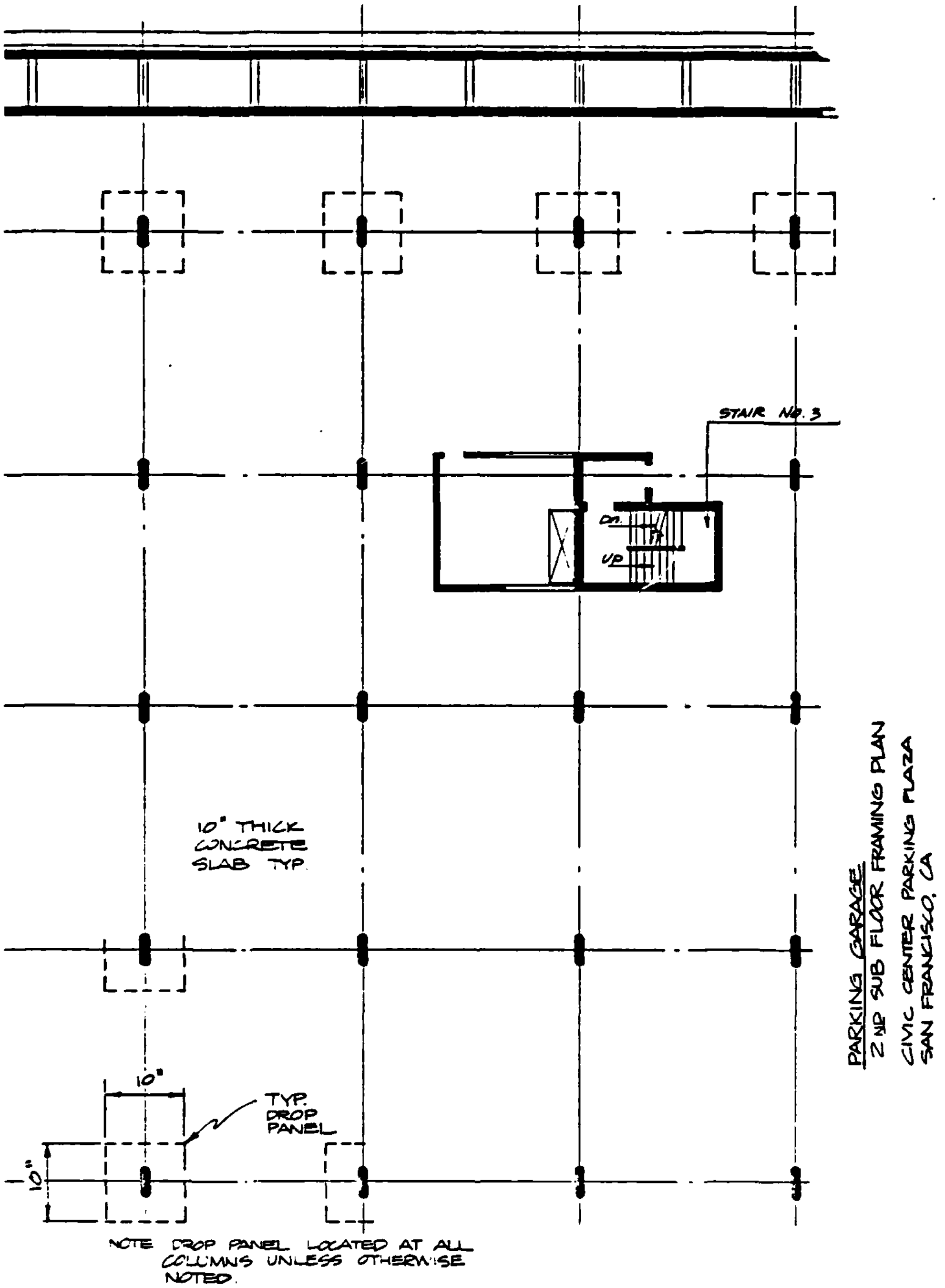


Fig. 5-2. First Floor Framing, Civic Center Parking Garage.

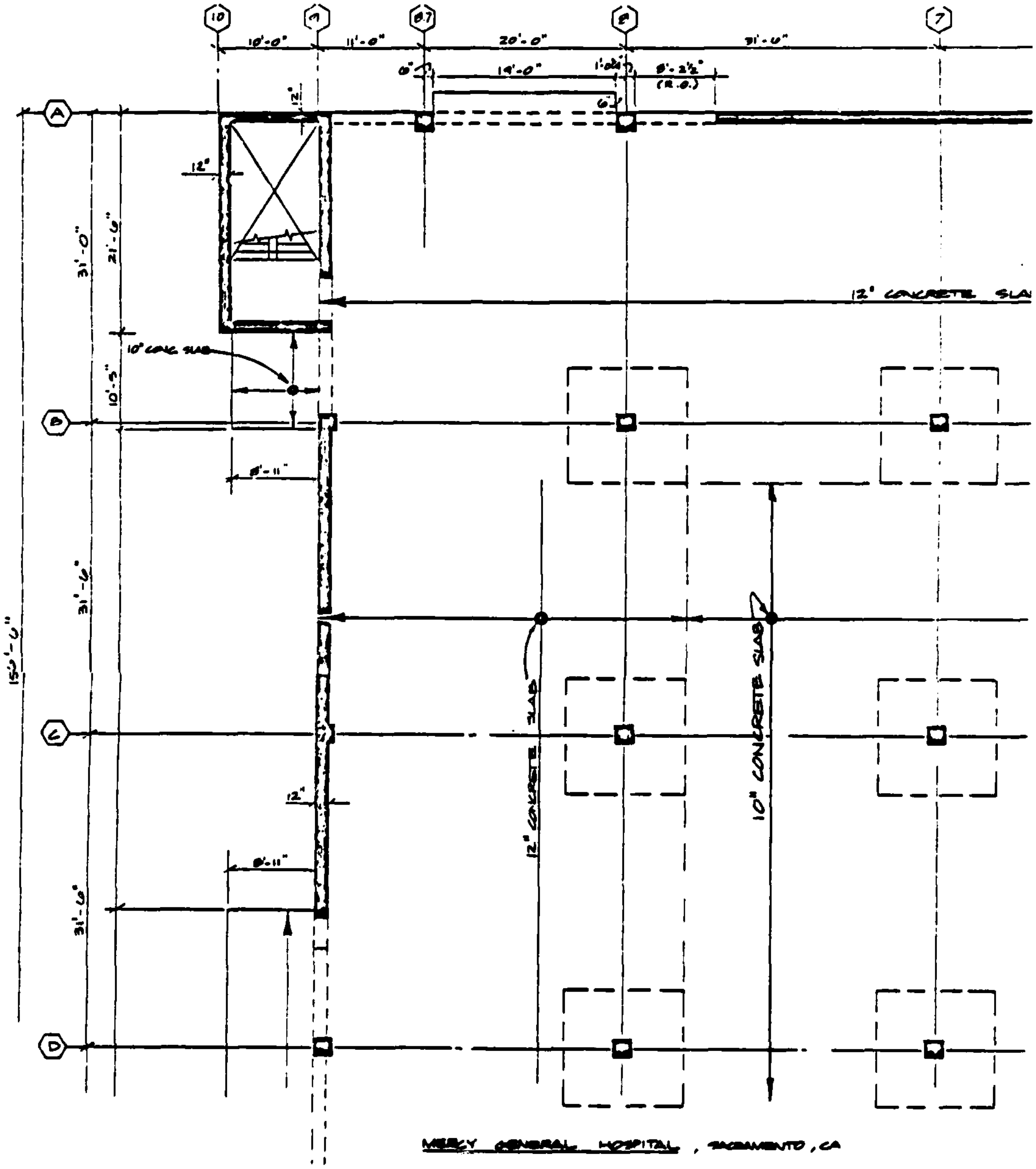


Fig. 5-3. First Floor Framing, Mercy General Hospital.

planners to rate basements without access to the structural drawings. However, the current structural engineering type of analysis does require information that can most conveniently be obtained from structural drawings.

These drawings provided the information (material strength, dimensions, reinforcing steel) required for the evaluation of the imposed vertical load capacity (strength) of the floor levels selected for shelter areas. These three structures all have either a basement (in SMCO and MGHS) or belowground levels (in CCPG). Of the several belowground levels in the CCPG structure, the lowest was selected for analysis. The spaces selected meet all the necessary qualifications for shelter space.

METHOD OF STRENGTH CAPACITY ANALYSIS

The standard (Uniform Building Code and American Concrete Institute Code, Refs. 54 and 36) methods of ultimate strength design as given in "Reinforced Concrete Fundamentals" by P.M. Ferguson, (Ref. 35) were used for the evaluation of the representative flat slab systems for shear capacities. These included: punching shear of the column at the drop panel; punching shear at the slab and at the drop panel interface; and slab beam shear at the drop panel interface. The standard procedures could not be used for the evaluation of the slab flexural capacity, since these procedures employ moment coefficients to provide design strength requirements in column strips and middle strips of the slab; therefore no total slab failure capacity can be found for an entire slab panel between column lines.

In order to estimate the total flexural capacity of the entire slab, a simple parallel (column line, mid-panel line) yield line system was used. The total negative moment capacity M_2 at the column line and positive moment capacity M_1 at the mid-panel line were used as the yield line flexure values in a total panel between columns. This parallel yield line system is essentially the lower-bound system since it neglects the strengthening effects of the column capital and drop panels. However, allowing for the real actual non-uniform load and the non-symmetrical configuratons of the slab in a given building, this flexural capacity is judged to be a good mean value estimate.

Non-Upgraded Capacity

The existing (non-upgraded) imposed blast pressure capacity is determined as follows:

(1) Find slab beam shear capacity on a one foot slab strip section at effective slab depth "d" from column face, and from drop panel face. Strength stress is $2\sqrt{f'_c}$ calculated without using the understrength ϕ factor.

(2) Find slab punching shear capacity at a perimeter d/2 from column face, and at d/2 from drop panel face. Strength stress is $4\sqrt{f'_c}$ without using the ϕ factor.

(3) Find slab flexural yield line capacity, using ultimate flexu strength capacity (without using the ϕ factor) of negative moment, M_2, and positi moment, M_1, of entire slab bay width. Yield line pattern is a parallel set of nega flexure lines over column lines and positive lines along the center of the bay w (See Ref. 35, pp 404-6).

All failure mode capacities are converted to uniform pressure loads (per square foot, psf) and the imposed blast pressure capacity, w_i is this pressure minus the dead load w_{DL} in psf.

Results are given in Table 5-1 for each structure.

Table 5-1
CALCULATED PUNCHING SHEAR AND FLEXURAL CAPACITIES IN psf
FOR NON-UPGRADED BASEMENT FLAT SLABS

Failure Mode	Building		
	SMCO	CCPG	MGHS
Beam Shear, w_B	1120	1265	1080
Punching Shear, w_P	670	1061	428
Flexure, w_F	120	131	128

Upgraded (Shored) Capacity

The upgrading methodologies described in the "Shelter Upgrading Manual: Key Worker Shelters" (Ref. 20), were used to upgrade the shelter areas to the maximum practical capacity. It was judged that 1/4 span (L/4) shore spacing would provide the highest imposed blast resistance compatible with retaining a reasonable proportion of usable shelter space. This scheme would provide approximately five feet of clear space between shores. However, since extra drop-panel support shores would also be provided, the space around the columns (under the drop panels) would be more restricted (about two or three feet of clear space).

Using the upgraded capacity rule given in Ref. 20, the original flexural capacity, w_F is increased by $(4)^2 = 16$ times by the L/4 shore spacing system. The imposed blast resistance for the upgraded structure then becomes

$$w'_F = 16\, w_F - w_{DL} - w_{SL}$$

where w_{SL} is the radiation soil cover load, taken as equal to 200 psf (and used only for the upgraded slab condition.)

The resulting upgraded values are given in Table 5-2.

Table 5-2
CALCULATED FLEXURAL CAPACITIES, w'_F
FOR UPGRADED (SHORED) BASEMENT SHELTER FLAT SLABS

	Building		
	SMCO	**CCPG**	**MGHS**
Expressed in psf	3370	4520	4270
Expressed in psi	23.4	31.4	29.7

DISCUSSION OF REPRESENTATIVE STRUCTURE ANALYSES

In the three representative flat slab structures, the shear modes of failure are at substantially higher vertical load values than the computed load capacity at flexural yield failure; and this flexural capacity w_F has approximately the following range in terms of design dead load w_{DL} and live load w_{LL}

$$(1.4w_{DL} + 1.7w_{LL}) \leq w_F \leq 2(w_{DL} + w_{LL})$$

For example: $w_{DL} = 150,\ w_{LL} = 50$

$$(295 \text{ psf}) \leq w_F \leq (400 \text{ psf})$$

with imposed blast load capacity

$$w_i = w_F - w_{DL} \qquad \text{as}$$

$$(145 \text{ psf}) \leq w_i \leq (250 \text{ psf})$$

While the shear capacities as computed by established design practice (Ref. 35) are larger than w_F, it is essential to recall, from the preceding sections on flat slab behavior, that these structural systems are subject to progressive collapse due to non-uniform, non-symmetrical load patterns and moment redistribution. The high shear capacities are based on symmetrical conditions and can be considerably reduced when these conditions are changed. Therefore w_F is judged to be the best indicator of strength in any actual structure under blast loading.

RESULTS OF STRENGTH EVALUATION

From the results of the strength evaluation of flat slab systems under vertical imposed blast pressure (modeled as a uniform load per square foot) the non-upgraded slab can take w_i of about 200 psf and the upgraded shored slab system can support about 4600 psf or about 30 psi. The casualty function estimates are based only on the computed strength capacity of a given flat slab system and the effects of structural failure. There are other factors for casualty evaluation such as:

effectiveness of closure of entry and openings, environment within the shelter, small debris and dust effects, radiation protection effectiveness, ability to exit from the shelter when structural debris exists on the shelter roof, and the effect of superstructure failure and collapse on the shelter roof. Factors of this sort are discussed later in this section and elsewhere in the report.

CONSIDERATION OF FAILURE OF OTHER STRUCTURAL COMPONENTS

The possibility that other structural elements, such as the external walls, might fail prior to the failure of the slab is discussed elsewhere in this report. In addition, the possibility that the response of the aboveground frame of the building to air blast loading might affect the basement shelter has been considered in a recent SSI research report (Ref. 55). However, for the specific flat slab structures considered, no detailed analysis was performed for any other structural components. The assumption was made that the flat slabs were the weakest structural component in these basements. This is considered reasonable because the columns and supporting walls are designed for the full dead load and live load of the entire superstructure of the building. The probable actual live load at any time is about 1/10 of the design live load, and may be even lower in a crisis situation.

CONSIDERATION OF OTHER CASUALTY MECHANISMS

As-built Basements

Other possible casualty mechanisms for the as-built case include initial nuclear radiation, primary blast, blast translation, thermal radiation, and residual nuclear radiation. For the three structures considered in detail in this section, it was considered reasonable to neglect blast effects, other than those causing slab failure, as well as thermal-radiation-caused skin burns or fires inside the shelters. Fires outside the shelters and residual nuclear radiation also have not been considered, not because they could not cause casualties, but because there is not a plausible method available to include them. Initial nuclear radiation has been considered to the extent of determining the free field values associated with the collapse overpressures for the assumed burst conditions. However, at the levels at which the as-built slabs will collapse, it would not cause casualties.

Upgraded Basements

In addition to the structural shoring assumed to be installed, it has also been assumed that the closures have been upgraded to prevent their failure prior to slab failure and that soil has been added for initial and residual nuclear radiation protection. As a result of this upgrading, casualties from primary blast, blast translation, thermal radiation inside the shelter, and initial and residual nuclear radiation are precluded. The cause of casualties will again be exclusively failure of the basement ceiling slab due to the air blast loading imposed upon it, with the possible exception as discussed in the next paragraph.

RESULTS OF CRACKING OF CONCRETE BY BLAST WAVE

Although it was not considered in the casualty estimates, cracking of the concrete cannot be ruled out, and could cause casualties under certain conditions. It is almost impossible to guarantee that localized shear or flexure cracks will not occur in the shelter roof cover. If these fissures do occur, and high pressure shock waves enter the shelter space, there can be severe or even lethal effects on the occupants. If it is recognized that 30 psi is equivalent to about 70 feet of water pressure, the situation may be clearer. It would be virtually impossible to assure a "water-tight" flat-slab shelter area if it were submerged to a depth of 70 feet. This lack of pressure integrity definitely needs to be considered when decisions are made concerning the viability of upgraded shelter areas. Many actual structures are stressed by foundation settlement and shrinkage, and would not require a great amount of extra load to develop severe cracking conditions.

To reiterate, it is quite feasible to upgrade a flat slab to resist high vertical pressure loads without general collapse, but it is not possible to assure isolation between the extreme pressure differences (outside versus inside shelter) during blast loading. As was discussed in Section 2 of the report, when there is a severe air blast environment outside the shelter, the seriousness of direct blast loadings inside the shelter is dependent primarily on the V/A ratio of the shelter, where V is the shelter volume and A is the cross-sectional area through which the blast wave enters the shelter.

RECOGNITION OF UNCERTAINTY AND VARIABILITY

The computed blast resistant capacities may be considered as the best or mean value estimate of the actual (random) capacity of the flat slab system that was analyzed. The natural uncertainties and variabilities involved in material strengths, slab configurations, simplified methods of analysis and evaluation, and in the performance of shoring systems (for the upgraded shelter) all cause the actual capacities to be random variables. These can be represented as normally (bell shaped) distributed random variables with mean values equal to computed capacities, w, and with standard deviations, σ, equal to 0.1w for the original (non-upgraded) capacity; and σ' equal to 0.15w' for the upgraded capacity. The higher variability for the upgraded capacity allows for the uncertainty due to limited test knowledge of the performance of shoring systems and uncertainty in the validity of the general upgraded capacity equation:

$$w' = 16w - w_{DL} - w_{SL}.$$

Using these values the 95 percent reliable lower bound values of capacity (such that probability of actual capacity above this value is 95 percent) is mean value - 1.65 σ.

For the non-upgraded case this is: $(1 - 0.165)\ w_F = 0.835\ w_F.$

For the upgraded case: $(1 - (.15)(0.165))w'_F = 0.75\ w'_F.$

Again, these are the values for which there is a 95 percent confidence that the actual random value of capacity will be greater or exceed these values.

A PROBABILISTIC MODEL FOR ACTUAL CAPACITY AND RELATED CASUALTY FUNCTIONS

Bearing in mind the fragility of slab systems under non-uniform load conditions, and non-symmetric support or span conditions, it was judged that the best estimate of the mean value of slab capacity is the flexural value, w_F, for non-upgraded systems, and w'_F for the upgraded case. Shear capacities are higher, but may be reduced by the non-uniform, non-symmetrical conditions of the actual structure.

If the variability of any actual capacity (as differentiated from the best estimated value) is represented by a coefficient of variation, and a normal bell-shaped probability distribution is assumed, then the probabilistic model of capacity is for:

Non-Upgraded Case

Actual w_i has mean value $w_F - w_{DL}$, and standard deviation

$$\sigma = 0.10\ w_i.$$

Upgraded Case

Actual w'_i has mean value $w'_F - w_{DL} - w_{SL}$, and

$$\sigma' = 0.15\ w'_i.$$

Using normal probability paper, the complete random model for capacity can be represented by a straight line passing through the mean value at 50 percent and rising to a value of mean plus one σ value at 84 percent. An example is shown in Figure 5-4 for the case where:

$$w_i = 200 \text{ psf}, \quad \sigma = 20 \text{ psf}.$$

$$w'_i = 4800 \text{ psf}, \ \sigma' = 720 \text{ psf}.$$

The horizontal probability axis at the bottom gives the probability that the actual capacity will be less than a value taken from the line. For example from Figure 5-4

$$P\,[\text{actual } w_i \leq 250 \text{ psf}] = 98\%$$

$$P\,[\text{actual } w'_i \leq 6000 \text{ psf}] = 93\%.$$

The top probability axis gives the complementary probabilities,

$$P\,[\text{actual } w_i \geq 250 \text{ psf}] = 2\%$$

$$P\,[\text{actual } w'_i \geq 6000 \text{ psf}] = 7\%$$

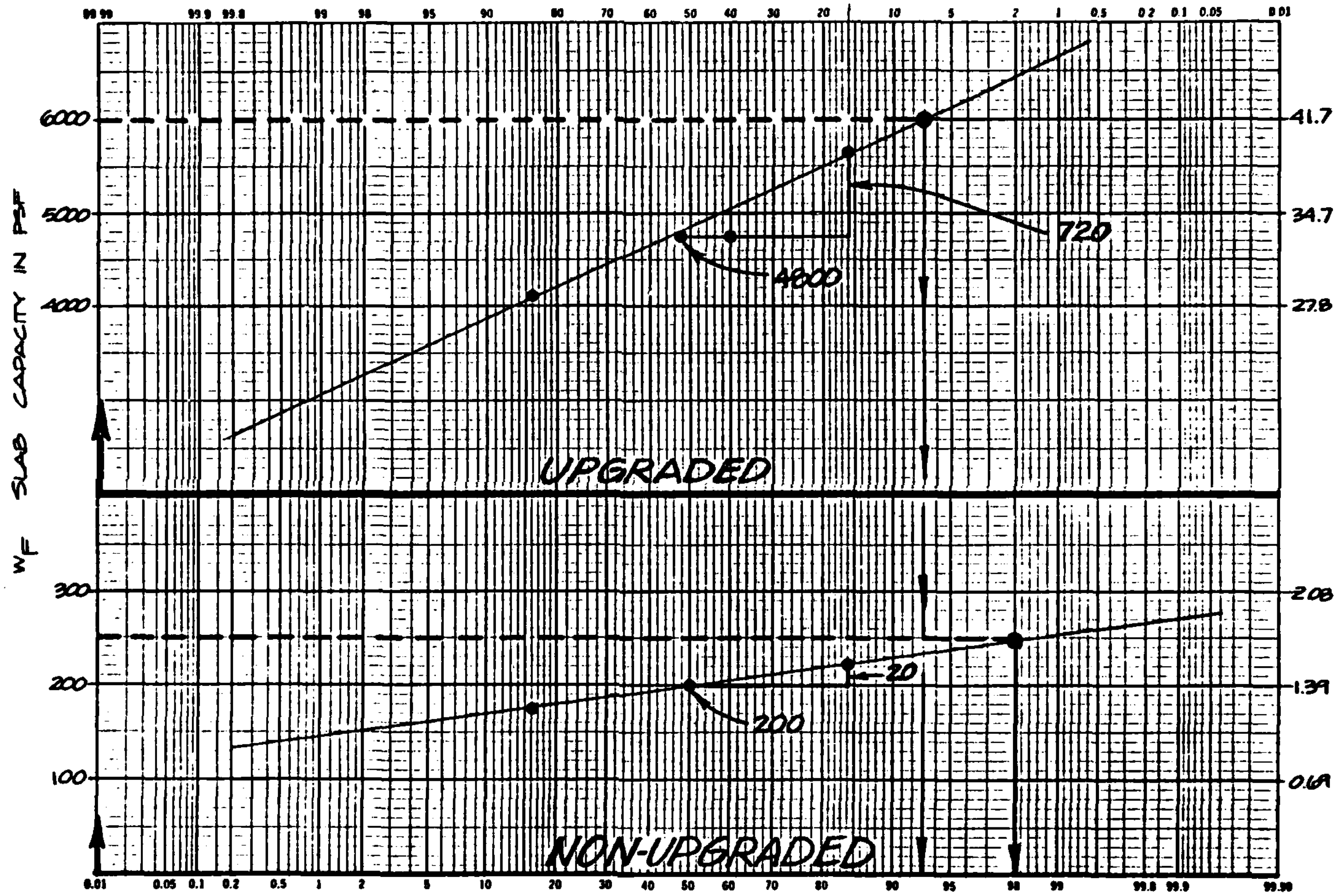


Fig. 5-4. Examples of Typical Casualty Functions.

CASUALTY FUNCTION

For given flat-slab capacity predictions or mean values, such as those shown on Figure 5-4

$$w_i = 200 \text{ psf}$$

$$w'_i = 4800 \text{ psf},$$

the plotted lines can be used as a casualty function or forecast of casualty rates.

Given a required blast load, w_B, and probability line for a given structure,

$$P [\text{ actual } w \leq w_b] = \text{Expected Percent Casualty Rate.}$$

This percent rate can be read directly from the bottom horizontal axis of Figure 5-4.

This relation is based upon the following interpretations of the probability statement.

Given 100 percent casualty rate of failure

$$C \mid w < w_B = 100\%$$

where $C \mid w < w_B$ is a symbolic notation for Casualty C given failure $w < w_B$. Then

$$P [C w < w_b] = P [w < w_B]$$

Using expected casualty rate as equal to this probability, then

$$\text{Rate} = P [\text{ actual } w < w_B]$$

This is based on the 100 percent casualty rate if slab failure ($w < w_B$) occurs. This may be an overly conservative assumption, but judging that extreme dust, debris, and shock pressure can result when the slab is breached, it may not be too far from reality.

GENERAL APPROXIMATE RULES TO ESTIMATE CAPACITIES

Given w_{DL}, w_{LL} the following approximations can be used,

$$w_{DL} = 10 \sqrt{w_{LL}}$$

$$w_F = 2(w_{DL} + w_{LL})$$

$$w'_F = 16\ w_F \quad \text{for 1/4 span shore system}$$

$$w'_i = w'_F - w_{DL} - w_{SL}$$

This is illustrated with sets of curves for the as-built and upgraded cases for representative live loads of 40, 75, 100, 150, and 250 psf (see Figures 5-5 and 5-6).

Application to Specific Designs

Figure 5-7 shows the applicable as-built and upgraded casualty functions with specific analytical results for the SMCO, CCPG, and MGHS basements superimposed on them. As noted earlier in the section, the computed load capacity at flexural yield failure has approximately the following range:

$$(1.4w_{DL} + 1.7w_{LL}) \leq w_F \leq 2(w_{DL} + w_{LL})$$

For clarity, only the upper bounds were plotted in the sets of curves in Figures 5-5 and 5-6. The lower bounds are also plotted in Figure 5-7. It is noted that the results for the non-upgraded cases fall between the upper and lower bounds, whereas the results for the upgraded SMCO fall within the bounds and the CCPG and MGHS are slightly above. That is, the analytical results indicate that their slab capacities with upgrading (shoring) are actually somewhat above that predicted on the approximate rule basis.

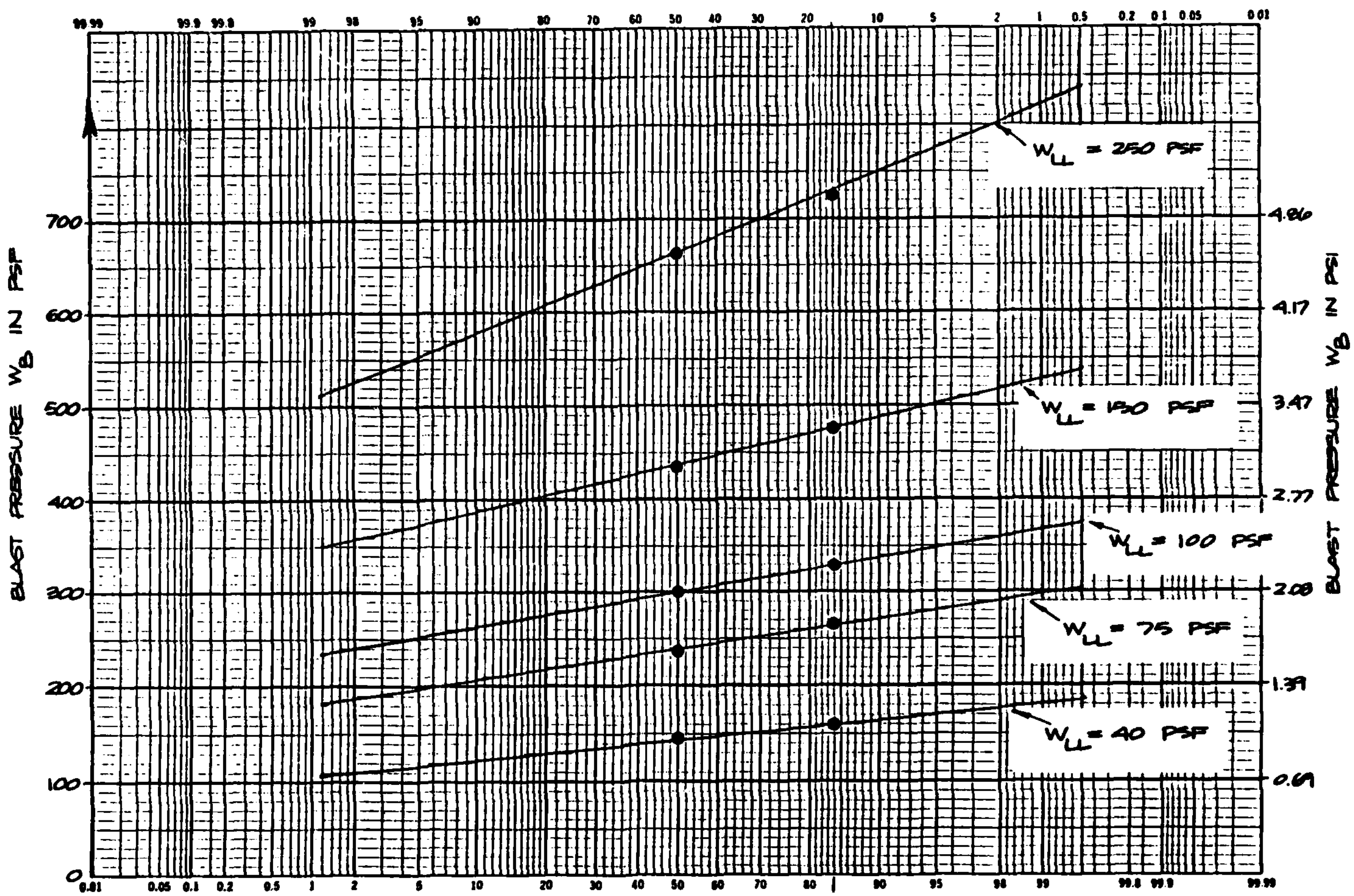


Fig. 5-5. Casualty Functions for Non-Upgraded Slabs at Stated Design Live Load Values.

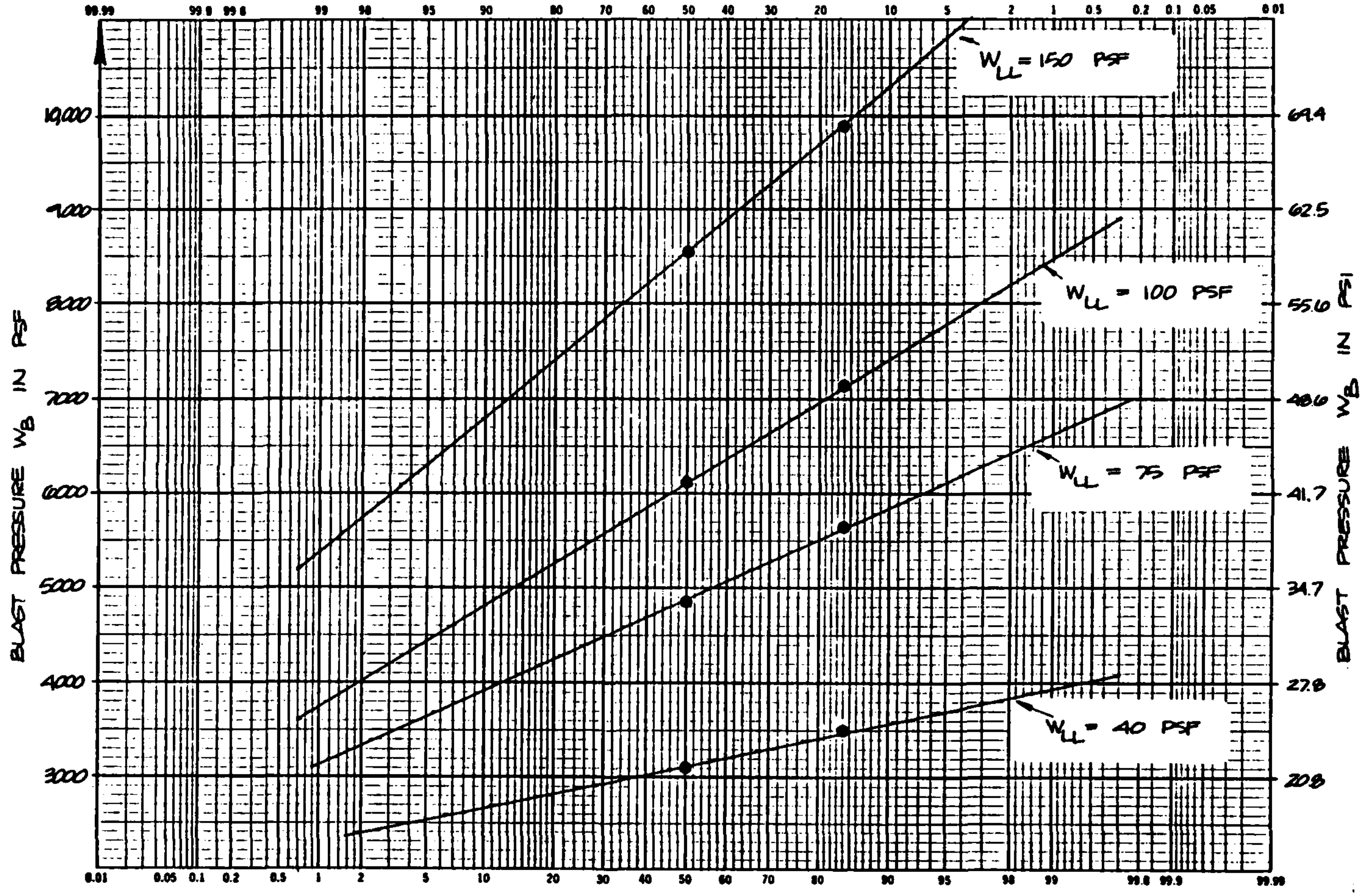


Fig. 5-6. Casualty Functions for Upgraded (L/4) Spacing) Slabs of Stated Design Live Load Values.

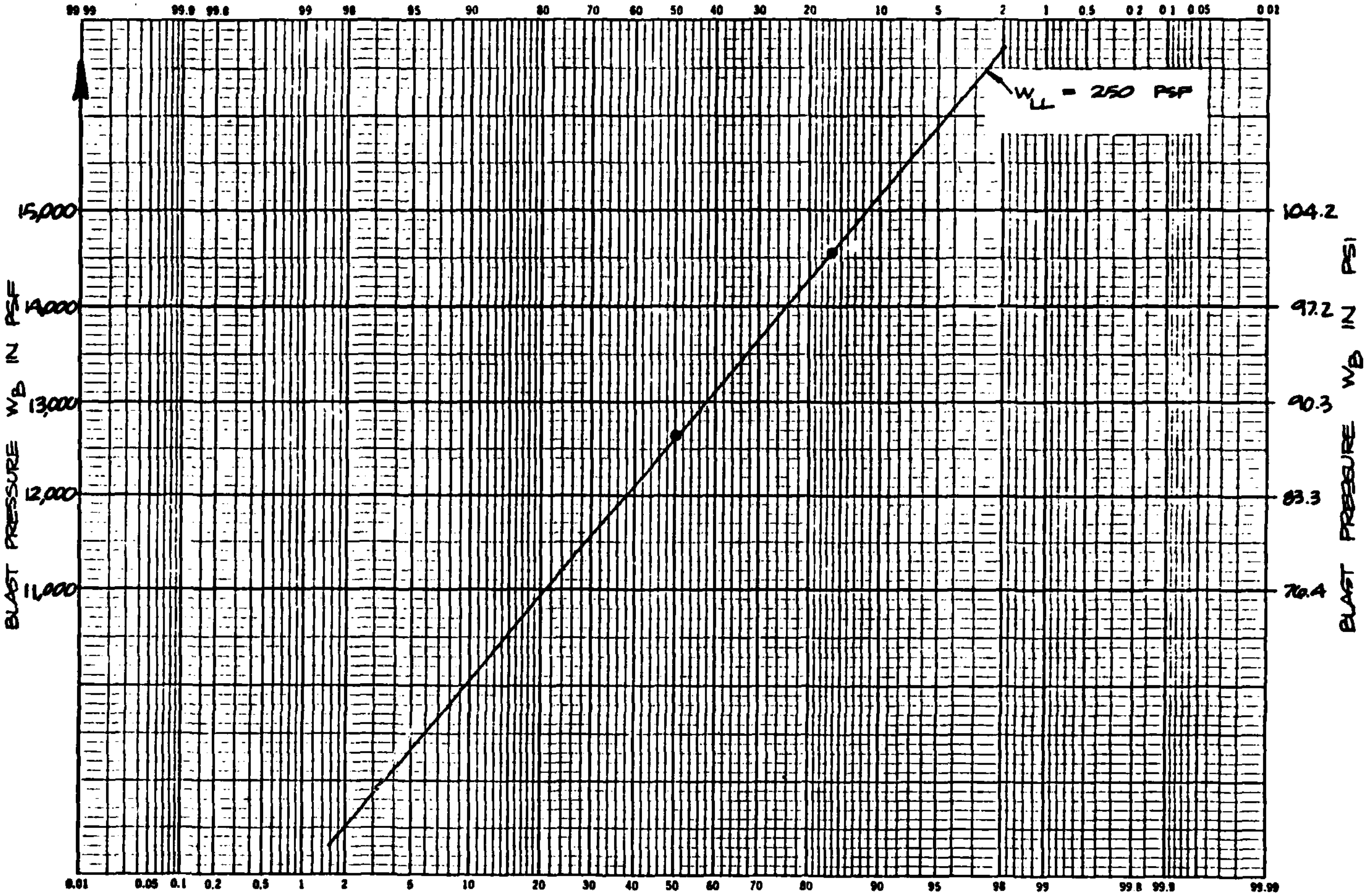


Fig. 5-6. (continued)

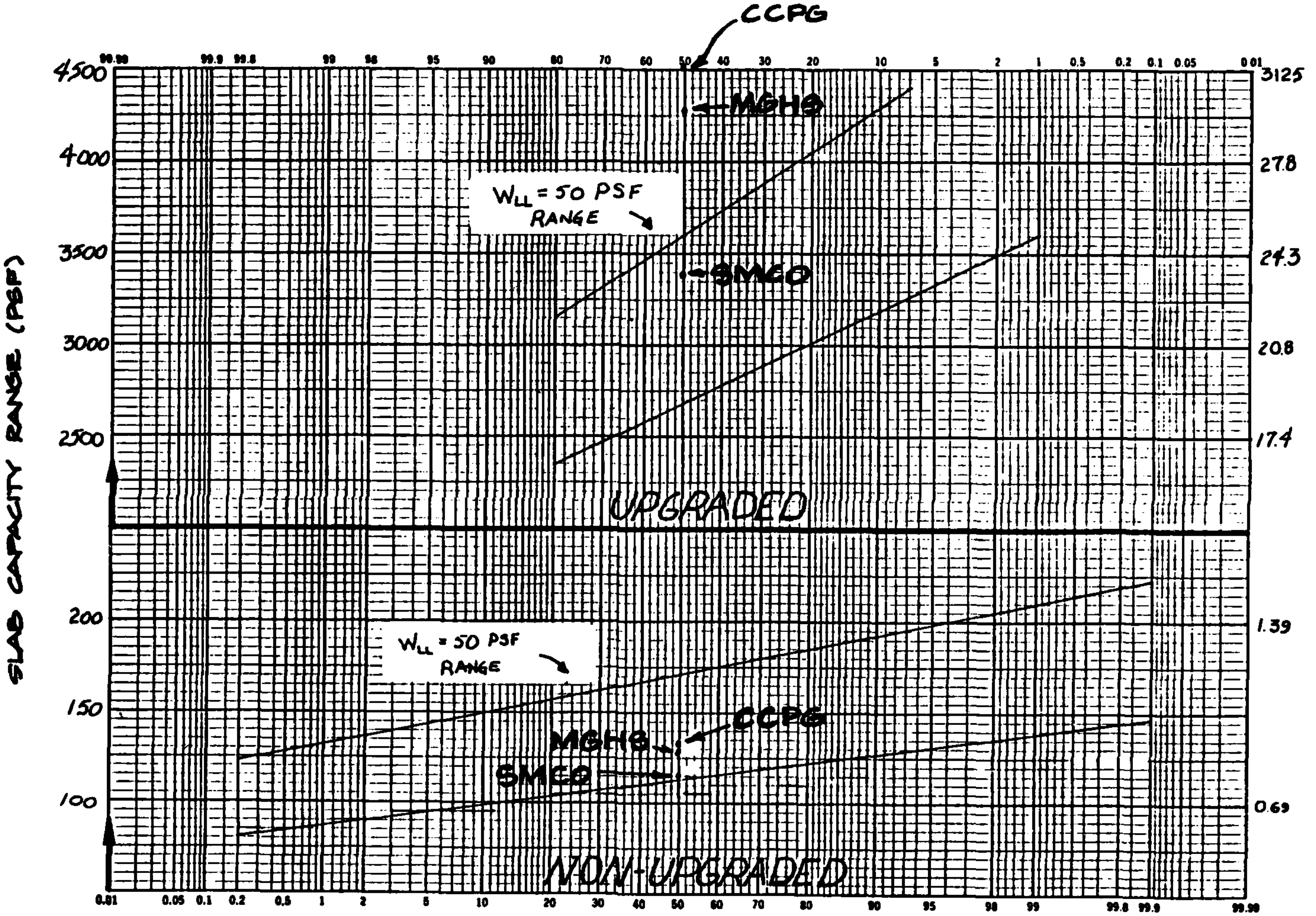


Fig. 5-7. Comparison of Approximate Casualty Functions With Specific Analytical Results.

Symbols and Notation for Section 5

w_{DL} = dead load in psf

w_{LL} = design (code specified) live load in psf

w_u = design strength = $1.4w_{DL} + 1.7w_{LL}$

w_v = load in psf at slab beam shear capacity ($2\ \sqrt{f'_c}$)

w_p = load in psf at slab punching shear capacity ($4\ \sqrt{f'_c}$), at either column or drop panel slab inter-face

w_F = load at flexural yield line mechanism capacity

w_i = imposed blast load capacity = strength capacity minus dead load ($w_F - w_{DL}$)

w'_F = upgraded capacity = 16 w_F for quarter point shoring system

w'_i = $w'_F - w_{DL} - w_{SL}$

w_B = given blast load level

w_{SL} = load of soil placed on slab for radiation protection in psf

Section 6
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

In the first year of this projected five-year program, a basis has been established for a damage function rating procedure that, when completed, should meet the needs of civil defense analysts desiring to perform comparative evaluations of U.S. civil defense shelter alternatives. In a modified form, this procedure is also expected to be responsive to the requirements of local civil defense planners. It should provide a simplified method that will permit them to determine the comparative ratings of the available local basements for upgrading to serve as civil defense shelters.

The damage function rating procedure, in its final form, is to be capable of taking into account all of the nuclear weapons effects that could be expected to cause casualties to those in the basement shelters under consideration. It should be usable for both risk area personnel shelters at 40-50 psi peak overpressure levels and host area shelters where the peak overpressures are not expected to exceed 2-3 psi. To provide flexibility in case of changing shelter requirements, it should also be usable from the lowest effects levels expected to cause casualties up to levels considerably above those currently specified for housing key workers in risk areas.

Ideally, the procedure should consider all of the significant features of the basement and its surroundings so as not to neglect any of the factors that could affect survival. These would include the structural elements of the basement, including openings into the basement and closures, as well as the building above the basement, and other objects in the vicinity, such as other buildings and the soil surrounding the basement. Measures to upgrade the basement for service as a shelter, such as the addition of shoring and closures and a layer of soil for radiation protection, shall also be included in the evaluation when applicable. In terms of weapon yields and burst conditions, the procedure should be usable for any anticipated strategic nuclear weapon yields and for either surface or air bursts.

In its present state of development, the procedure is more limited than the conceptual system that has just been described. The capability to predict overpressures that would cause structural collapse of the floor slab above the basement is comparatively well developed for both as-built and upgraded cases. Examples utilizing this capability are included in the present report for as-built and upgraded flat slab basements. Interim techniques for determining the survival of basement walls are also described and illustrated in the present report. The casualty functions resulting from the predicted structural failures are less well established, since the theoretical and experimental basis for these functions is very limited.

The capability to provide casualty functions for specific basements that take into account other casualty-causing nuclear weapons effects such as initial nuclear radiation, thermal radiation, and fires resulting from the attack, is less well developed. In principle, such effects can be predicted, if sufficient details are provided as to the burst, environmental, and target variables. However, a practical way of including these effects for the present purposes that will not require an unreasonable effort to input the conditions assumed or to compute results for the multiplicity of possible cases, remains to be developed.

CONCLUSIONS

The following conclusions are based on the results of the first year of the Damage Function Rating Procedure Research Program.

1. The capability to predict the overpressure levels at which structural collapse of the as-built or upgraded first floor slab will take place is comparatively well developed. This conclusion is applicable not only to flat slabs but to the other types of floor slabs scheduled to be analyzed in the remaining options of the research program.

2. The capability to determine levels of failure due to blast loading for other basement structural components including walls, framing, connections, and closures is less well developed.

3. In cases where the survival of those in the basement shelter is governed by the collapse of the first floor slab, the capability to predict casualties as a function of peak overpressure is reasonably well developed.

4. Especially for as-built basements, the capabilities to predict the levels inside the basement of primary blast, blast winds, initial nuclear radiation, thermal radiation, and other casualty causing effects are not well developed. Estimates can be made for specific simple geometries, but data gathering and computational efforts for realistic basements are prohibitively difficult.

5. For upgraded basements, the situation can be simplified to a certain extent by assuming that the upgrading for such effects as initial and residual radiation and the closing off of openings into the basement is adequate to prevent casualties, at least up to the overpressure at which structural collapse of the shored first floor slab takes place.

6. Since existing basements in general have very limited structural strength and inherent radiation shielding properties, upgrading will usually be required to enhance survival against the nuclear effects levels specified in current FEMA planning documents. However, for any specific basement, the upgrading design that may be chosen and installed in an emergency period is not known in advance. Because the upgrading designs will depend on the future local availability of materials for such purposes as shoring, improving radiation protection, and installing closures, it is not practical to completely specify them in advance of the emergency period. This limits the degree of certainty that the actual crisis period upgrading of a specific shelter will provide the survivability assumed in peacetime rating procedures.

7. If the procedure is to consider all reasonable possibilities, it could require rating a large number of alternative upgrading options. Another approach would be to attempt to determine the potential for upgrading of the as-built basement, and to refrain from the production of numerous damage/casualty functions for the various feasible upgrading options.

RECOMMENDATIONS

The following recommendations are offered, based on the the results of the first year's work:

1. The future development of the damage function rating procedure should be directed toward attainment of a practical capability to provide a credible casualty function for any potential basement shelter, to include not only the effect of slab failure but other potential structural damage and personnel casualty mechanisms.

2. In order to attain this capability, future efforts should emphasize practical methods to include other pertinent contributors to the casualty functions for basements besides fallout radiation and the collapse of the first floor slab. Depending on the extent and nature of the upgrading and other factors, the following may make significant contributions to the casualty function for a particular basement:

   a. Failure of structural elements other than the first floor slab.
   b. Initial nuclear radiation
   c. Primary and tertiary blast
   d. Thermal radiation (burns and fires)
   e. Secondary fires (from air blast)

3. The basic suitability of a basement to serve as a shelter is dependent upon many factors. The rating procedure for field use should attempt to quantify the factors involved so that reasonable decisions can be made. One significant determination is the practical potential for successfully upgrading the basement during a crisis period to survive specific effects levels. This should not be obscured by bland assumptions made without a realistic survey basis that certain levels of upgrading can be done successfully.

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APPENDIX A

REVIEW OF PREVIOUS SURVIVABILITY RESEARCH

REVIEW OF PREVIOUS SURVIVABILITY RESEARCH

INTRODUCTION

There has been a great deal of research that is related in some way to the present program to develop damage function and casualty function rating procedures. This includes much of the previous Civil Defense and Nuclear Weapons Effects research, especially that related to the survival and upgrading of shelters and that having to do with the casualty producing characteristics of the various nuclear weapons effects. Other categories of relevant research include studies relating to building structural failure mechanisms, and to injury production from various types of peacetime accidents. This discussion will be limited to previous FEMA (including DCPA and OCD) sponsored research on people survivability in civil defense shelters.

Referring to the FEMA work unit system, research work on shelter damage function and casualty function determination has been done under the 1600 - Shelter Systems Studies Project. Within that project, individual work units of interest have been accomplished under Task 1610 - Shelter Concept Studies and under Task 1620 - Evaluation of Partial Shelter Systems. A variety of individuals and organizations have carried out research in this subject area. Of the most direct relevance for the present purposes is the major portion of the overall work in the area that was performed by Dr. A. Longinow and his associates of the IIT Research Institute. It will be described briefly in the paragraphs that follow. However, for a more complete picture, the reader is referred to References 1-18, which discuss specific cases studied and results obtained over approximately a 15-year period in much more detail than will be possible here.

REVIEW OF INDIVIDUAL SURVIVABILITY STUDIES

The relevance for our purposes of the individual survivability studies undertaken by Dr. Longinow and his co-workers varies appreciably. This is true for

several reasons: they had differing objectives; the types of structures analyzed were different in the various reports; the studies were accomplished over about a 15-year period, so the analytical techniques utilized evolved accordingly; and the assumptions used in the studies also differed. For this reason, References 1-18 have been considered individually for the contribution that they can make to the present program to provide improved casualty functions for as-built and upgraded basements.

In Tables A-1 through A-12, relevant FEMA (DCPA, OCD) sponsored people survivability studies by Dr. A. Longinow and various co-authors at IIT Research Institute are considered individually. The tables summarize for each report: the general objectives of the study; the types of shelters and upgrading included; the peak overpressure ranges considered; the yields and heights of burst assumed; the nuclear effects included; the analytical methods used; and the structural failure mechanisms and personnel casualty mechanisms assumed. They also include an evaluation of the relevance of the report for present purposes. A few of the references are not summarized. Reference 1 was primarily concerned with cost comparisons for shelters. Reference 4 contains information on analyses of NFSS buildings, which has been consolidated with information in Reference 7. Reference 14 is a review of previous studies, which have been considered individually here. Reference 17 describes load tests of a wood floor over a home basement, so is outside of the main area of interest. References 6 and 10 concern survivability research but were not research final reports per se and are not included. (Note that the page numbers and report sections indicated in Tables A-1 through A-12 refer to the reports being summarized. Also, the summaries and interpretations provided, although we have attempted to follow the original reports closely, are ours rather than those of the original authors.)

Table A-1

Civil Defense Shelter Options for Fallout and Blast Protection (Single Purpose) by A. Longinow and O.J. Stepanek, IITRI, June 1968 (Ref. 2)

General Objectives of Study:
To develop data on shelter concepts, costs and protective capabilities of shelters capable of being grouped in shelter complexes and deployed in urban and/or peripheral regions. "Permanent" (single-purpose) structures requiring the skills, specialized equipment and communication and supply routes of the fabrication and construction industries were studied. These include: Reinforced concrete arch structures, Steel arch structures, Reinforced concrete rectangular structures, and Timber rectangular structures. (pg. S-1)

Types of Shelters and Upgrading Included:
The two basic structures considered were the arch and the rectangular type. The basic unit of each was designed to accommodate 500 persons at approximately 10 sq ft of floor space per person. The arch is of the two-level type having an internal radius of 17.5 ft and an overalll length of 82 ft. The basic rectangular unit consists of exterior and interior walls forming a gridwork, with the roof member spanning in two directions in the case of concrete and in one direction in the case of timber. (pg. 7) (Since these are single purpose shelters, upgrading is not an issue.)

Peak Overpressure Ranges: Each of the basic shelters was designed for fallout radiation alone as well as for fallout and 10, 20 and 30 psi free-field incident overpressures and associated effects of thermal and prompt nuclear radiation. With respect to fallout, a protection factor of 1000 was used. (pg. 16).	**Yields and Heights of Burst:** Not specified.
Nuclear Effects Included: Fallout radiation, blast, thermal radiation, and prompt nuclear radiation were considered in the design. (pg. 16). However, only blast seems to have been used in the actual analysis of the shelters. (pg. 82)	**Analytical Methods Used:** A simplified dynamic analysis of the structures was used. It is described on pp. 82-94 of the report.
Structural Failure Mechanisms: Not specified.	**Personnel Casualty Mechanisms:** Not specified.

Relevance to Present Effort:
Very little. The shelters analyzed were not of the as-built/upgraded basement type. The analytical techniques were in an early state of development.

Table A-2

Civil Defense Shelter Options: Deliberate Shelters, Volumes I and II by A. Longinow, J. Kalinowski, C.A. Kot and F. Salzberg, IITRI, December 1971 (Ref. 3)

General Objectives of Study:

1. Investigate the survivability potential for people located in selected classes of deliberate personnel shelters when subjected to the effects of nuclear weapons.
2. Determine sheltering costs for several feasible shelter options.
3. Select a rating system, which includes "people survivability" and "sheltering costs", whereby the performance of personnel shelters in a nuclear weapon environment may be rated and compared in a consistent and rational manner. (pg. S-1, Vol. I)

Types of Shelters and Upgrading Included:

Single Purpose Shelters (Low Level Weapon Effects Design) - four reinforced concrete arches, four steel arches, and four reinforced concrete rectangular shelters. Single Purpose Shelters (High Level Weapon Effects Design) two reinforced concrete arches. Dual-Purpose Shelters - three basement shelters, population 550 persons; three basement shelters, population 1100 persons; three parking garage shelters, three expressway grade separation shelters, and one subway passenger station (pg. 12, Vol. I). Since the shelters discussed in this report were designed either exclusively as shelters, or the designs were slanted for use as a shelter in addition to some other use, upgrading in the sense required for the basements we are considering is not included.

Peak Overpressure Ranges:

Single Purpose Shelters (Low Level Weapon Effects Design) - fallout only, 10, 20 and 30 psi. Single Purpose Shelters (High Level Weapon Effects Design) 100 and 150 psi. Dual-Purpose Shelters - 5, 25, and 50 psi designs, except subway station was conventional use design. (pg. 12, Vol. I)

Yields and Heights of Burst:

0.2, 0.5, I, and 10 Mt weapons. (pg. S-4, Vol. 1)

Nuclear Effects Included:

Air Blast only.

Initial and fallout radiation and primary and secondary fires are mentioned but were not used to determine the survivability ratings. (pp. 8, 18, Vol. I)

Analytical Methods Used:

When the structure is subjected to overpressure, the point of incipient failure of the shelter structure is determined. This is the point at which it has yielded so that plastic hinges are fully formed in key structural elements, such as a roof slab or an arch shell. Deflections in these components are several times their yield value. It is postulated, however, that these components are still connected and are capable of supporting their own as well as the initial surcharge dead load. The overpressure level at which the structure will experience catastrophic collapse is also determined. These points are connected by a straight line. It is assumed that there are 100 percent survivors at the overpressure point of incipient failure and 0 percent survivors at the overpressure point of catastrophic collapse. These points are connected by a straight line to produce a survivability rating for a shelter. (pp. 12-13, Vol. I).

Structural Failure Mechanisms:

Depends on the structure. See Chapter 2 of Volume II. and pg 13, Volume I)

Personnel Casualty Mechanisms:

Persons in the shelters are assumed to be killed when the collapsing shelter falls on them. (pg. 17, Vol. I)

Relevance to Present Effort:

Slight. The structures are not the as-built or upgraded basements we are to analyze. The analytical methods used were in an early stage of development compared to later reports by Longinow.

Table A-3

People Survivability in a Direct Effects Environment and Related Topics by A. Longinow, G. Ojdrovich, L. Bertram, and A. Wiedermann, IITRI, May 1973 (Ref-5)

General Objectives of Study:
This report describes a deterministic, computerized methodology for predicting the survivability (relative safety) of people located in conventional (NFSS) buildings when subjected to the direct effects of megaton range nuclear weapons. Individual effects considered include thermal radiation, prompt nuclear radiation, primary and secondary blast and debris. The computational process is described and illustrated by means of example problems. Related topics include: distribution of blast-initiated debris, analysis of special permanent and expedient shelters relative to blast effects and a cost and survivability comparison of above- and below-grade shelters. (pg. iii)

Types of Shelters and Upgrading Included:
Although examples are given in Chapter 2 of upper story and basement survivability curves for various specific (as-built) NFSS buildings, the report emphasis is on describing methodology rather than carrying analyses through in detail. In Chapter 3, debris distribution from a simplified two story building with no basement is analyzed. In Chapter 4, cost and survivability comparisons are made for a basic building with no basement and no shelter, the basic building with blast shelter at grade, the basic building with basement area but no shelter, and the basic building with shelter below grade. In Chapter 5, a set of 8 home basement and free standing permanent and expedient shelters is analyzed.

Peak Overpressure Ranges:
For the typical problem used to illustrate the prediction methodology, the range of overpressures of interest is defined as 0 to 16 psi. (pg. 23) For the debris distribution illustration, the building is assumed to be located at the 5 psi range. (pg. 53) For the cost and survivability comparisons, both shelters were designed for the dynamic effects of a 15 psi blast overpressure. (pg. 79) Expedient shelter blast resistance ranged to 10 psi for moderate damage and to 13 psi for collapse (pg. 94).

Yields and Heights of Burst:
The model is said to be capable of generating weapon effects when weapon size, height of burst, and range are put into it. (pg. 2). For the debris trajectory example, a 1 Mt weapon is specified. (pg. 53). For the cost and survivability example, shelter distance to ground zero examples for 1 Mt and 20 Mt were listed, but this was related to a peak incident overpressure of 15 psi. (pg. 81) The expedient shelter effects were related to megaton range nuclear weapons (pg. 91)

Nuclear Effects Included:
The model is described as considering only the direct (prompt) effects which occur in the Mach region of nuclear weapons. These effects are thermal radiation, prompt nuclear radiation and primary and secondary blast. (pp 14-15). On page 109, it is stated that the prompt nuclear radiaton dose prediction routine needs to be revised since original data are no longer current and new data are available.

Analytical Methods Used:
The computer program described in the report is a deterministic procedure with an input routine that accepts data on the weapon size, height of burst, and range from ground zero. Building description is input in terms of geometry and physical properties. Dose prediction routines determine the intensities of individual nuclear effects and casualty mechanisms. A routine is provided to relate each of the computed dose intensities to corresponding casualty criteria. (See pp. 17-37 and Appendix C).

Structural Failure Mechanisms:

Not provided in any detail.

Personnel Casualty Mechanisms:
Burn casualty (whole body) from thermal radiation, radiation casualty (whole body) from prompt nuclear radiation, blast casualty (pulmonary hemorrhage) from primary blast, and impact casualty (head, whole body), blunt or penetrating debris casualty (head, thorax, abdo[illegible] or limbs), and acceleration casualty (whole body), [illegible] from secondary blast. (pg. 15).

Relevance to Present Effort:
This report has limited relevance since it is concerned primarily with upper stories of buildings, expedient upgrading of basements of the types we are concerned with is not covered, and the model described is no longer current.

Table A-4

Survivability in a Direct Effects Environment (Analysis of 50 NFSS Buildings) by A. Longinow, IITRI, July 1974 (Ref. 7)

General Objective of Study: The objective of the study was twofold: 1. To analyze 25 existing buildings with the object of determining the extent of protection afforded in each when subjected to the direct effects of megaton range nuclear weapons and	2. To combine the results with those of another set of 25 buildings previously analyzed in a similar fashion and on the basis of combined statistics to develop a preliminary classification (ranking) system for conventional buildings in terms of protection afforded. (Abstract)
Types of Shelters and Upgrading Included: Separate survivability estimates are made for upper stories and basements of 50 NFSS buildings, although only 36 of the buildings had basements. Table 1 on	pp 12–14 lists the buildings with their survivability estimates, first floor types, and other information. Upgrading was not considered.
Peak Overpressure Ranges: Survivability estimates for people in upper stories are plotted for the range of 0–20 psi and for people in basements for the range of 0–40 psi, (pp 6–11, 15).	**Yields and Heights of Burst:** A 1 Mt surface burst (pg. 2)
Nuclear Effects Included: For upper stories – thermal radiation, prompt nuclear radiation, and blast (pg. 1) For basements – blast and prompt nuclear radiation (pg. 2)	**Analytical Methods Used:** The reader is told on page 2 that analysis methods used *in this study are described in detail* in another Longinow report (Ref. 3; See Table A–2)
Structural Failure Mechanisms: Not given in any detail. However, it appears from the information provided that upper story survival was governed significantly by wall failure levels and in basements was largely governed by floor slab failure.	**Personnel Casualty Mechanisms:** For upper stories, thermal radiation, prompt nuclear radiation and impacts produced by dynamic pressures and debris are included in the survivability estimates. (pg. 1) For basement areas, impacts produced by the collapse of the overhead floor system and prompt nuclear radiation are the two mechanisms considered. (pg. 2)

Relevance to Present Effort: The results for the 36 as–built basements analyzed are of some interest, although the detailed analyses are not included, which limits their utility. No upgrading was considered. Also, Dr. Longinow has stated (Ref 15 (See Table A–10)) that he has updated his analytical methods since this report was written.

Table A-5

Casualties Produced by Impact and Related Topics of People Survivability in a Direct Effects Environment by A. Longinow, E. Hahn, A. Wiedermann, and S. Citko, IITRI, August 1974 (Ref. 8)

General Objectives of Study:
Predicting the survivability (relative safety) of people located in (the upper stories of) conventional buildings when subjected to the direct effects of megaton range nuclear weapons. The emphasis is on impact casualties produced by the effects of blast. Casualty-producing effects considered include (1) dynamic pressures associated with the passage of the blast wave, which can cause people to lose balance, be rotated, translated terminating in impact on hard surfaces, and (2) debris produced by the breakup of structural and non-structural components when interacting with the blast wave. Other topics discussed in the report include a classification of shelter spaces, analysis of a (gable roof) fallout shelter against the effects of blast, feasibility of using large limestone mines as shelters and the analysis of an emergency operating center against the direct effects of nuclear weapons. (Abstract)

Types of Shelters and Upgrading Included:
In the main portion of the report, upper stories of buildings are considered as-built. (pp. 2-1 through 2-95). A classification system for shelters in upper stories of buildings is described on pp 3-1 through 3-14. In Chapter 4, a general analysis is made of an EOC located in a police administration and public service building. The EOC is in the lower level, but the basement is partially below and partially above grade. Since it would be difficult to harden for blast, only general suggestions are made for upgrading it. The fallout shelter is a wood frame structure and is mostly above grade. Large limestone mines are discussed in Appendix B.

Peak Overpressure Ranges:
For the analysis described in chapter 2, overpressures used were for a 1 Mt weapon and ranged from 2 to 20 psi in increments of 2 psi. (pg. 2-5) For the EOC, the range of interest is indicated as 1 to 15 psi (pg. 4-9) and overhead floor system failure overpressures up to 25 psi are listed. (pg. 4-28) For the mines, 20 psi (pg. B-1)

Yields and Heights of Burst:
A I Mt weapon for the impact analysis (pg. 2-5). A single megaton-range nuclear weapon for the EOC (pg. 4-1). A 1 Mt surface burst for the mine analysis (pg B-1).

Nuclear Effects Included:
Blast for the impact casualty analysis. (pg. 2-2). Blast, plus limited discussion of other possible effects for the EOC (pg. 4-9). The "BUILDINGS computer program described in Appendix A and intended to predict the survivability of people located in the upper stories of conventional buildings, has provision for considering translational effects produced by dynamic pressure, debris effects, prompt nuclear radiation, and thermal radiation. (pg. A-5). Blast for the mines (pg. B-1)

Analytical Methods Used:
For the analysis of impact casualties to which people in the upper stories of buildings may be subjected when exposed to the blast effects of nuclear weapons, catalogs of people trajectories and of debris trajectories were generated and compared for interactions. Impact velocities between people and debris and of people with floor, wall and ground surface were determined and compared with casualty criteria to determine the number of survivors. (pp. 2-2 through 2-17, and Appendix A) Details of the analytical methods used for the EOC are not provided.

Table A-5 (continued)

Structural Failure Mechanisms:	Personnel Casualty Mechanisms:
Simplified wall crack patterns were developed, partially based on experimental results from the URS shock tunnel tests. These were used to generate debris piece patterns. (pp. 2-13 through 2-29). For the fallout shelter, diffraction and drag loading effects are considered and modes of failure include rafters failing in bending and longitudinal shear, columns buckling and crushing, ridge beam failing in bending, rafter notch failing in compression and front and back walls failing in longitudinal shear and bending. (pp. 5-1, 5-4).	Impact with debris, and with floor, wall and ground surface (pg. 2-3). Debris from structural failure for fallout shelter (pg. A-6). Incident and reflected blast wave for mines (pg. B-10).

Relevance to Present Effort:
The methodology used by Dr. Longinow for determining impact casualties is of interest to the present effort, although it concerns upper stories of buildings, rather than basements.

Table A-6

Debris Motion and Injury Relationships in all Hazard Environments by A. Longinow, A. Wiedermann, S. Citko, and N. Iwankiw, IITRI, July 1976 (Ref. 9)

General Objectives of Study:

To produce casualty (injury and fatality) relationships for people located in conventional buildings when subjected to hazards produced by man-made and natural disaster environments. Although the emphasis is on the direct effects produced by megaton-range nuclear weapons, some consideration is given to debris effects produced by a tornado.

Types of Shelters and Upgrading Included:

Conventional basements of the one-way slab and two-way slab (flat plate and flat slab) types were considered. The one-way reinforced concrete slabs considered included two basic types, i.e., simple span simply supported and two-span continuous over a central support. Design parameters were varied over the following range: Span length (simply supported) - 12, 16, 20 ft. Span length (two-span continuous) - 16, 20, 24, 28 ft. Design live load - 50, 80, 125, 250 psf. Ultimate compressive strength of concrete - 3, 4 ksi. Yield strength of reinforcing steel - 40, 60 ksi. A clear ceiling height of 8 ft was kept constant. (pp. 53-54) For the square two-way slabs considered here, various combinations of flat slab panel span and live load were considered for flat slabs (plates) without drop panels, for flat slabs with drop panels, and for flat slabs with drop panels and column capitals. All combinations of span = 16, 20, 24, and 28 ft and live load = 50, 80, 125, and 250 psf were considered for one or more types of flat slab. Story height was assumed to be 12 ft. (pp. 67-68) The basement designs were considered in the as-built condition with no upgrading.

Peak Overpressure Ranges:

The analyses were performed by increasing overpressures until slab failures were produced, rather than by determining effects as specific overpressures. Upper bound overpressure for ultimate collapse of the strongest one-way slab considered was 24 psi, and for the strongest flat slab was 13.5 psi. (pp. 81, 92)

Yields and Heights of Burst:

A single megaton range nuclear weapon in its Mach region. (pg. 54)

Nuclear Effects Included:

Air blast. (pg. 51)

Analytical Methods Used:

Slabs were analyzed to identify reasonable collapse mechanisms and to determine corresponding collapse overpressures when subjected to the blast effects of a single, megaton-range nuclear weapon in its Mach region. Collapse mechanisms were identified based on yield-line theory, available experimental data, and engineering judgment. (pp. 54-66 and 73-76)

Structural Failure Mechanisms:

Symmetric and unsymmetric collapse modes were assumed for one-way slabs resulting from plastic hinges developing at midspan. (pp. 54-66) For the two-way slabs, it was assumed that failure in flexure would occur with yield lines forming along the lines of maximum moment or in shear due to punching at the columns. (pp. 73-76)

Personnel Casualty Mechanisms:

Debris from the breakup of the overhead basement slab. (pg. 80) Flow induced translational effects in basement shelters were considered in a separate chapter (Chapter 4) of the report, but the results weren't integrated with those discussed here.

Relevance to Present Effort:

Moderate. A portion of the analysis presented considers flat slab basements, the structural type we are considering in the first year of the present effort. However, Dr. Longinow has since updated his analytical techniques from those presented here, the basement designs considered are simple, idealized cases, and the only personnel casualty mechanism considered is debris from the breakup of the first floor slab.

Table A-7

Relative Structural Considerations for Protection from Injury and Fatality at Various Overpressures by A. Longinow and A. Wiedermann, IITRI, June 1977 (Ref. 11)

General Objectives of Study:
Producing casualty (injury and fatality) relationships for people located in conventional buildings when subjected to the direct effects produced by nuclear weapons. People survivability estimates of multi-story buildings subjected to blast effects of megaton range nuclear weapons are presented. Results are for full basements with two-way reinforced concrete overhead floor systems supported on steel beams. The transient velocity field that may exist in such basements is modeled and used to determine the response of individuals located within. (Abstract)

Types of Shelter and Upgrading Included:
To study survivability in basement with two-way reinforced concrete overhead floor systems supported on steel beams, typical square interior panels were designed based on all combinations of the following parameters:

Span: 12, 16, 20, 24, and 28 ft.

Ultimate compressive stress of concrete: 3000 and 4000 psi; Yield strength of reinforcing steel; 40,000 and 60,000 psi; Design live load: 50, 80, 100, 125, 200 and 250 psf. No upgrading was considered. (pg 8)

Peak Overpressure Ranges:
Upper bound of survivability estimate extends to about 7.5 psi. (pg. 17)

Yields and Heights of Burst:
A single megaton-range nuclear weapon. (pg. 11)

Nuclear Effects Included:
Blast. (pg. 7)

Analytical Methods Used:
Overpressures producing failure in the slab and the supporting beams were determined using procedures described in Chapters 7 and 8 of the "Air Force Design Manual; Principles and Practices for Design of Hardened Structures" by N.M. Newmark and J.D. Haltiwanger and in Chapter 7 of "Introduction to Structural Dynamics" by J.M. Biggs. (pg. 11)

Structural Failure Mechanisms:
Shear and flexural failure of the slabs and beams, and failure of the beam connections through exceeding the combined shear capacity of the bolts, bearing capacity of the beam web, bearing capacity of column web or flange, or bearing capacity of simple connection support angles. (pp. 11-12)

Personnel Casualty Mechanisms:
Impact produced by spalled chunks of concrete from the overhead slab and the collapse of the slab itself. (pg. 14) A separate portion of the report considers casualties caused by the transient velocity field produced by the air blast. (pp. 19-92)

Relevance to Present Effort:
Limited, although the basement type considered here by IITRI is scheduled to be analyzed as part of the third year's effort. Only a brief chapter summarizing the analysis is presented in this report. A separate longer section of the report discusses flow induced translational effects on persons in basement shelters. However, the shelters considered are different from those analyzed for slab failure and the results for the shelters analyzed structurally do not include any casualties due to translation of people by blast winds.

Table A-8

Survivability in Crisis Upgraded Shelters, by A. Longinow, IITRI, February 1978 (Ref. 12)

General Objectives of Study:

The study was concerned with determining the survivability potential of people remaining in high risk areas when subjected to a nuclear weapon attack. Specifically, it was concerned with: Reviewing techniques that may be used for upgrading shelters in high risk areas during the crisis period; determining the "people survivability" potential of shelters upgraded in this manner; and thus producing criteria for projecting "people survivability estimates" for high risk population centers. (pg. 2)

Types of Shelters and Upgrading Included:

(1) Basement of a reinforced concrete "flat plate" office building. This is a four-story reinforced-concrete flat plate structure with reinforced concrete walls, a basement and a brick exterior. This building contains a personnel shelter located in a portion of the ground floor (basement). The ground floor is partially above and partially below grade. The shelter was designed to resist a blast overpressure of 4 psi. This design overpressure applies to the peripheral walls, the overhead (flat plate) slab and closures. The walls of the shelter envelope are windowless. Upgrading consisted of installing timbers around the columns to strengthen the structure against failure from punching shear at the columns.

(2) Basement of a 10-story steel framed apartment building with masonry walls and a full basement. It was designed in accordance with the Chicago, Illinois Building Code. This study considered the sheltering potential of the basement in its as-built and upgraded states. Upgrading consisted of providing closures for stairwell and elevator shafts and increasing the structural resistance of the beam-to-column connections with timbers; of the edge beams with timbers; and of the slabs with a timber crib.

(3) Emergency Operating Center, Livermore, California (basement). This is a one-story load-bearing reinforced-concrete masonry structure with a full basement. The basement has a reinforced concrete overhead slab and reinforced concrete peripheral walls. Interior basement walls consist of reinforced concrete masonry. Upgrading consists of closing off the openings into the basement.

(4) Hamilton Air Force Base, Building 424 (basement). This is a three-story reinforced-concrete frame building with reinforced concrete floor slab. The building has a basement which is partially above grade and has numerous windows. Upgrading consists of closing off all openings and providing intermediate supports for the beams.

(5) Middlefield Parking Garage (underground portion). This structure consists of a two-story wood frame building with street level and underground parking areas. The underground garage is fully buried and is located primarily below the street level parking area. Its roof system consists of one-way reinforced concrete joists supported by reinforced concrete girders spanning between circular reinforced concrete columns. Upgrading consists of strengthening the joists by providing a line of midspan supports. Further upgrading would be to also strengthen the girders by timber cribs, columns, or some combination.

(6) West Pavilion, Stanford University Hospital, Stanford California (basement). The West Pavilion 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 building has a reinforced concrete frame with exterior columns and interior reinforced concrete load-bearing walls. The floor system consists of transverse reinforced concrete tube slabs but with solid slabs along transverse column lines. Upgrading consists of providing closures for each of the openings.

Note: Each shelter was evaluated in its as-built and upgraded conditions to the extent made possible by available data. (pp. 6-7, 11, 51, 69, 74, 79, 84)

Table A-8 (continued)

Peak Overpressure Ranges: Survivability levels ranged from under 2 psi to approximately 30 psi for the various buildings. (pg. 93).	**Yields and Heights of Burst:** A single megaton range nuclear weapon exploded at the surface. (pg. 7)
Nuclear Effects Included: Blast and prompt nuclear radiation. However, it is noted in the report that the procedure used for calculating the intensity of prompt nuclear radiation at the location of the structure is not current. Recent studies performed for DCPA have produced a more up to date procedure. For this reason, prompt nuclear radiation hazards estimated in this report may be more severe than is actually the case. (pg. 7)	**Analytical Methods Used:** (1) For the office building basement, determined the shear capacity and flexural capacity to produce yielding and to produce collapse of the flat plate floor system (pp. 15-33). (2) For the apartment building basement, determined overpressures at which the floor system will fail first either in flexure of the slab or the supporting beams or shear failure of the connections. (pp. 39-61) (3) through (6) The other shelters discussed in this report had been analyzed previously by H. L. Murphy, and Longinow used his results for collapse loads. (pg. 7).
Structural Failure Mechanisms: (1) As built - shear punching at the columns (pg. 31). Upgraded - flexure of the slab (pg. 33). (2) As built - shear of beam to column connections (pg. 52). Upgraded - flexural failure of the slab (pg. 61).	**Personnel Casualty Mechanisms:** Debris from the collapse of the floor slab was the only personnel casualty mechanism contributing to the people survivability estimates, except that ionizing radiation made some contribution in the upgraded apartment building basement and Livermore EOC casualty estimates. (pp. 16, 63, 70, 75, 81, 85)
Relevance to Present Effort: The basement analyses presented, although not analyzed by Longinow's current techniques, are pertinent to the kinds of structures we are expected to be able to analyze.	

Table A-9

Survivability on the Fringe of High Risk Areas by A. Longinow, IITRI, October 1978 (Ref. 13)

General Objectives of Study:

To produce data on the basis of which questions such as the following could be answered: l. What level of shelter is required in host (low level of risk) areas? 2. What level of protection is required in fringe areas i.e., areas in direct vicinity of high risk areas? The study was concerned with the development of survivability functions for people in regions with overpressures of 2 psi and less, caused by megaton range nuclear weapon detonations. (pg. 1)

Types of Shelters and Upgrading Included:

1. Basement of a school classroom building with a precast/prestressed overhead floor system consisting of hollow core deck sections. Corridor and peripheral walls in the basement are of reinforced concrete and are 9 inches thick. The sheltering potential of the basement was evaluated for four conditions: as-built; as-built plus l ft of soil cover for fallout protection; upgraded without soil cover; and upgraded, with soil cover. The upgrading involved blocking all openings and placing a timber bracing (support) system in all basement classroom areas halfway between the longitudinal corridor walls and the longitudinal basement walls. 2. Basement of a wood framed, single family residence. 3. Special purpose, home basement shelters (concrete block shelter, lean-to shelter, rigid frame shelter, and reinforced concrete block shelter). 4. Special purpose, outside shelters (aboveground A-frame shelter, plywood box shelter, wood grate roof shelter, and gable roof shelter). (pp. 7, 10-14)

Peak Overpressure Ranges:

Somewhat in excess of 2 psi, as well as below 2 psi. 1-2 psi and 2-5 psi (pp. 3,6)

Yields and Heights of Burst:

A single megaton range nuclear weapon exploded near the ground surface. (pg. 10)

Nuclear Effects Included:

Air blast (pp. 14, 40, 49)

Analytical Methods Used:

Dynamic load carrying capacity of the school basement precast, prestressed overhead floor unit was calculated. This was based on flexural and shear failure as a result of airblast and static loadings for the specified as-built and upgraded cases. (pp. 17-27)

Structural Failure Mechanisms:

Shear and flexural failure of the precast/prestressed overhead floor system. The possibility that the entranceway closures fail, but the floor system remains, is mentioned but not analyzed in detail since it is stated that the effects of closure failure would be expected to be less severe. (pg. 17)

Personnel Casualty Mechanisms:

The primary casualty mechanism was found to be debris from the breakup of the overhead floor system. The mode of failure was found to be shear of the individual precast, prestressed concrete units used in the construction of this building. People were assumed to be uniformly distributed in all classroom and corridor areas. (pg. 14)

Relevance to Present Effort:

Precast systems are scheduled to be studied as part of the fourth year's effort. The other types are outside our scope of work.

Table A-10

Probability of People Survival in a Nuclear Weapon Blast Environment by A. Longinow, IITRI, May 1980 (Ref. 15)

General Objective of Study:
This study was concerned with updating the current methodology for predicting the survivability of people in basement type shelters when subjected to the prompt effects of nuclear weapons, and of developing casualty functions for them. The current methodology (published in 1974) was updated in probabilistic terms and a computer program was written and is included in the report. (pg 1)

Types of Shelters and Upgrading Included:
(1) Three single-purpose shelters designed to resist overpressures of 20, 30 and 40 psi produced by the detonation of a 1 Mt weapon.
(2) One dual-purpose shelter designed to resist 15 psi from a 1 Mt weapon.
(3) One expediently upgraded one-way slab type basement shelter designed to resist 250 lb/sq ft. The upgrading concept consisted of a timber framework which isolated a square slab with a center to center span of 16 ft in each direction. With this arrangement, the original one-way slab becomes a two-way slab in the enclosed region.(pp. 2, 43)

Peak Overpressure Ranges:
See above. Some of the damage and casualty function curves extend to 150 psi peak overpressure.

Yields and Heights of Burst:
A 1 Mt weapon near the ground surface. (Abstract)

Nuclear Effects Included:
Air blast.
Prompt nuclear radiation was considered in the study. However, due to the complexity of the problem, it was not possible to include this effect in the updated methodology on the same level of detail as was done for the airblast effects. Nuclear radiation was therefore neglected. (pg. 1)

Analytical Methods Used:
Determine the probability of roof slab collapse for a given airblast loading, and on this basis predict the probability of people survival against the effects of slab collapse and blast pressures. The roof slab collapse analysis takes into account the variability of material and geometric parameters of the slab and the variability in the peak overpressure and the positive phase duration of the blast load.

Structural Failure Mechanisms:
Analysis is limited to rectangular, reinforced concrete shelters whose roof slab (first floor slab) is the weakest structural component of the shelter. Walls, foundations and the closure are not considered. (pg. 113) Slab failure mechanisms are shown in sketches on pp. 35, 62, and 67.

Personnel Casualty Mechanisms:
The probability of survival calculation considers the following casualty mechanisms:

1. Debris from the collapse of the slab,
2. Primary blast. (pg. 113)

Relevance to Present Effort:
The single-purpose shelters and the dual-purpose shelter analyzed in the study, since they were designed to survive specified levels of airblast loading, are somewhat outside the scope of the present study. The expediently upgraded basement is directly relevant, although the upgrading scheme considered is somewhat unusual.

Table A-11

Damage Functions for Upgraded Shelters by A. Longinow, M-Y. Wu, and J. Mohammadi, IITRI, January 1982 (Draft) (Ref. 17)

General Objectives of Study:
This study was concerned with predicting the probability of survival of people located in expediently upgraded conventional basements when subjected to the blast effects of a 1 Mt weapon detonated near the ground surface. Two categories of basements are considered, i.e., basements of engineered buildings and basements of single family residences. (pg. 1)

Types of Shelters and Upgrading Included:
(1) Low rise engineered buildings with basements, with the first floor at grade, and the basement walls not directly exposed to the blast load. The first floor slab was designed as a one-way system for live loads in the range of 50 to 250 psf. A total of twelve separate cases representing three different span lengths (12, 16, and 20 ft) and four different design live loads (50, 80, 125, and 250 psf) were considered. Each of the twelve basements was analyzed as expediently upgraded using four different upgrading schemes. As used in this study, an expedient upgrading scheme involves supporting the first floor slab and blocking off all openings into the basement. (pp. 1, 2, 36)

(2) Four conventional wood frame residences with basements. These are real buildings whose plans were obtained from engineer/architect offices. Each building was evaluated as upgraded using the studwall scheme. Then, two of the basements were reevaluated using the post and beam concept. The process was repeated by assuming that 1 ft of soil would be placed over the first floor for radiation protection. Placing 2 ft of soil would significantly affect the strength of the floor system. The case involving 2 ft of soil was, therefore, not considered. (pg. 2)

Peak Overpressure Ranges:
Varies - up to about 50 psi in some cases. See Appendix C.

Yields and Heights of Burst:
A 1 Mt weapons detonated near the ground surface. (pg. 1)

Nuclear Effects Included:

Air blast (pg. 40).

Analytical Methods Used:
The analysis procedure consists of two parts. The first part is a probabilistic structural analysis which determines the probability of shelter failure (collapse). The second part is a probabilistic people survival analysis which makes use of the probability of structural failure results. (pp. 2-3)

(1) For the reinforced concrete slabs, probabilities of failure based on flexure and shear when acting independently of each other were determined. Since it is not known how these failure modes correlate, the actual failure probability was bounded. Upper and lower bounds on the failure probability of the slab were computed and shown on curves of probability vs free-field overpressure in psi. (pp. 40-49)

(2) For the wood frame residences, calculations were made leading to the determination of the probability of failure of the expediently upgraded floor system consisting of joists, girder, columns, and stud walls as described in Appendix A of the report.

Table A-11 (continued)

Structural Failure Mechanisms:	Personnel Casualty Mechanisms:
(1) Shear and flexure failure of the slab for the reinforced concrete first floor slabs. (pp. 28-29) (2) For the wood frame houses, failure of the expediently upgraded floor system consisting of joists, girder, columns, and stud walls. (Appendix A)	(1) For the reinforced concrete basements, casualty mechanisms included debris from the collapse of the overhead slab and primary blast. (pg. 40) (2) For the single family residences, the only casualty mechanism considered was debris from the breakup and collapse of the floor system into the basement area. (pg. 85)

Relevance to Present Effort:
The analysis of reinforced concrete one way slabs is relevant, although we are to include other casualty mechanisms besides debris from the collapse of the overhead slab and primary blast. Longinow recommends including in the computer program other structural components (flat slabs, flat plates, one-way slabs, beams (steel, reinforced concrete), columns (steel, reinforced concrete) composite steel and concrete systems, and masonry systems), other weapons effects hazards (prompt nuclear radiation, fallout radiation, ground shock, and fires), and improved casualty data. (pg. 53)

Table A-12

Assessment of Combined Effects of Blast and Fire on Personnel Survivability, by A. Longinow, T.E. Waterman, and A.N. Takata, IITRI, June 1982. (Ref. 18)

General Objectives of Study:
(1) to perform a preliminary analysis of hazards to sheltered personnel in a blast–fire environment produced by the detonation of a 1 Mt nuclear weapon near the ground surface, and (2) to lay the basic ground work for developing a consistent, formal methodology for estimating the probability of people survival in a blast–fire environment. (Abstract)

Types of Shelters and Upgrading Included:
(l) Conventional basement of the "TEAPOT" house strengthened to provide additional blast protection. This includes strengthening the floor system over the basement with additional supports for joists and girders, blocking of windows and doors leading into the basement and mounding the structure with soil up to the first floor level. A mechanical ventilation system is also assumed to be provided. (2) Preengineered (slanted) dual–purpose shelter. In this case, instead of a wood joist floor system over the basement, the residential building is assumed to have a reinforced concrete slab. The peripheral walls are concrete block as is the case with the TEAPOT house. Window wells and doors are adequately blocked off, the structure is mounded with soil to the first floor level and a mechanical ventilation system is provided. (3) Expedient, single purpose buried pole–type shelter placed in an open area behind a residence in the rearmost portion of the back yard. (pg. 126)

Peak Overpressure Ranges:
(1) >3.5 psi (up to less than about 10 psi) – Severe damage (buildings destroyed). (2) 2.0 to 3.5 psi – Moderate damage (buildings standing with major wall/roof damage). (3) <2.0 psi – Negligible damage (broken windows or none). (pg. 69, 130)

Yields and Heights of Burst:
A l Mt nuclear near–surface burst (pg. 1)

Nuclear Effects Included:
Blast, prompt nuclear radiation, thermal radiation, fallout radiation (pg. 130) Only blast and thermal effects discussed in any detail.

Analytical Methods Used:
A computerized airblast debris analysis program previously used by IITRI (pg. 16). The IITRI fire model (pg 68). Blast/fire interactions based on the McAuliff and Moll study, as modified by IITRI (pg 72). Some of the results are presented in general terms only. (pp. 126–131)

Structural Failure Mechanisms:
Structural collapse considered but not defined in detail. Also considers fire in structures/debris above or around shelter. (pg. 130)

Personnel Casualty Mechanisms:
Primary blast (ruled out due to <10 psi). Injury from debris due to structural collapse. Dynamic pressure (ruled out for these basements). Prompt nuclear radiation (listed but apparently not considered significant at these overpressure levels). Fire – heat, toxic gases. Fallout radiation (listed but not discussed). (pp. 126–131)

Other Features:
Only very general comments are made as to how the shelters described are damaged or how the persons in them are injured. No damage or casualty functions are presented. (pp. 126–131)

Relevance to Present Effort:
Limited. As stated in the report, this is a preliminary analysis of hazards to sheltered personnel in a blast–fire environment. (pg 132) Only very general comments are made as to how the shelters described are damaged or how the persons in them are injured. No damage or casualty functions are presented. (pp. 126–131)

GENERAL COMMENTS ON LONGINOW STUDIES

The tables just presented have summarized currently demonstrated capabilities for the determination of casualty and damage functions for basement and other shelters, the types of shelters analyzed, the nuclear weapons effects considered, the structural failure mechanisms, and the casualty mechanisms considered and other significant aspects of the IITRI work. It is reiterated that the work on determination of damage functions and casualty functions performed by Dr. A. Longinow and his associates at IITRI was broader in scope than the present effort. Thus, portions of it are not directly pertinent. Some of it concerned shelters in the upper stories of buildings, single purpose shelters, expedient shelters apart from buildings, shelters in wood frame structures, and other situations not directly related to determining casualty functions in as-built or upgraded basements of the types specified in the present scope of our work. However, much of this research is also of interest to this program since it indicates Dr. Longinow's methods for handling various aspects of the determination of casualty functions for shelters.

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APPENDIX B

BASEMENT STRUCTURAL SYSTEMS

BASEMENT STRUCTURAL SYSTEMS

INTRODUCTION

This Appendix includes a general discussion of building elements and of the connections between those elements. This discussion is based on the state of the art in prediction of building element failures and recent work on failure predictions using computer analysis. Some of this material has been published in two earlier SSI reports, Refs. 1 and 2. As presented here, it has been modified to present those portions that are applicable to basements of the types we are considering.

STRUCTURAL SYSTEM ELEMENTS

The majority of basements to be ranked will be framed construction types. Figure B-1 illustrates one method of categorizing building construction types. Mass construction is found in some older buildings, but in general is no longer used, except for poured-in-place concrete in many foundation areas of both concrete and steel framed structures. For this reason, concrete construction appears on Figure B-1 under both mass and framed construction types. Typical framed concrete construction details are shown in Figure B-2 for two types, flat slab and beam-and-girder construction. Figure B-2 also shows typical methods to upgrade basement shelters with shoring and soil for radiation protection shielding over the basement ceiling.

There are seven primary structural elements that are important to the integrity of a basement structure and are, therefore, to be included in the development of a rating procedure. Figure B-2 shows many of the elements listed below, which are described in more detail in subsequent paragraphs:

- o Ceiling (basement)
- o Floor (basement)
- o Exterior walls (both backfilled and above grade)
- o Interior walls (bearing and non-bearing)
- o Framing system (beams, girders, columns)
- o Connections (between system elements)
- o Openings (doors, windows, stairwells, ducts)

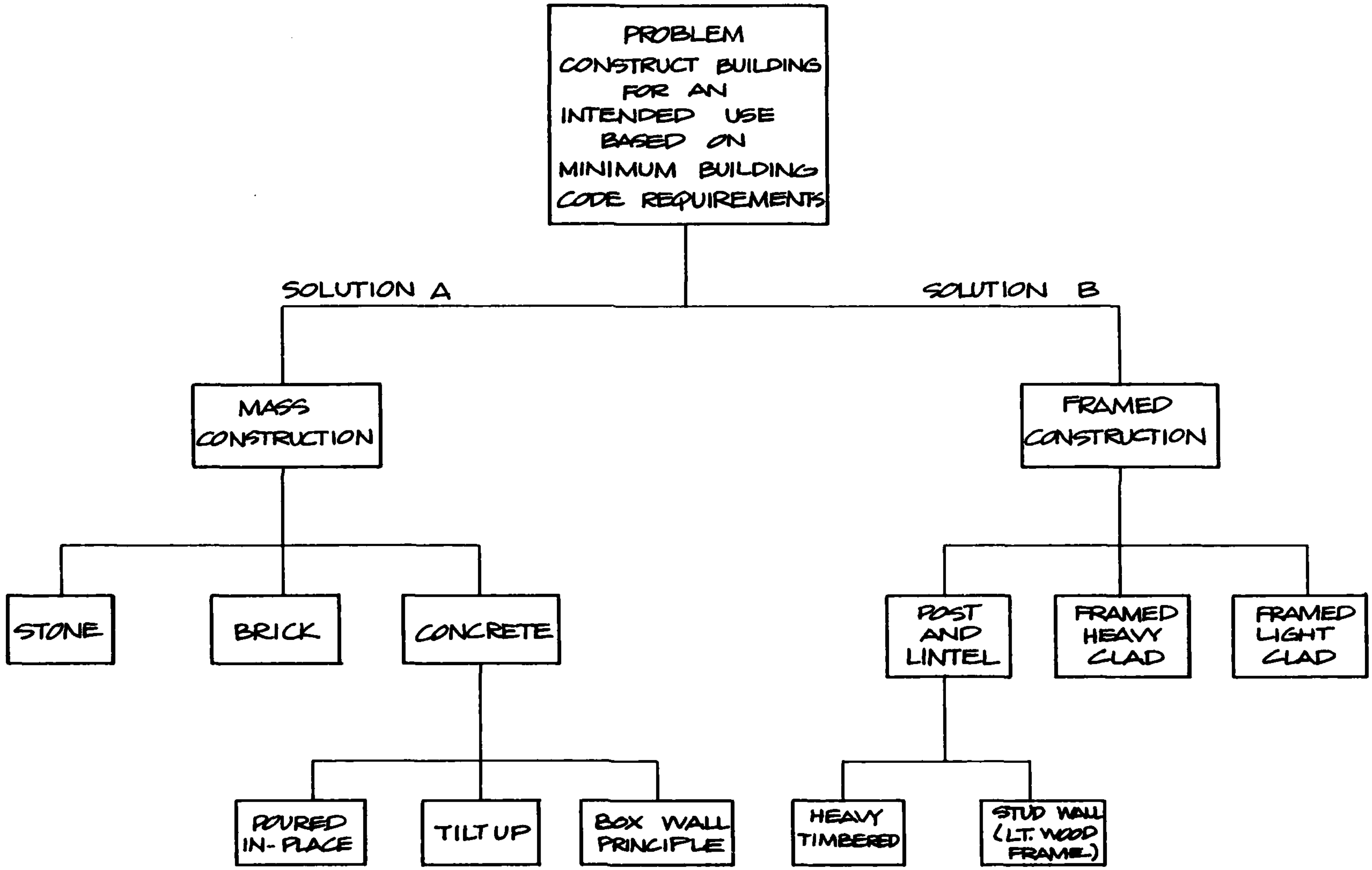
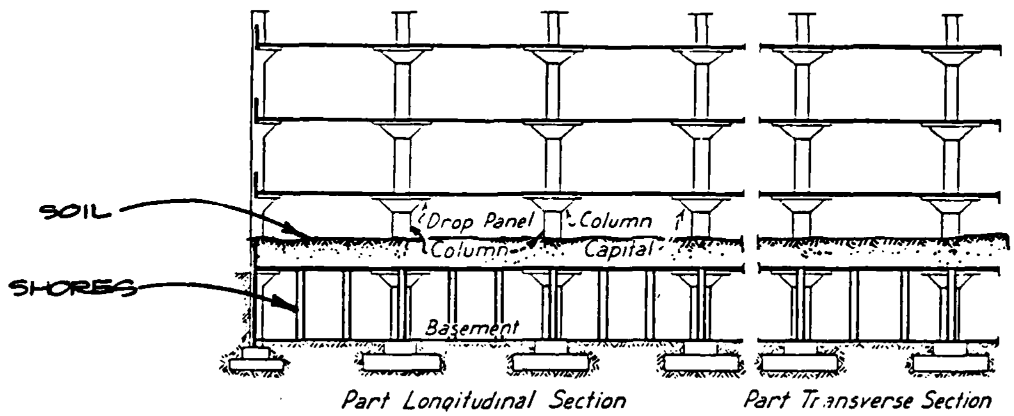
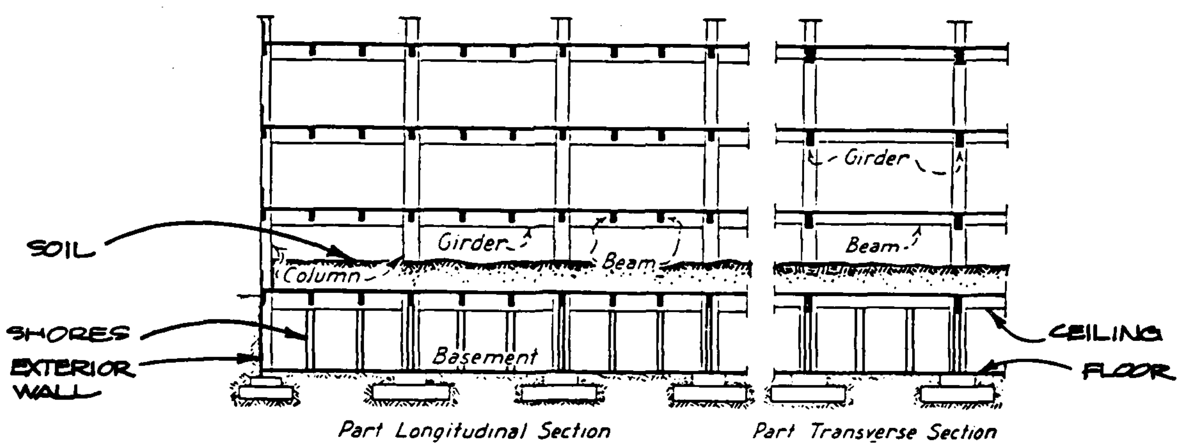


Figure B-1. Building Type Principles. (Ref. 7)



a. Flat Slab Construction.



b. Beam and Girder Construction.

Figure B-2. Types of Reinforced Concrete Framing.

Many of the above listed structural elements have been investigated previously by SSI and others (Refs. 3, 4, 5, 6, 7). The most reliable data from these investigations are based on shock tunnel tests (Ref. 3) and theoretical analyses based on the tests, which were used to predict modes of failures.

In addition, SSI has recently performed some "pioneer" efforts on building frame collapse response (Ref. 2) and building collapse from demolition (Contract EMW-C-0582), which will provide valuable input data to survivability of structures. Both of these programs are oriented toward analyzing failure modes of structures and include data on how these failures affect the integrity of potential basement shelter areas.

One of the most important factors to be considered in the ranking of potential basement shelters in buildings is the original intended use of the structure. With this information, and the approximate date of construction, the design live load can be determined using the building code in effect at the time of construction. With the original intended use determined and a visual inspection of the basement to ascertain the principal type of construction, it is possible to determine to some extent material specifications and properties. These data may then be used in assisting the engineer to predict failure modes of the elements and/or connections between elements. This approach was used to some extent in the frame response program described earlier, to provide input to the computer collapse analysis.

Many of the buildings that have viable potential basement shelter spaces are older buildings. Rarely are detailed plans available for analysis, and SSI has had extensive first-hand experience in analyzing basement areas using visual inspection and field measurements.

PREDICTION METHODOLOGY

Much of the effort during the first year has been devoted to gathering and analyzing data to be included in the building index portion of the damage function rating procedure. The data analyzed to date have been obtained from a number of sources, including the extensive laboratory testing programs conducted by SSI in the past, present SSI research efforts, and published test data by others.

The data gathering and analysis will continue throughout the program; however, the primary emphasis has been directed at developing well-defined lists of the building element types, to be ranked and weighted as to their individual performance under blast loading. This is the necessary second step prior to combining these elements into complete structures for ultimate evaluation and rating. Further SSI test programs will continue to provide significant amounts of required data. Other sources will be utilized when appropriate. The methodology that will be used in developing the ranking and weighting of the various structural elements is discussed in more detail below. It should be kept in mind that some structural systems outlined may be eliminated as potential basement shelters as the program progresses.

Building Elements

It is intended that the building index portion of the damage function take into account each of the primary structural elements that make up a building, such as the floors, framing connections, and walls. The connections that occur between these elements are not ordinarily considered structural elements, but must be addressed early in the index development for reasons that are discussed in the sections on connections and joint resistance functions. The building index will be the foundation for the damage function prediction model, and will require development in a manner that will permit the planner to rate and select the best basement with a minimal amount of effort. The methodologies that we intend to use in developing the rating procedure for each of these primary structural elements are outlined below.

Floor Systems - The rating of each type of floor system will follow the general evaluation and selection process developed for use in the shelter upgrading manuals (Refs. 4 and 5), and will be based largely on the "intended use" of the floor. With few exceptions, most buildings constructed during the past 50 years were designed using some published building code. These codes recommend the minimum design loads for floors for each category of occupancy. A typical listing of these recommended loads is shown in Table B-1.

A particular building may have been designed using one of three or four national codes, or the design may have been based on a local code, which is usually an adaptation of one of the national codes with minor revisions. The particular code used is ordinarily not a problem with respect to the rating of floor systems, since the recommended load requirements are quite similar in all codes, and the various occupancy categories can be conveniently grouped by live loads.

TABLE B-1: DESIGN INFORMATION — RECOMMENDED MINIMUM FLOOR LIVE LOADS

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Apartments (see Residential)	
Armories and drill rooms	150
Assembly halls and other places of assembly:	
Fixed seats	60
Movable seats	100
Platforms (assembly)	100
Bowling alleys, poolrooms, and similar recreational areas	75
Corridors:	
First floor	100
Other floors, same as occupancy served except as indicated	
Dance halls and ballrooms	100
Dining rooms and restaurants	100
Dwellings (see Residential)	
Garages (passenger cars only)	50
Grandstands (see Reviewing stands)	
Gymnasiums, main floors and balconies	100
Hospitals:	
Operating rooms, laboratories	60
Private rooms	40
Wards	40
Corridors, above first floor	80
Hotels (see Residential)	
Libraries:	
Reading rooms	60
Stack rooms (books & shelving at 65 pcf)	
but not less than	150
Corridors, above first floor	80
Manufacturing:	
Light	125
Heavy	250

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Office buildings:	
Offices	50
Lobbies	100
Corridors, above first floor	80
File and computer rooms require heavier loads based upon anticipated occupancy	
Penal institutions:	
Cell blocks	40
Corridors	100
Residential:	
Multifamily houses:	
Private apartments	40
Public rooms	100
Corridors	80
Dwellings:	
First floor	40
Second floor and habitable attics	30
Uninhabitable attics	20
Hotels:	
Guest rooms	40
Public rooms	100
Corridors serving public rooms	100
Corridors	80
Reviewing stands and bleachers	100
Schools:	
Classrooms	40
Corridors	80
Sidewalks, vehicular driveways, and yards, subject to trucking	250
Skating rinks	100
Stairs and exitways	100
Storage warehouse:	
Light	125
Heavy	250

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Stores:	
Retail:	
First floor, rooms	100
Upper floors	75
Wholesale	125
Theaters:	
Aisles, corridors, and lobbies	100
Orchestra floors	60
Balconies	60
Stage floors	150
Yards and terraces, pedestrians	100

DESIGN LOAD GROUPS

(as used in this manual)

LIGHT:

40 to 60 psf

MEDIUM:

80 to 125 psf

HEAVY:

150 to 250 psf

A second factor that will enter into the rating system will be the type of construction. A precast floor system, for example, designed for a "light storage" occupancy, 125 psf (see Table B-1), would not necessarily have the same collapse load as a cast-in-place concrete floor designed for an identical occupancy. The reason for this is inherent in the design methods used and the manufacture and/or selection of the construction materials; i.e., different design methods are required for different materials, many incorporating different parameters and safety factors. A similar comparison could be made for a concrete on steel deck system or a one-way slab on steel beams.

Wall Systems - A considerable amount of previous work has been performed on the blast resistance of wall systems without backfill (Ref. 3), and current investigations under other SSI contracts are addressing the survival of basement walls. Based on the data available to date, the rating methods that we intend to use to develop a wall rating index for basement walls will be based on the mass of the wall and type of backfill. The wall mass rating may be modified, however, as data are obtained on the effect of blast loads on basement walls.

Initially, the rating index for basement walls will be very preliminary because of the lack of usable data in these areas. It is anticipated that the investigative programs conducted and/or completed under other SSI contracts during the next few years will provide substantial input toward the development of this index. The completion of the testing and analysis of the performance of the basement test walls at the MILL RACE event, supplemented by tests of 1/20th scale models of similar walls in the 12-in. shock tube, will provide some insight into wall response. This testing and analysis are also discussed in Section 4 and Appendix C. As data are developed and become available from the ongoing investigation, they will be incorporated into the damage function rating procedure.

Connections - As stated previously, there are seven elements important to basement shelters. It is our intent to include them all in the index, but only four basic structural elements: floors, framing systems, connections, and exterior walls are discussed in any detail in this report. As this program progressed through the first year, it became apparent that the performance of many types of primary structural elements was directly related to the performance of their associated connections. In several cases, full-scale tests of structural systems (Ref. 1) indicated that the integrity of the connections controlled the load-carrying

capabilities of the structural elements. Therefore, a large portion of this appendix deals with connections.

During the first year's effort, considerable data were developed under other SSI programs with respect to structural connections, and it is anticipated that these data will be substantially increased during subsequent years. Included in the first year's program was a comprehensive survey and analysis of the expected performance under blast loading of concrete structural connections. It is the intent to continue this analysis and include steel connections under future options; thus the discussion of steel in this report is more brief in scope.

It is realized that the rating index to be developed for connections will, in all probability, only be used to modify the building index. However, because of the importance of connections to the overall prediction model, it is expedient to address structural connections early in this program and to begin to rank connection performance as the data become available.

CONCRETE STRUCTURAL SYSTEMS AND CONNECTIONS

This section of the report is a review of the types of concrete slabs and concrete connections and connection systems that most directly affect the performance of potential basement shelters. The total damage function rating procedure program effort is 75% concrete structure oriented; thus, a detailed discussion on concrete is provided. Based on system types, concrete construction can be divided into two basic categories — cast-in-place and precast. Each of these categories is discussed separately.

Cast-in-Place Concrete

This type of construction, as the name implies, consists of concrete transported to the building site in a plastic state, placed into or on top of forms, and vibrated and compacted in and around the reinforcing steel. Descriptions of the various construction types are based on the different design philosophies employed. These different design methods may result in a somewhat different appearance and performance characteristic for each system type. However, from the standpoint of an investigation of connection integrity under blast loading, many of these systems have much in common and their differences are, in many cases, technically insignificant.

Reinforced concrete floors are defined as a slab supported so as to effect a successful, efficient, and economical transfer of the floor loads to the columns, and then to the foundation through the footings. The slab may be supported by reinforced concrete beams and/or girders, masonry or reinforced concrete walls, or directly on the columns. Following is a description and evaluation of the various types of typical cast-in-place concrete slabs and their related connections.

One- and Two-Way Slabs - When a slab has its principal reinforcement in one direction and is supported on two sides by beams or walls, it is defined as a one-way slab. If the principal reinforcement is in both directions, it is a two-way slab, and therefore, one-way slabs are really only special cases of two-way slabs. If the ratio of the long side to the short side of the slab is 2 or greater, the slab is probably designed as if all the bending is in the short direction, hence a one-way slab. If this ratio is less than 2, bending must be assumed in both directions, and the slab is designed as a two-way slab. Obviously, the use of this ratio is only for the purpose of defining the type of system and, in fact, the ACI Building Code (Ref. 8) does require investigation of the design of the reinforcement in the long direction of a slab supported along its four edges even though the ratio of long side to short side may be greater than 2 (Ref. 9).

The beams, columns, and outer walls are cast monolithically with the slabs. For this reason, the slabs usually are designed to be continuous over the beam supports, with the end spans tied into the edge beams and/or walls for moment resistance. Typical connections associated with these types of slabs are shown in Figure B-3. This degree of "fixity" at supports provides a certain amount of redundancy in the system, which permits a redistribution of stresses under severe loadings. These slab systems perform well structurally as shelters when properly upgraded. The MILL RACE high explosive test in 1981 contained a basement structure with the floor above consisting of two bays of a 6-in. thick, two-way slab, shored, and subjected to a 40 psi blast environment. The performance of this shelter area was quite good, with no severe structural damage noted (Ref. 10). A number of tests on one-way slabs, both shored and unshored, have been conducted by Scientific Service, Inc. (Refs. 11 and 12), and the U.S. Army Waterways Experiment Station (Ref. 13).

One-Way Joist Slabs - One-way joist construction is a special case of the one-way slab. Since the concrete located between the neutral axis and the tension face

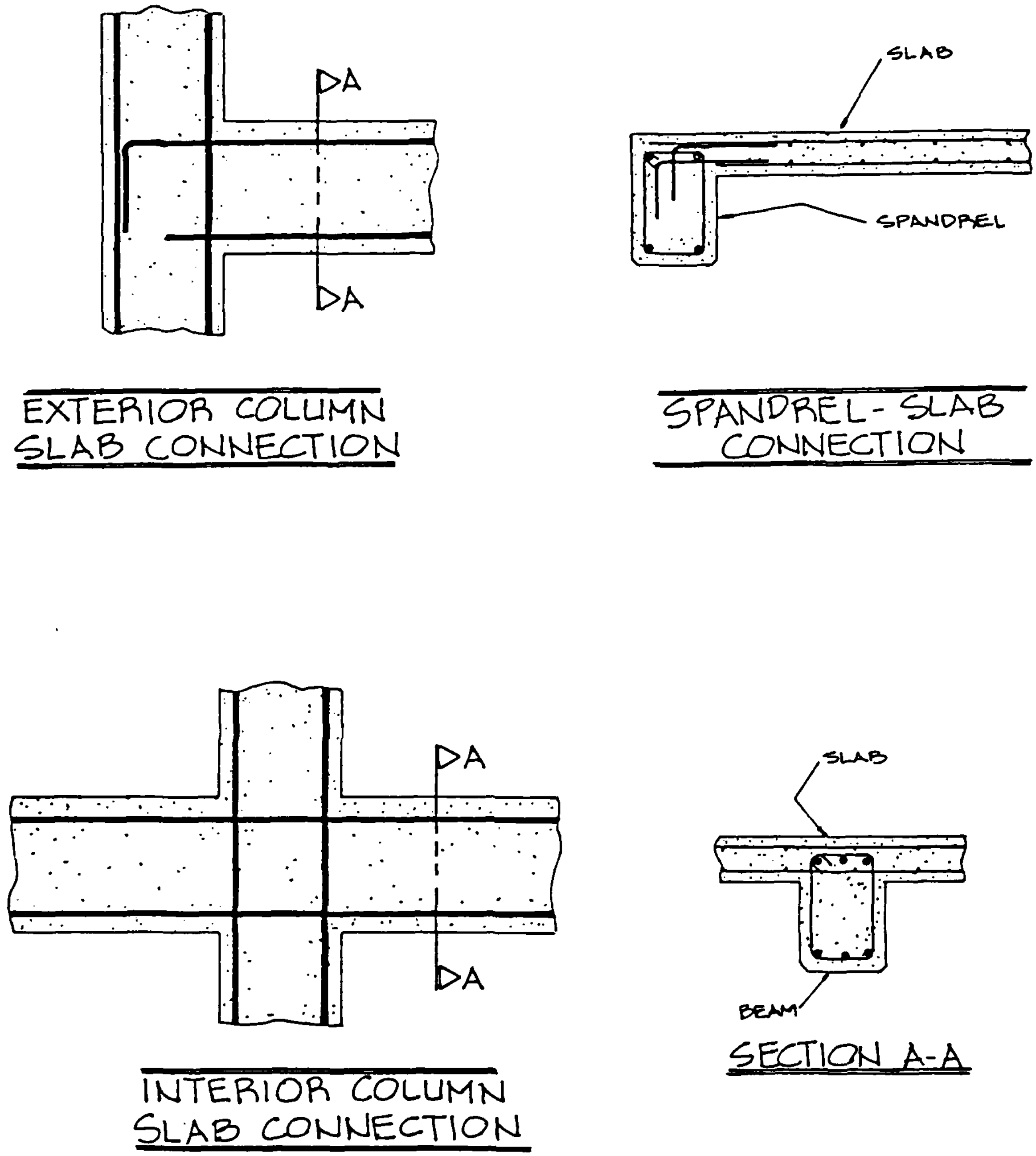


Fig. B-3. Connections for One- and Two-Way Slabs.

of a solid reinforced concrete slab does not significantly contribute to the flexural strength, but is effective in resisting portions of the shearing stresses, it is possible, under certain load and span conditions, to eliminate a large portion of the concrete on the bottom side of the slab, leaving only ribs or joists. The bottom of these joists corresponds to the bottom of a nearly equivalent solid slab. This configuration saves concrete and thereby reduces the weight of the slab. The reinforcing steel, which is ordinarily distributed rather evenly throughout the solid slab, is now concentrated in each of the joists; less reinforcement is required, however, since the dead weight of the system has been reduced.

Since the formwork for this type of construction is more complicated and thereby more expensive, the construction industry has developed standard sizes that may be rented or purchased. Standard form sizes typical for this type of construction result in void spaces between joists of either 20 or 30 in. wide, and may be obtained in depths of 6 to 20 in. (Ref. 14).

The one-way joist system is cast monolithically and has much of the redundancy of the one-way slab discussed above, and if properly upgraded, the joists and beams and their related connections would be expected to perform well. This type of construction is shown in Figure B-4. However, a question remains with respect to the slab portion between the joists. These slabs are typically 3 to 4½ in. thick and are minimally reinforced. Tests have been conducted on slabs of this thickness and span, but supported on all four edges, and the results of these tests indicated good performance, primarily because of the presence of membrane action. It is doubtful, however, that this membrane action would be as effective in a slab supported on only two edges, and further investigative effort in this area is required.

Flat Slabs and Flat Plates - A flat slab is a concrete slab reinforced in two directions so that it transfers its loads directly to supporting columns; i.e., it has no beams or girders to transfer the loads to the columns, such as one- and two-way slabs. In order to better resist the stresses concentrated immediately surrounding each column, this type of construction typically uses a flared column capital and often has a thickened slab, or drop panel, around the column. The configuration of these connections is shown in Figure B-5.

A flat plate is also reinforced in two directions and transfers its loads directly to the columns, but is constructed without column capitals or drop panels. These

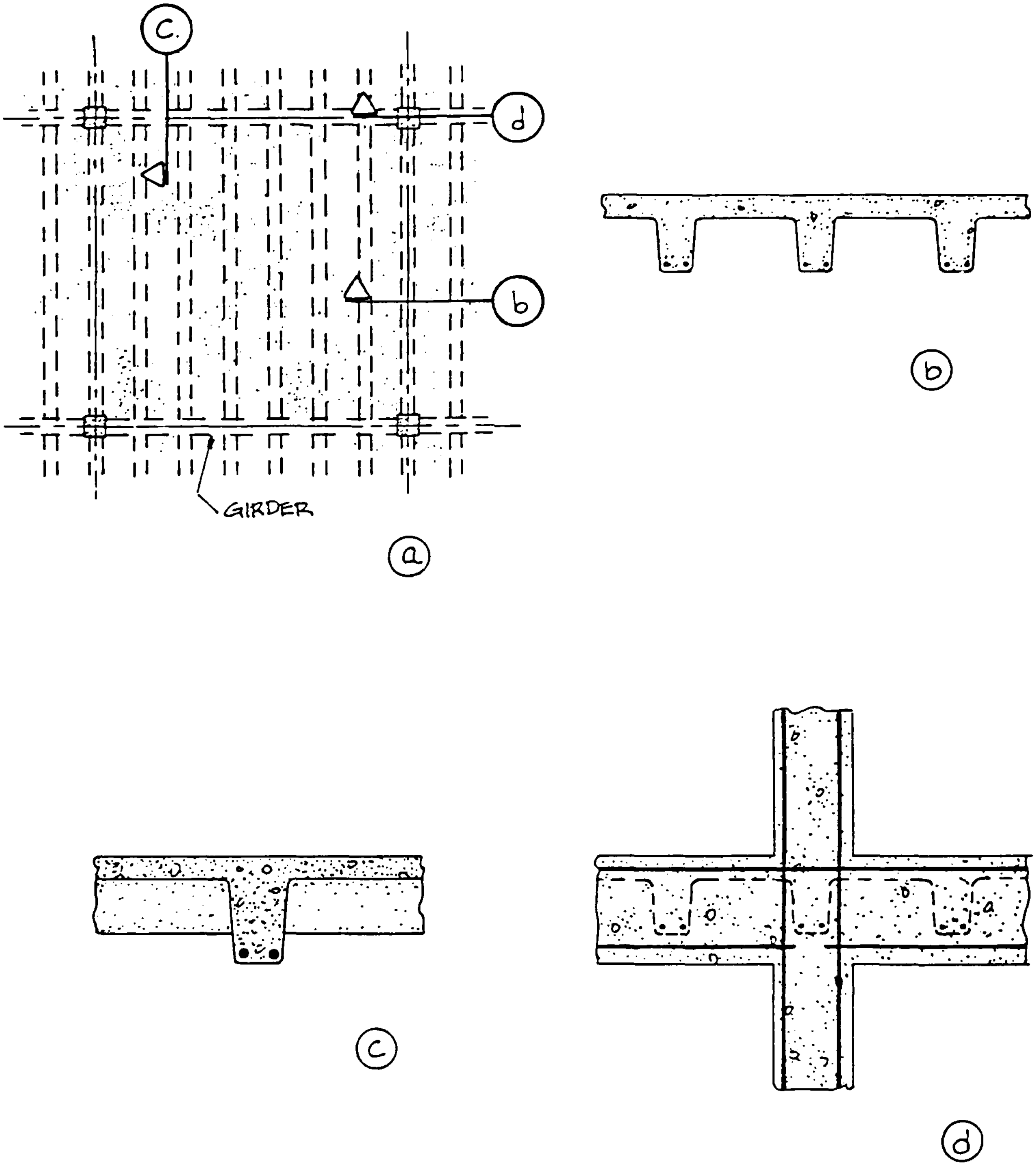
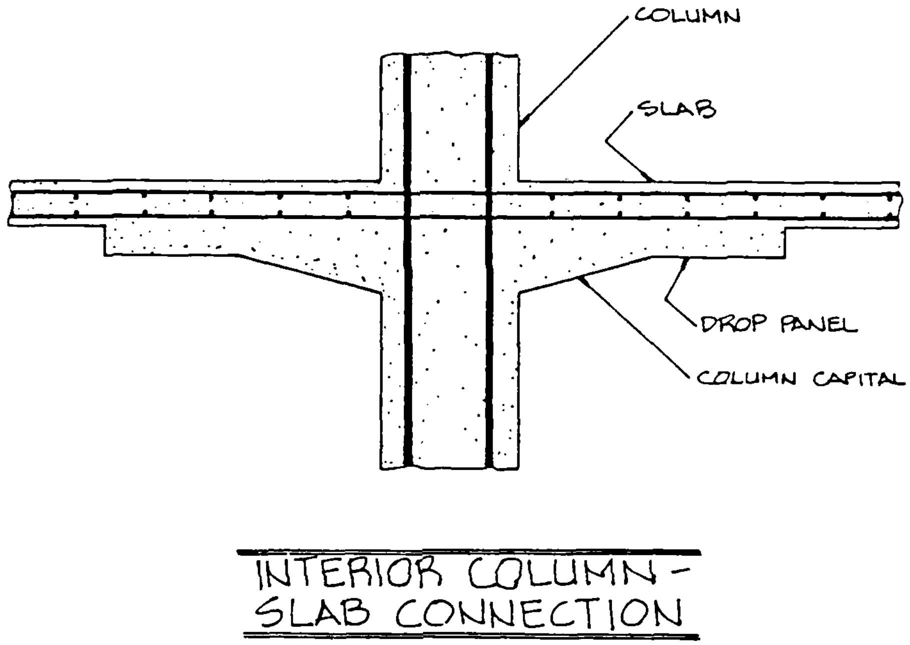


Fig. B-4. One-Way Reinforced Concrete Joists.



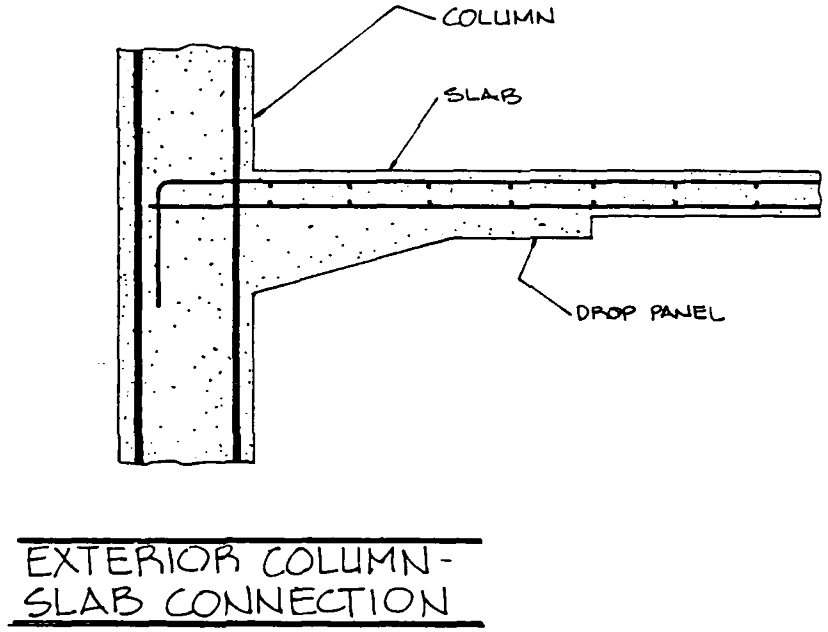


Fig. B-5. Flat Slab Connections.

connections are illustrated in Figure B-6. Since no special accommodations are made at the column, either by thickening the slab or by the addition of capitals, the shearing stresses at these locations limit the load for which flat plates are feasible. However, they are extensively used because of their flat uninterrupted ceilings, particularly as ceilings in areas where partitions are to be installed. At the present time, a considerable number of flat plates use post-tensioned reinforcement. This type of construction will be discussed in more detail later.

As was the case with the one- and two-way slabs, flat slabs and plates are typically cast monolithically with the columns, and sometimes the walls. They are designed to be continuous over the column supports, and their end spans are tied into the walls and/or edge beams providing moment resistance. This type of design and construction again provides a certain amount of redundancy in the system that permits stress redistribution prior to collapse under severe loadings. When upgraded properly, the flat slab type of construction would be expected to perform well as a shelter, and the connections would not require any special consideration. Full-scale field tests of this type of construction subjected to 40 psi have been conducted successfuly (Ref. 10). Recent information from the demolition industry indicates that flat slab construction is difficult to demolish and may, therefore, be one of the better construction types for basement shelters.

Flat plate construction would not be expected to perform as well as flat slab, however, because of the design parameters previously outlined. Under severe loading, a shear failure would be anticipated adjacent to and around the columns; i.e., the column would punch through the slab. This column/slab connection would probably be the critical failure mechanism for this type of system even with the slab portion properly upgraded. This type of failure mode is clearly indicated in a nine-bay, one-quarter scale model test conducted by the USAE Waterways Experiment Station in February, 1982 (Ref. 15). Accordingly, in order to consider flat plate construction as a viable shelter option, particularly in risk area basement shelters, an upgrading method must be developed and verified by test for these critical stress areas adjacent to the column supports.

Waffle Slabs - Waffle slab construction consists of rows of joists at right angles to each other with solid heads at the columns. For design purposes, waffle slabs are considered flat slabs, with the solid heads at the columns performing the same function as drop panels. The economic basis for this type of construction is

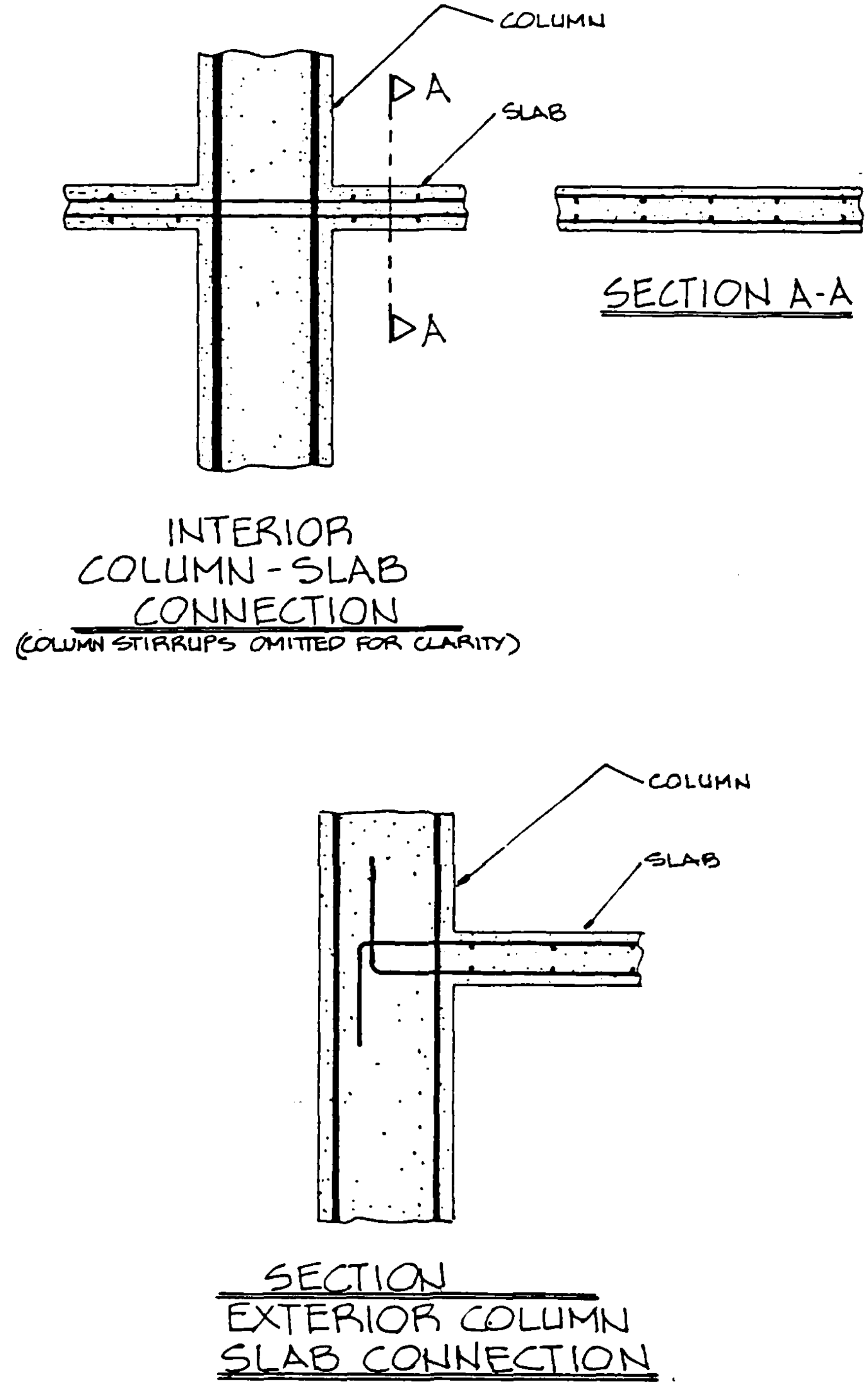


Fig. B-6. Flat Plate Connections.

the same as was described for one-way joist slabs; i.e., a reduction in concrete, weight, with a corresponding reduction in required reinforcing steel. As with the flat slab, the principal reinforcement is in both directions, but in this case located in the joists, and the slab is cast monolithically with the columns and walls. The standard forms available for waffle slabs are either 30 in. or 19 in. square, the former providing 36 in. on center joist spacing with 6-in. wide joists, and the latter providing 24 in. on center joist spacing with 5-in. wide joists. The 6-in. joists have standard depths of 8 to 20 in. and the 5-in. wide joists, 4 to 12 in. (Ref. 14).

This slab system would be expected to perform well as a shelter, similar to the flat slab, if properly upgraded, and the connections would not require special consideration. The slab portion between the joists, similar to the one-way joist slab, typically ranges from 3 to 4½ in. thick. Unlike the one-way joist, however, test data indicate that this thin slab section should not be a detriment for use as a shelter, because of the membrane action developed by support at all four edges. U.S. Army Waterways Experiment Station (reports unpublished) on full scale waffle slabs and small sections of the thin slab sections from waffle slabs. This type of construction is shown in Figure B-7.

Post-Tensioned Slabs - In the last 15 years post-tensioned construction has increased some 350% in the United States, primarily because of the desire to obtain reduced structural system depth and longer spans, at an economical cost (Ref. 16). At this point, a very brief description of post-tensioning theory and construction methods is in order for those who are unfamiliar with this type construction. The strength properties of concrete are such that it is a good material in compression, but relatively weak in tension; i.e., the compressive strength is approximately ten times the tensile strength. In the design of concrete slabs using normal reinforcing steel, the steel is located in areas where tensile stresses are anticipated, thus utilizing the concrete in compression and the steel for tension. Post-tensioning is a method by which compressive forces are induced into the slab in order to take advantage of the natural compressive strength of concrete. This is accomplished by placing elongating wires or strands, called tendons, placing concrete around the tendons, allowing the concrete to attain its desired strength, and securing the tendons by filling a duct that surrounds the tendon with grout and anchoring the tendons at their ends (bonded post-tensioned), or by anchoring the tendons at their ends only (unbonded post-tensioning).

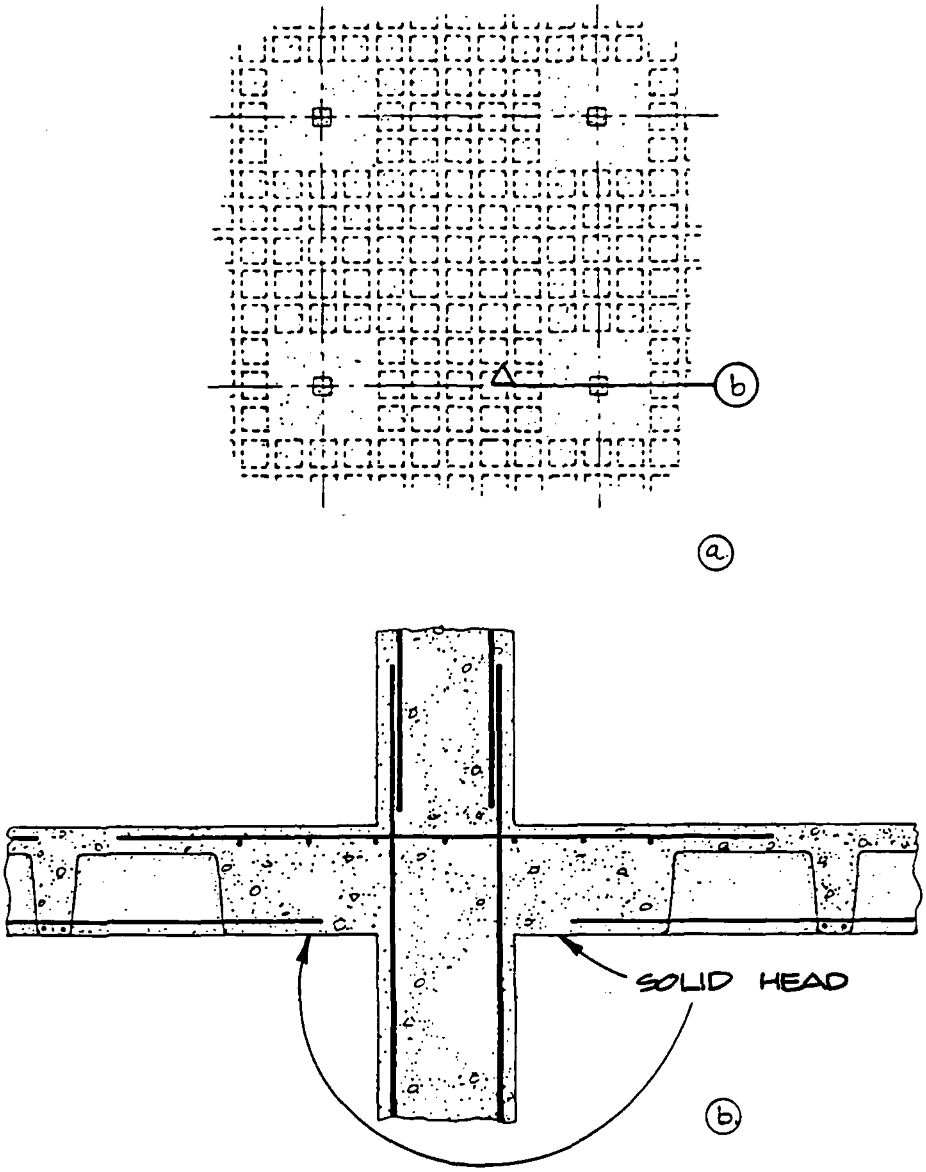
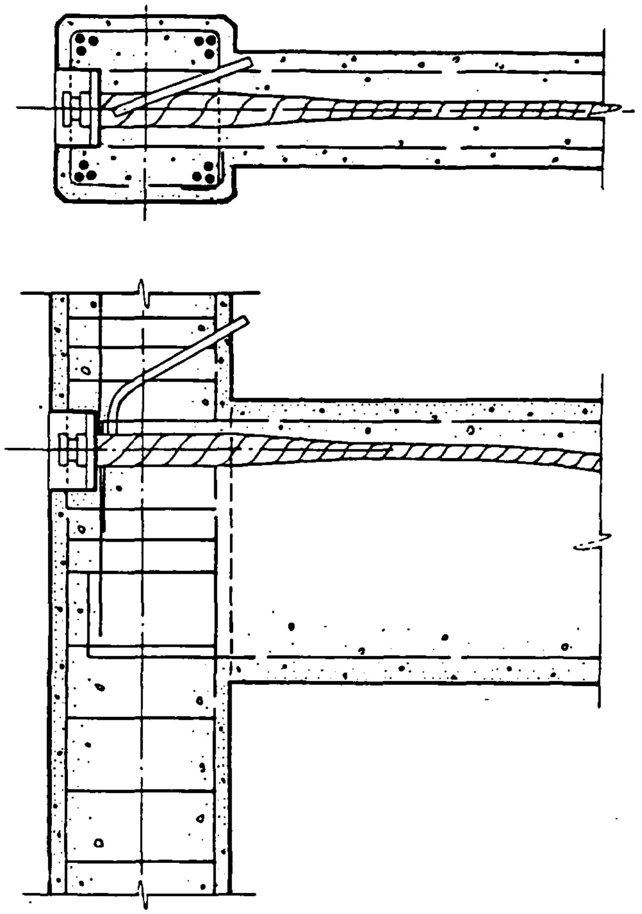


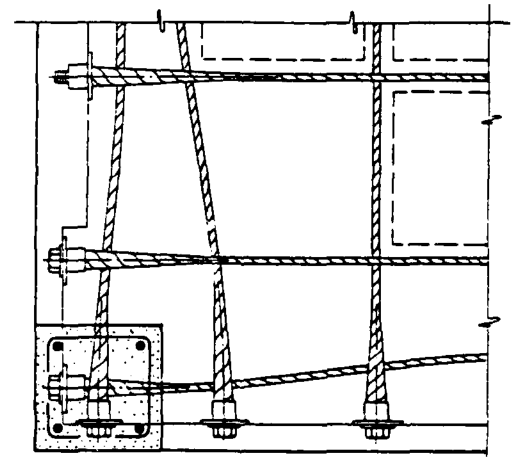
Fig. B-7. Waffle Slabs.

In post-tensioned slab construction, the load-carrying capability of the slab is obtained by placing a series of tendons accurately on the slab formwork prior to placement of the concrete. These tendons may be several bays or more in length and spaced at specific intervals, and are typically placed in a parabolic profile to accommodate both the negative and positive moments in continuous spans. If the tendons are to be "unbonded", they are greased and paper wrapped or plastic covered, and if later to be "bonded", placed in a flexible duct, in order that they do not bond initially and may slide easily through the slab to accommodate the later stressing operation. Once the concrete is placed and reaches sufficient strength, each tendon is elongated the calculated amount, thus tensioning each to a design stress. To hold the tendon at this final stress level, anchoring devices with steel jaws are utilized at each end of the tendon. If the tendons are to be "bonded", the duct is pumped full of grout. These anchoring devices are later covered with concrete for fire proofing and corrosion protection. Typical tendon anchorage details are shown in Figure B-8.

There are inherent problems in upgrading and utilizing post-tensioned slabs as viable shelter options. When tendons are unbonded and a tendon failure occurs in one span, many spans may be affected. As stated in the "Post-Tensioning Manual" (Ref. 16), "A catastrophic loading such as might occur from an explosion or a severe earthquake which resulted in a failure in one bay of a beam or one-way slab with unbonded tendons could result in a progression of the failure throughout all bays of a multi-bay building." These types of progressive collapses have occurred in the past. One notable example is illustrated by the parking structure at Bailey's Crossroads, Fairfax County, Virginia, which, in 1973, totally collapsed when debris fell on a portion of the slab from an adjacent high-rise construction collapse (Ref. 17). Building code revisions have, to a small degree, addressed this problem in one-way unbonded post-tensioned slabs by requiring secondary means of carrying loads; i.e., additional bonded tendons or reinforcing steel. It is not clear whether these code requirements are adequate, and when and if improved, the changes may need to be extended to two-way slabs. Two-way slabs are now excluded from this revision on the assumption that a loss of load-carrying capability would occur in only one direction, an assumption that is highly speculative. This code change was instituted in 1976 and probably was not implemented in building design methodology to any great degree until at least 1978. Accordingly, the great majority of existing structures that would now be considered for use as shelters do not have the advantage of even this limited code provision.



ANCHORAGE AT COLUMNS / WALLS



TENDON ANCHORAGE AT CORNER COLUMN

Fig. B-8. Post-Tensioned Connections. (Ref. 16)

A review of the current literature on failure analysis investigation indicates that post-tensioned construction, particularly unbonded, has a relatively poor performance record. Significant problems that have been reported include loss of precompression of the slab due to creep and shrinkage of the concrete, a condition that is approximately three times greater in unbonded post-tensioned slabs than in conventionally reinforced slabs, and corrosion of end anchorages, which are highly susceptible to stress corrosion if not adequately protected.

In order to accommodate the plastic deformation or creep of the concrete slab with time, as well as from shrinkage and thermal volume changes, Ref. 16 recommends a number of connection details that minimize restraint; one such detail is shown in Figure B-9. Although these suggested details serve this purpose, they further degrade connection integrity with respect to blast survival, and accordingly, their use in basement shelters.

Post-tensioned slabs are currently used in many structures, and it is expected that their use will increase; therefore, it is necessary to include this type of construction for consideration as a shelter option, even though they may be unsuitable. At the present time, however, a slab of this construction would not be considered a viable candidate for basement shelter upgrading. An investigative effort is required in order to identify the specific problem areas and to develop and test potential upgrading methods.

Precast Concrete

Precast concrete is defined as concrete that is cast in some location other than its final position in a completed structure. The location may be a plant that specializes in the manufacture of particular elements, or actually the building site, where a contractor may use temporary forming methods to produce the elements on a one time only basis. In either case, the common characteristic of precast concrete is that the individual elements, or building components, must be located in their final position, and connected and/or secured to supports, footings, or each other, in order to perform as designed. Static and dynamic load tests of buildings, or portions of buildings, as well as the investigation of building failures, have shown that in many cases the connections associated with precast concrete are the weak link in this type of construction.

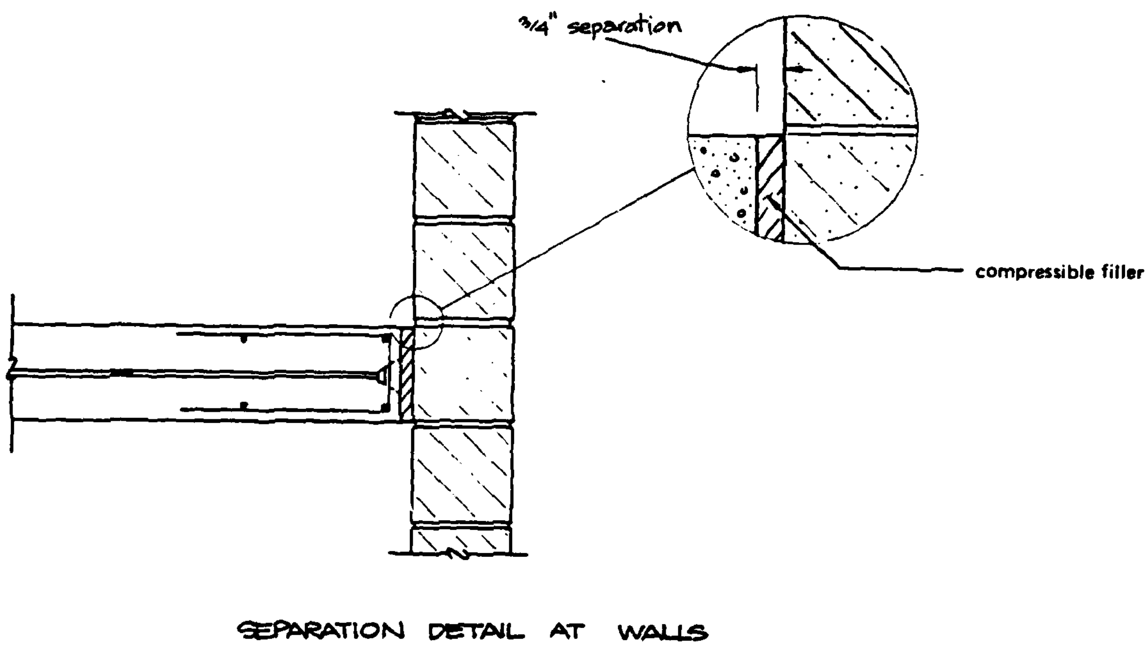


Fig. B-9. Post-Tensioned Connections. (Ref. 16)

In the construction of a total building, precast concrete components may be used in combination with structural steel and/or cast-in-place concrete, or they may used to construct nearly the entire structure. These components may be structural precast (i.e., they perform a function in the building necessary to its structural integrity), or architectural precast, or a combination of both. The precast components may be pilings, single or multi story columns, beams and girders, single and double tees, hollow-core or solid slab floor or roof members, or wall panels consisting of solid units, single or double tee and hollow-core members, or insulated sandwich units. The wall panels may be designed to be either load bearing or non load bearing components (Ref. 18). Other types of precast components are curtain wall cladding, stairway units, sun shades, and spandrel beams. It is obvious from this partial list that the uses of precast concrete, and the shapes and configurations in which it may be obtained, are limited only by economics and the imagination of the architect. However, for the purpose of evaluating basement areas in buildings constructed of this material, we need be concerned only with components used as the main structural elements; i.e., the elements required to support the intended design loads, and subsequently when upgraded, the blast loading. Accordingly, we will confine the investigation to elements such as walls, floors, columns, beams and girders, and their related connections.

The primary reinforcement in precast concrete structural components may be mild steel reinforcing or prestressing strands. However, in precast slabs, unlike the required differentiation between mild steel reinforcing and post-tensioning in cast-in-place concrete slabs, both of these reinforcing methods perform very similarly when loaded to failure loads (Refs. 11, 12 and 19). The types of connections used for each reinforcing method are the same and are independent of which design method is used. Although the design methodologies for prestressing and post-tensioning are very similar, the construction methods are different. Prestressing results in fully bonded strands that do not depend on end anchorages or grouting for integrity, and the cutting or destroying of one strand at a location along its length does not result in loss of prestress, or load-carrying ability, for the full length of the strand.

Since precast concrete structural components of identical configuration may be used in constructing different building elements, they will be combined for evaluation with respect to their use in the building; i.e., floors, walls, columns, and beams and girders.

Floors - The components that are used in floors and roofs are described below and illustrated on Figure B-10 (Refs. 18 and 20):

Solid Flat Slabs - These are usually fabricated in depths of 3 to 6 in., and may be reinforced with mild steel or prestressed. The width of the units is restricted by shipping and the width of the precaster's casting beds, but typically varies from 8 to 12 ft. These slabs would typically be used in spans of from 13 to 24 ft, and are usually covered with a structural reinforced concrete topping. See Figure B-10(a).

Hollow-Core Slabs - Precast prestressed hollow-core slabs are manufactured by commercially franchised processes using specialized forming machinery. Six principal processes are used in the United States to produce slab widths from 2 to 8 ft, depths from 4 to 12 in., and core configurations such as round, rectangular, or elliptical. They may be installed side by side in a floor, or positioned up to 3 ft apart with the space in between spanned with metal decking. Typical spans for these units range from 18 to 42 ft. See Figure B-10(b)(c).

Single Tees - The configuration of these prestressed units, as the name implies, consists of a horizontal slab, or flange, with one vertical stem located at the mid-width of the flange. Single tees vary in width from 6 to 12 ft, and in depth from 16 to 48 in. The flange is typically a minimum of 2 in. thick at the outer edges, and increases in depth toward the stem. Single tees usually span greater distances than double tees, up to 120 ft, however, due to economic considerations, somewhat shorter spans are in typical use. See Figure B-10(d).

Double Tees - These prestressed units are plant cast in steel forms and derive their name from their cross-sectional appearance, a horizontal slab with two vertical stems symmetrically spaced. Common variations to this section result when they are cast without the slab portion on the outside of one stem, an "F" slab, or without a slab on the outside of both stems, a channel slab; these modifications, however, do not have any bearing on this investigation. Double tees vary considerably in cross-sectional dimensions, and thus, in span and load-carrying capability. Width is typically 4 to 12 ft, and depth, 10 to 41 in. The

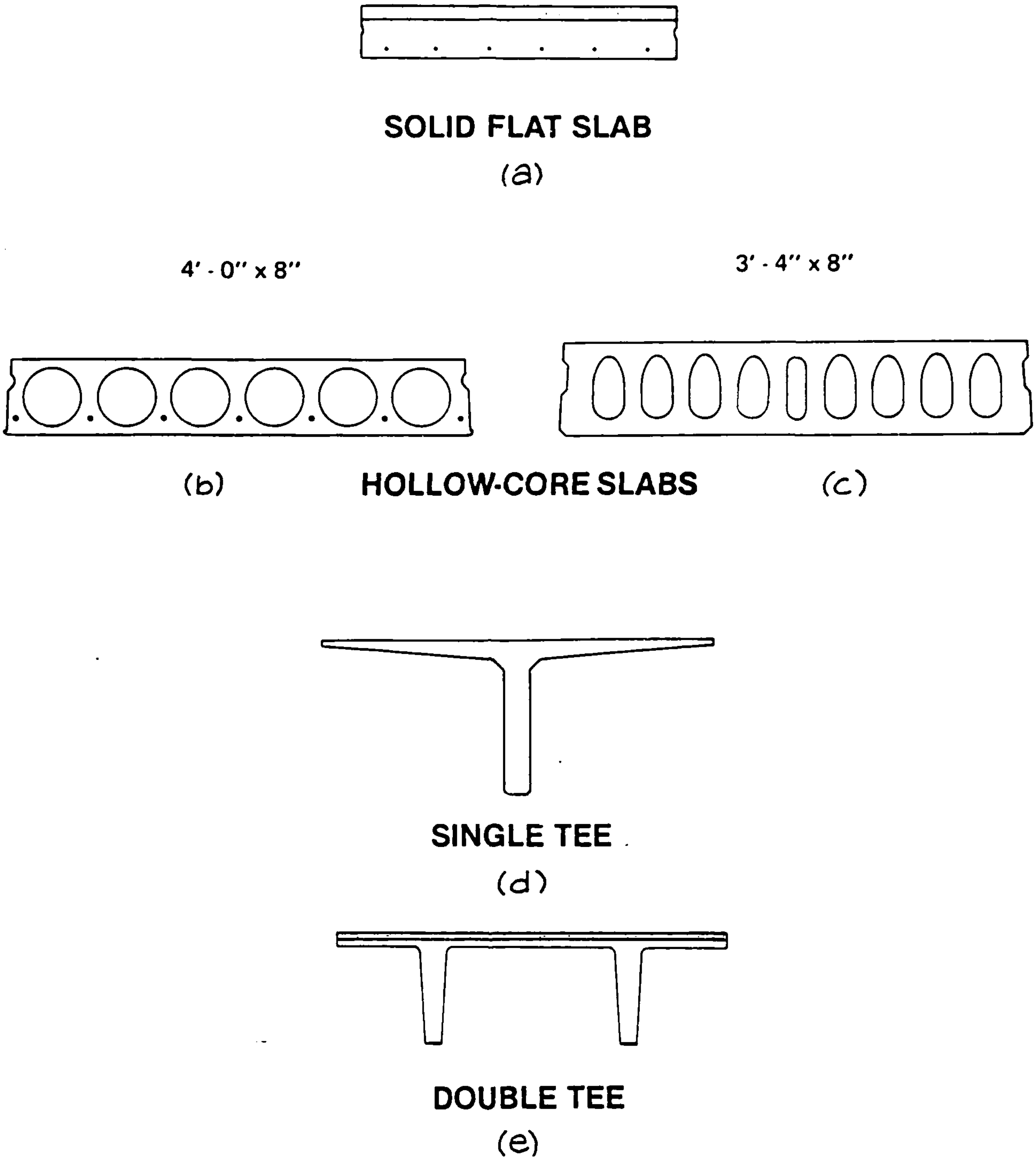


Fig. B-10. Floor Slabs. (Ref. 18)

horizontal slab, or flange, thickness is typically 2 in. throughout its width, but may be increased to $2\frac{1}{2}$ in. to 4 in. because of structural requirements. Spans of up to 90 ft are possible with the deeper sections, but considerations of handling, transporting, and erecting these units result in maximum economical spans of 50 to 60 ft. See Figure B-10(e).

During the previous discussion of cast-in-place concrete floors, the fact that this type of construction was cast monolithically with and over its supports, whether beams, columns, walls, etc., resulting in a continuous structural system, was judged to be advantageous with respect to selection as a shelter option. The primary reason for this judgment was the redundancy of that type of construction. That is, the system has multiple load paths, and when overloaded or subjected to unsymmetrical loading for which it was not designed, it has the capability to redistribute the resulting stresses to a stronger load path. The same cannot be said of a non-monolithic, non-continuous system such as precast concrete. Although some types of precast floor systems lend themselves to partially continuous designs, by and large, this type of construction is a simple span. Each individual precast unit, although it may have some ability to transfer loads to immediately adjacent units by weldments or grout joints, must perform independently of the other spans and/or bays.

The normal volume changes occurring in concrete as a result of shrinkage, creep, and temperature must be relieved at the bearing ends in simple span precast concrete components, instead of being distributed throughout the structure, as would be the case in monolithically cast-in-place slab systems. Because of this, as well as the fact that simple span flexural members undergo rotation at the bearing ends, fixed connections are not recommended for precast concrete floor and roof units. The design of these end connections varies with the type of unit. Solid and hollow-core slabs normally rest on bearing pads, usually of felt, asbestos-cement, or hardboard positioned on the supporting walls or beams and have no positive connection, as shown on Figures B-11, B-12, and B-13. Single and double tees are also erected this way, using bearing pads of elastomeric, laminated fabric, or frictionless (Tetrafluorethylene) material (Ref. 21) at each bearing end, or using welding at one end, thus achieving a semi-fixed connection at one end, with the bearing pad at the other. Examples of these connections are shown in Figures B-14, B-15, and B-16. This "welded at one end" type of connection is currently widely used, even though Ref. 18 states ". . . axiomatically that the bottoms of double tees and other stemmed

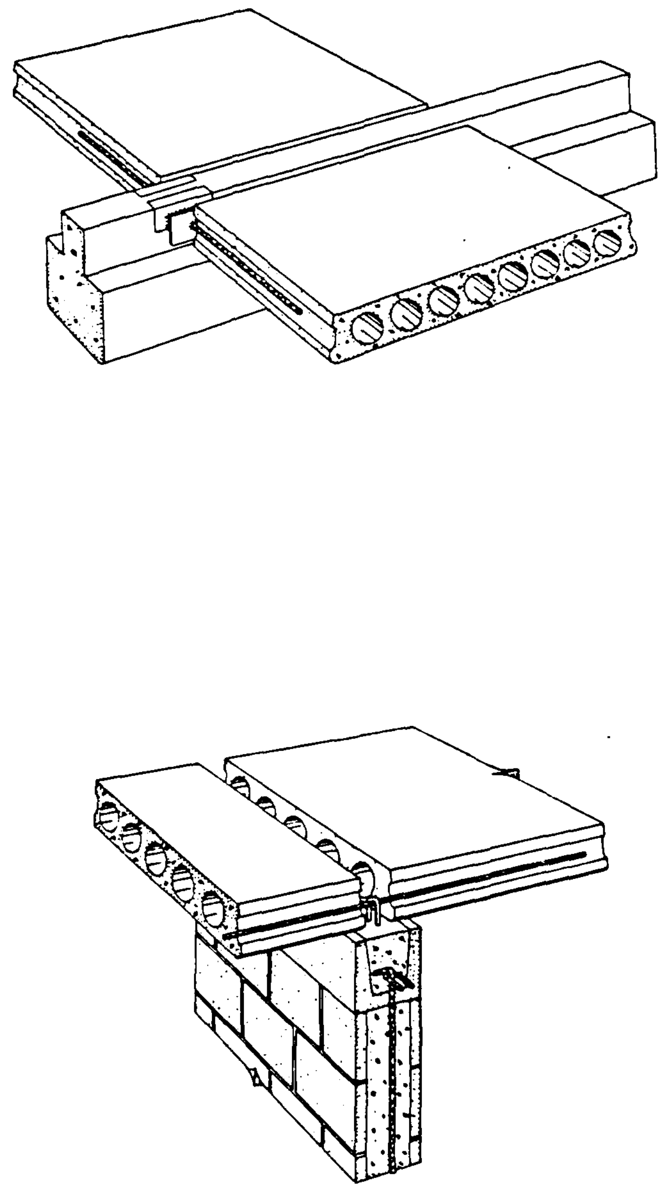


Fig. B-11. Hollow-Core Slab Connections (Untopped). (Ref. 21)

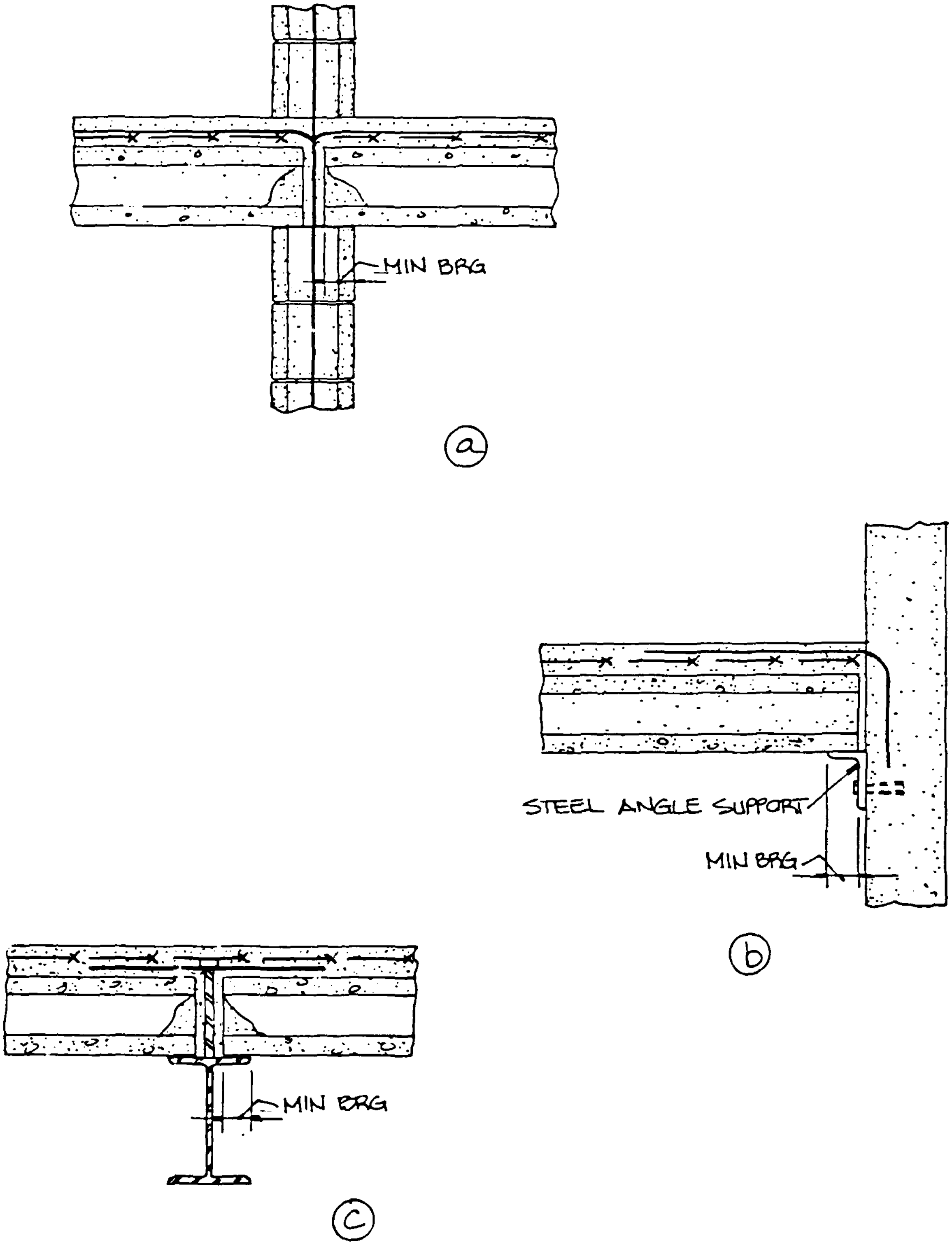


Fig. B-12. Hollow-Core Slab Connections (Topped).

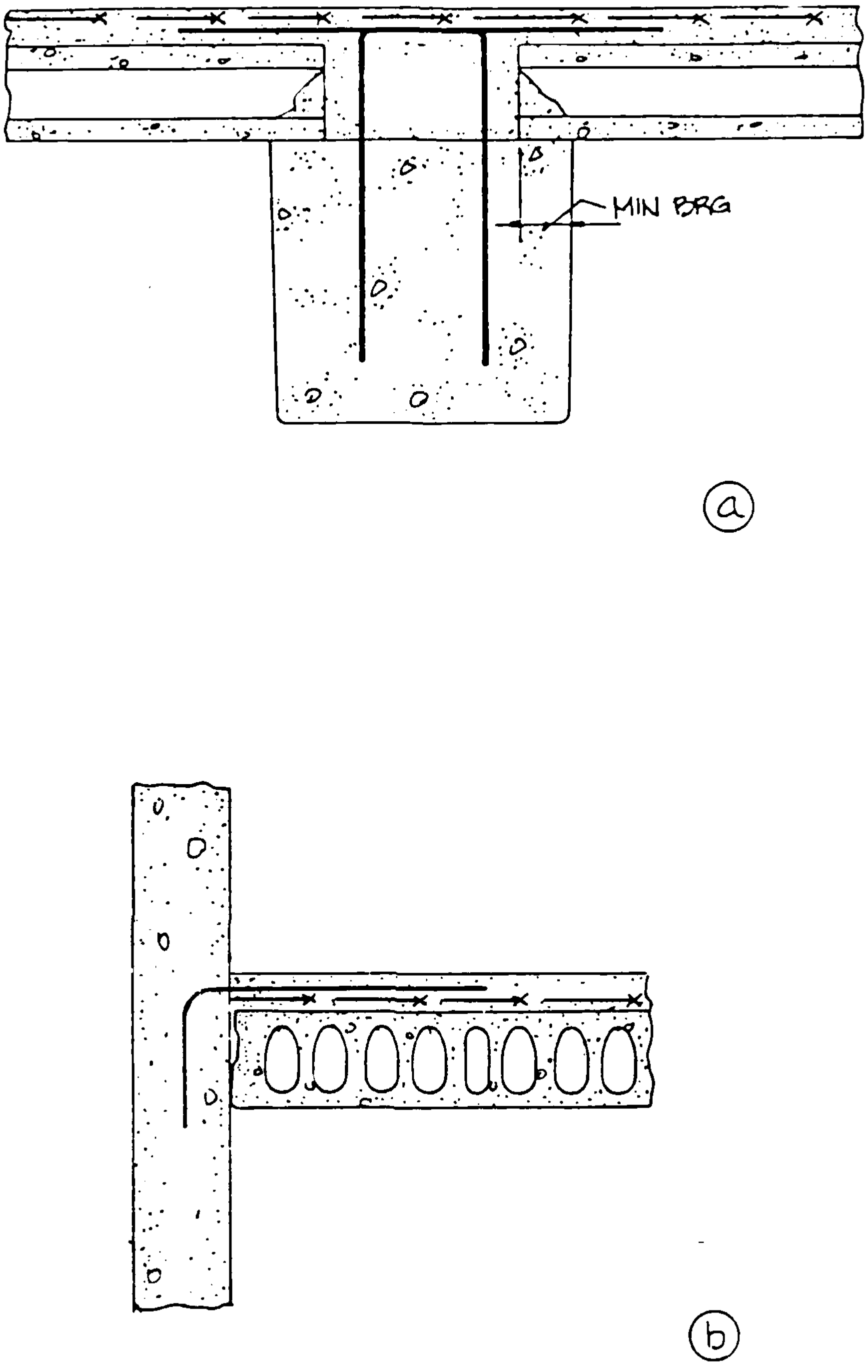


Fig. B-13. Hollow-Core Slab Connections (Topped).

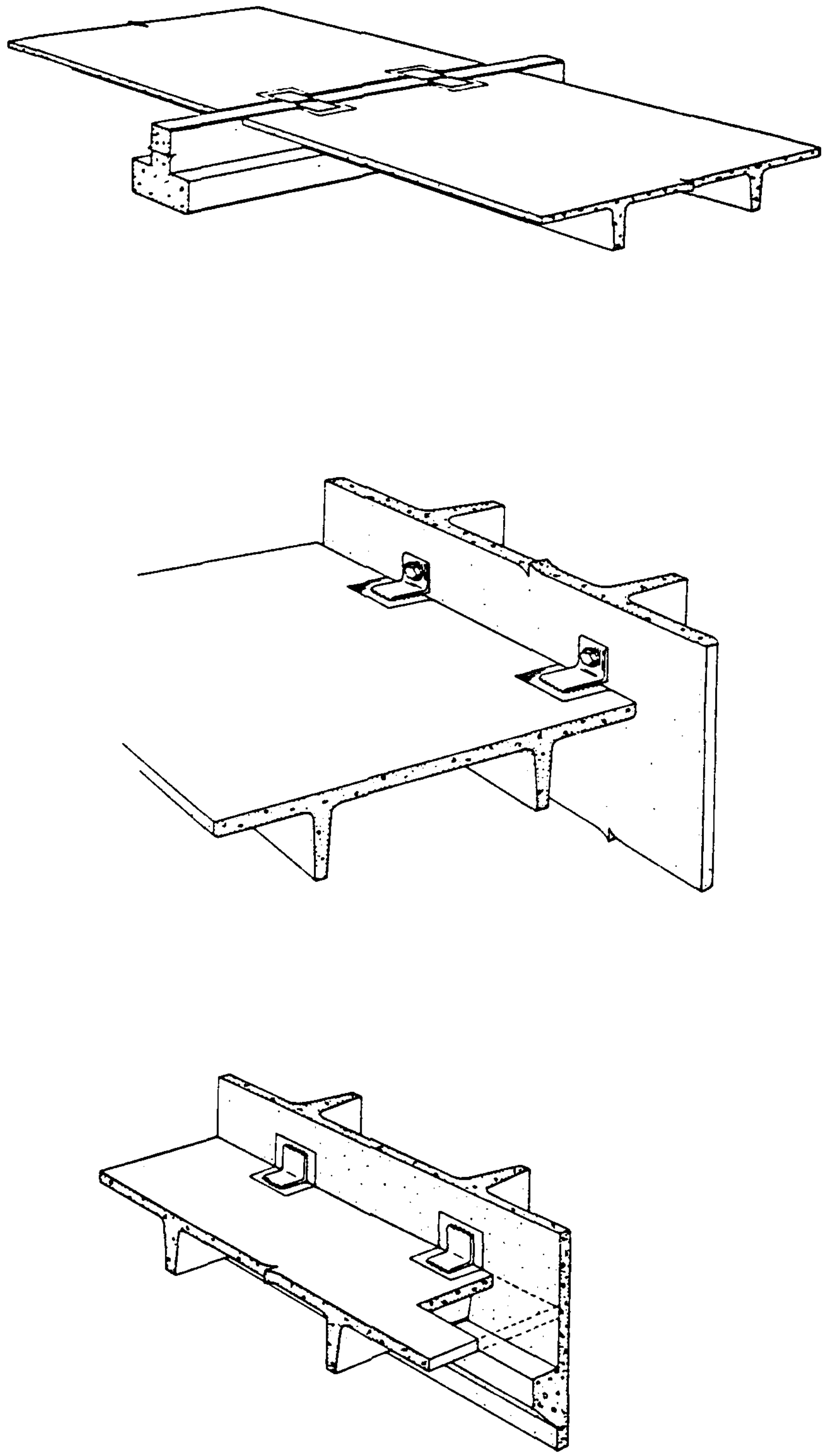


Fig. B-14. Double Tee Connections (Untopped). (Ref. 21)

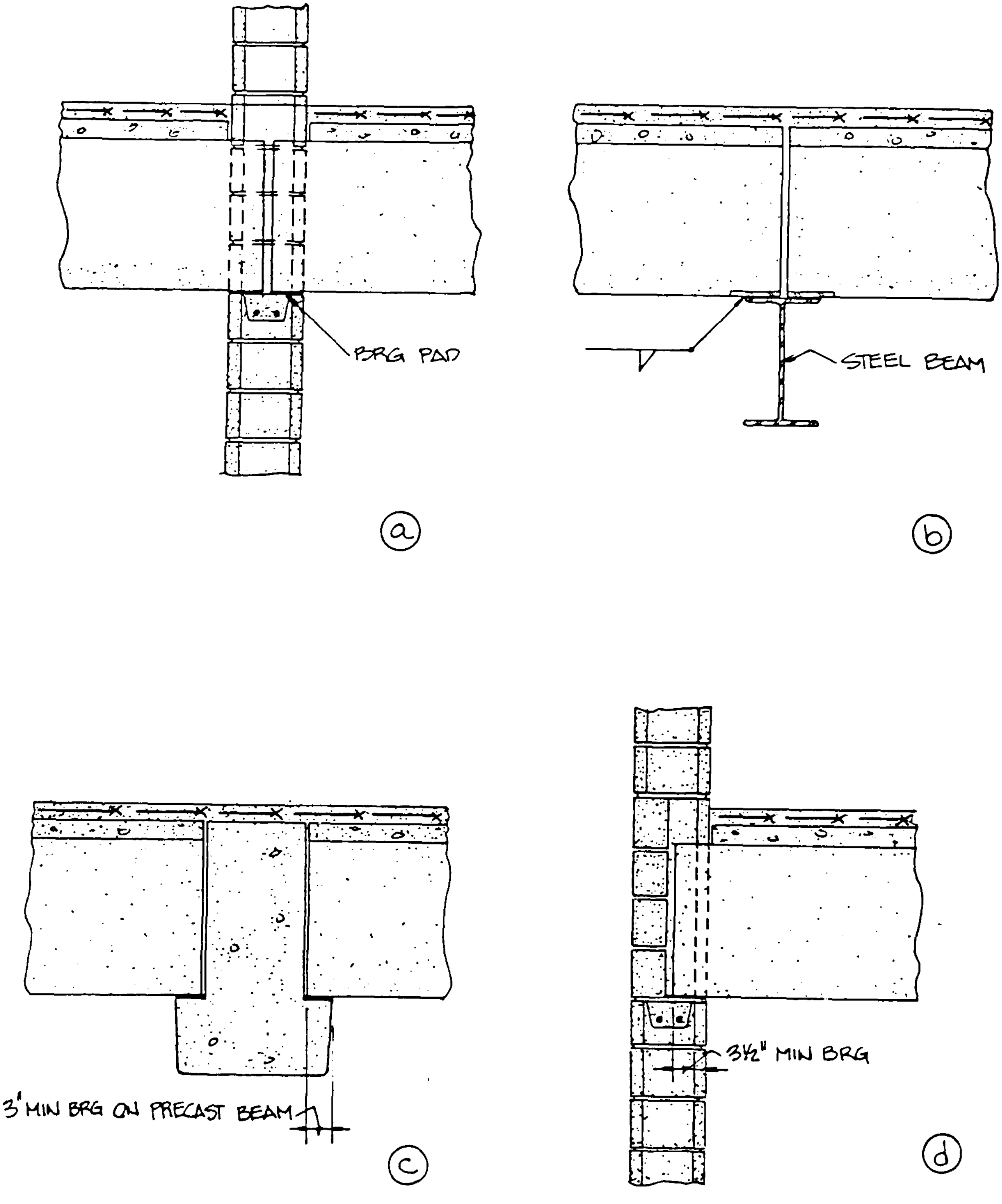


Fig. B-15. Double and Single Tee Connections (Topped).

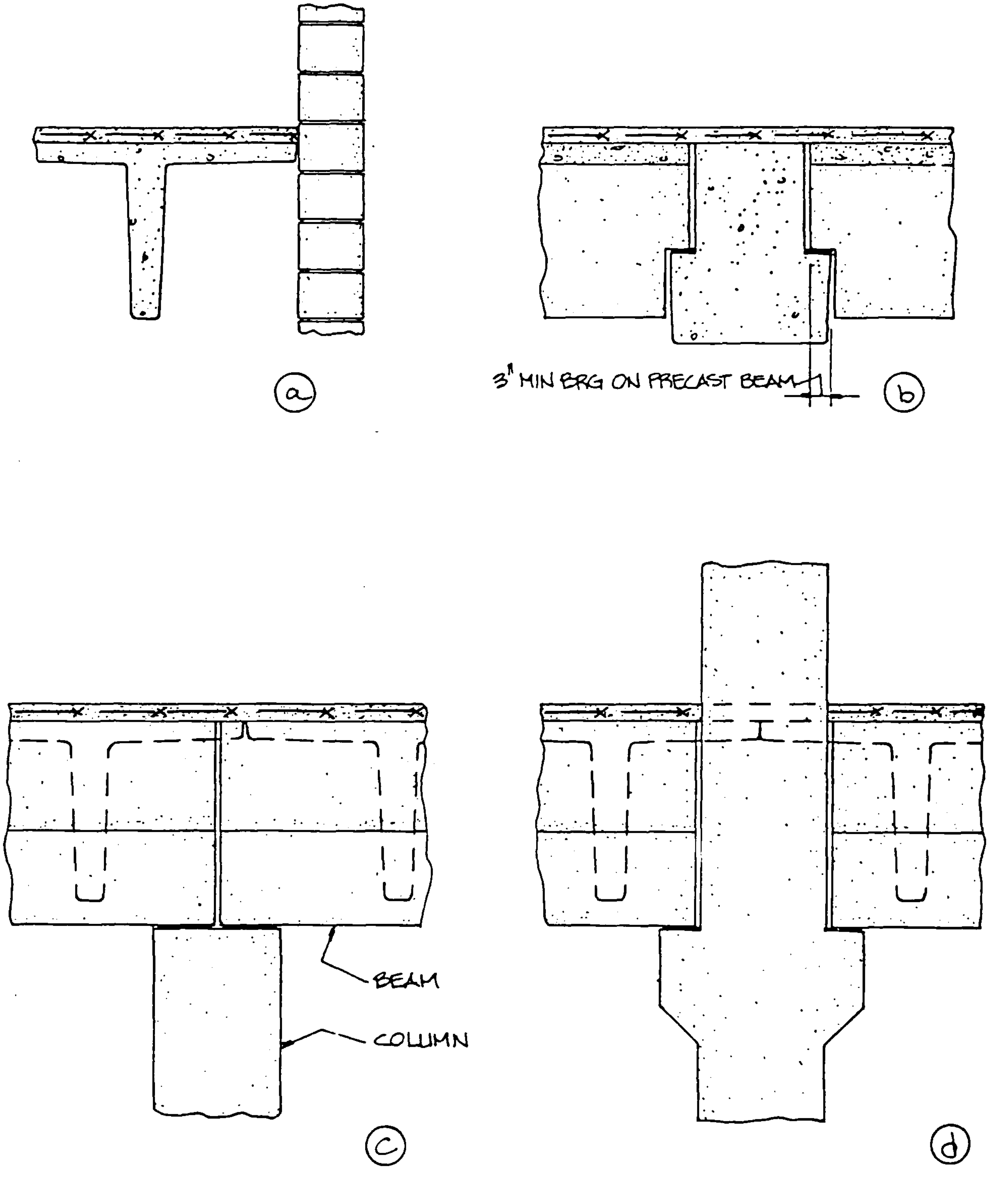


Fig. B-16. Double and Single Tee Connections (Topped).

prestressed concrete members are never welded at their supports: they are left to 'float' free on neoprene . . . ." In structures older than 15 years, it is not uncommon to find stemmed floor and roof units with both ends welded, a practice that has been generally discontinued. As might be expected, restraining these units to this degree against volumetric changes can result in considerable distress that, in several cases, has resulted in collapse. One of the more recent notable examples was in Antioch, California, where a partial collapse of a high school auditorium/gymnasium roof occurred after 21 years of service (Ref. 22). Although there were several contributing factors associated with the collapse, it was concluded that the added stresses resulting from volumetric changes triggered the collapse.

Another support configuration common in double and single tees is that of a notch, or "dap", at the ends of the members. This is where the connection is recessed, or dapped, into a member during casting as illustrated in Figure B-16(b). The purpose of this type of connection in stemmed members is to provide a level floor or roof above when members of different depths frame into one another, or to accommodate architectural requirements with respect to floor to ceiling heights or overall height of the structure. These types of connections present special problems to designers because of the several potential failure modes that must be investigated separately. It is generally accepted by the design profession that many of these notched connections were inadequately reinforced prior to 1970, a view that is supported by continuing evidence of distress in this area in older structures. In the last ten years, the design approach to this connection has been significantly revised, and structures constructed during this time probably contain adequate reinforcing. However, because of the special problems that must be addressed in its design and fabrication, these types of connections require investigation with respect to probable failure modes and possible upgrading techniques, prior to the consideration for use in shelters.

All of the above floor elements may be installed in a building with or without structural reinforced concrete topping. When topping is used, it is applied at the building site on top of the previously installed precast floor elements. The topping serves several purposes in this type of construction. It may be used to assure a level floor surface, since prestressed precast units generally contain inherent camber, or upward bow, particularly if the spans are long, and to eliminate the problem of differential camber between adjacent units. It may be used to increase the

structural load-carrying capacity of the units by designing the topping to perform compositely with individual slab units. However, its primary use may be to serve as a lateral load carrying diaphragm in areas of the country where seismic design is a prime consideration. When topping is used, it is normally 2 to 4 in. thick.

If concrete topping is not used over the tee units, the differential camber problem is solved and the lateral diaphragm developed by the welding of embedded plates cast into the flange edges. Untopped solid and hollow-core slabs, depending on the design and fabrication method, use either welded edge connections, or achieve the leveling and transfer of the lateral forces by use of the grout keys between units. This type of construction is prevalent in the eastern and southern areas of the country.

It is obvious from the above discussion that individual precast concrete floor elements have very little positive resistance to large dynamic loadings in any direction except downward. Horizontal loads can only be resisted by bearing friction, which is negligible if the elements are properly installed with elastomeric or frictionless bearing pads, and upward loading is resisted only by gravity. When the units are untopped and tied together and the resulting diaphragm secured properly to shear walls, additional resistance is achieved horizontally, particularly if the units are seismically designed. The primary problem in the use of this material with untopped floors in buildings that are considered for use as basement shelters is the previously mentioned one of redundancy. None of the connection methods outlined above is strong enough to transfer large dynamic loading throughout the system. Individual units would be expected to fail, and would, in all probability, drag adjacent units and parts of the supporting structure down with them. Evidence of this type of behavior was observed during a test of untopped hollow-core units subjected to 40 psi at the MILL RACE event (Ref. 10). Without considerable further testing and evaluation, and development of further upgrading methods, it would not be recommended that structures with untopped precast concrete floors be considered for use as shelters.

Although there are no test data for substantiation, it would be expected that precast floors topped with structural concrete would perform better. The topping is continuous over a number of the units and is connected to the vertical lateral resisting elements (usually shear walls). The topping would assist in holding the units together and in resisting movement away from supporting elements. It is

possible that the use of some form of topping that could be expediently applied would serve to some degree as an upgrading method for untopped systems. Testing of these units with topping is required in order to develop the required survival parameters for these systems so that they may be included as possible shelter options.

Beams and Girders - The differentiation between the terms "beam" and "girder" is not a significant one with respect to this discussion. If any distinction is required, it is usually required only for clarification during discussion of a number of similar, but different size, structural members in a particular building. In those cases, the beams are the smaller members and may be supported by girders. The design, fabrication, erection, and performance of beams and girders are essentially similar, and in order to minimize confusion, we will use only the term girders to be consistent with the terminology of the precast concrete industry.

Precast concrete girders may be fabricated using either prestressing strands or conventional mild steel steel reinforcing as the primary reinforcement. Generally, because of the economics involved, the majority of girders produced today are prestressed. This was not true 20 years ago. However, the method of reinforcement has no bearing on an investigation of connections, nor does it significantly affect the anticipated performance under a large, one-time, dynamic loading.

Girders are normally used in a structure to support other precast concrete floor or roof components, such as the solid and hollow-core slabs and the tee units described previously. Their purpose is to transfer the vertical loads from these components to the columns and/or wall systems. Although they may be manufactured in many cross-sectional shapes, they are usually classified in three general categories: rectangular, ledger, and inverted tee. They are normally fabricated in depths up to 48 in. for use in buildings, with larger sizes and additional shapes ("I" shapes, etc.) common in bridge construction. The standard cross sections are shown in Figure B-17.

The end bearing details and connections for girders are basically the same as those used for tee units, as described previously. Some of the more typical are shown on Figure B-18. They may be bearing on elastomeric or other types of bearing pads, or may have a welded connection at one end, and frequently have notched, or dapped, ends, as shown on Figure B-18(c). The connection shown in Figure B-18(d) consists of reinforcing bars projecting into tubes cast into the ends of

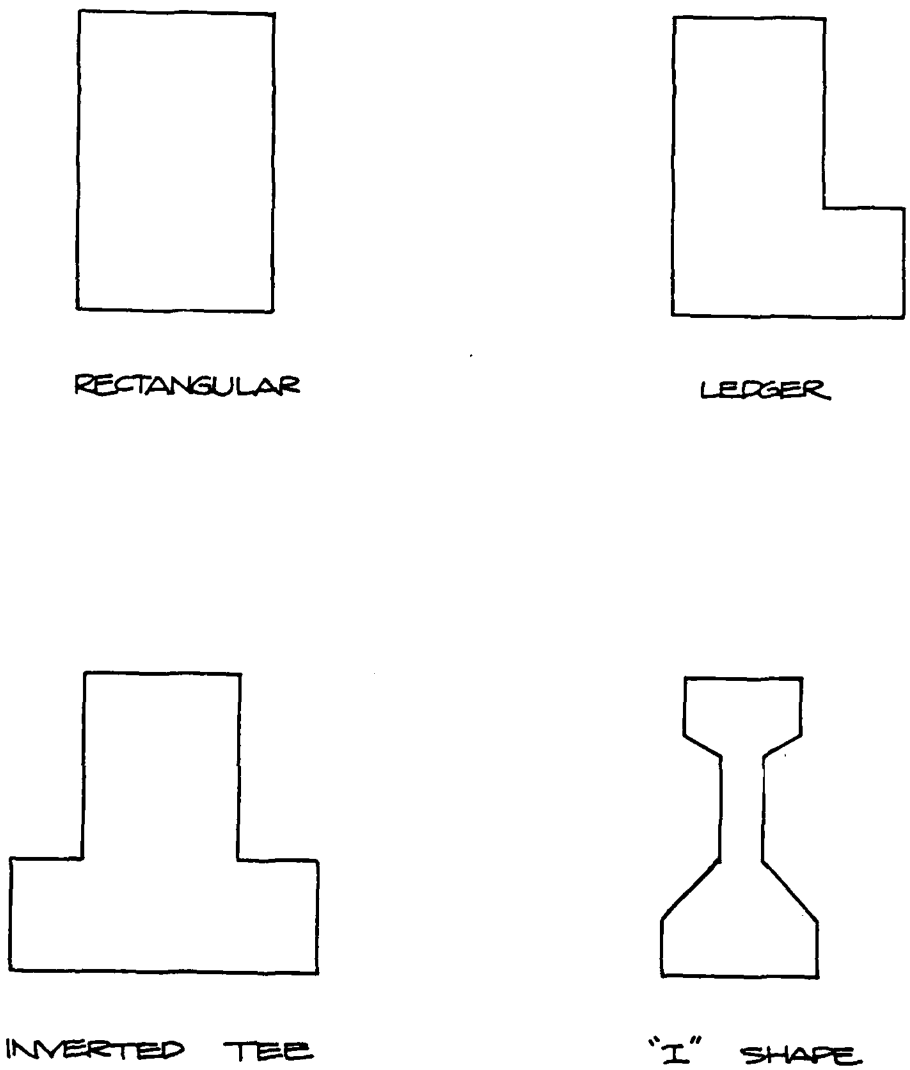


Fig. B-17. Girder Shapes.

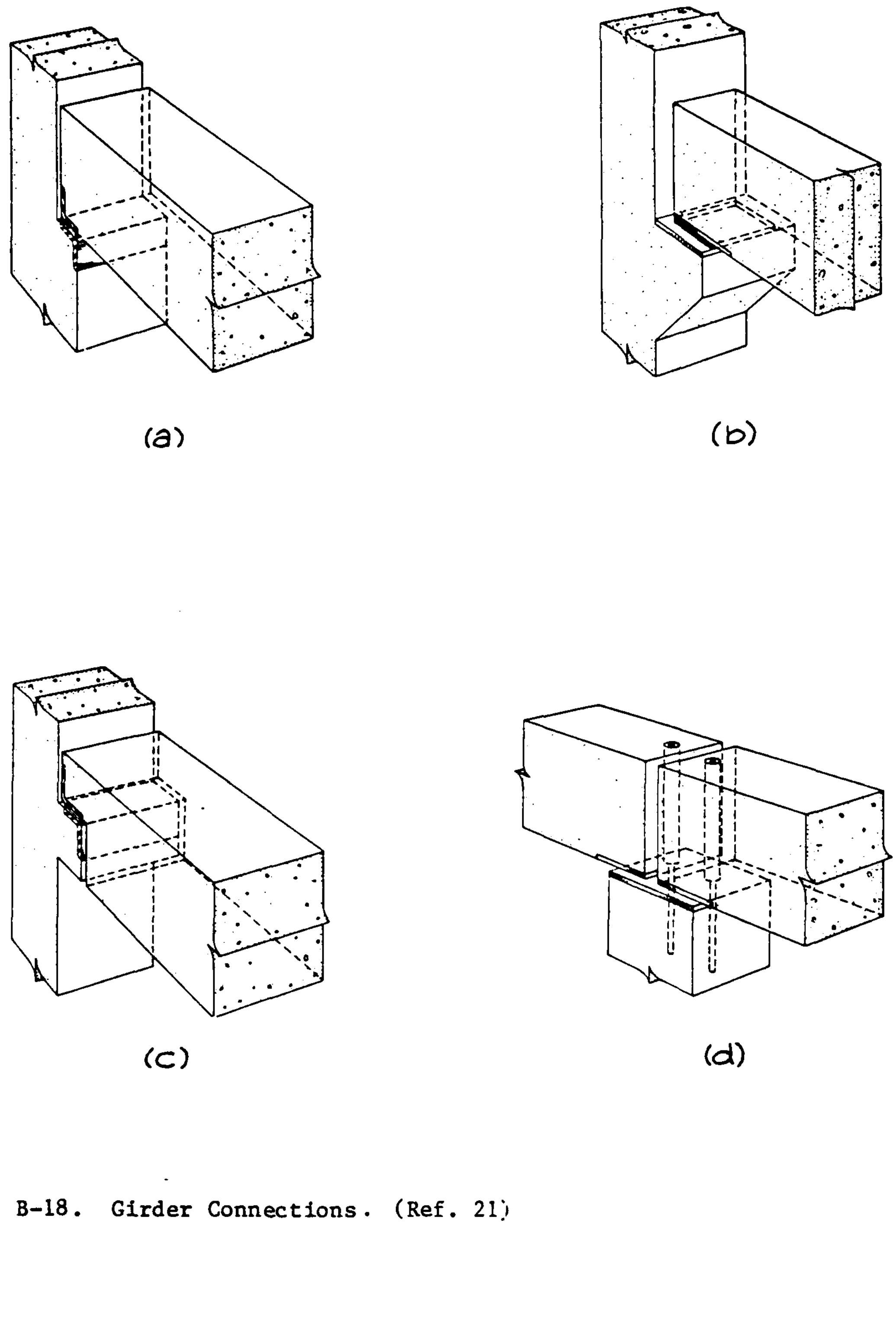


Fig. B-18. Girder Connections. (Ref. 21)

the girder, and is not designed to provide significant restraint. Precast girders are rarely designed with continuity at intermediate supports, but when the precast floor elements are topped with structural concrete, they are usually designed compositely with the topping.

Since precast concrete girders have many of the same connection characteristics and are designed similarly to tee sections, it would be expected that their performance under blast loading would also be quite similar. This is true to some degree. Girders supporting floor units that have structural topping will certainly perform better than those supporting untopped units. However, unlike single or double tees, girders have much more stability and mass, and do not rely on the relatively thin top flanges for integrity. For example, single tees are extremely unstable unless lateral support is provided by adjacent units or walls, a fact that is taken into account during their erection. Girders, on the other hand, are relatively stable whether supporting a floor or not, and their mass alone makes them resistant to being displaced by much lighter precast floor or roof units. Two precast girders were tested at the MILL RACE event (Ref. 10). Each of these girders carried untopped hollow-core slabs on either side, and was supported at its ends on concrete corbels without any positive connection. Both girders were shored to partially withstand the 40 psi overpressure. Although the slabs on either side failed almost completely, the girders remained in their original location with only slight damage.

Areas with respect to precast girders requiring investigation include the notched end bearing configuration, and the development of upgrading techniques to resist horizontal translation. Existing hardware, with some modification, now used on bridge girders to resist lateral and vertical motion caused by earthquakes, may hold promise in this latter area as an upgrading method.

Columns - The performance of precast concrete columns and their related connections under blast loading cannot be discussed in the same context as other precast concrete elements for several reasons. First, from a practical standpoint, precast concrete columns cannot be upgraded, nor can their primary connections, at the base or column splices, be strengthened or upgraded to any degree. Second, the ability of a precast column to perform as designed; i.e., to support the precast elements resting on or connected to it and remain vertical, is to a large degree dependent on the performance of these elements and their connections. The state of the art in using precast columns is such that the elements are normally used only as

vertical load resisting elements in buildings (Ref. 18), and thus, have little resistance to lateral loading. In a completed structure, the precast floor elements, tees or slabs and girders, provide some degree of lateral support for the columns. However, if these elements either partially or wholly collapse, the columns would not be expected to survive.

Precast concrete columns may be of any cross section, but square or rectangular cross sections are the most frequent. They normally range in size from 10 in. by 10 in. to 30 in. by 30 in., and seldom are longer than about 50 ft (5 stories). If the columns are more than one story in height, corbels will be located on one or more sides in order to provide support for the precast girders. Precast columns may be either prestressed or conventionally reinforced with mild steel. Usually mild steel reinforcing predominates.

The typical base connection for precast concrete columns is achieved by use of a bolted base plate (Ref. 20). This connection consists of a steel bearing plate, with predrilled holes, cast on the bottom of the column and welded to the vertical main reinforcing steel of the column. Anchor bolts are cast in the footing, and during erection the column is positioned over the bolts and set to the desired vertical alignment and elevation by the use of leveling nuts on the bolts. Figure B-19 shows several different configurations of these connections. The column is braced in this position and the space between the bottom of the plate and the top of the footing is drypacked, usually with a non-shrink, high strength grout. The design of this base connection is based on the vertical loads that occur in service and any erection loads; as mentioned above, the connection has little ability to resist lateral loads. This connection design is an important consideration in the performance of this type of building when subjected to lateral loading, particularly in relatively stiff shear wall buildings (Ref. 18). The proper design and construction of bolted base plate connections, using the proper elastomeric bearing pads at the beam/corbel interfaces, allow sufficient drift movements to be accommodated in the total structure without distress.

If columns require splicing, either to achieve a longer column or to provide for some type of discontinuity, the connections are similar to the base connections, and are shown in Figure B-20.

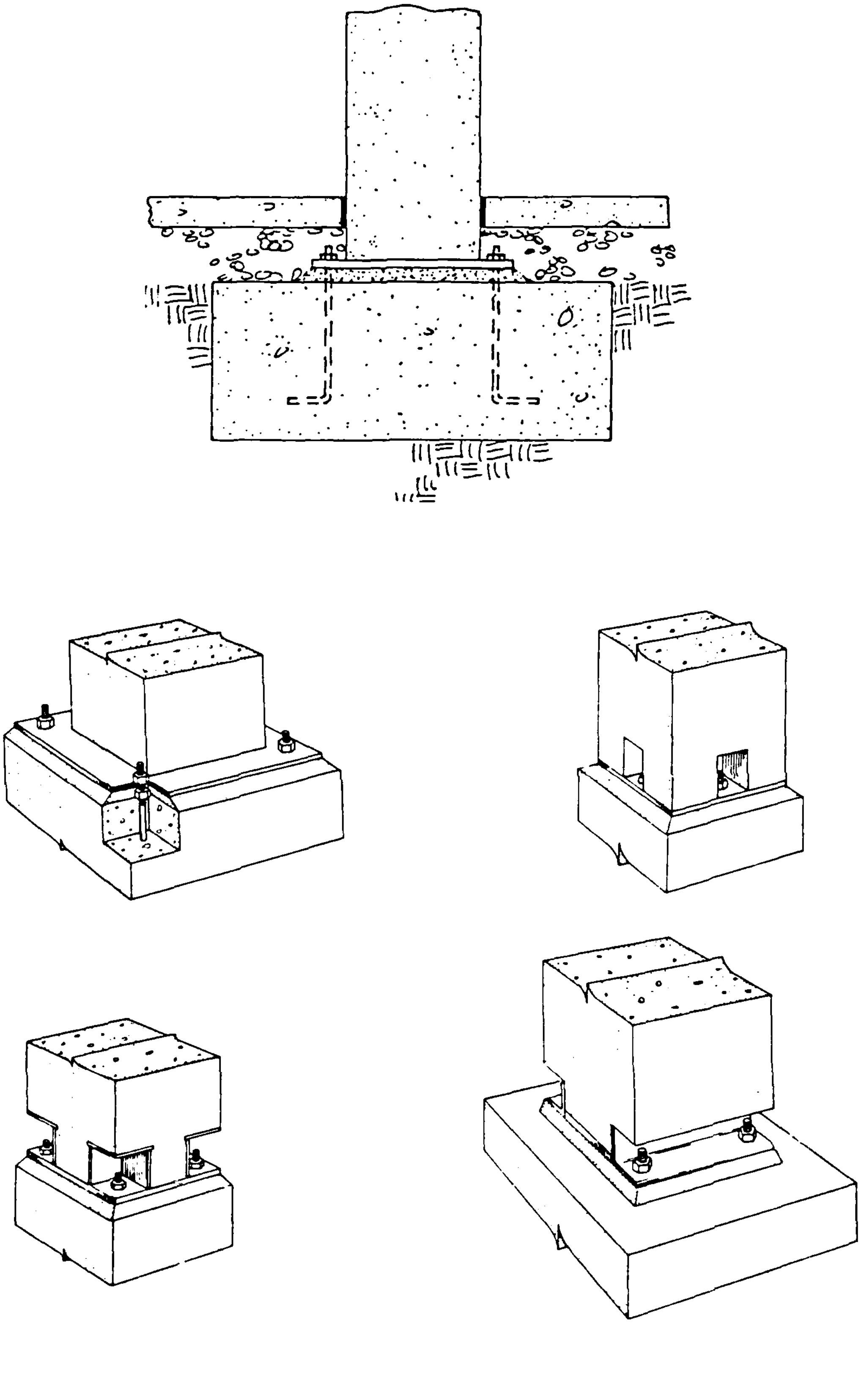


Fig. B-19. Precast Column Base Connections. (Ref. 21)

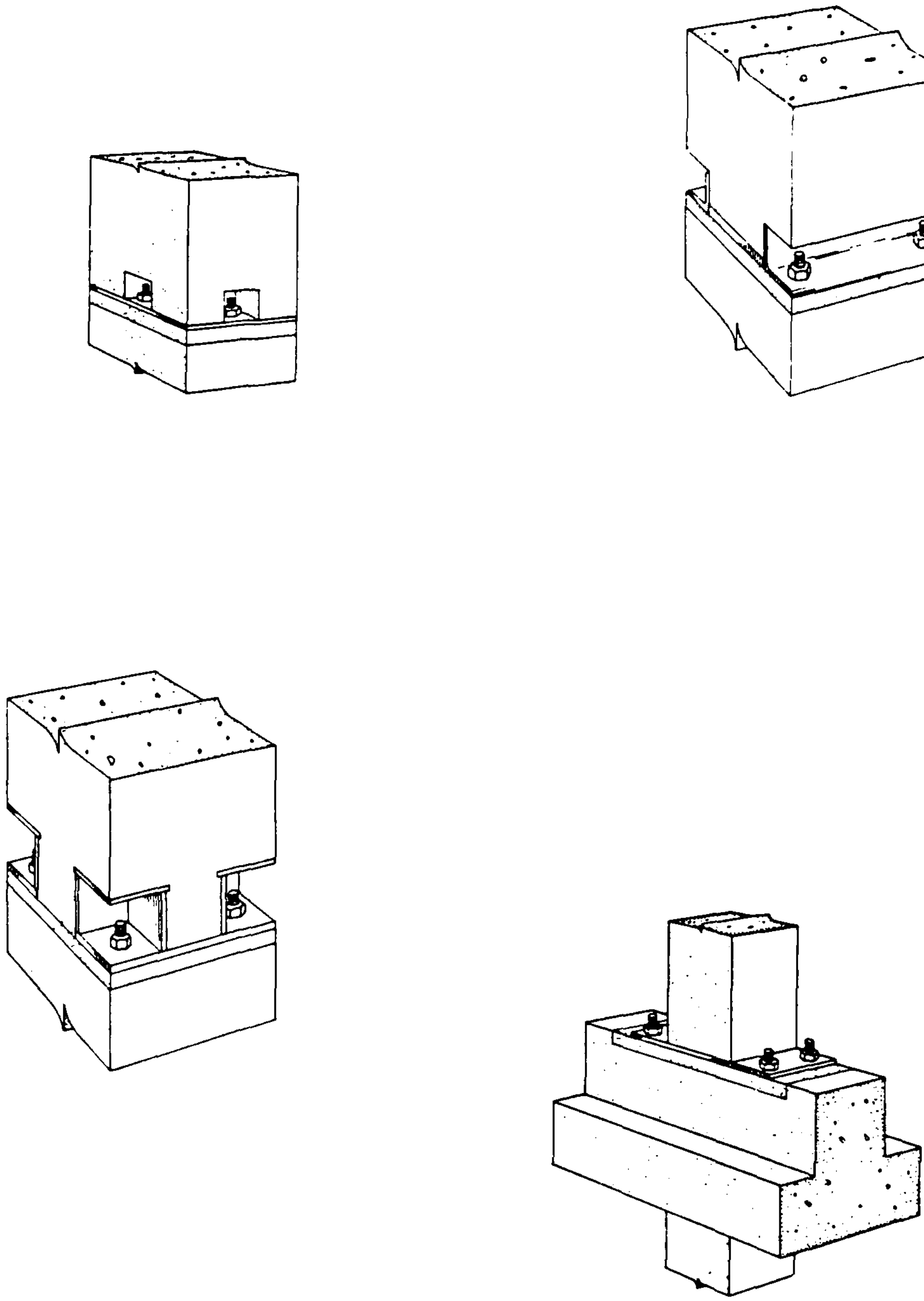


Fig. B-20. Precast Column-to-Column Connections. (Ref. 21)

The primary difference between cast-in-place and precast columns in performance under unusual loadings is significant, particularly if loads are applied laterally. A cast-in-place column is cast around reinforcing steel protruding from the footing. This steel is lapped with the longitudinal column steel, and even if not designed as a moment resisting connection, it provides considerably more structural redundancy than the bolted plate connection by distributing transient stresses to the footing. Cast-in-place columns in precast buildings are not uncommon, and, in general, they would be expected to provide greater resistance and perform better than comparable precast columns under blast loading. Figure B-21 shows two typical cast-in-place column connections.

Wall Panels - Precast concrete wall panels are modular elements used to form the above-grade envelope of precast buildings. These wall panels fall into three basic categories (Ref. 18):

(1) Non-load bearing cladding panels - designed to support only their own weight and wind or seismic forces normal to the panel - may be spandrel panels, solid wall panels, or window panels.

(2) Non-load bearing shear walls - designed to transfer lateral wind or seismic forces from the horizontal diaphragm to the foundation or other elements.

(3) Load bearing wall panels - designed to support vertical loads from the building framing system, and may also be designed to transmit lateral forces to the building foundation.

Precast wall panels are not often found in basement areas and, therefore, are not discussed in detail here. They have provided valuable insight to wall response for test purposes. Should further research reveal that the above-grade installation of these panels somehow affects the casualty functions for basements, the data may be incorporated under later program options.

Areas that require consideration for further research and investigation are the horizontal panel joint connections, particularly those that indicate the most promise of integrity and possible upgrading; i.e., at the foundation and at the basement ceiling level. Precast wall panels are gaining more acceptance. Based on their popularity, two precast wall panels were tested at the MILL RACE event as basement walls at a 40 psi overpressure. The panels survived the test. These panels were specifically designed and constructed so that the connections were [illegible]

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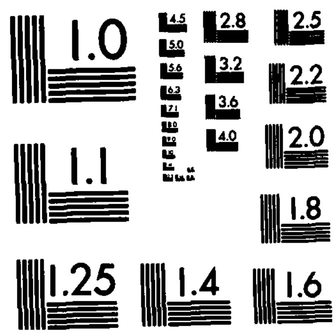
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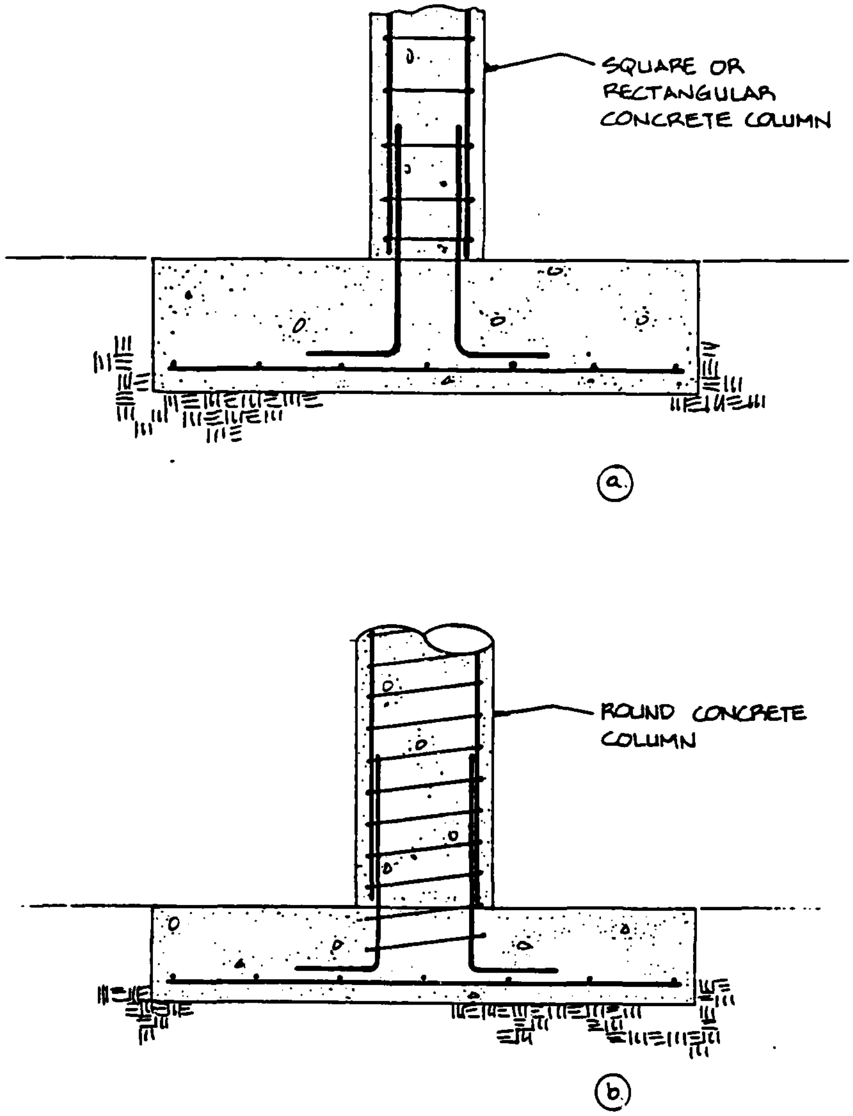


Fig. B-21. Reinforced Concrete Cast-in-Place Column Base Connections.

factor in evaluating the test results (Ref. 10). Since precast wall panels are increasing in use throughout the country, tests are required on these wall panels with realistic connection conditions, particularly at the floor or ceiling level in basement structures at overpressures greater than 40 psi.

Commentary - As one reviews the above discussion on precast concrete elements, connections, and the resulting building systems, several facts become apparent: (1) Although the individual structural precast components are limited in number and can be generally grouped together for evaluation, the methods and hardware used for attaching and securing these components together vary greatly and are difficult to categorize. (2) A building constructed primarily of precast concrete components has little redundancy, and therefore may be unable to adequately redistribute transient stresses caused by overloading throughout the structure. (3) A precast concrete structure may only perform as well as the performance of the weakest components and/or connections. Isolating the collapse of one or more of these weak components might not be possible, thus a progressive collapse of a significant portion of the structure may result.

The above facts appear to severely discredit precast concrete construction for consideration for high overpressure basement shelters. However, the economics of this type construction are such that a large number of these buildings are now in place, and the industry is expected to grow substantially in the future. Accordingly, this type of construction cannot be excluded from the overall basement shelter program. Ways must be found to evaluate these structures and their component connections, and to upgrade, if possible, at least the critical connections and/or floor slab portions of the building to allow their use as basement shelters. Two current FEMA/SSI programs — studying the demolition of existing buildings and the response of building frames to dynamic lateral loading — may yield some useful data in this regard. We are not optimistic that precast concrete is going to be a significant portion of the basement shelter program, when subjected to high blast overpressures.

Summary for Concrete Construction

The expected performance of cast-in-place concrete floor slabs subjected to blast loading, with several exceptions, is generally good. Two-way slabs and flat slabs, in the shored condition, have been subjected to 40 psi overpressure in field tests with good results (Ref. 10). A number of static tests have been conducted on one-way slabs, both shored and unshored (Refs. 11 and 12), as well as dynamic tests

(Ref. 13), all indicating satisfactory performance. Waffle slabs subjected to dynamic loading have performed well, as have one-way joists. Tests, as well as analysis, indicate somewhat less performance might be expected from a flat plate system, but the deficiencies appear to be correctable with proper upgrading techniques. Post-tensioned slabs appear to be questionable with regard to their performance under blast loading, and additional research is required to determine if any upgrading scheme is practical for these systems.

When evaluating cast-in-place concrete buildings as a total system, one would expect that, with the proper upgrading, their performance would be superior to the majority of building systems. The method of construction, the casting of the floor or roof, columns, and sometimes the walls, monolithically, developing to some degree moment resisting connections, gives these systems the distinct advantage of redundancy, and thereby the ability to redistribute stresses resulting from transient overloads throughout the structure. Unfortunately, precast concrete structures lack this redundancy. In general terms, their overall performance as shelters would be suspect. Tests on hollow-core floor slabs have been conducted both statically (Ref. 19) and in the field at 2 and 40 psi overpressures (Ref. 10), with mixed results. Because of the large number of these types of systems in existence, as well as the increasing use of this method of construction, a considerable amount of investigative effort is in order to determine possible upgrading techniques and methods of hardening, or improving, connections.

One approach that holds promise in the evaluation and subsequent upgrading of buildings is that of attempting to tie the lateral design criteria to the anticipated building performance. One common theme that was prevalent in the preceding discussion on concrete construction was that the integrity of the connections, and therefore the building system, was tied more or less directly to the design methodology. If a building is designed to withstand significant lateral loading, either because of code requirements or for other reasons, its performance as a shelter would be expected to be superior to one that is designed and constructed without such requirements. This would be true with all types of structures, but concrete structures, and particularly those of precast design and construction, are uniquely sensitive to lateral load design requirements.

Cast-in-place concrete buildings designed to accommodate large lateral loading may be ductile moment-resisting frames in which the connections are specifically

designed to prevent failure in the joints and confine any yielding to the flexural members (girders) rather than the columns, thus, with the exception of unbonded post-tensioned systems, providing additional resistance to possible catastrophic collapses. Precast concrete buildings normally use shear walls as lateral load resisting elements, and large lateral load requirements result in substantial diaphragm connections between the floor or roof and the shear walls, between the walls and the foundation, and even in the non-structural architectural element connections. These connections, depending on the lateral load requirements, vary considerably throughout the country, and run from virtually non-existent in many areas to quite substantial in others. Although precast concrete buildings may be judged to be far down on the list of viable shelter options, the ability to evaluate these structures as to their relative performance is important to the overall shelter program.

Lateral load requirements for buildings are based on the anticipated horizontal loading resulting from wind or earthquakes. Basically, therefore, the question of defining these requirements is, to a great degree, based on the geographical location of the building. It would appear then that by using published maps, such as those in Chapter 23 of the Uniform Building Code (Ref. 23), indicating the various zones of wind and seismic activity, one would have a simple guide to the design parameters used in particular areas. Unfortunately, however, the solution to the problem is not that straightforward. The UBC is only one of a number of building codes used in this country, and each may have slightly different risk zone maps, and more important, may require different design requirements for an identical risk zone. The reasons for these design discrepancies are primarily psychological and not technical. Portions of the country that have experienced severe wind or earthquake damage, particularly in recent history, are much more conscious of the effects of these events, and emphasize their design philosophy accordingly.

For example, the Uniform Building Code is one of the better building codes with respect to lateral force design. It was developed initially in California, an area of high seismic risk and with a history of recent catastrophic earthquakes. This code is widely used, with only minor variations, throughout California as well as neighboring states. In recent years, this code has been adopted for use in some midwestern and eastern states and cities, but it is significant to note that even in identical seismic zones (zones of supposedly equal risk) the design methodology used for buildings may be considerably different. Sacramento, California, and Charleston, South Carolina, for example, are both located in seismic risk zone 3 according to this code, a zone

that might expect "major damage" from an earthquake. However, probably because of its proximity to San Francisco, both geographically and politically, buildings in Sacramento are designed with more attention to and emphasis on lateral force design, than those in Charleston. Each city has a different building code, which specifies a different approach to lateral force design, even in the same seismic risk zone.

Even with all the complexities involving different codes and design requirements, it is believed that connecting lateral load requirements to expected building performance is a viable approach, and it will be considered in the development of a prediction methodology for the evaluation of shelter options. When viewing this concept on an overall national basis, there is obviously considerable diversity between geographical regions and political jurisdictions; this is not, however, a technical problem with respect to the shelter evaluation process. Only about a half dozen building codes are used throughout the United States, these are updated periodically, and records are available in all cities, counties, states, as to which of them was in use at the particular time. For example, there were no requirements for consideration of lateral forces as a result of earthquakes in the building codes used in southern California prior to 1933 and in northern California prior to 1948. Accordingly, buildings built prior to those years in those particular areas would require downgrading when evaluated as shelters, with these evaluations becoming progressively more positive as the codes in these areas recognized the seismic risk and reflected it in their design procedures.

Another area that will be incorporated into the evaluation of structures for use as shelters, and that ties directly into the lateral force design parameters, is that of the "importance factor" assigned to buildings. In several of the current building codes, the Uniform Building Code for example, both wind and seismic design take this factor into account. The factor is based on how essential the facility is in performing services during an emergency. Structures such as hospitals, fire and police stations, disaster operation centers, and buildings where the primary occupancy for assembly is large, say more than 300 persons, are included. Although there are not a large number of these structures and this factor may not be significant in the overall number of buildings it affects, these types of buildings are usually large and well-built and should be judged accordingly and included in the prediction methodology.

STEEL CONSTRUCTION

This section of the report is a cursory review of steel frames and connections, which will affect the performance of basement areas as shelters. Steel framed structures are numerous and generally are found in conjunction with concrete floors on steel deck systems and one-way and two-way slabs on steel beam systems. The majority of steel framed buidings with potential basement shelters are expected to be slab-on-beam systems.

Steel Frame Connections

Steel construction is predominant in high-rise buildings and often is found in medium rise and industrial buildings. Thus, the probability is high that shelter survey teams will encounter potential basement shelters constructed with structural steel members.

All steel structures are framed construction, with an exterior cladding. In some instances steel-framed buildings have a fire protection covering on the steel members, normally a rough textured cementitious type of material or gypsum wallboard, or tile. In these instances, the steel members and the framing connection elements will not be visible for inspection.

There are three typical methods for connecting steel framed members:

- Rivets
- Bolts
- Welds

Riveted steel connections were the primary method for erection of steel structures until the early 1950's. In 1947, the Research Council on Riveted and Bolted Structural Joints was formed, primarily because maintenance of bridge structures required occasional replacement of rivets, which was time consuming and expensive. The Council developed the first structural bolt specification, which was issued in 1951. The specification had its 10th revision in 1976. From a construction standpoint, bolted steel connections in buildings were not common prior to 1955.

For welded connections, the first treatise "Design of Welded Structural Connections" was copyrighted in 1961 by the James F. Lincoln Arc Welding Foundation. Since that time welded steel building frames and connections have become prevalent in the steel construction industry.

Details of various types and configurations of riveted, bolted, and welded frame elements are shown on some of the following pages. It should be emphasized that modern construction techniques, with emphasis on the economics of shop fabrication of steel members and elements, have resulted in a combination of bolted and welded framing connections. Riveted connections, though popular prior to 1955, are no longer used in modern steel construction. From a structural analysis standpoint, riveted and bolted connnections are structurally equal, although there is mounting evidence they do not respond similarly when overstressed to failure.

Connection Types - In many instances, detailed structural analyses of steel connection elements are more readily performed than comparable concrete elements because concrete reinforcing steel is not visible for inspection during a shelter survey. However, steel framing connections may also not be available for inspection owing to the fire protection on the structural members and connections. Details of typical shear and moment connections are provided in Figures B-22 through B-25. Note that welds and bolts are common on the same connection and that bolts are often used to temporarily fasten elements together for field welding. Figure B-26(a) is a detail showing a typical welded connection to clarify the difference between shear welds and moment welds. Figure B-26(b) is a detail showing a typical welded steel column-base plate connection.

Welded steel frame connections are generally considered as rigid connections, while bolted and riveted connections may be classified as rigid, semi-rigid, or flexible (simple beam). The analysis of response of riveted or bolted connection elements will determine the resistance characteristics of the connection and the connected members. When stressed to failure, a plastic hinge will occur either in one of the members or the connections, and failure will occur in the least resistant element of the joint.

JOINT ELEMENT RESISTANCE FUNCTIONS

Introduction

The analysis of building structures under blast loading requires a knowledge of the effective elastic limit moment capacity, M_p, for the beam and column joints, according to a newly developed analysis method described in Ref. 2. This section of Appendix B provides methods for engineering evaluation of these capacities for the

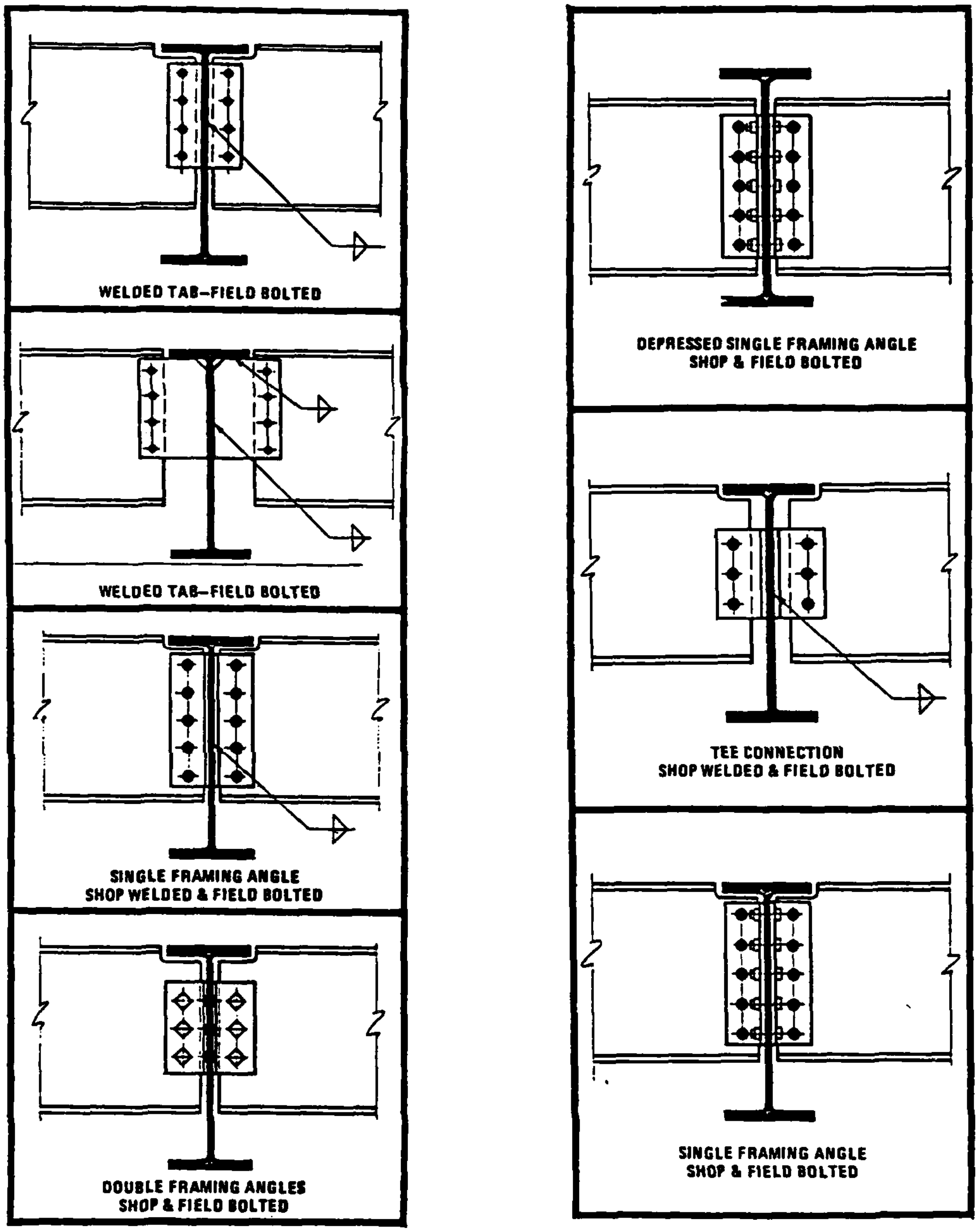


Fig. B-22. Shear Connections - Beam to Girder. (Ref. 24)

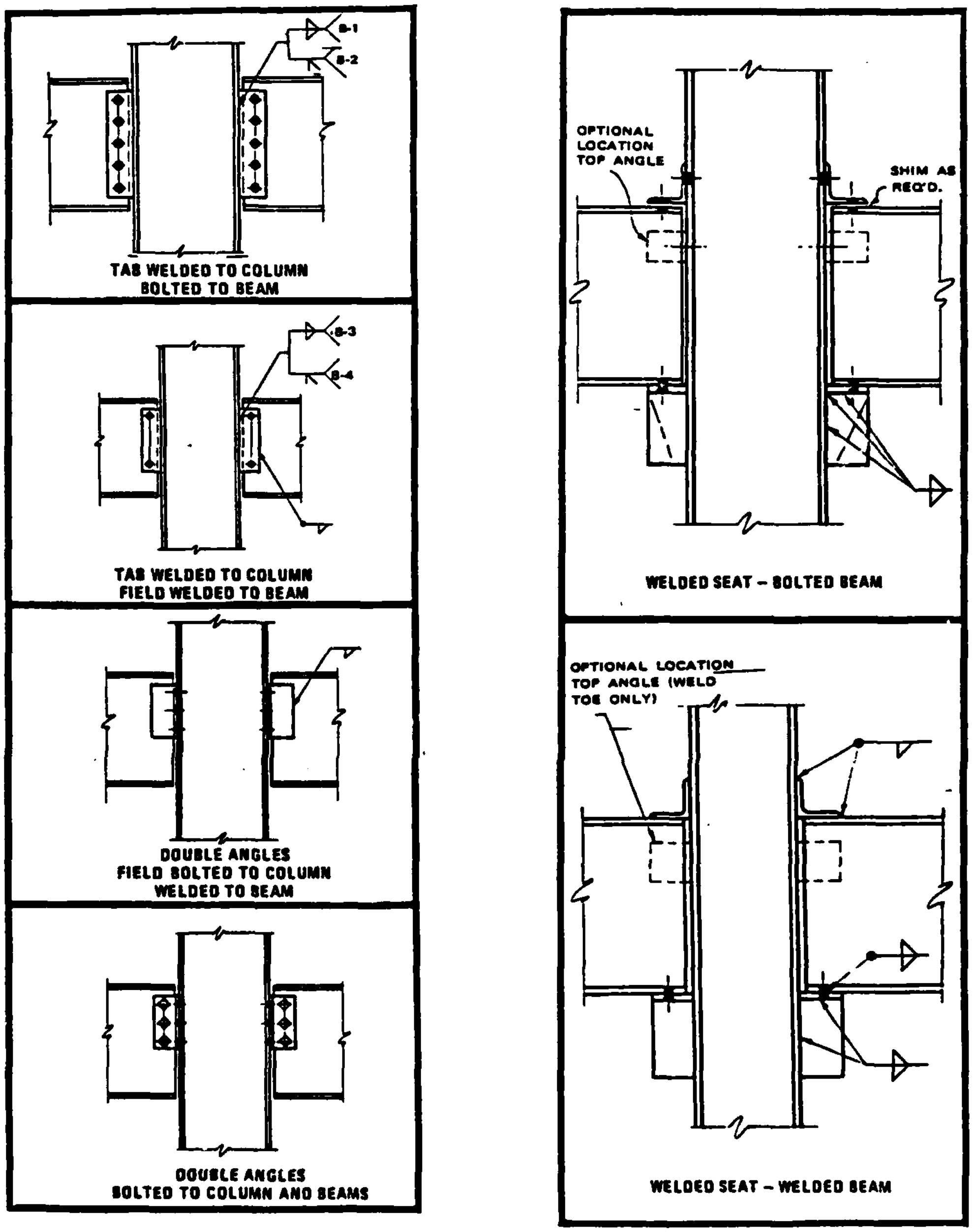


Fig. B-23. Shear Connections - Beam to Column. (Ref. 24)

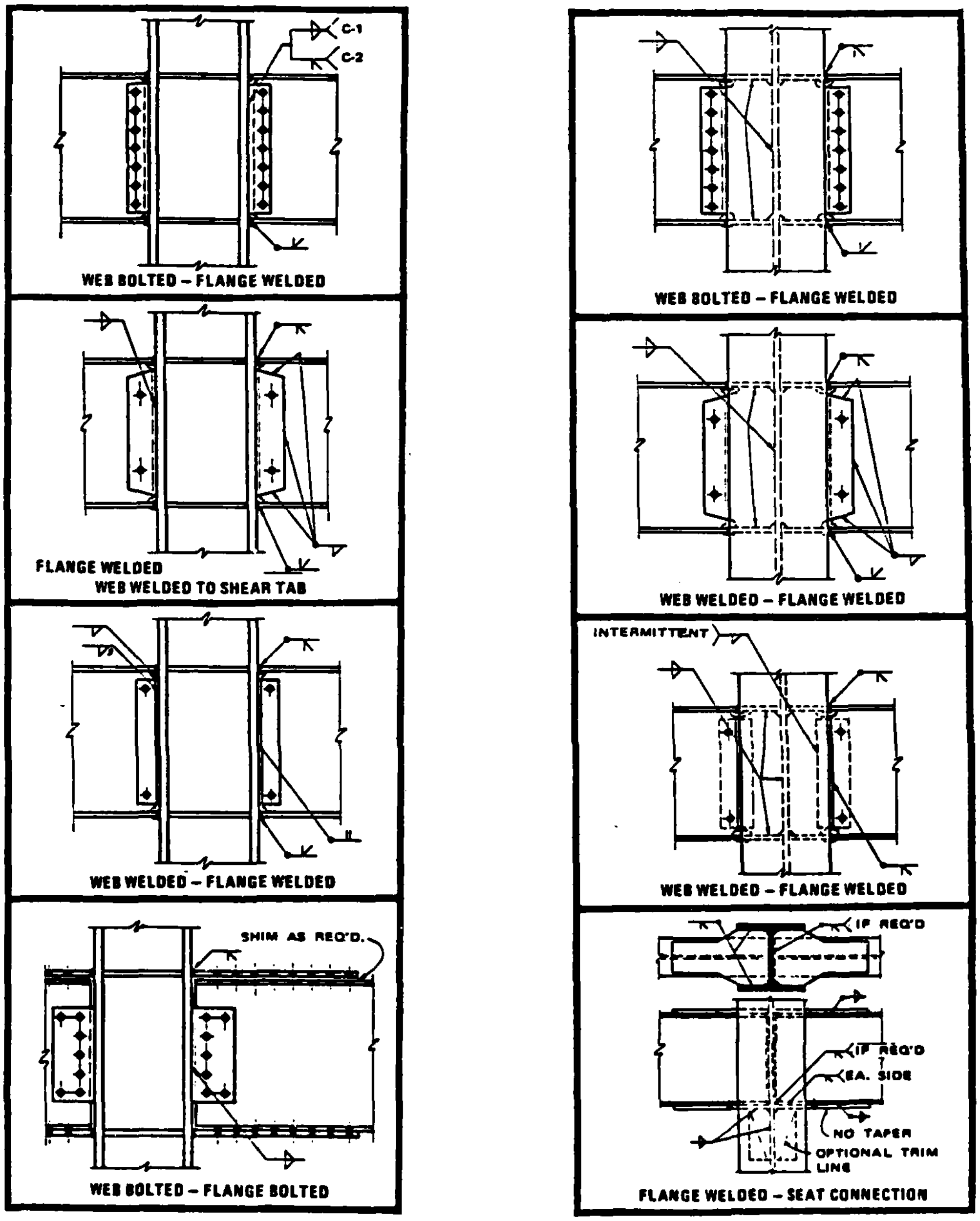


Fig. B-24. Moment Connections - Beam to Column Web. (Ref. 24)

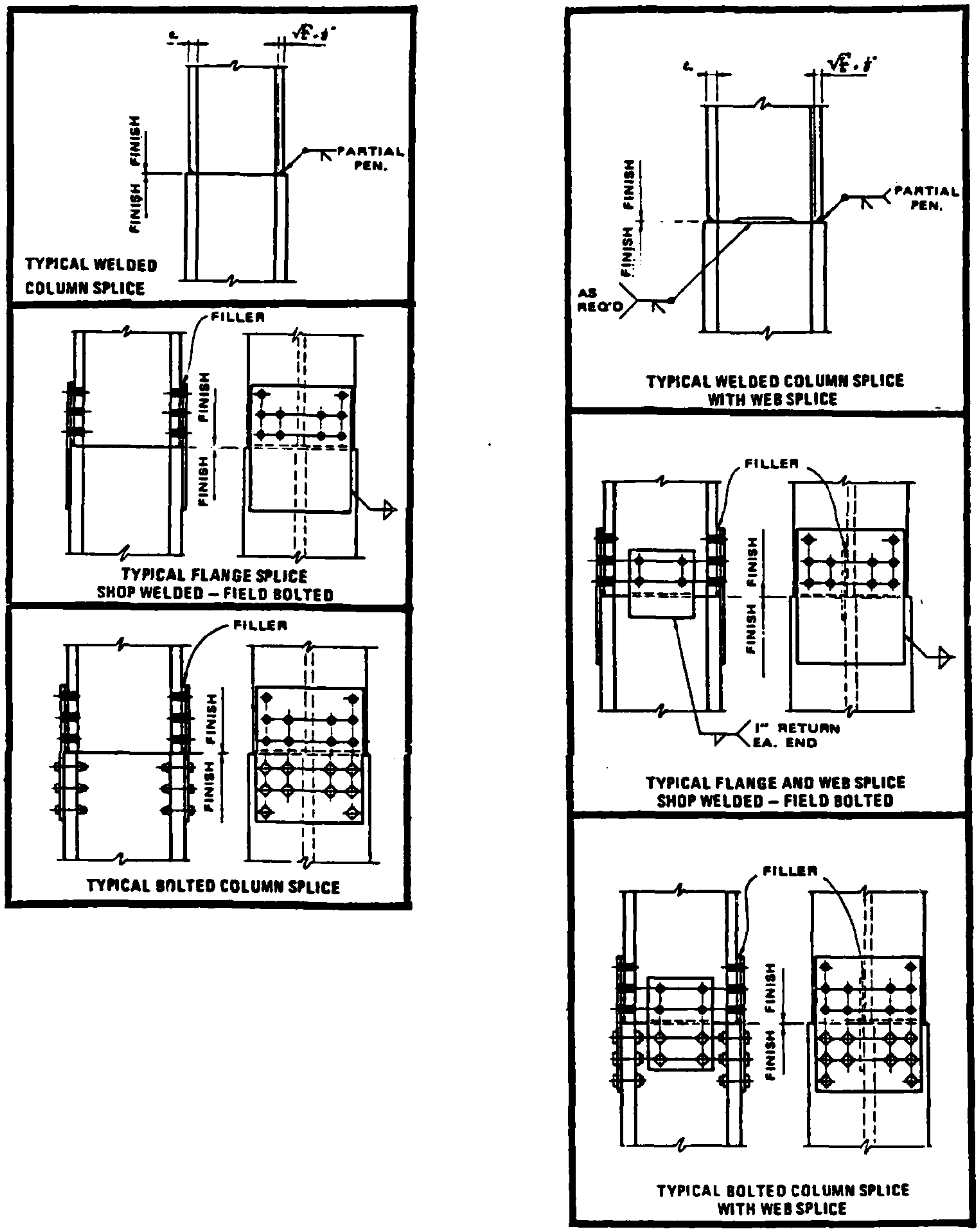
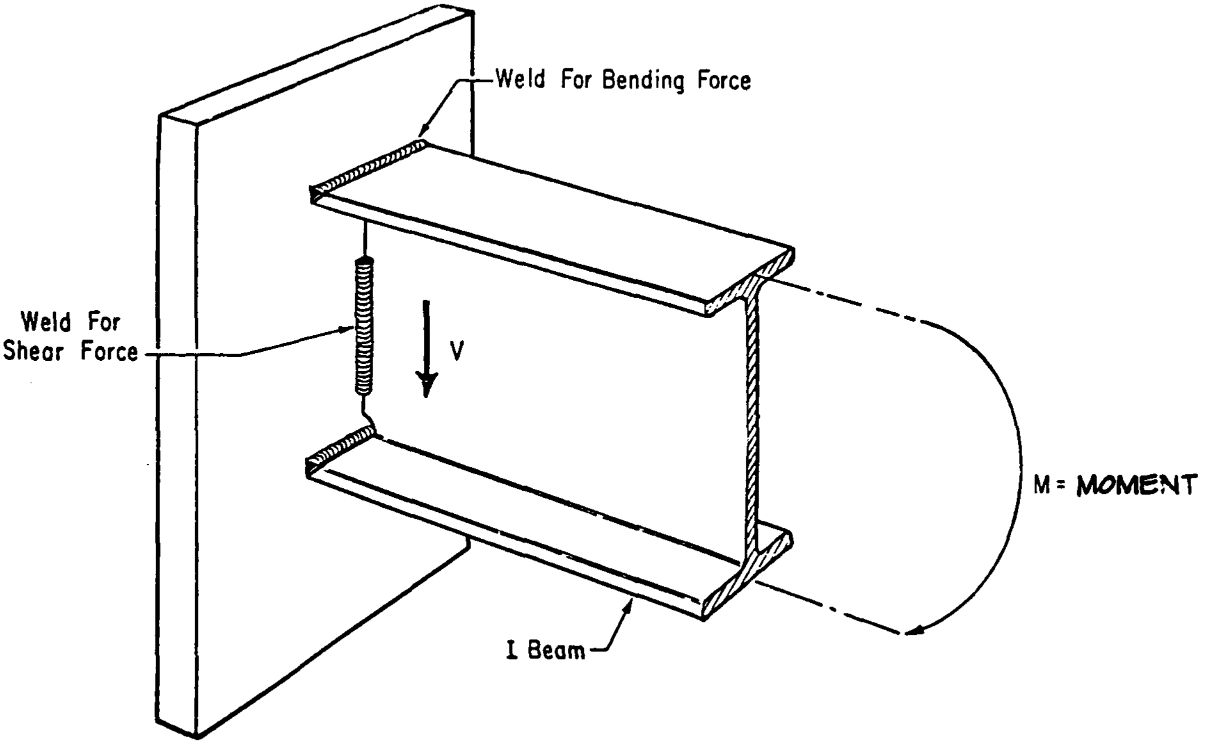
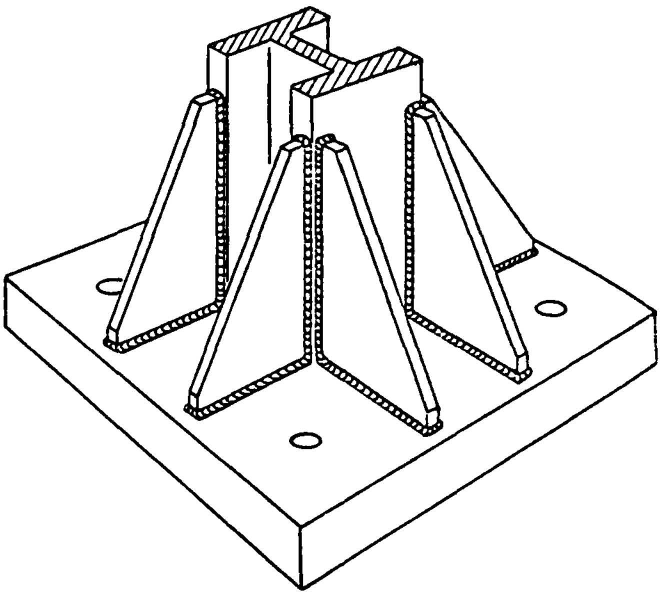


Fig. B-25. Column Splices - Flanges and Web. (Ref. 24)



a. Shear and Moment Welds.



b. Column Base Plate Connection.

Fig. B-26. Typical Welded Connections.

structural elements in both steel and reinforced concrete frame buildings and is limited to the beam and column elements of frames. The principal objective of the analysis is to determine the effect of the frame column failure at the ground floor (basement shelter ceiling) slab. Both the steel and reinforced concrete member capacities will be evaluated on an ultimate strength design basis.

Structural Steel Construction

The AISC Manual (Ref. 25) provides strength information for beams, columns, welded and bolted connections, and splices. Part 2 of the manual gives the specific plastic design or strength values. (See interaction diagram, Figure B-27, and Table B-2 for all definitions):

Beams, $M_p = ZF_y$,

or $M_p = SF_y$ for older non-compact sections.

Columns,

$$P/P_{cr} + (C_m M)/\ (1 - P/P_e)\ M_m \leq 1$$

where $P_{cr} = 1.7\ AF_a$, $C_m = 0.4$ for reversed curvature, $M_m = M_p$. (Strengths for connections and splices are 1.7 times the corresponding AISC Manual Part 1 allowable stress values.)

With respect to beam flexural capacities, older construction may have steel (built-up or rolled) sections that are non-compact, such that they would buckle before developing full plastic capacity. For these sections the M_p value should be taken as SF_y, for the tabulated or calculated section modulus, S.

In any given structure, and particularly in older structures, column splice details (such as shown in Figure B-28) generally constitute the weak link in column M_p values. This has been verified in explosive building demolition. If these splices are weak in flexural resistance and are located near the column base, then their estimated M_p capacity should be used at these locations. The transverse shear resistance of the splice may also be a weak link and should be investigated.

Also, particularly in older construction, the interior beam column connections may be simple web or flange clip angles with rather minimal M_p values (see Figure B-29). This type of detail should be identified, and the appropriate estimated M_p value should be used in the analysis.

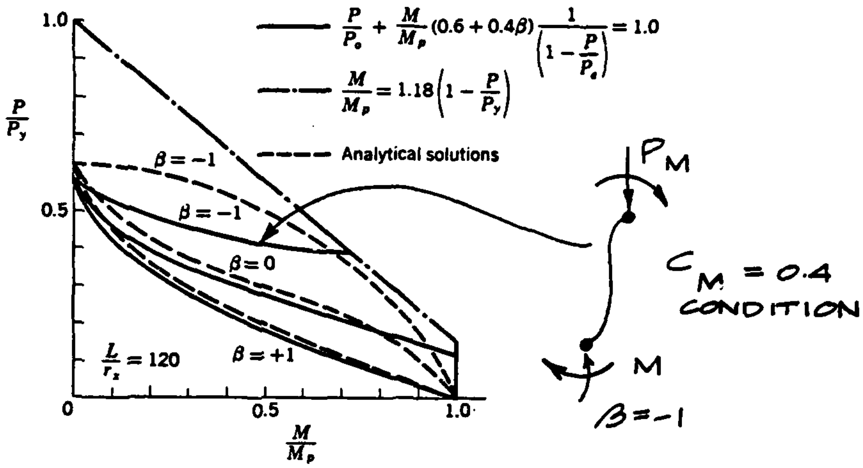


Fig. B-27. Normalized Interaction Diagram.

TABLE B-2: AISC MANUAL, PART 2 DEFINITIONS

A	Gross area of an axially loaded compression member
C_m	Coefficient applied to bending term in interaction formula for prismatic members and dependent upon column curvature caused by applied moments
F_a	Axial compressive stress permitted in a prismatic member in the absence of bending moment (kips per square inch)
F_y	Specified minimum yield stress of the type of steel being used (kips per square inch)
M	Factored bending moment (kip-feet)
M_m	Critical moment that can be resisted by a plastically designed member in the absence of axial load (kip-feet)
M_p	Plastic moment (kip-feet)
P	Factored axial load (kips)
P_e	Euler buckling load (kips)
P_{cr}	Maximum strength of an axially loaded compression member or beam (kips)
S	Section modulus
Z	Plastic section modulus

Column-Related Design

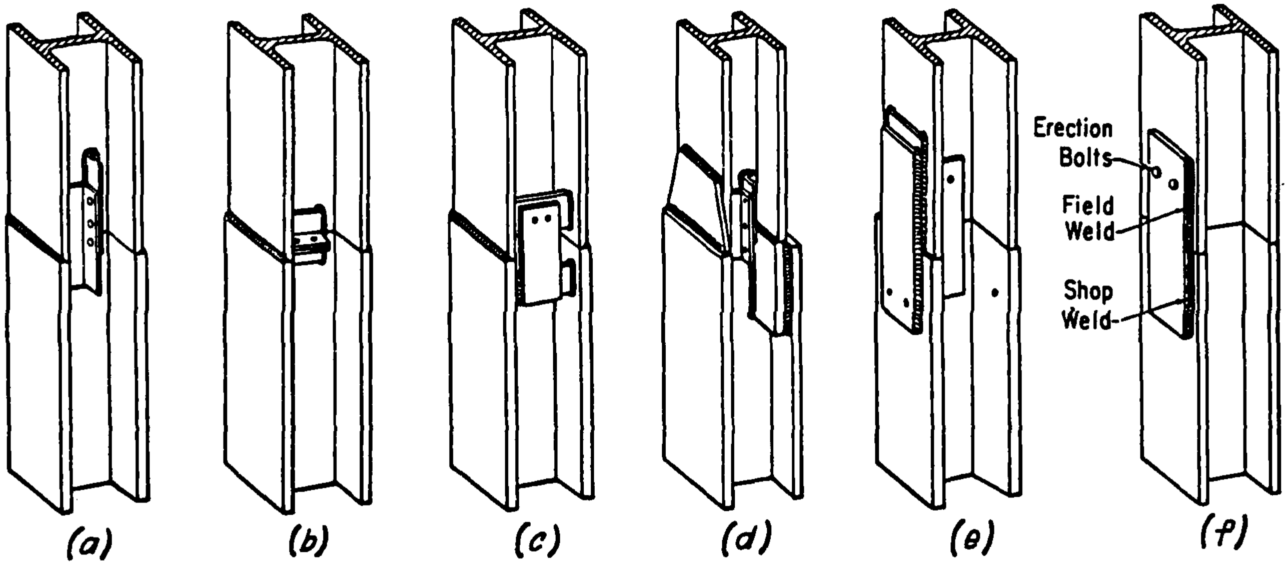


Fig. B-28. Typical Steel Column Splices.

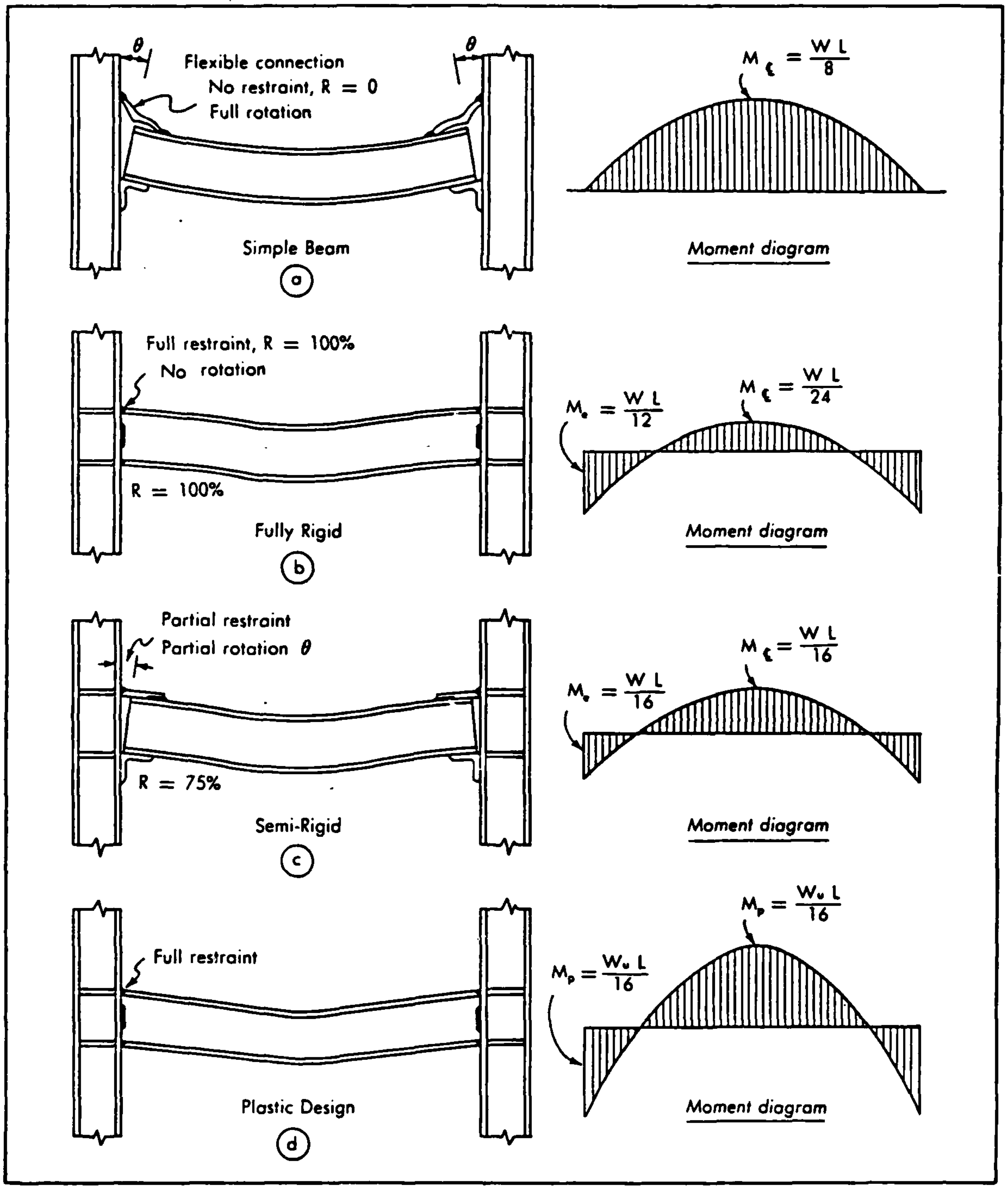


Fig. B-29. Typical Steel Beam Connections.

Recommended ductility ratios, $\mu = M_F/M_p$ are 8 to 10 for fully developed sections or rigid connections, where M_F is the failure moment. Other than for the case of weak splice details, the shear capacity of a steel section will not be less than the shear necessary to develop the M_p value.

Reinforced Concrete Frame Construction

By far the most prevalent forms of construction are the flat slab (or waffle slab) and the two-way slab systems, see Figure B-30a and B-30b. Emphasis in this report is on Figure B-30a. The CRSI Handbook (Ref. 14) provides most of the strength information required for these systems along with the beam and column section capacities for frame elements.

For the majority of slab system frame structures, the equivalent frame is defined by a frame strip along each column line, having a width equal to the bay width perpendicular to the column line. Figure B-31 shows the general flexural section configurations. The flexural strength, M_p, of these beam sections can be taken from the CRSI Handbook using the assumption that strengths for F_y = 60 ksi steel with the ϕ factor are equal to strengths for F_y = 40 ksi without the ϕ factor. The most realistic estimate of M_p would be without the ϕ factor multiplier.

For a suitable approximate estimate of beam capacities, M_p, the following procedure can be applied to the construction elements in Figure B-31.

- o Assumption: Positive steel area equals one-third negative steel area at the column face

  Negative $M_u = (\text{Neg. } A_s)F_y(0.8h)$

  Negative A_s = top steel

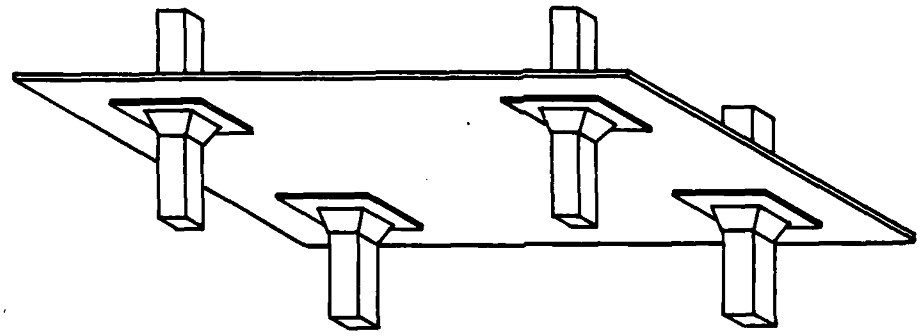
  Positive M_u = 1/3 Negative M_u

- o Assumption: Negative dead load moment equals one-third Negative M_u (see Figure B-32)

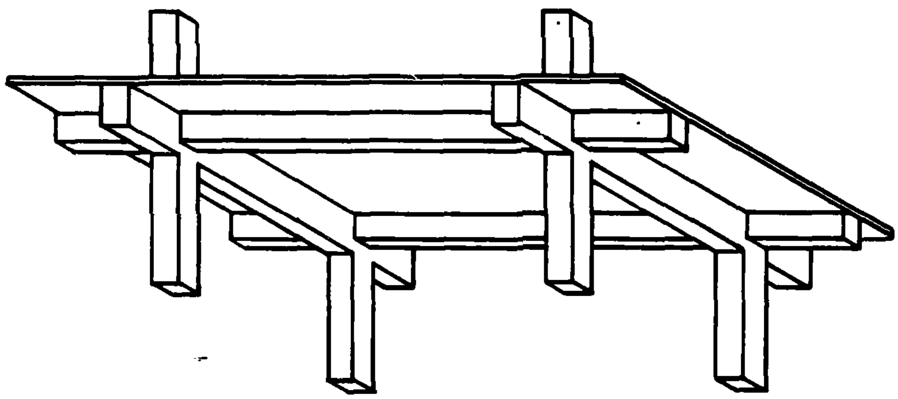
  Negative M_p = Negative $M_u - M_{DL} = 2/3\ M_u$

  Positive M_p = 1/3 Negative $M_u + M_{DL} = 2/3\ M_u$

  Therefore, Negative M_p = Positive M_p

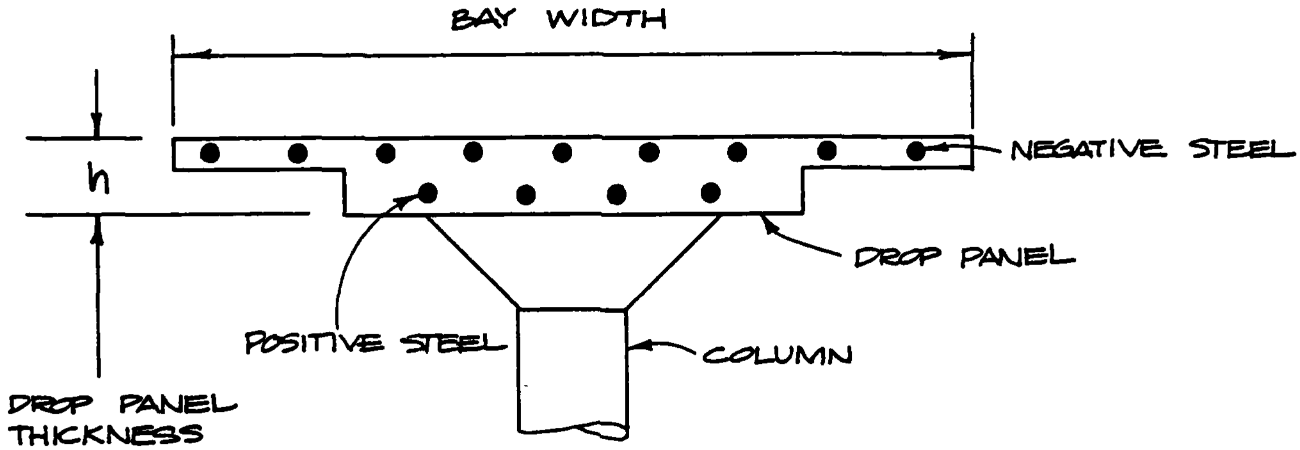


a. The Flat Slab

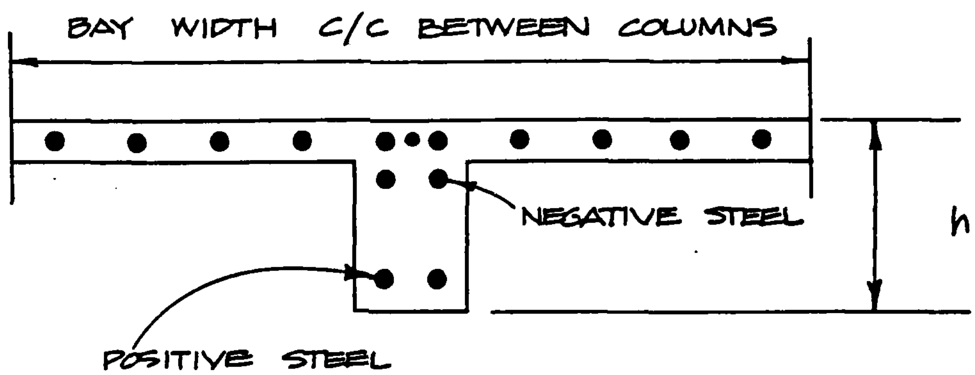


b. The Two-Way Slab

Fig. B-30. Examples of Slab Systems.



a. Two-way Slab.



b. Flat Slab.

Fig. B-31. Slab System Beam Sections.

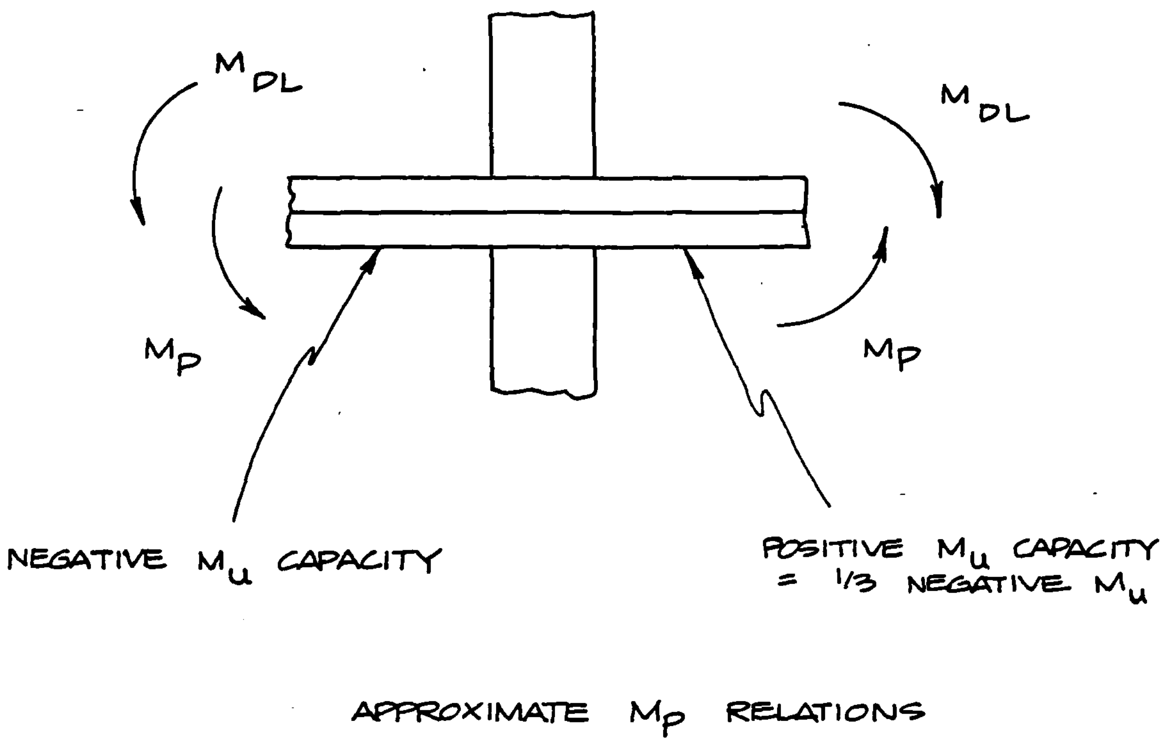


Fig. B-32. Approximate M_p Relations.

For column M_p values, under the effects of axial and flexural load interaction, the interaction tables of the CRSI Handbook provide that $M_p = P_u$ times the eccentricity e. These values are for $F_y = 60$ ksi with the ϕ factor and can be assumed to be equal to the M_p for $F_y = 40$ ksi without the ϕ factor. Figure B-33 shows an example interaction curve.

It is important to recognize that columns may fail in shear; this capacity can be estimated by

$$V_u = 4\sqrt{f'_c}\, A_c \approx 250\, A_c \text{ lb}$$

where A_c is column section area in square inches.
The shear stress of $4\sqrt{f'_c}$ represents the presence of shear stirrup steel in the form of column ties.

Also, reinforcing steel splices may provide a weak link if the spliced bars are not staggered; one-half M_p might be used when this splice condition is present.

Recommended ductility ratio $\mu = M_F/M_p$ is 5 for fully developed sections of rigid frame concrete joints. It can thus be seen that concrete is approximately half as ductile (more rigid) than steel with ductility ratios of 8 to 10.

EFFECTS OF SHORING

When they are shored, the frame response of the structures will be much different than in their unshored (as-built) condition.

Joints at the Ground Level Slab Intersection

Although the joints that exist at the ground level; i.e., at the superstructure of the basement, are similar to other joints in the structure, they behave much differently. The reason(s) for this behavioral difference is that the upgrading of the basement structure will greatly change (enhance) the floor stiffness. The slab portion of the structure is to be shored, which will increase its stiffness relative to the as-built, pre-upgrading stiffness. The dead load will also be increased two to three times owing to the depth of soil needed for radiation protection. This shoring will be allowed for in the upgrading index. In addition, the slab restricts lateral

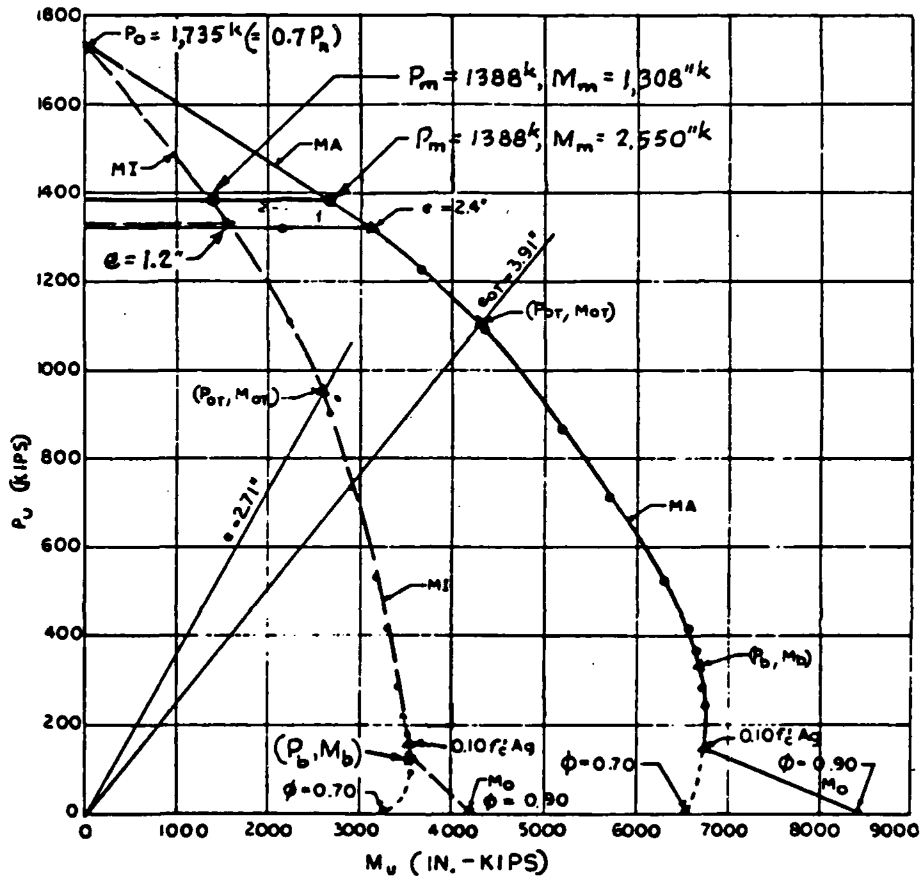


Fig. B-33. Example Interaction Curve for Concrete Columns. (Ref. 14)

motion at the ground level, if the slab and basement walls are poured integrally. Typical upgrading schemes that illustrate these differences are shown in Figure B-2, previously described.

Close-in Shoring

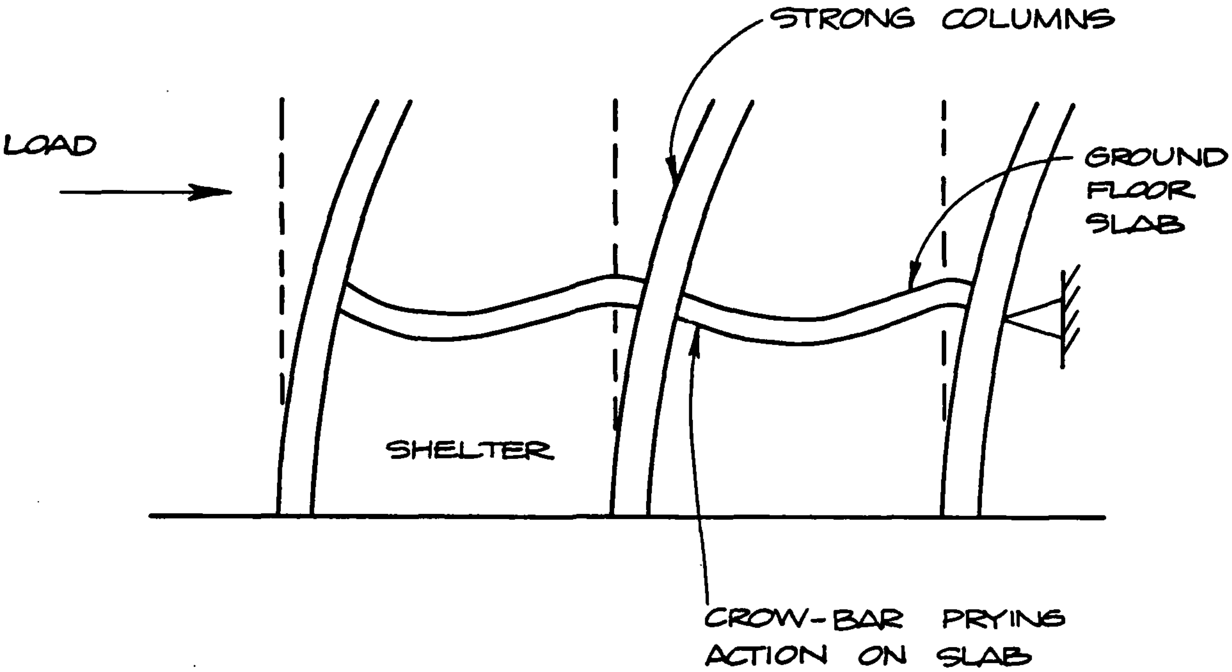
For cases where there are high moments induced in the ground-level slab; either due to heavy-debris loads or due to the "crow-bar" prying action of deflecting strong steel frame columns, the shores may be placed "close-in" to the columns in order to assist the slab or a weak steel frame connection in resisting the imposed column movement, see Figure B-34.

DEVELOPMENT OF FAILURE MODES

The Role of Plastic Analysis Techniques

During the late 50's and early 60's, plastic design of framed structures was a primary part of structural engineering. The concept of plastic design had the potential for economies in building design and construction and for the development of a more uniform factor of safety throughout the structure (i.e., a more uniform probability of collapse for each of the various components). However, the evolution of plastic design of framed structures has declined substantially since the advent of sophisticated computer programs. These computer programs permitted rapid economical linear elastic analysis of even the most complex frame structures, and hence, the simple "plastic mechanism" analysis advantage of plastic design has been outmoded. Also, the material economies of plastic design, inherent in member section capacities and in moment redistribution, have been incorporated into building code provisions. Hence, present design is performed by use of elastic analysis for stresses, and section design by approximations of plastic capacity.

For the purposes of the frame response history analysis, however, there was a specific need to identify the different mechanisms of frame collapse and the respective collapse loads. This "mechanism" analysis is based on the resistances of the beams and column elements for a typical frame and is determined by the methods described in the section on joint element resistance functions and the particular computer model. The form of analysis and prediction of failure modes in this section follows the classical plastic design techniques. By the use of the computer, the fundamental theories of limit design will predict the most probable failure modes for



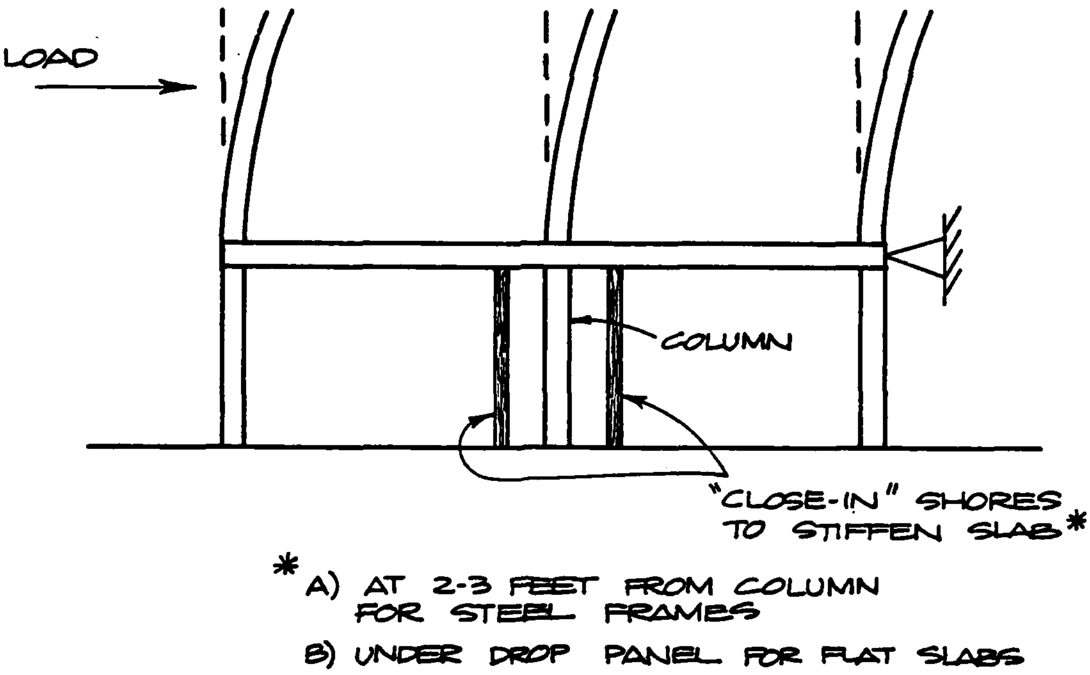


Fig. B-34. Shoring Countermeasure Against High Induced Moments in Ground-Level Slab.

a building frame. The initial fully elastic response of the framed structure, however, may show some anomalies, in that higher moments may be achieved at levels other than those predicted by the limit design failure mode analysis. These moments will be elastic failures, however, and as successive trials approach the mechanism of collapse, classical plastic design will eventually dominate the failure mode.

Figure B-35 shows a representation of a typical framed structure model. The hinge formation in this type of structure, if the basement is properly shored, will be forced to occur in the column above the floor level. At the column-beam intersection in joint "A", at the first level, a yield hinge development in the column will occur as long as the column is continuous through the joint and the beam has some moment resistance. Because of the relative stiffness of the floor system, it is very possible that in the elastic analysis initial yielding may develop in the beam at M_b prior to complete hinge formation in the column labeled M_p. However, since side sway is prohibited at this ground level, a hinge will ultimately develop at M_p, and only small deformation damage will occur in the M_b area. For the interior joint "B", M_b occurs twice, M_c is on the basement side of the column, and M_p is on the column above the first floor. Here again, if the column is continuous through the joint, and M_b has any value at all, the hinge will form at M_p. In concrete frames this would nearly always be the case, particularly if the column steel is spliced above the floor level, thereby creating a zone of weakness.

As discussed previously, many steel frames have essentially pin connections or seated connections at the beam-to-column connection, and the column is indeed continuous through the joint. This results in a lack of moment capacity in the beam, but this deficiency may be overcome by locating shores near the columns. Placing of shores near a column generates significant moment resistance in a member, even if it is nominally pin connected. This resistance occurs because the shear capacity of the joint times the short lever arm distance to the shore generates sufficient moment to cause the hinge to occur above the floor level. The goal is to stiffen the basement shelter sufficiently, using shores, to cause any failures to occur in the column above the basement ceiling.

For reasons discussed previously, the basement selected should include a reinforced concrete floor cast monolithically with the wall system in order to resist side sway at ground level. Ordinarily, precast concrete floor systems do not meet this stiffness criterion.

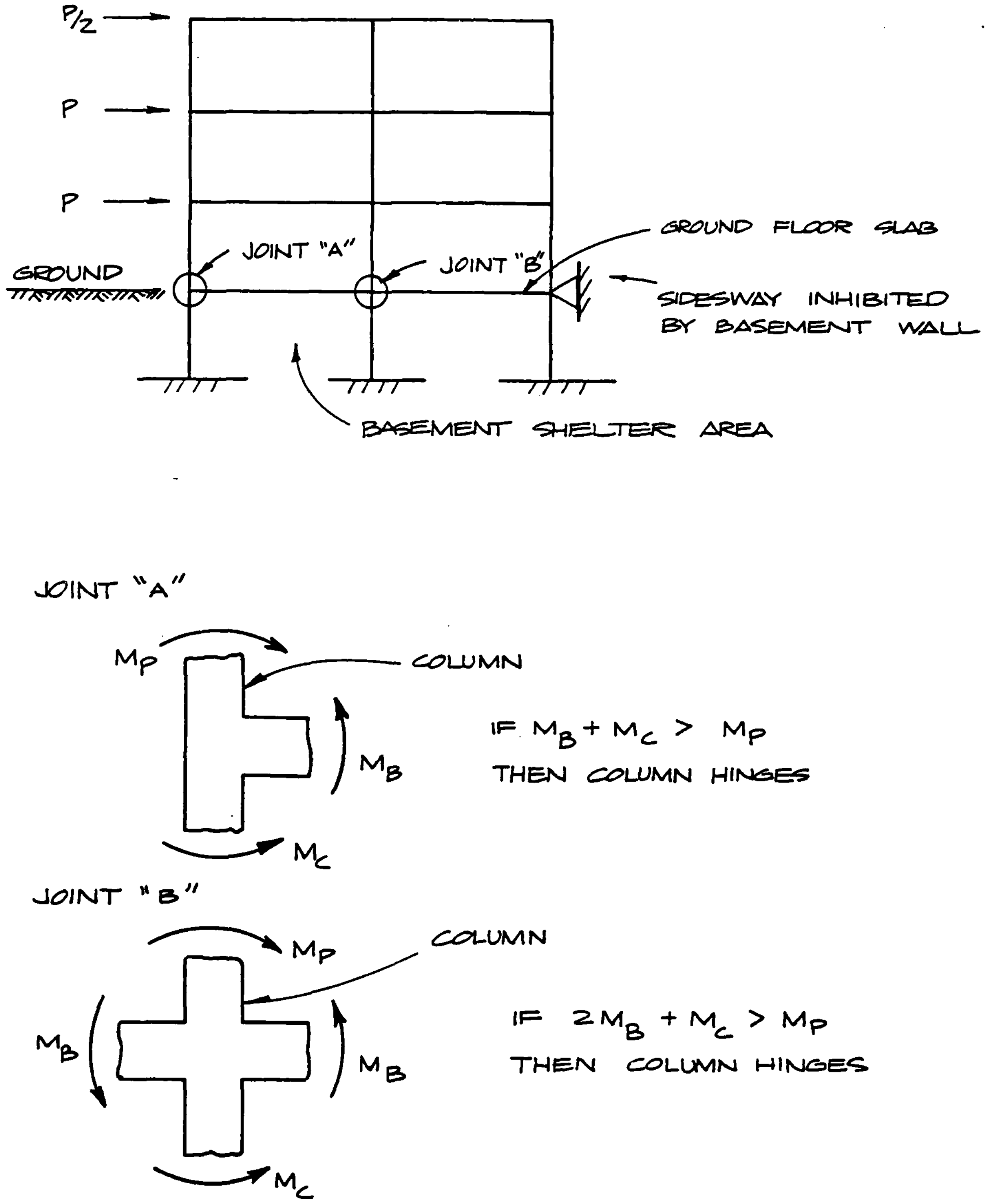


Fig. B-35. Hinge Formation in Typical Structure.

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APPENDIX C

BASEMENT WALL RESPONSE AND PERFORMANCE UNDER DYNAMIC LOADING CONDITIONS

BASEMENT WALL RESPONSE AND PERFORMANCE UNDER DYNAMIC LOADING CONDITIONS

INTRODUCTION

One of the elements that will determine the integrity of basement shelters in terms of damage or survivability of shelterees is the basement walls. The majority of exterior basement walls are of cast-in-place infill concrete, cast integrally with the exterior columns, or they may be cast separately prior to backfill, yet are rigidly connected to the exterior supporting columns. Precast basement walls are not common, but are known to exist. They would be expected to be more prevalent in steel framed structures than in classic concrete construction.

In order to determine the strength of basement walls under the influence of blast loads, precast walls are the simplest and most expedient to test, both in the field and in laboratory environments. Initial testing of basement walls under SSI research programs was based on the use of precast walls. The design and construction of basement walls is based on expected soil loads, surcharge loads, and loading effects caused by saturated retained soils.

The following discussion covers research in the field and laboratory procedures by SSI covering some of the parameters that affect the response of basement walls under blast loads. The conclusions drawn are not to be construed as final, because there is a significant need for further research in soil-structure interaction, particularly under dynamic loads.

Part of the SSI research program has been conducted as laboratory research using 1/20th scale models of the walls constructed for the MILL RACE event in September 1981 (Ref. 1). These walls were tested in a 12-inch shock tube, at varying static and dynamic pressures, with some exceeding 40 psi.

PERFORMANCE OF BASEMENT TEST WALLS AT MILL RACE

The MILL RACE tests included basement walls of three types that were designed and constructed according to published coefficients of at-rest soil pressures. There were two 8-inch thick unreinforced and ungrouted concrete masonry block walls, two 8-inch thick minimum reinforced and fully grouted concrete masonry block walls, and two 8-inch thick concrete walls reinforced with temperature steel only. The walls were approximately 8 feet high and were 6 feet 3 inches to 6 feet 9 inches in width, simply supported at top and bottom with clearance (no support) at the sides. The walls were backfilled with an uncompacted cohesionless granular material in a vertical excavation. Thickness of the backfill was approximately 2 feet and height of the backfill was approximately 7 feet. The remaining one foot of wall height was buried under uncompacted native caliche soils, and an additional 18 inches of this material covered the basement ceiling area. This soil extended beyond the limits of the building walls as a fill slope (see Figure C-1).

The ungrouted, unreinforced masonry block walls were designed to fail. For comparison purposes, the curve in Figure C-2 is taken from Ref. 2, and is an extreme probability distribution for simply supported unreinforced and ungrouted 8-inch concrete masonry block beams, derived from shock tunnel static test data. The distribution indicates that 50% of the walls should fail at 1.4 psi, and 95% of such walls should fail at 2.0 psi. In fact, none of the MILL RACE walls failed, although the two minimally reinforced precast concrete walls cracked near the midpoint. Since the masonry block walls did not fail, it is concluded that either they received substantially less than 2.0 psi horizontal overpressure from the 40 psi surface air blast load (unlikely), or other factors affected the response of the walls to the blast loads. A discussion of some of these factors and some laboratory tests conducted to determine them is included.

Soil properties, dynamics, geometry, and failure probability are all factors that need to be considered in assessing this apparent anomaly between predicted and field conditions at MILL RACE. Of these, the effect of soil conditions is probably the most discussed and least understood factor in the design of basement walls. The following paragraphs present some theoretical background for an understanding of

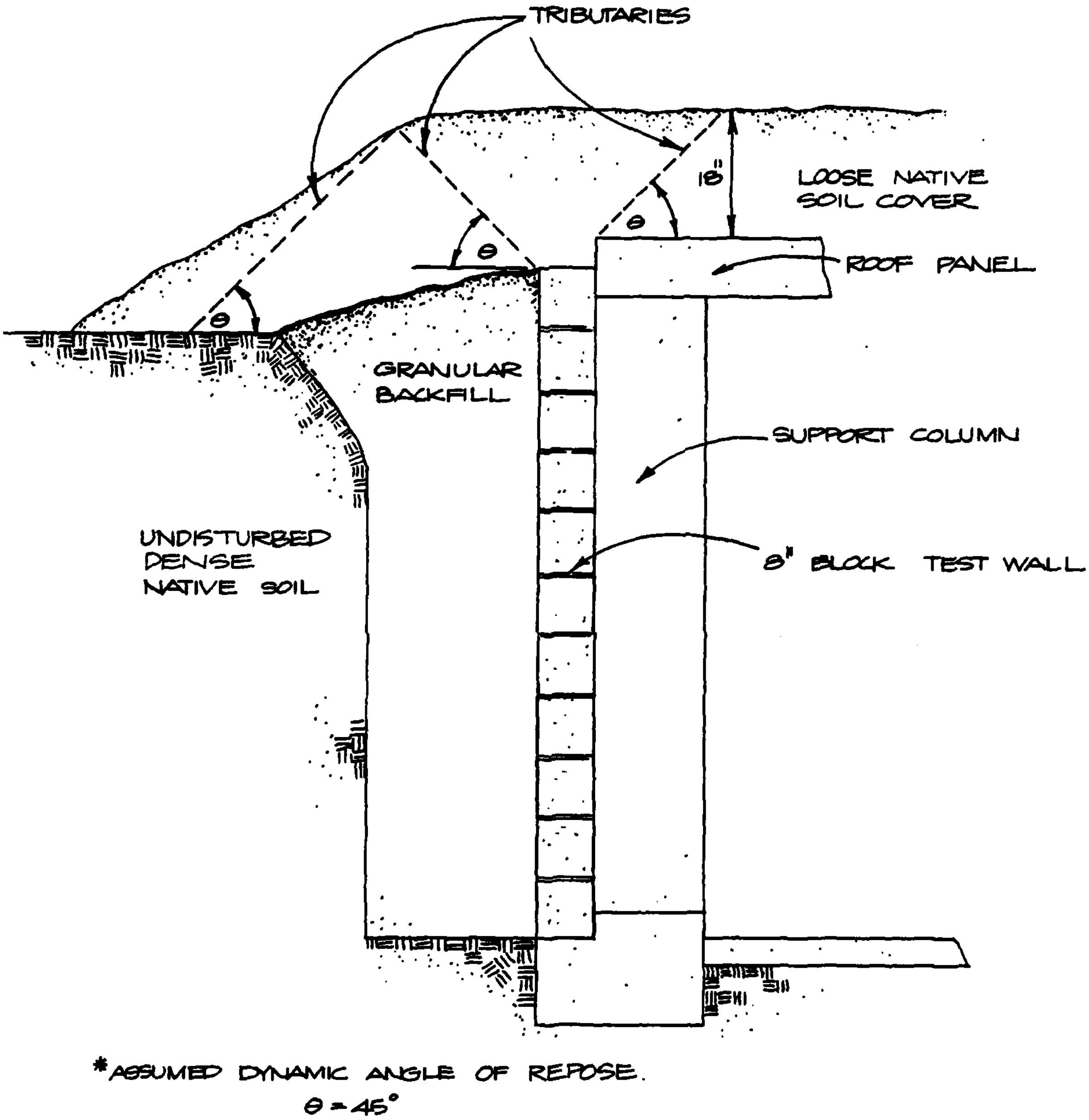


Fig. C-1. Details of MILL RACE Masonry Block Wall Construction.

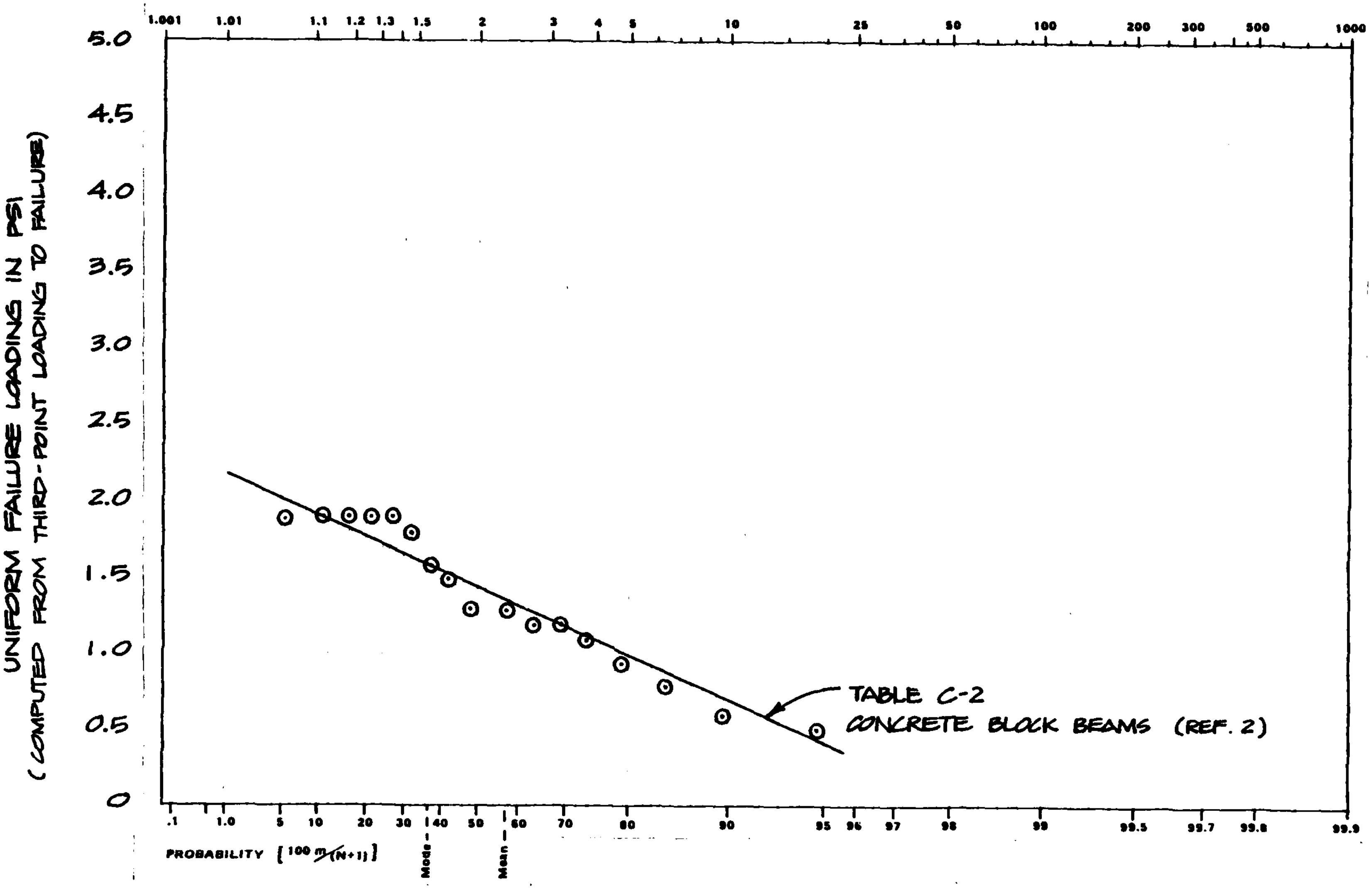


Fig. C-2. Probability of Survival.

soil/structure interaction and the effects of earth pressures acting against basement walls (Refs. 3, 4, 5, 6).

Background Theory

Static Lateral Pressures - Basement walls are typically supported at the top and bottom as shown in Figure C-3. The wall is restrained and not free to move. It acts as a simply supported beam with a span of H. Earth pressure acting on the wall is approximated by at-rest pressure:

$$P_o = K_o \gamma z$$

where P_o is at-rest pressure,

K_o is coefficient of at-rest earth pressure,

γ is unit weight of soil, and

z is depth from ground surface.

Typical values of K_o are tabulated in Table C-1.

The values presented above indicate that soil type and construction method are factors affecting earth pressure. Cohesionless materials such as sand or gravel are generally preferable for backfilling because of better drainage properties. Good drainage properties result in lower earth pressures being produced, provided there is positive drainage away from the cohesionless backfill materials. Normally, basement walls are built in an open excavation with sloping sides. When the walls and floors are completed, the drainage systems and backfilling are constructed. If backfill is well compacted, the at-rest pressures in the backfill may be very large, particularly if saturated, and this pressure would govern the design.

For cases where the ground water table is higher than the footing of the walls, water pressure should be added to the at-rest earth pressure, which is calculated based on buoyant unit weight of soil, γ_b. Design surcharge load, q, should be added when appropriate. Figure C-4 illustrates the combined lateral pressure caused by soil weight, surcharge load, and ground water.

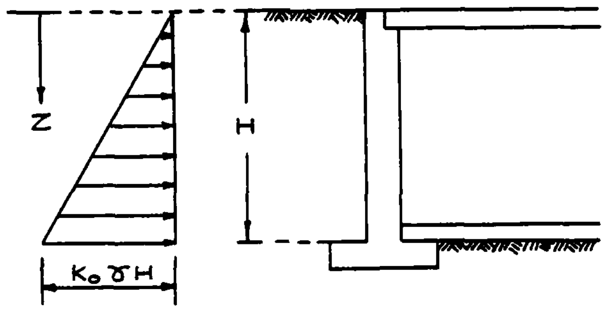


Fig. C-3. Earth Pressure on Basement Wall.

TABLE C-1: COEFFICIENT (K_o) OF AT-REST EARTH PRESSURE

Soil Type	K_o
All types of normally consolidated soils	$1 - \sin \phi$
Loosely placed sand	0.5
Compacted sand	0.5 to 1.5
Hand compacted clay	1.0 to 2.0
Machine compacted clay	2.0 to 6.0
Overconsolidated clay	1.0 to 4.0

Note: ϕ is the effective friction angle of the soil, and may be determined by the direct shear test, ASTM D-3080.

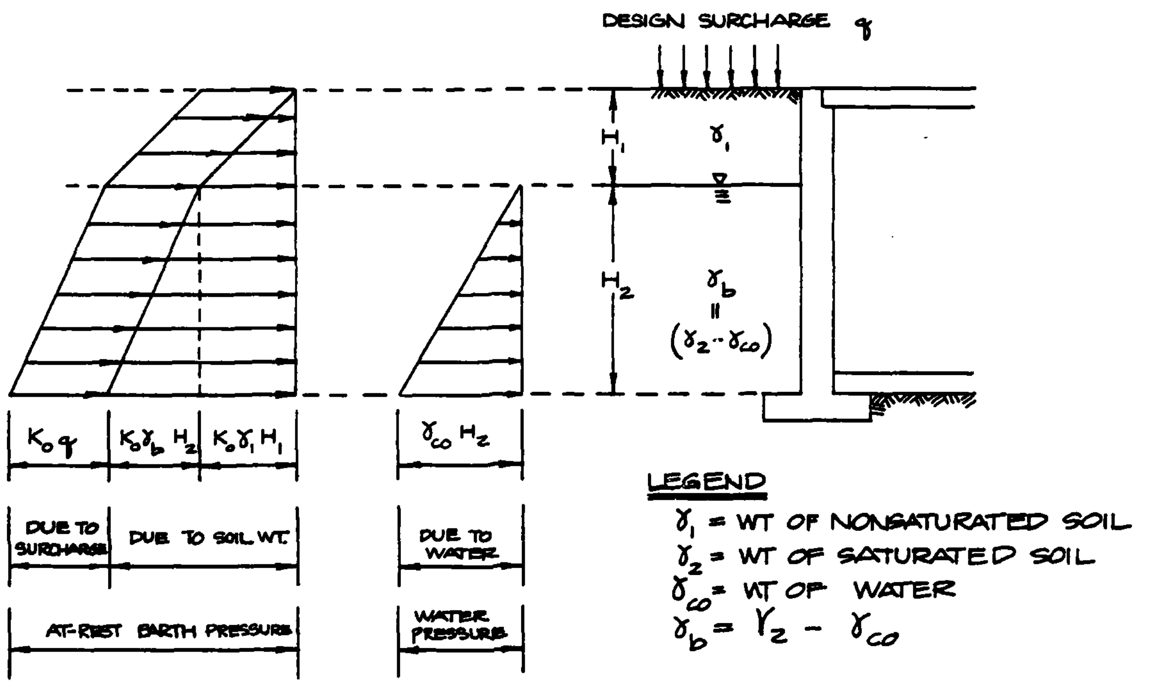


Fig. C-4. Effect of Ground Water and Surcharge.

Dynamic Lateral Pressures - Air blast from nuclear explosions will impose earth pressures in addition to static loads on the walls of basement shelters. Figure C-5 shows an air blast sweeping over the ground surface from a near surface explosion. The shock wave in air induces stress waves in the ground, and these stress waves in turn create dynamic stresses on buried structures.

The behavior of buried structures during blast loading has been considered as similar to static loading because the lengths of stress waves generally are long compared with the depths of the structures. Stress amplification by reflection of stress waves from the structures is relatively small, and soil-structure interaction produces substantial damping in the soil adjacent to the structures.

For underground protective structures with roof surface at grade level, the roof or closure will exclude air blast from the interior of the structures, and therefore the walls act as a rigid retaining wall with a dynamically applied surcharge. As a first estimate, dynamic lateral stress is generally taken from static conditions wherein the lateral stress is equal to the vertical stress at the corresponding depth, multiplied by K_o. In this case, K_o should be evaluated in terms of total stress instead of effective stress; typical values are presented in Table C-2.

TABLE C-2: VALUES OF K_o FOR USE WITH AIR BLAST LOADING

Soil Type	$\mathbf{K_o}$
Dry Granular Soils	0.5 or less
Saturated soils	1.0

Actual values of K_o can vary considerably with depth along a vertical wall below the surface depending on how the wall is supported and the backfill material.

Above Ground vs Below Ground Shelters - For resisting the air-blast loading, buried sidewalls are superior to similar walls exposed above grade. Data presented in Table C-3 were taken from Ref 7, to compare dynamic pressures acting on vertical walls due to a large nuclear explosion.

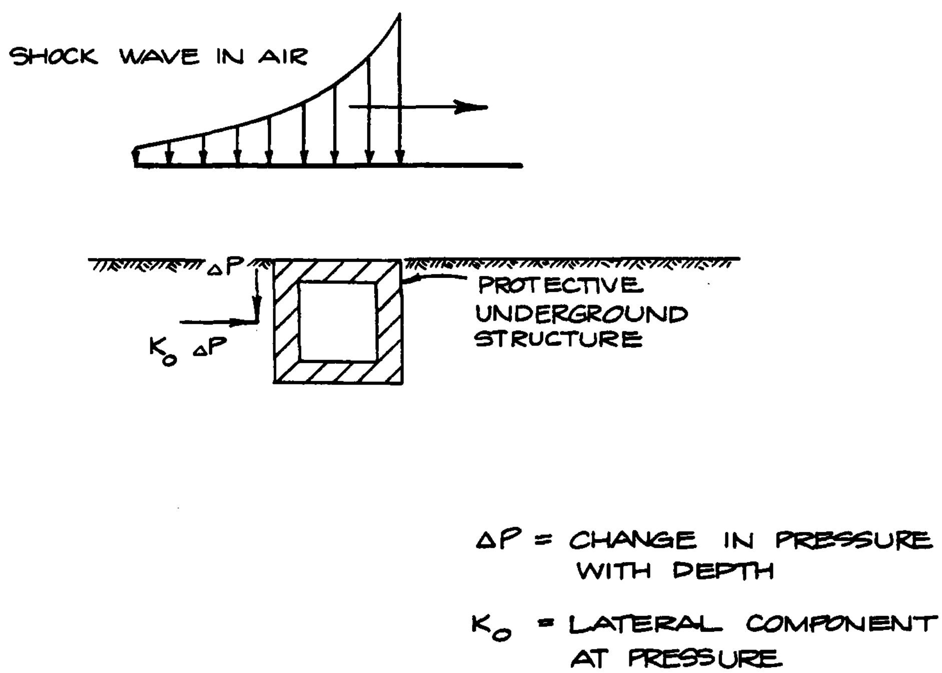


Fig. C-5. Effect of Surface Shock Wave on a Buried Structure.

TABLE C-3: WALL PEAK PRESSURE MULTIPLYING FACTORS

(1) Above Ground Walls - Peak pressure on vertical walls amplified by reflection.

Free Field Peak Overpressure (psi) (P_s)	Actual Peak Press. (psi) On Vertical Walls (Reflected Press.) (P_f)	Approximate Multiplying Factor On Reflection (P_f/P_s)*
2	4	2
10	25	2.5
20-25	60-80	2.96-3.18
40	146	3.65
60	251	4.18

* Numbers in this column from equation $P_f/P_s = 2(7P_o + 4P_s) / (7P_o + P_s)$ and the relationship is greater than or equal to 2.0. P_o = atm., P_s = incident, P_f = reflected

(2) Below Ground Walls - Peak pressure on vertical walls attenuated by soil characteristics, i.e., ratio of horizontal to vertical soil pressures

Soil Type	Approximate Multiplying Factor of Overpressure Applied to Vertical Walls (K_o)
Cohesionless soils, dry or damp	0.25
Cohesive soils, not saturated:	
Stiff	0.33
Medium Stiff	0.50
Soft	0.75
Saturated Soils	1.0

This comparison shows that sidewalls of shelters below ground are subject to overpressure forces that are considerably less than the forces on aboveground shelters. It therefore appears that cohesionless backfill material will produce lower dynamic lateral pressures on the wall, in addition to providing better drainage that also prevents backfill saturation (which affects dynamic lateral pressures adversely).

Analysis of Mill Race Test Results

The Table C-3 relationships for the ratio of horizontal to vertical pressure (K_o) are based on the best data published in the literature to date (Ref. 8), and historically have been used to predict the response on basement walls for dynamic loads. The MILL RACE test walls were designed according to these published K_o ratios; however, these data in most cases have not been verified by field testing. As noted earlier, it could be concluded that the masonry block walls, which did not fail, received substantially less than 2.0 psi horizontal overpressure from the 40 psi surface air blast load. However, if this were true, the effective K_o for cohesionless soils, from part 2 of Table C-3, was apparently less than 0.05 (i.e., less than 20% of the published value), or other factors prevented wall failure. However, the data in Table C-3 for loads on below ground structures were developed from experiments pertinent to semi-infinite, homogenous, isotropic soil conditions. Real world backfill conditions involving several dissimilar soils, and geometries have not been assessed in such experiments. Significant research effort is needed to determine the role and impact of a number of factors influencing the response. Some preliminary laboratory work recently completed by SSI on this subject is included herein.

The factors of interest are all those that may affect the soil pressures on a vertical wall resulting from an overpressure load on the soil surface above. All these factors that are amenable to experimental investigation include:

- o Load transfer in-plane to the top of the wall
- o Density of backfill (relative compaction)
- o Particle size and shape of the backfill
- o Interface between backfill and basement walls
- o Lateral extent (width) of backfill
- o Interface of backfill with native soil

- o Deflection of the wall under load
- o Effect of wall deflection on transmission of pressure through backfill
- o Degree of soil saturation

At MILL RACE, the wall loading was undoubtedly also affected by the depth of loose fill that lay over and adjacent to the structure, and the influence of the dynamic in-plane vertical load in the wall transmitted through this material as a result of the passing shock wave.

Because the structure wall and undisturbed native soil were both stiffer than the loose cover material and the granular backfill, there would be a tendency for active arching (Ref. 9) to take place. In this arching the overpressure load would be transferred to the structure wall and to the undisturbed native soil through a process somewhat like punching. In effect a tributary will develop in the soil cover because of the stiffer "member" beneath it. The angle of the failure plane, θ, in the soil cover that defines the tributary is essentially the dynamic angle of repose in loose native soil (see Figure C-1). Such information has not been made available for the MILL RACE site soils. However, stable berms constructed of loose native soil indicate an angle of repose that may be considered pertinent to dynamic conditions. Unfortunately, even this angle of repose has not been measured, but from MILL RACE site photographs it would seem to be between 35 and 45 degrees, and this range is adequate for a simple analysis.

The steeper 45 degree angle will provide a conservative estimate of the span of the two tributaries that will cause the loose backfill to be bridged. This portion of the load is transferred to the adjacent wall and to the native soil at the respective interfaces. At 45 degrees the horizontal projection of each tributary area is equal to the height of the cover; i.e., 18 inches. At the top of the backfill cavity, the total span from wall to native soil is nearly 3 feet. Hence, it is entirely possible that virtually the entire load that was expected in the granular backfill was transferred to adjacent materials through the arching action of the soil cover.

It is of interest that the two reinforced concrete wall panels that exhibited posttest cracks were adjacent to a ramp that was excavated by the construction

contractor in order to provide truck access to the excavation. Where the ramp was excavated there was no stiffer soil member immediately adjacent (within 2 feet) of the backfill. Therefore, no transfer of load (active arching) at the backfill interface took place. As a consequence, a substantial portion of the load would have been transmitted to the panels adjacent to the ramp (See Figure C-6). The cracks observed in the reinforced panels suggest an equivalent uniform pressure loading of approximately 4.0 psi.

LABORATORY SIMULATION OF ACTIVE ARCHING

In order to try to define at least some of the parameters that may have resulted in the non-failure of the 8-inch block walls at MILL RACE, a laboratory test program was initiated by SSI. This program under Contract EMW-C-0701 consisted of laboratory testing of wall panels at one-twentieth scale. Test panels were fabricated to model the dimensions and stiffness of the 8-inch non-reinforced block wall. The testing procedures were designed to include application of static and dynamic loads in a 12-inch shock tube, to determine the breaking strength of the panels and to measure other factors affecting lateral loads on buried walls.

One test was designed specifically to measure the effects of active arching and to determine the formation of the tributaries. Figure C-7 shows the results of this static load test. The laboratory test configuration is shown with the approximate location of the backfill and earth cover at MILL RACE superimposed on the laboratory sketch. The sketch relationship is referenced to the wall thickness dimension.

This single static test on a one-twentieth scale MILL RACE wall panel is insufficient to derive a mathematical relationship for all soil types. However, some preliminary conclusions can be drawn from the tributaries formed.

1. The tributaries that formed over the wall indicate the active arching transferred some of the static load as an in-plane load to the wall. This action caused the wall to perform as a rigid or semi-rigid arched wall rather than as a simply supported (beam) wall panel.

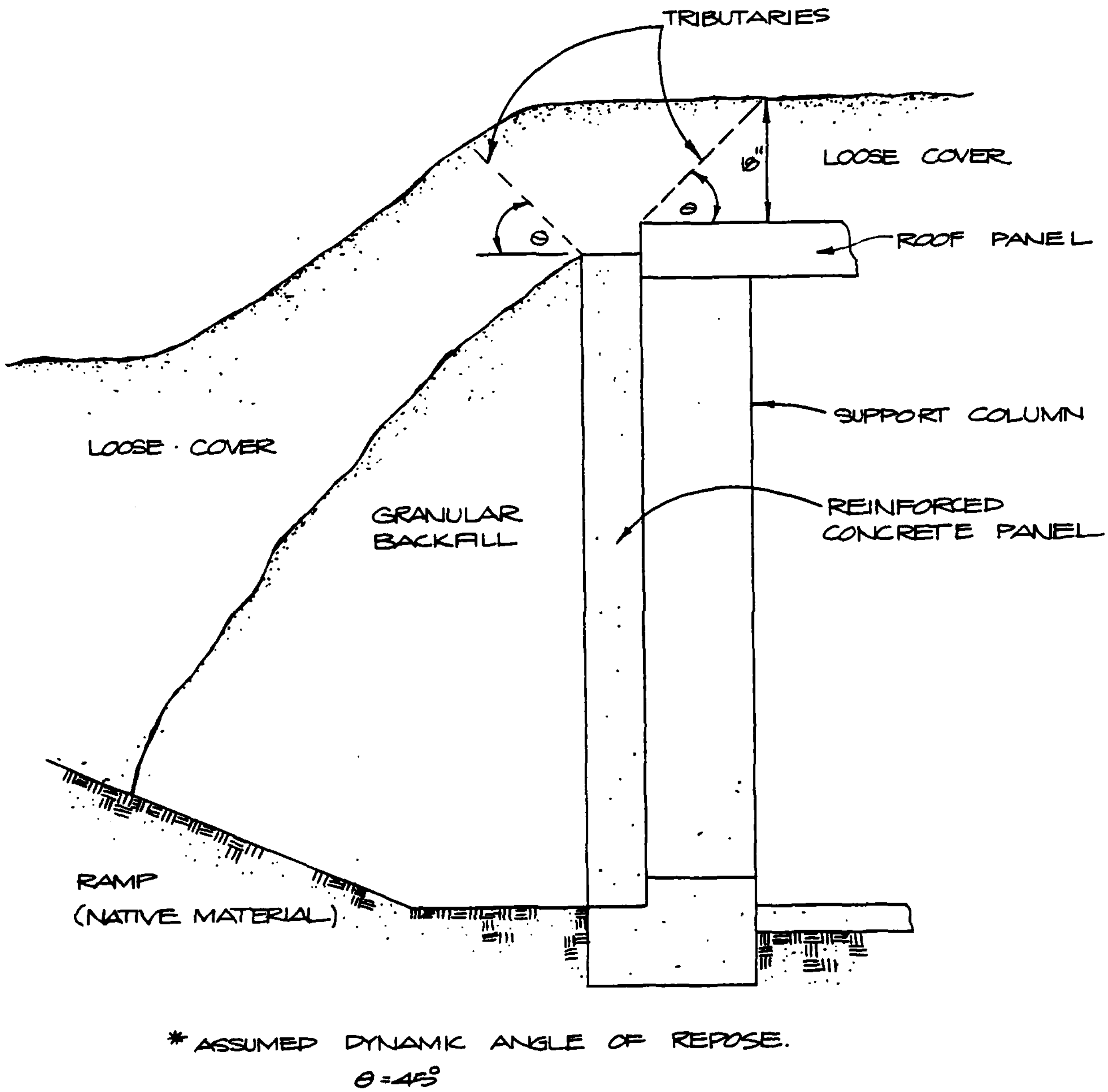


Fig. C-6. Details of MILL RACE Minimally Reinforced Concrete Walls.

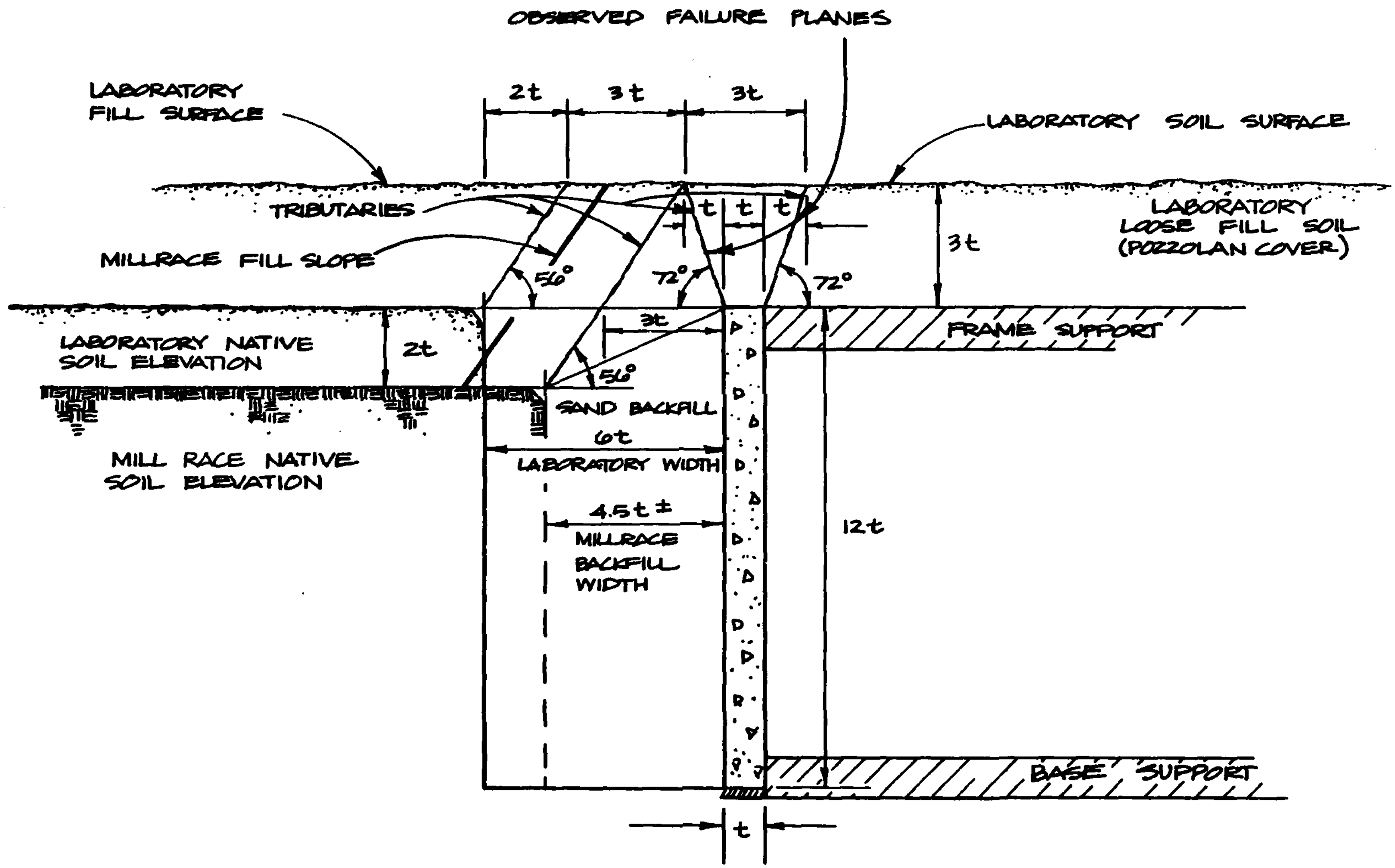


Fig. C-7. Laboratory Simulation of Soil Arching and Tributaries (1/20 scale) with MILL RACE Test Soil Configuration Superimposed.

2. The tributaries formed at the contact of native soil and backfill and the outer wall tributary reduced the effective area of dynamic load transfer to approximately one-half the wall backfill thickness owing to active arching.

Assuming similar formation of tributaries when superimposing on Figure C-7 the MILL RACE backfill and soil cover conditions (from Figure C-1), it appears that active arching may have occurred to the extent that perhaps only a small fraction of the vertical load occurred in the backfill. Further, the fraction of this transferred to the wall for a likely K_o value (25%) indicates the vertical load there is less than 5.6 psi. That is, referring to Figure C-2 the 50% probability of failure of the wall is equivalent to an out-of-plane uniform load of 1.4 psi, and therefore, at a 50% failure probability, the equivalent load on the block walls was less than 1.4 psi corresponding to a vertical load in the backfill that is less than $1.4/K_o$ (about 5.6 psi).

In-Plane Preload Considerations

AT MILL RACE, there was the distinct possibility that the tributaries formed in the soil thickness above the wall resulted in a transfer of load from the surface blast to the wall, and thus effectively increased the rupture strength on the tension side of the wall. Preload influence on increased rupture strength was analyzed and reported in Ref. 10. This analysis has been applied to the unreinforced, ungrouted concrete block wall at MILL RACE.

We first consider the "Elastic Bending" in the MILL RACE walls. Note in this initial effort only the static case will be considered; i.e., $dy/dt = 0$. The wall will be analyzed as a vertically mounted simply supported beam 1 inch wide (Figure C-8). Let

L = 96 in. long; t = 8 in. deep; w = 1 in. wide

*W = 24 lb/in. of width; i.e., consider a 1-in.-wide beam

M = bending moment $\quad$ A = area (in.^{2}/in. of width)

p = static pressure in psi $\quad$ P_v = preload in lb/in.

σ_r = rupture modulus in psi $\quad$ H = shear at centerline crack

I = moment of inertia ($t^3/12$ in in.^{4}/in.)

*(ungrouted 8-in. lightweight concrete block unreinforced = 24 lb/in.
grouted 8-in. lightweight concrete block unreinforced would be 51 lb/in.)

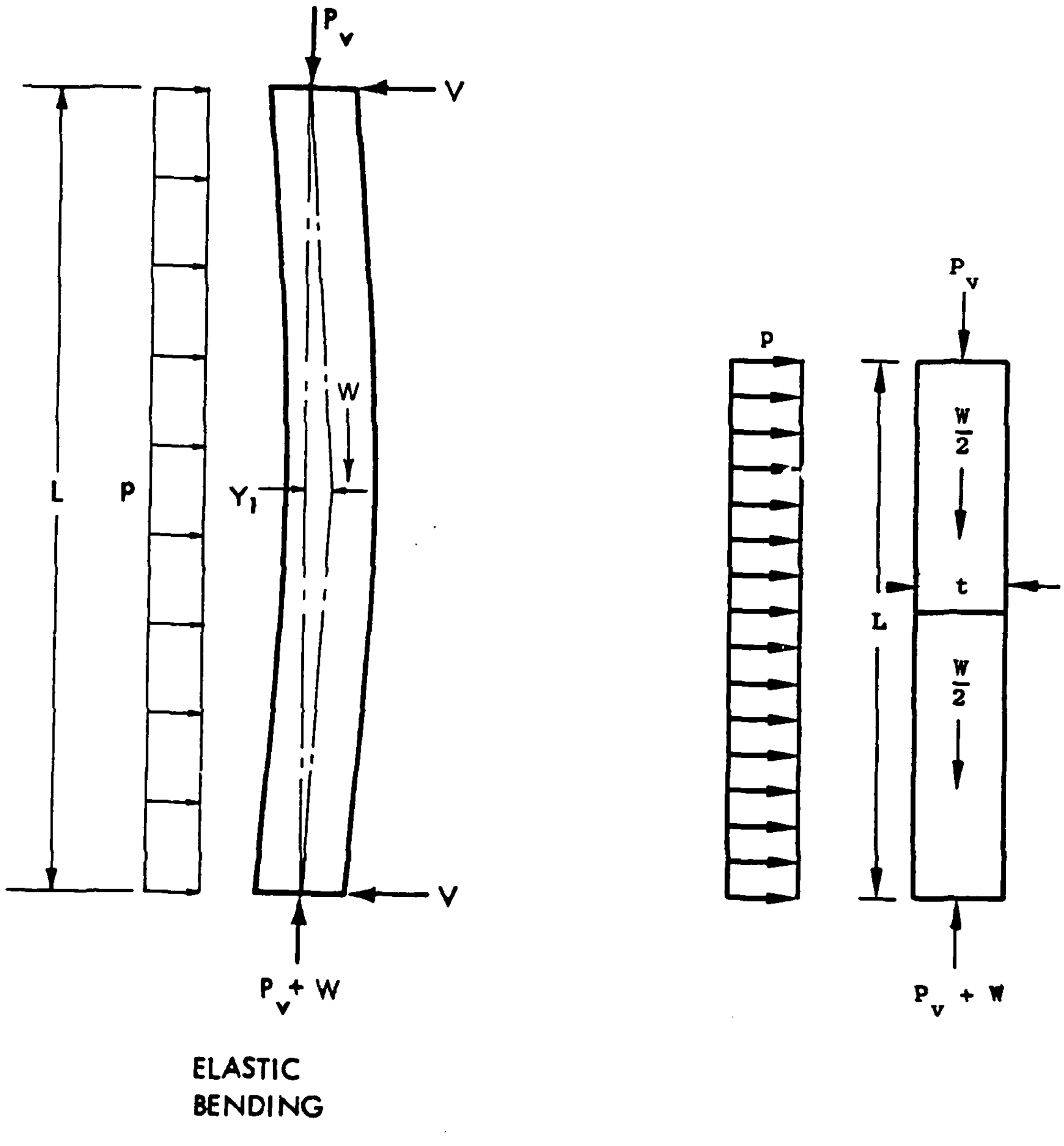


Fig. C-8. Wall Failure Model.

From mechanics the modulus of rupture is:

$$\sigma_r = (Mt/2I) - (P/A)$$

where $$M = (pL^2/8)$$

and $$P = P_v + W/2$$

and P is the total of the preload, P_v, plus half the weight of the wall, if fracture at the center of the beam is assumed. One can then write the equation of uniform lateral failure load as

$$p = \frac{8\sigma_r + P_v + W/2}{6(144)} \qquad \text{(Eq C-1)}$$

Example computations for determining the effects of in-plane preload on the out-of-plane failure load for three values of the modulus of rupture are presented in Table C-4. These three values were selected to bracket the concrete block walls and the scale model walls. The data in the table (developed from Equation C-1 with W at 24 lb per unit inch of 8-inch wall) are presented graphically in Figure C-9.

TABLE C-4

INCREASE IN LATERAL PRESSURE TO CAUSE FAILURE AS A FUNCTION OF THE IN-PLANE PRELOAD, P_v

σ_r (psi)	P_v 0 psi	1 psi	10 psi	40 psi
50	0.48	0.50	0.75	1.59
100	0.94	0.97	1.22	2.05
150	1.40	1.43	1.68	2.51

The preload per linear inch of wall is determined from the tributary configuration from Figure C-7 as three times the preload, P_v; i.e., for each 1 psi applied directly to the wall as an in-plane load, the tributaries transfer an additional 2 psi, for a total of 3 psi applied load. Thus a 1 psi load applied at on the ground surface results in

1 psi x 1 in. x 8 in. x 3 = 24 psi per linear inch of wall

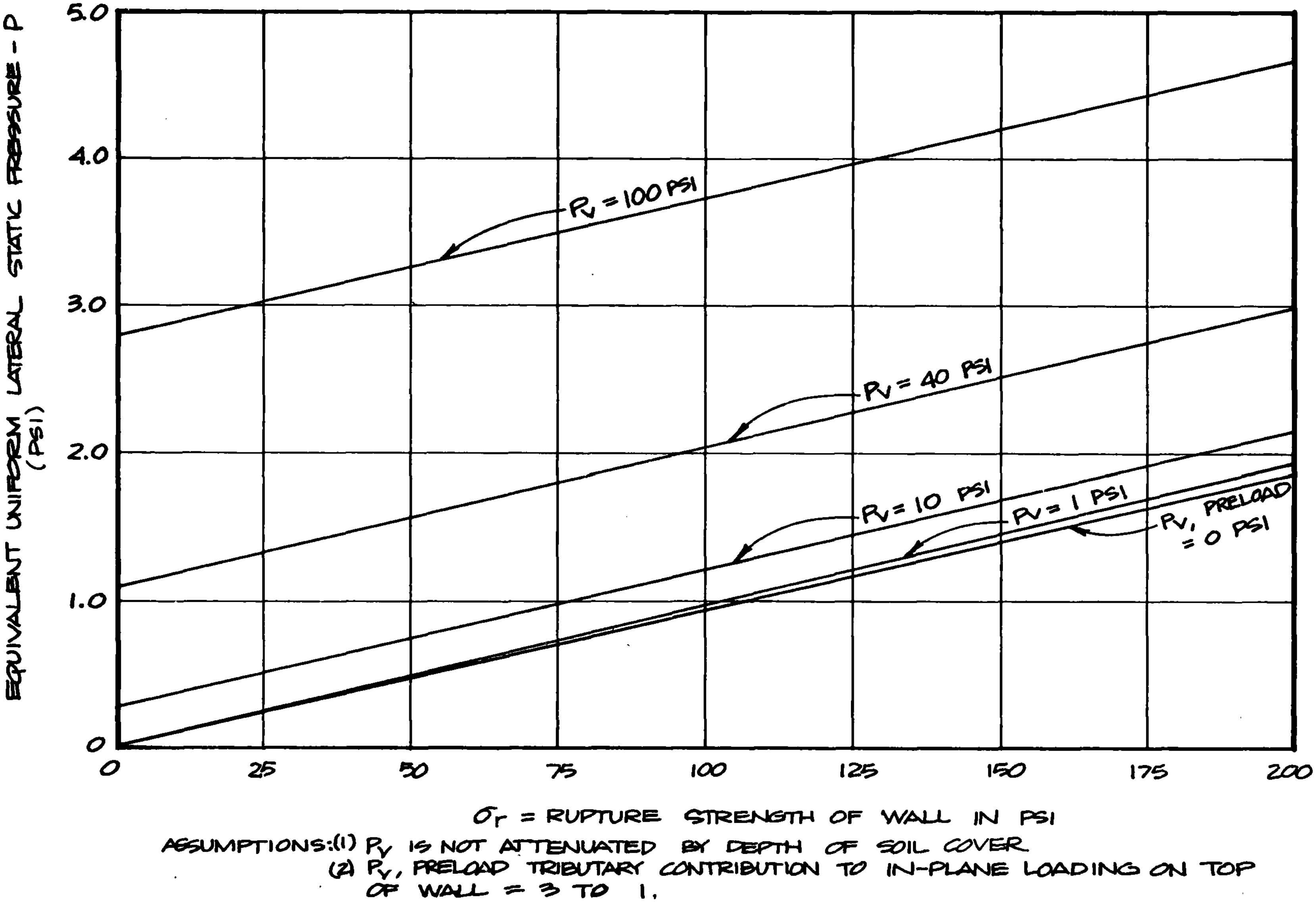


Fig. C-9. Effect of In-Plane Loading on Failure Pressure of 8-Inch Ungrouted Unreinforced Block Wall With Typical 18-Inch Soil Cover for Radiation Protection.

The allowable working rupture stress for unreinforced masonry block in flexure is 10 psi from Table 24-B of the Uniform Building Code (Ref. 8). If we assume there is a minimum factor of safety of 5 built into the working stress limitations on concrete block, the actual rupture strength, σ_r, would be more on the order of 50 psi. For σ_r = 50 psi and P_v = 0 psi, from Table C-4, p = 0.48 psi. This value corresponds nearly to the 95% probability of survival (where 0.48 psi intersects the curve in Figure C-2). This same curve indicates the mean (at 37th percentile) rupture strength is 2.7 times larger, or 135 psi. The application of in-plane preload forces along the top edge of the wall does increase the resistance to static load failure significantly. Using the data generated in Table C-4, at a modulus of rupture of 50 psi (characteristic of 8-in. concrete block), apparently a 40-fold increase in preload increases the resistance to lateral failure by a factor of 1.59/0.50 = 3.2.

From a practical standpoint, a basement shelter that is constructed with ungrouted, unreinforced 8-inch thick concrete block walls, if subjected to a 1 psi blast overpressure, would easily survive the blast loading on the ceiling with minimum midspan shoring, and wall failure would have a minimal probability of occurring (5%). However, when subject to a 40 psi blast wave, the ceiling would require extensive quarter point shoring to ensure survival of the ceiling, yet the wall panel vulnerability is only increased to a 50% probability of failure, even without shoring. Without a statistical sample of 10 to 20 walls this might not be observed; i.e., one or two walls might fail or not fail.

This analysis did not consider the effect of the attenuation of a surface blast wave with depth in the soil adjacent to the wall. It considers only that a dynamic blast force can be made to act on the top of the wall (in-plane) to increase strength because it will coincide with the shock wave traveling through the soil that induces the failure force, P, on the wall.

Laboratory research was also conducted at 1/20th scale to determine the effect of in-plane loading on rupture stress. The configuration of the test apparatus to support the test walls is shown in Figure C-10. The inherent failure probability of the set of test walls when they are simply supported on two edges and they are subjected to uniform out-of-plane loads and no in-plane loads (P_v = 0) was first

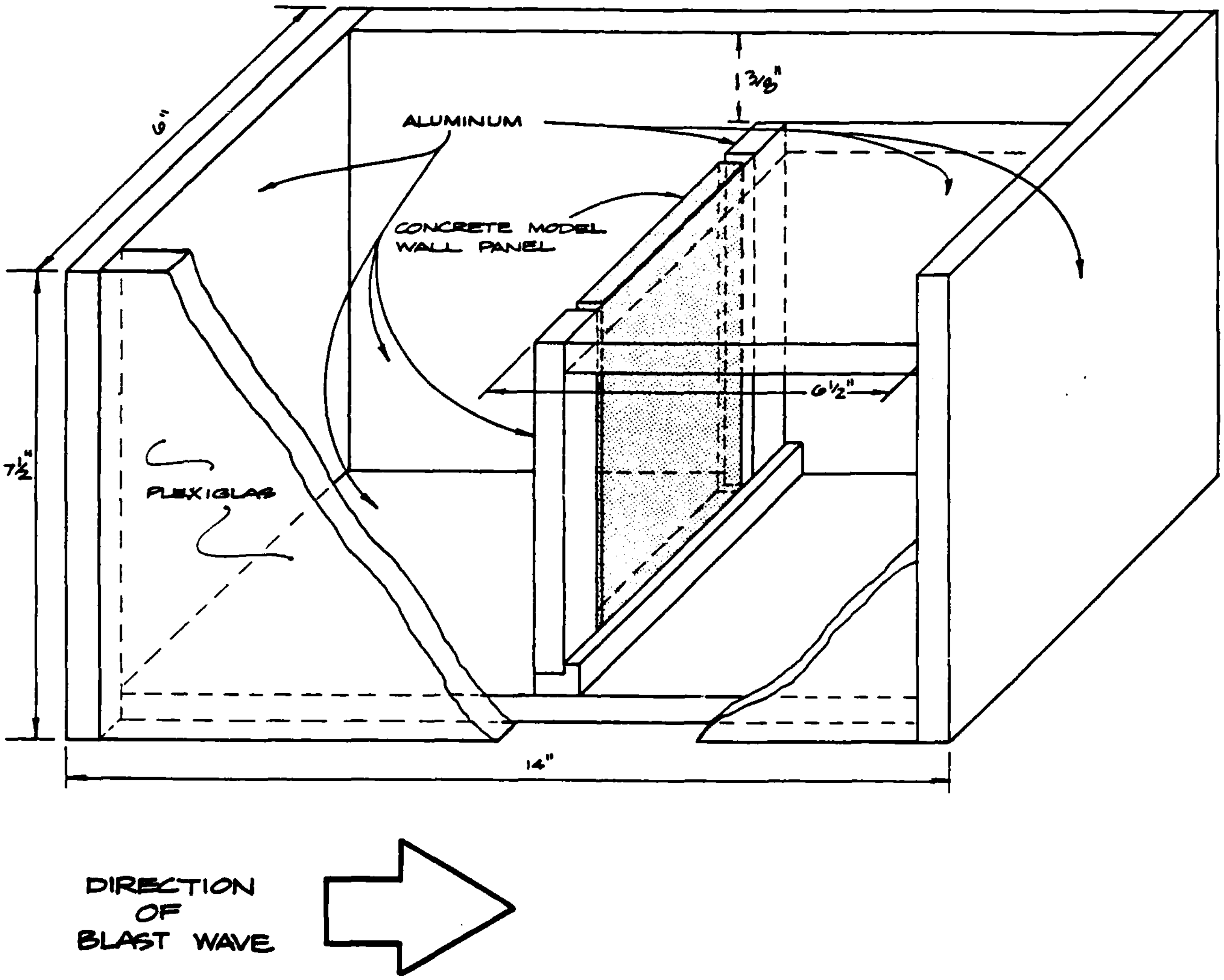


Fig. C-10. Shock Tube Wall Test Apparatus.

determined. In this case, failure is in bending as illustrated by the reference test configuration diagram, and the accompanying curve in the lower portion of Figure C-11, indicating probability of survival (and failure) versus uniform pressure, p. With the inherent failure probability of the set of test walls determined, tests were subsequently conducted on walls from the same batch in the wall test apparatus (Figure C-10), with loading as shown in the upper sketch of Figure C-11. The entire below-grade assembly was mounted in the test apparatus box, which was equipped with a transparent side wall. Then, as the static overpressure on the surface was gradually increased, observations of wall cracking and collapse could be made. The upper curves in Figure C-11 are a plot of the data obtained on the cracking and collapse probabilities for static loading on the surface. Comparison of these upper curves with the reference test curve in the lower portion of Figure C-11 shows that for whatever reasons, the 95% probability of surviving a uniform loading on the surface (which will not result in a uniform loading on the below-grade wall) for the geometry illustrated by the upper sketch in Figure C-11 is 13 times the probability of surviving collapse under a uniform out-of-plane load with no in-plane load. This difference increases to 23 times at the 50% probability of survival, and is different at each percentile because the two lines representing failure probability are not parallel. (They would be parallel only if the same variation applied to both configurations.) The existence of this difference is very important design information -- but it will not be truly valuable until it is clear just what factors are principally responsible. Subsequent to these static tests, dynamic tests were initiated utilizing the SSI shock tube to develop pulses simulating 40 psi surface loadings (at 1/20th scale) for both nominal 1 kt weapons and nominal 1 Mt weapons. Results of these tests will be described in subsequent reports.

Application of In-Plane Loading to Basement Shelter Construction

What are the ramifications of in-plane loading on basement walls? The data developed indicate in-plane loading increases the rupture strength of all shelter basement walls. This effect correspondingly increases the survivability of the shelterees. There are a number of methods that might be applicable to increasing in-plane loads, thus increasing wall rupture strength. These methods are easily adapted to construction of underground key worker or dual use shelters (see Figure C-12). But, their application to existing buildings is limited based on the

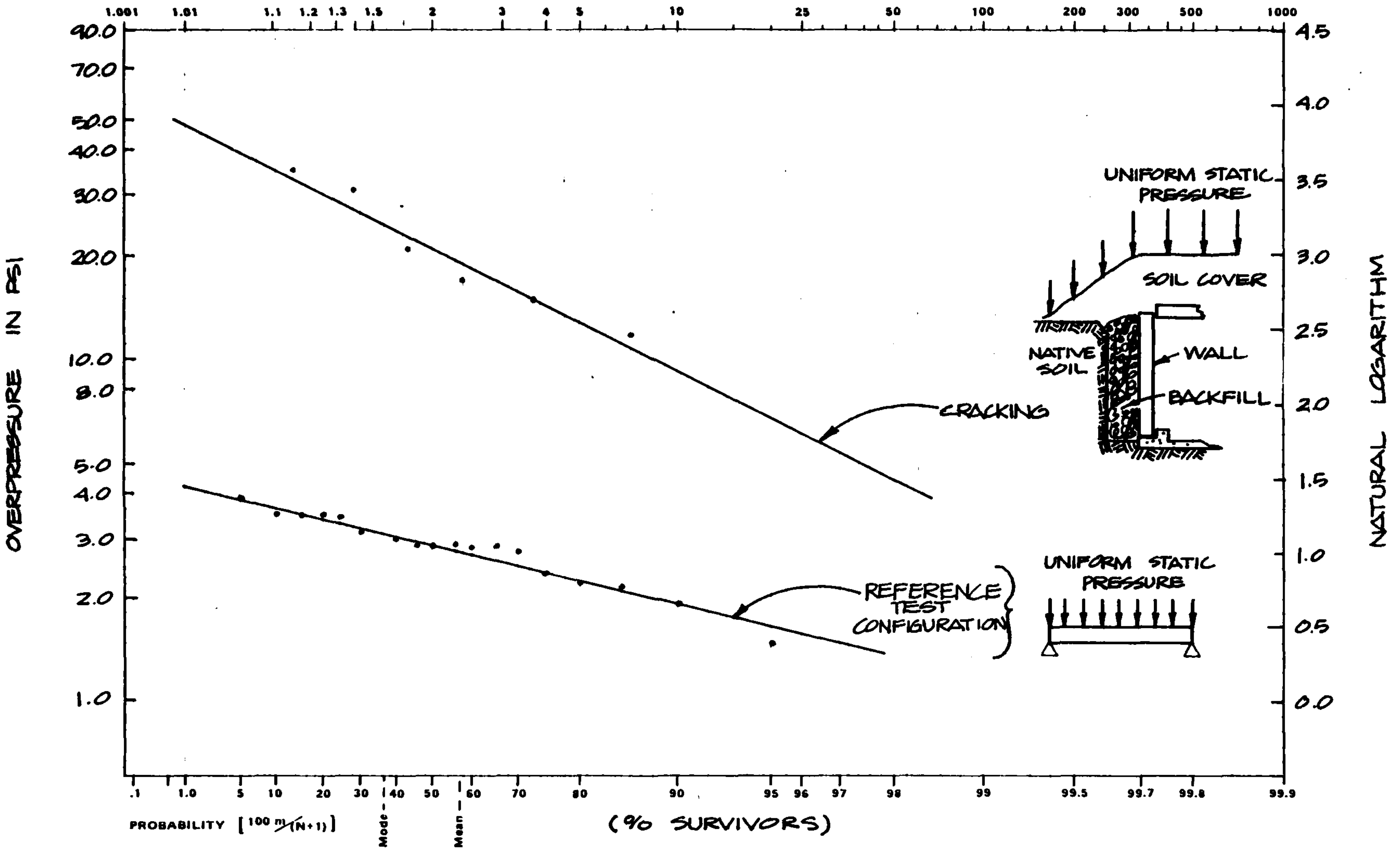
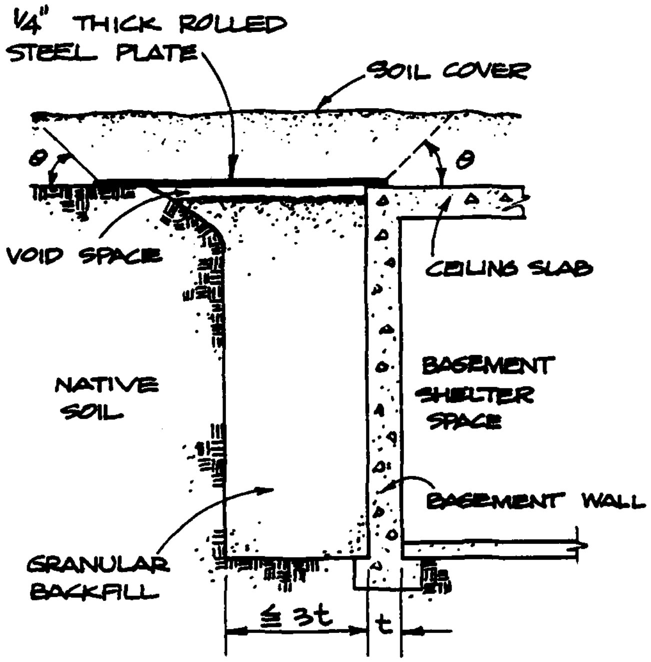
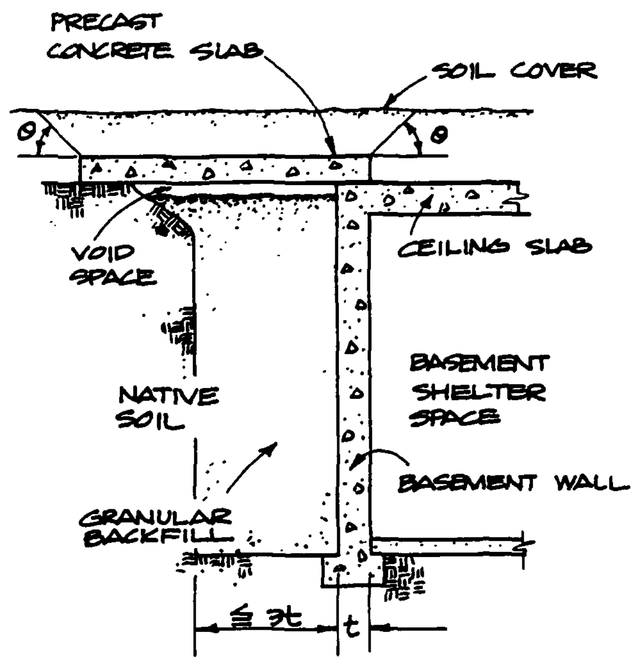


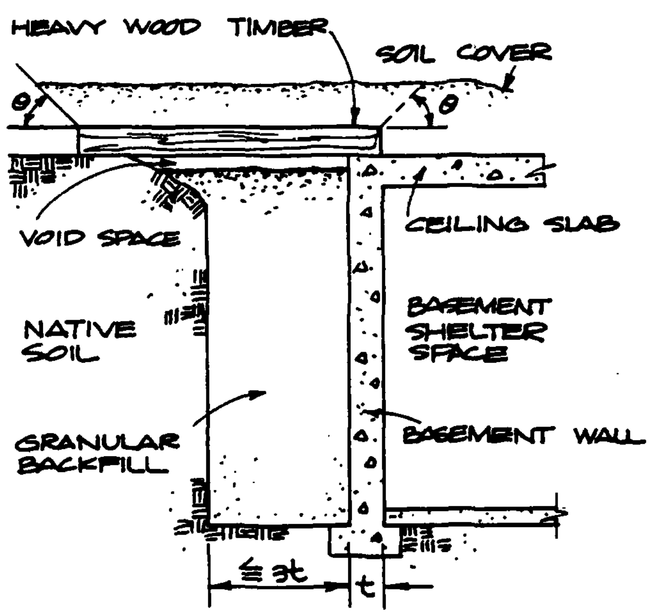
Fig. C-11. Probability Distribution Curves Governing Survival (Failure) of Below-Grade Walls versus Uniform Loading.



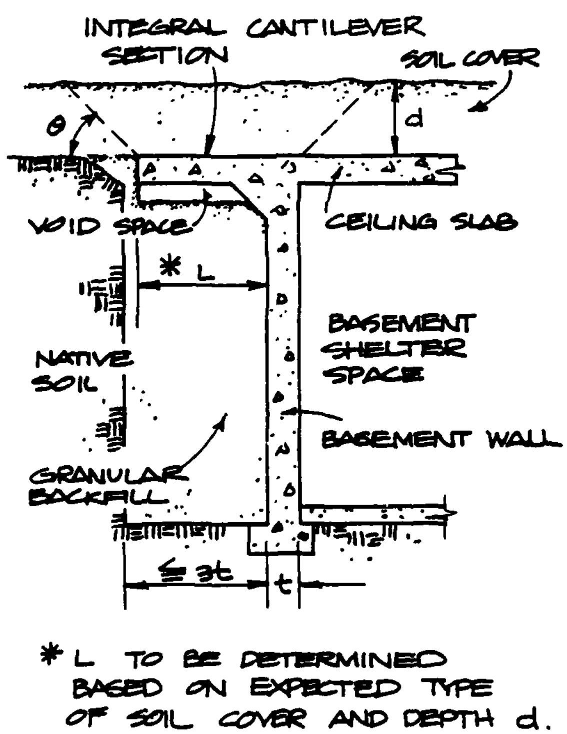
a. Use of Steel Plate



b. Use of Precast Concrete Clad



c. Use of Heavy Wood Timber



d. Use of Integral Cantilever

Fig. C-12. Transfer of Dynamic Blast Load to Wall as an In-Plane Load.

construction detail configuration of the exterior columns and walls above the basement ceiling. If an exterior lip or offset exists between the upper story exterior walls and the lower basement story exterior walls (see Figure C-13), i..-plane load transfer is possible. It is noted that existing buildngs are more difficult to analyze in terms of the capability to develop in-plane wall loadings. It is proposed that each potential basement shelter must be inspected in detail, to determine whether in-plane load transfer is a viable alternative. Should the application of in-plane loads not be a viable alternative, then the basement area may require downgrading in the building index working procedure.

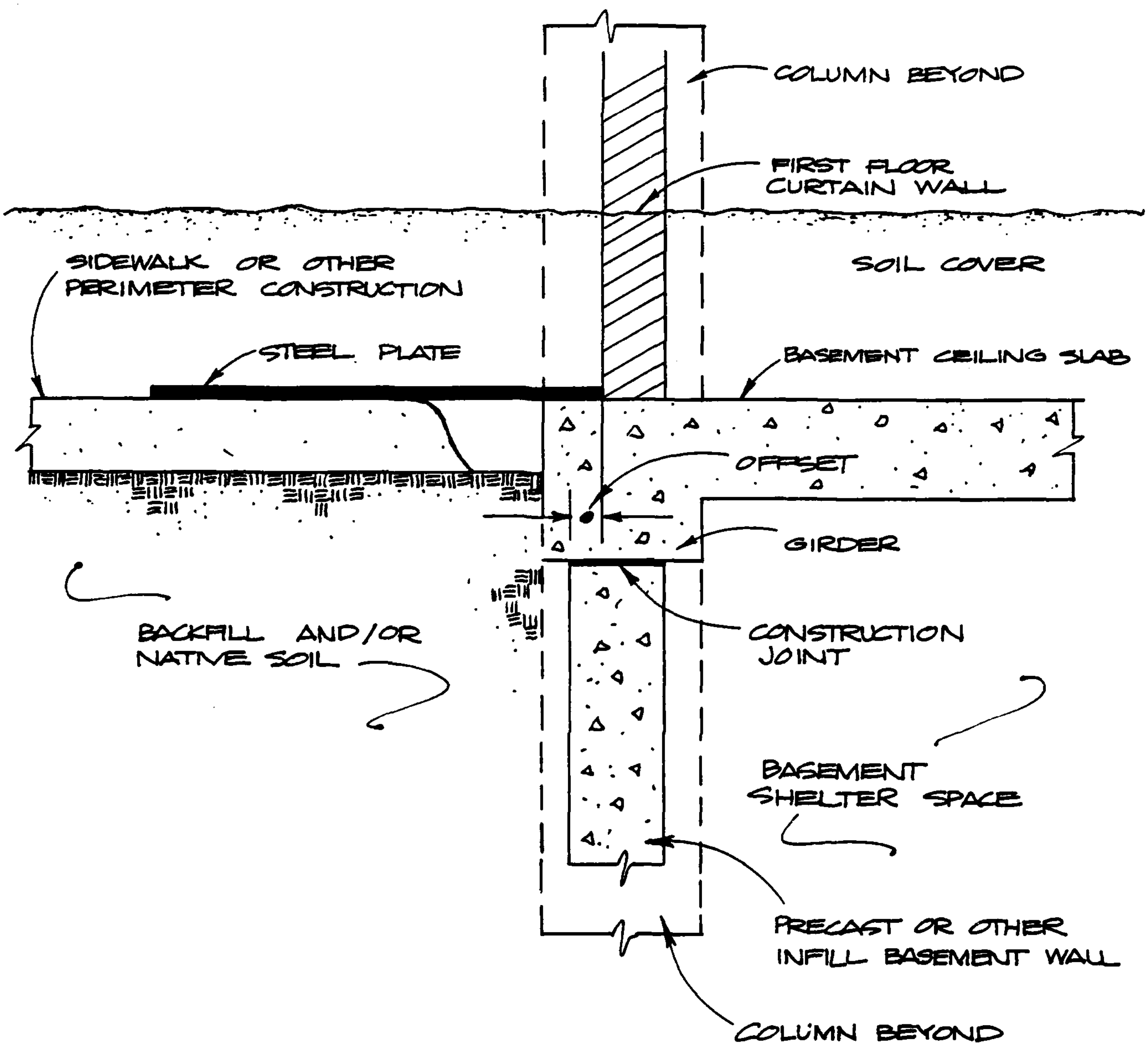
There are other advantages to developing in-plane loading in basement walls. Initially an intimate knowledge of the type of backfill adjacent to the wall becomes less important. The blast loading on the load bridging devices shown in Figure C-12 is transferred to the basement walls and native soil, and thus, there is less opportunity for the lateral load on the walls to develop.

Secondly, the in-plane loading techniques can be applied to multiple key worker or dual use shelters to increase the rupture strength, reduce perimeter backfill requirements, and provide for utility tunnels with environmental plumbing and ducts to the shelter complex (Figure C-14).

Figure C-14 is an idealized shelter complex, but the development of in-plane loading transfer methods theoretically provides two advantages:

1. Transfer of vertical blast loads to the shelter walls, via the natural tributaries developed in the soil, and
2. Reduction of the dynamic blast load on the ceiling of the shelter owing to the active arching and transfer of load due to formation of the tributaries described in (1) above.

Much further research is needed in both the field and the laboratory to determine the formation of tributaries for a variety of soil types and attenuation of shock waves with depth in these same soils.



NOTE: FOR CLARITY REINFORCING STEEL NOT SHOWN.

Fig. C-13. Transfer of Dynamic Blast Load to Exterior Girder and Basement Exterior Wall in a Multistory Building With Upper Walls Offset From Edge of Girder.

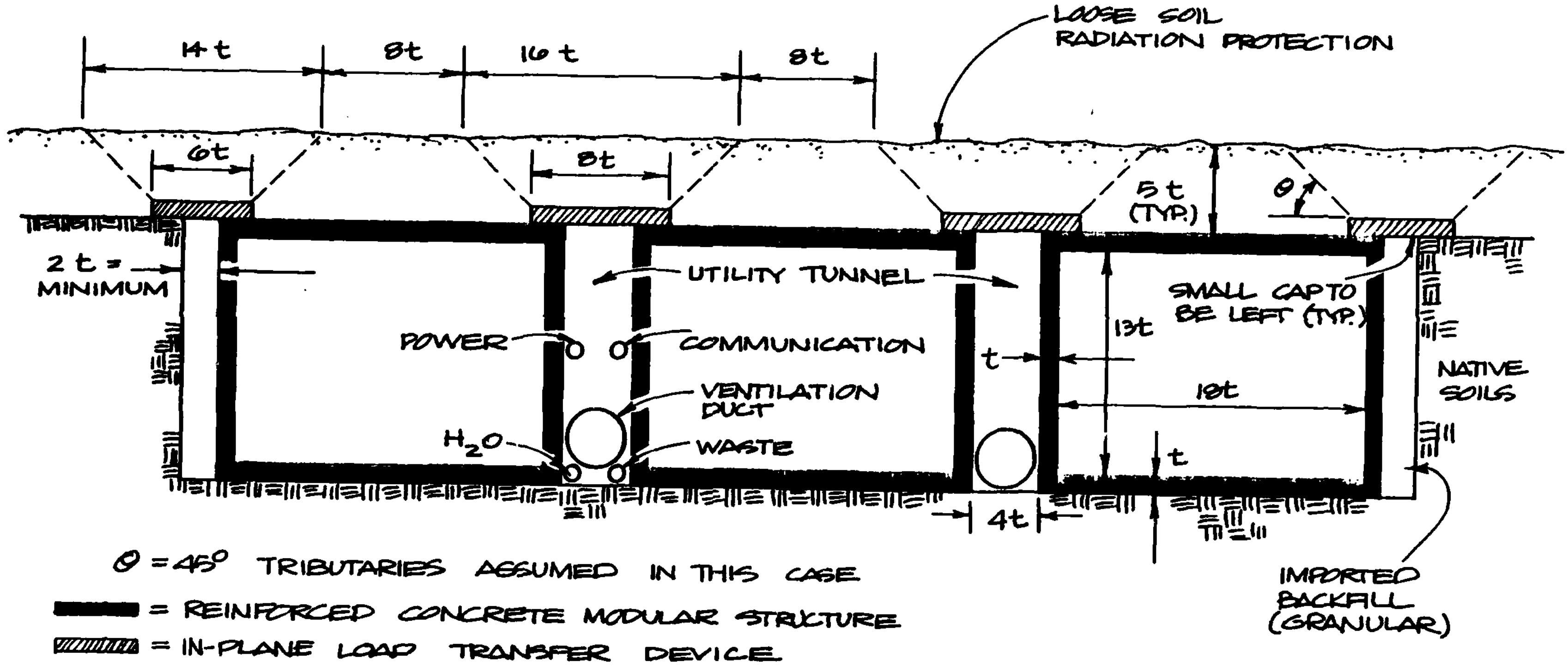


Fig. C-14. Idealized Multiple Key Worker or Dual-Use Shelter Showing Utility Tunnels for Environmental Functions Providing In-Plane Loading Enhancement and Active Arching Opportunities.

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APPENDIX D

CONCEPT OF POTENTIAL FOR UPGRADING

CONCEPT OF POTENTIAL FOR UPGRADING

GENERAL

In this appendix to the report, the concept of "potential for upgrading" is discussed and illustrations are given of some of the factors influencing this potential. In general terms, what is meant by "potential for upgrading" for a basement is the capability of the basement to be modified to serve as a shelter from nuclear weapons effects. This potential is related to the already existing ability of the basement to serve as a shelter in the condition in which it exists at present, as well as to other factors. However, it will be more feasible to upgrade some basements to survive a specified overpressure level, or to provide a specified fallout protection factor, or to provide specified levels of protection against other nuclear weapons effects, than it will be others. A basement that would have provided minimal protection in the as-built condition might when upgraded be superior to another that had been better in the as-built condition. In addition, factors having nothing to do with structural strength must also be considered.

The utility of the concept of potential for upgrading is in the guidance that it can provide to the local planner when he or she is rating basements that are being considered for use as personnel shelters. There are numerous factors that affect the suitability of a basement to serve as a shelter (see Table D-1). This includes "administrative" factors such as size and availability for shelter purposes as well as those that would directly affect the survivability of people. These latter include but are not limited to the structural characteristics of the first floor slab of the structure being considered. Examples of other factors affecting survivability include the sizes and locations of openings into the basement, the nature of the building above the basement, the nature of the building's contents, the characteristics and locations of other structures in the vicinity, the location of the basement in relation to potential targets, the presence of hazardous materials nearby that might present a contamination hazard in case of attack, the presence of reservoirs, tanks, pipelines, or other potential flooding sources in the vicinity, etc. The distribution of openings is important, not only because closures will be needed for them, but because it will be essential to have more than one exit, in case of blockage of one by debris,

TABLE D-1

SHELTER POTENTIAL RANKING TABLE

Factor	Ranking
Sufficiently distant from likely targets	Yes/No
Population in locality needing sheltering	Yes/No
Reasonable access to the shelter available	Yes/No
Sufficient floor area available	Yes/No
Two or more adequately separated entrances	Yes/No
Capable of being adequately ventilated	Scale
Temperature limitations can be met	Scale
Flooding danger acceptably low	Yes/No
Hazardous material spill danger acceptably low	Yes/No
Fire danger acceptably low	Yes/No
Danger from debris falling on shelter acceptable	Scale
Ease of achieving required nuclear radiation protection	Scale
Ease of achieving required blast protection	Scale

and it will be essential to provide ventilation for the shelter because otherwise the air will become too hot and carbon dioxide will build up to unacceptable limits.

Some of the factors to be considered in determining a basement's potential for upgrading may be of such a nature that they can rule the basement out of further consideration. That is, they may be of a "go" or "no go" nature. For example, the basement may be so small as to preclude its use, or access to it may be extremely difficult, or there may be other reasons to eliminate it from further consideration. In other instances, factors will serve to increase or decrease the ease of upgrading the basement to some desired level. These sorts of factors would be determined by relatively simple rating scales. For example, the structure in the as-built condition may not have sufficient mass to provide much protection against nuclear radiation. So, a larger quantity of added earth will need to be placed on and around the basement than for another basement being considered. If there is no nearby source of readily available soil because the nearby areas all contain buildings, paved surfaces, etc, this will also adversely affect the suitability of the basement for use as a shelter.

In some respects, the shelter potential ranking system resembles systems already in use by shelter survey personnel. However, it would go well beyond them, not only in that it would consider additional factors not considered by the present techniques, but also that it would consider the factors currently being taken into consideration more comprehensively.

BASIS FOR RANKINGS

The general basis for the factors listed in the table is as follows:

Target Distance

While civil defense shelters are not expected to be targets themselves, it is highly desirable to avoid placing them in close proximity to potential targets. This does not mean that they must not be in risk areas. Because of the necessity for providing shelters for key workers in risk areas, current planning is to position shelters for approximately four million persons in the risk areas. However, these areas include locations where the overpressures and other nuclear effects levels will

probably be extremely high, too high for survival in an upgraded basement type of shelter. On the other hand, expected nuclear weapons effects in other portions of the risk areas may be sufficiently low to permit survival of persons in basements with a reasonable amount of upgrading. Distinguishing between these two kinds of areas may be very difficult for local civil defense planners. First, we really don't know what the Soviet plans are for nuclear attack on the United States. Secondly, official U.S. estimates of Soviet capabilities and scenarios giving possible Soviet targeting of this country are highly classified. However, local planners can be told what general kinds of things are expected to be targeted and can be given guidelines on desirable minimum safe separation distances from them. They can then use their knowledge of the local area to avoid establishing shelters near most potential targets.

Population Requiring Sheltering

Current crisis relocation planning is intended to provide the details of where the risk area population will relocate to in the host areas during a crisis and of which essential facilities will continue to operate in risk areas. When this planning has been completed to a sufficient degree of detail, the numbers of people requiring shelter in each specific location should be known. It should then be possible to match up the requirements with the potential shelters in existing structures, possibly in an iterative way. That is, if there is a surplus of shelter in one small area and deficits in adjacent areas, adjustments can be attempted to match up supply and demand. If there is still a deficit, either expedient shelters will have to be constructed during the crisis period for the surplus population, or further changes in population location will have to be made. There may also be situations where there is more shelter available than is needed. For example, this could happen in urban areas that have been effectively evacuated and where key worker shelter requirements are already met. It could also happen in remote host areas where there are numerous potential shelters and a low number of evacuees from risk areas.

Reasonable Access

An otherwise suitable basement might be in a location where there weren't adequate roads to permit people to reach it within the established time limits. Alternatively, the normal utilization of the building might be such that access to it might reasonably be expected to be severely restricted because of peacetime theft, security, or other reasons.

Usable Floor Area

One basic requirement that must be met by potential shelters is that they have sufficient available area to accommodate specified numbers of persons, assuming that at least 10 square feet of floor area will be provided for each person to be sheltered. This is necessary because, except for shelters in private residences, trained shelter managers and other trained personnel and special equipment for functions such as radiation monitoring are to be provided. It would be impractical to provide these for very small capacity shelters.

Separated Entrances

In view of the likelihood that exits may be blocked by debris, or made non-functional for other reasons, it is a reasonable precaution to require at least two entrances for each shelter, separated by some minimum distance from one another.

Ventilation Adequacy

Shelters will ordinarily be occupied by closely spaced people for extended periods of time. If ventilation is not provided, body heat will cause temperatures within the shelter to climb to unacceptable levels. In addition, the carbon dioxide levels in the room air will reach hazardous levels. Dangerous levels of other contaminants are also possible. This means that it is mandatory to provide ventilation for the shelter. Furthermore, since electrical service to the building in which the shelter is located may be disrupted, it will not be possible to rely on electrically powered ventilation systems, unless there is a reliable source of emergency power with an ample supply of fuel. Thus, the shelter must be provided with emergency ventilation and with openings to the outside for air supply and exhaust purposes. Part of the initial evaluation of potential shelters will be for the purpose of establishing whether it will be feasible to adequately ventilate the portion of the basement expected to serve as a shelter.

Temperature Limitations

As has just been mentioned, one of the most important reasons for providing ventilation is to keep shelter temperatures within safe habitation limits. However, in some circumstances, there are heat sources within or adjacent to the proposed shelter that could not be eliminated to help maintain temperatures at acceptable levels. For example, in the case of a critical facility, nearby heat producing equipment might have to be kept in operation, and this would mean that an otherwise usable basement would be too hot to consider.

Flooding Danger

In assigning people to basement shelters, the risk that the basements could be flooded as a result of an attack should be kept in mind. This could occur if dams, storage tanks, pipelines, or other potential sources of large amounts of liquid are damaged in the attack. Where possible, shelters where the probability of flooding is remote are to be preferred.

Hazardous Material Danger

In determining the suitability of a potential shelter, another consideration that may be important in some circumstances is the location of the structure in relationship to hazardous material storage areas. If hazardous materials that are susceptible to being released, especially gaseous and liquid types, are stored or handled nearby, the structure would not be as appropriate for selection as a shelter as some other choice without this potential problem.

Fire Danger

If there is a sufficient inventory of potential shelters to provide some flexibility, structures where the danger from fire is relatively high should be avoided. In the nuclear attack situation, fires hazardous to the shelter inhabitants might result from the flammable nature of either the contents or the structure of the building in which the proposed shelter is located. Another possibility is that the fuel loading in the general area is high and the possibility that the area would be involved in an area fire is also high.

Debris Danger

Depending on the location of a potential shelter in relation to other structures such as high-rise buildings, large smokestacks, or other sources of debris, it may be unsuitable for consideration as a shelter. If the air blast could reasonably be expected to topple such structures over onto the basement in question, it may not be wise to designate it as a shelter. According to Ref. 1, "This consideration essentially eliminates the use of basements of multi-story buildings and requires that shelters be located at least one building height away from any nearby structure." The advice just quoted is for key worker shelters, and it may be more feasible to consider such shelters in areas where the peak overpressures would be expected to be much lower.

Nuclear Radiation

The nuclear radiation protection scale will consider both the inherent protection from nuclear radiation afforded by the existing structure and its surroundings and the degree of difficulty of providing some specified degree of protection, based on difficulty of adding needed amounts of soil or other shielding materials, numbers and sizes of openings to be closed, etc.

Air Blast

The air blast protection scale will consider both the inherent protection from blast effects afforded by the existing floor slab, walls, and other structural components and the degree of difficulty of upgrading the structure by shoring, providing effective closures, etc.

OTHER POSSIBLE FACTORS

The listing presented in Table C-1 is considered preliminary. Based on additional field experience and future research, it may be found desirable to modify the items listed or the criteria for acceptability. A recent study (Ref. 2), for example, contains a listing of 23 "Critical Items for Shelter Habitability", as determined by a group of experts. These are broken down into three main categories of Environment, Shelter, and Personal, as shown in Table C-2. Those assigned the highest numerical rating were considered most critical. Although some of the items are independent of the shelter selected, provision of others could be facilitated substantially by the choice of more suitable basements as shelters. It may be possible to accommodate additional items of this sort in the rating system, without unduly complicating the procedure.

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TABLE D-2

CRITICAL ITEMS FOR SHELTER HABITABILITY

		Mean Rating
Environment	(6 items; mean rating, 7.5)	
1.	Radiation sensing equipment	9.0
11.	Air movement/ventilation	7.5
12.	Temperature	7.3
20.	Humidity	7.0
21.	Illumination	7.0
22.	Square feet per person	7.0
Shelter	(11 items; mean rating, 7.6)	
2.	Waste and sanitation facilities	8.5
4.	Medical facilities	8.0
5.	Quality of leadership	8.0
6.	Water amount	8.0
7.	Communication equipment	7.8
8.	Strength of leadership	7.7
15.	Package ventilation kit (PVK)	7.3
16.	Access to outside communication	7.2
17.	Failure of vital instruments	7.2
18.	Shelter ready for occupancy	7.0
23.	Amount of leadership	7.0
Personal	(6 items; mean rating, 7.5)	
3.	Presence or absence of family members	8.2
9.	Type of sickness	7.6
10.	Overall health	7.5
13.	Fear of unknown	7.3
14.	Length of stay - known/unknown	7.3
18.	Pre-existing personal problems	7.1

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DAMAGE FUNCTION RATING PROCEDURE FOR FLAT SLAB BASEMENT SHELTERS — Unclassified

December 1982

Scientific Service, Inc., Redwood City, CA
Contract EMW-C-0678, Work Unit 1622D — 257 pages

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The report includes: a descriptive listing of basement structural systems and other pertinent basement parameters; a description of the characteristics of typical flat slab basement designs; a review of applicable casualty data and prediction models for nuclear warfare casualties; a summary of previous research on development of casualty functions; a description of the current status of the damage and casualty function development procedure; casualty function predictions for representative flat slab basements; and conclusions and recommendations.

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