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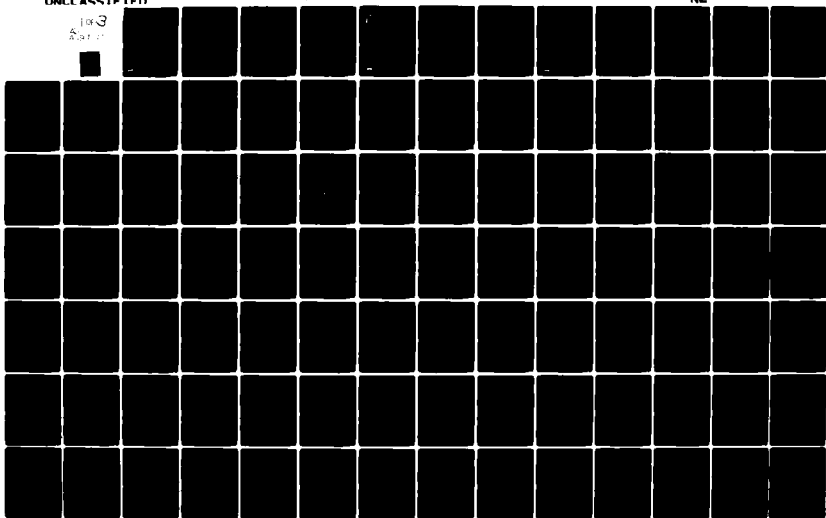
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**UPGRADING BASEMENTS FOR COMBINED NUCLEAR
WEAPONS EFFECTS: PREDESIGNED EXPEDIENT OPTIONS II.**

Final Technical Report

July 1980

Contract No. DCPA01-77-C-0227
FEMA Work Unit No. 1155C

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A1-22	In 4th line from bottom: Correct spelling to "static".
A2-9	In footnote: Insert parenthesis to change to "... table) is ...".
A2-20.1	In 4th column, heading: Correct spelling to "Length".
B2-8	In Figure B2-1A: Correct arrow for "Cap Block" to run to block that is on top of "Post".
D1-13	In definition of "ksi": Change "kip" (within parentheses) to "ksi".

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20 ABSTRACT

By: H. L. Murphy*

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. The purpose of upgrading such basement spaces is to provide shelter when needed by persons in: host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using selected target aiming points and Mt-range bursts; and, risk areas, where shelter is needed that is within 15-minutes travel time of each key worker's place of work and provides potential shelter for 30- to 50-psi air blast ranges, in terms of peak free field overpressure.

Chapters of this report's main text are devoted to discussions of: background; general principles applicable to upgrading basements; closures for all basement shelter openings/apertures, in terms of principles for providing them; needs to be met in strengthening the structure over shelter candidate basements; some techniques and materials that can be used for such structure strengthening; and, shelters for key workers. In general, the main text of this report is intended for the artisan, the appendices having the more extensive, technical data and discussions.

The titles of the appendices are: Blast-Resistant Design/Analysis General Approach; Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication; Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; Plywood Use for Closures - Design; Wood Beam and Column Design - Simply Supported; Home Basements Upgrading in Host Areas; Blast-Resistant Design/Analysis of Steel Members; and, Structural Steel Local Availability and Use for Blast Shelter Upgrading.

The suggestions, guidance and technical help of M. A. Pachuta, G. N. Sisson, and D. W. Bensen, FEMA, are gratefully acknowledged, as are the contributions of former colleagues C. K. Wiehle, E. E. Pickering and J. E. Beck.#

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UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDESIGNED EXPEDIENT OPTIONS II

Summary

Final Technical Report

July 1980

By: H. L. Murphy
Consulting Civil Engineer*

For: Federal Emergency Management Agency
Washington, D.C. 20472

Contract No. DCPA01-77-C-0227
FEMA Work Unit No. 1155C

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SUMMARY

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. It is recommended that any serious user of this report have copies of the reports covering the first two phases at hand. The purpose of upgrading such basement spaces is to provide shelter when needed by persons in host or risk areas or elsewhere. As stated in the second phase report, some work results were reported there for which the work was actually performed under this third phase's funding.

The first two phases of work were performed under a policy guidance that called for exploiting the inherent ultimate blast resistance of the slab over the basement selected for upgrading consideration, that is, upgrading work that would strengthen the remainder of the floor support system to a level equaling the inherent strength of the floor slab. The hoped for result was shelter adequate for, say, 8 to 15 psi blast free field overpressure. Exceptions were made, of course, such as the case of a large basement cut in half by a long-way interior corridor, and covered on each half with a rather long span, one-way slab running between corridor walls and outer walls; the basement cover slab's potential blast resistance could be upgraded considerably by simply running added support walls parallel to the interior corridor and supporting each existing one-way slab at its midspan.

In this (third) phase of the project work, shelter guidance for selection of candidate basements for upgrading has been re-oriented to meet the CRP (Crisis Relocation Plans). The CRP requires shelter in two kinds of specific geographic areas, which are located by study of selected attack areas in the United States and relative air blast ranges from the targets:

1. Host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using the selected target aiming points and Mt-range bursts. Circumstances have required, in certain cases, a policy exception to provide shelter at and beyond the 3 psi range; the appendix on Home Basement Upgrading for Host Areas is specifically aimed at meeting this shelter need.

2. Risk areas, where shelter is needed that: (1) is within 15-minutes travel time of each key worker's place of work; and (2) provides potential shelter suitable for 30 to 50 psi air blast peak free field overpressure. Such shelter is discussed in Chapter 7.

The remaining chapters of this report's main text are devoted to a more complete background discussion; a discussion of general principles applicable to upgrading basements; a discussion of closures for all

basement shelter openings/apertures in terms of principles for providing them; a discussion of the needs to be met in strengthening the structure over shelter candidate basements; a discussion of some techniques and materials that can be used for such structure strengthening; and, a discussion of shelters for key workers as just mentioned. In general, the main text of this report is intended to be understandable to the non-engineer/architect and to provide help, for such a person as an artisan semiskilled in carpentry, in dipping briefly into several of the technical appendices to use them without having to either study or understand completely each entire appendix.

The titles of the appendices are: A. Blast-Resistant Design/Analysis General Approach; A1. Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication; A2. Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; A3. Plywood Use for Closures - Design; B1. Wood Beam and Column Design - Simply Supported; B2. Home Basements Upgrading in Host Areas; D1. Blast-Resistant Design/Analysis of Steel Members; and, E1. Structural Steel Local Availability and Use for Blast Shelter Upgrading.

Acknowledgments

Through suggestions and guidance, the technical help of M. A. Pachuta, G. N. Sisson, and D. W. Bensen, U. S. Federal Emergency Management Agency, was freely given and is gratefully acknowledged. Similarly acknowledged are: the work of a colleague, J. E. Beck,¹ in assisting specifically in preparing Appendix A2, as well as contributing pieces, plus advice in technical discussions, used in many places in this report; considerable work earlier in this project phase by another colleague, C. K. Wiehle; and the work of E. E. Pickering² in preparing Appendix E1.

¹ Formerly at SRI International and currently the principal of James E. Beck and Associates, Palo Alto, California.

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Chapter 1

INTRODUCTION

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. Reports covering the first two phases are References [1 and 2].¹ It is recommended that any serious user of this report have copies of both references at hand.² The purpose of upgrading such basement spaces is to provide shelter when needed by persons in host or risk areas or elsewhere. As stated in the footnote on page 1 of the second phase report [Ref. 2], some work results were reported there for which the work was actually performed under this third phase's funding.³

The first two phases of work were performed under a policy guidance that called for exploiting the inherent ultimate blast resistance of the slab over the basement selected for upgrading consideration, that is, upgrading work that would strengthen the remainder of the floor support system to a level equaling the inherent strength of the floor slab. The hoped for result was shelter adequate for, say, 8 to 15 psi blast free field overpressure. Exceptions were made, of course, such as the case of a large basement cut in half by a long-way interior corridor, and covered on each half with a rather long span, one-way slab running between corridor walls and outer walls; the basement cover slab's potential blast resistance could be upgraded considerably by simply running added support walls parallel to the interior corridor and supporting each existing one-way slab at its midspan.

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- Host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using the selected target aiming points and Mt-range bursts. Circumstances have required, in certain cases, a policy exception to provide shelter at and beyond the 3 psi

¹ Brackets are used herein to indicated sources in the References list at the end of this report (main text).

² Available for purchase from NTIS, Springfield, Virginia 22151.

³ For example, original Appendices A1 and A3 herein.

range, meaning that Appendix B2, Home Basement Upgrading for Host Areas, is specifically aimed at meeting this need for shelter against 2-3 psi air blast peak free field overpressure.

- Risk areas, where shelter is needed that: (1) is within 15-minutes travel time of each key worker's place of work, and (2) provides potential shelter suitable for 30 to 50 psi air blast peak free field overpressure. Such shelter is discussed further in Chapter 7.

The remaining chapters of this report's main text are devoted to a more complete background discussion; a discussion of general principles applicable to upgrading basements; a discussion of closures for all basement shelter openings/apertures in terms of principles for providing them; a discussion of the needs to be met in strengthening the structure over shelter candidate basements; a discussion of some techniques and materials that can be used for such structure strengthening; and, a discussion of shelters for key workers as just mentioned. In general, the main text of this report is intended to be understandable to the non-engineer/architect and to provide help, for such a person as an artisan semiskilled in carpentry, in dipping briefly into several of the technical appendices to use them without having to either study or understand completely each entire appendix.

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Chapter 2

BACKGROUND

The Introduction section of the second phase report [2] has been updated and is used below.

Because of their inherent shelter potential, basements of substantial buildings having a concrete slab first floor (supported by either steel or reinforced concrete beam-girder-column systems) are the natural choice for relatively high degrees of protection. The first floor (floor over basement) concrete slab ordinarily has a rather high degree of air blast (collapse) resistance, because for normal use it must withstand individual point loads, as well as general area loads. The supporting beams and girders become progressively weaker, in that order (in ultimate or collapse strength), however, as the tributary areas served increase in size and thus the effect of normal use point loads decreases for these heavier supporting members. Columns may or may not be relatively weaker depending on the height of the building. It is frequently found that the first floor slab itself will resist about 10 psi peak "side-on" air blast pressure or more, but that this potential is degraded rather seriously by weaker beams and girders, and sometimes columns. In addition most basements have exterior openings, and all have interior openings, permitting air blast to enter (assuming that the air blast wave passes through or destroys the building above the basement). Thus, the general principle to be followed is to exploit the relatively high floor slab resistance, through closing openings as well as applying strengthening measures to the other portions of the first floor system and possibly to the basement column system. If the first floor slab does not have an acceptable level of inherent blast resistance, upgrading the basement may still be well worth considering but should be balanced against the cost (in materials, elapsed time and manpower used) of alternate schemes for shelter (e.g., expedient/field shelters; constructed R/C (reinforced concrete) and/or corrugated metal ducts (see Chapter 7); mines, etc.).

The measures available for upgrading existing basement space for shelter purposes may be categorized as either "expedient" or "engineered." Expedient measures are those that can be accomplished in a relatively short period of time (say two to three days) during a crisis build-up period by building occupants using readily available materials. Expedient measures may be pre-engineered with resulting designs distributed in advance in "how-to-do-it" drawings and instructions. Engineered measures also require longer periods of time and the services of professional engineers for evaluation and design, perhaps tailored to a specific building or a specific type of building.

Upgrading measures considered include prevention of air blast entry into the shelter space, reduction of air blast loading on exposed areas, strengthening of floor system structural members, provision of debris

protection, provision of "last resort" shelter in case of floor system collapse, and other protective measures. Both closed and open shelter situations were considered as were post-attack considerations.

For the expedient case, the most common vulnerability problems were examined and principles of protection are given. Specific building features requiring protection are illustrated and suitable methods of protection and materials are presented. The degree of protection afforded by the various methods and materials are given. Suggested local sources of materials and required tools are also given. The expedient section is prepared in "how-to-do-it" illustrative manner so as to permit ready application by non-engineer building occupants and other untrained personnel.

For the engineered case, the air blast resistance characteristics of suitable basements in existing buildings are described along with upgrading principles and techniques. Methods of evaluating individual buildings for basic first floor system air blast resistance are discussed. Upgrading design guidance for various building features is given. Specific detailed evaluation and design procedures for the more complex upgrading problems are given in appendices. Several examples of existing buildings are also given, with basement upgrading measures applied.

It is intended that this report, together with the first- and second phase reports [1,2] serve as a basic reference and guidance for civil defense planners, building owners, occupants charged with upgrading shelter space for themselves, engineering enterprises, and others concerned with the air blast upgrading of existing buildings before or during a strategic population relocation, or other civil defense shelter program. The information contained herein will also be useful for expedient upgrading, on an opportunity basis, of buildings used for temporary shelter in the population relocation or "host" areas.

Many of the matters mentioned above were covered in the first-phase report [1] for which an overview is provided by including that report's Contents, Table and Figures lists; see Table 1. Figures 1 through 11 of the earlier report [1] provide schematics or concepts for closures and structural strengthening; the later work reported in Reference [2] and hereinafter includes engineered/predesigned data for use in closures and in structure strengthening.

For technical readers such as civil engineers and architects interested in strengthening of basements for combined nuclear weapons effects shelter, recommended reading includes the Appendices herein plus References [1 and 2] for existing basements, and References [3 and 4] for basements under design or planning.

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Open Versus Closed Shelters

Some vulnerability problems of open and closed shelters are discussed starting on page 6 of Reference [1], as indicated in its Contents list reproduced herein in Table 1. Certain matters, however, are worth brief mention here.

Open shelter requires that all materiel in the shelter be fastened with sufficient strength to resist the anticipated entering blast wave, or else such items of materiel may well become missiles causing injury or death. An alternative is to surround the materiel items that pose potential missile hazards with a blast resistant barrier, so that the blast wave does not strike the items. Dealing with such potential missile hazards can be expensive (in terms of materials, elapsed time, and manpower), varying with the type of structure: for example, a parking garage might be provided with such missile hazard reduction on a cost-effective basis, whereas a potential shelter space with many pieces of equipment of varying degrees of fixity could well dictate against considering open shelter.

Closed shelters can be provided with aperture closures (see Chapter 4) and, within the range of air blast peak free field overpressures considered in this study, that is up to 50 psi, materiel will generally not require fastening in place; exceptions would be made in the case of overhead items by insuring that the fastenings are adequate to take any shock or vibration hitting the overhead structural members.

For host areas, where shelter against air blast up to 2 or 3 psi (see Chapter 1) are contemplated, work in anchoring materiel items might be expected to be modest in cost.

Subject to these brief comments, this report generally contemplates the use of closed shelters, unless specifically mentioned otherwise. A parking garage, for example, could well be the exception because of the problems of providing closures for the vehicle entrances in contrast to the cost of anchoring down or removing materiel items.

Appendices

The appendices of this report include several that are new to the work and others that are extracts, revisions, or provided totally by reference to a previously published appendix. Because of the manner of their development, the appendices are not numbered in the order of their use (as perhaps they should be).

Chapter 3

UPGRADING EXISTING BASEMENTS - GENERAL

This report is concerned with upgrading work for nuclear effects shelter in existing building basements, that are fully or partially below ground level and that can be bermed with soil up to the level of the first floor slab. If floor-level berming simply cannot be accomplished for the entire basement, the lessened radiation protection, as well as the need for increased blast protection of exposed wall portions (peak reflected pressure from a blast wave hitting a wall head-on is approximately three times the peak blast free field overpressure), must be considered. Generally, use of only part of the basement for closed shelter is more costly (in terms of available resources) than use of the full basement, because of the need for blast resistant interior walls around the partial basement shelter.

Radiation protection upgrading, whether for fallout or initial nuclear and thermal, usually requires simply additional mass in or on the floor over the shelter, plus of course the berming mentioned above. This can be bagged material or loose soil (easily placed on the floor slab), in sufficient amount for the required level of protection generally stated in full policy statements only briefly mentioned earlier. Use of part of the (ultimate/collapse) strength of the overhead slab will, of course, reduce the strength available for blast resistance. Calculations involved in such portioning of the overhead floor strength are illustrated by examples in Appendix B2 on upgrading home basements, but would be similar for a floor over higher blast level shelters.

Blast resistance needs to be considered in upgrading can generally be described under the following three categories:

(1) Peak air blast free field overpressures up to 2 or 3 psi: As briefly mentioned earlier, this level of blast resistance applies to host areas used under the CRPs (Crisis Relocation Plans). In home basements, one can assume that the interior pressures will have a long enough rise time to reach the peak pressure, to allow handling the blast loading as a statically applied but very short duration load - stated another way, to consider the blast load as an impact load only in terms of duration, not in terms of an instantaneously applied load. Further, one should consider the ultimate/collapse strength blast resistance of the floor over the basement, not with the usual design factors of safety. These matters are discussed in more detail in Appendices A and B2.

(2) Blast resistance equalling a free field overpressure that will fully tax the inherent blast strength of existing slabs over the basement shelter to be upgraded: Although this level of blast resistance is not current but earlier policy, as discussed above, this level may represent the most that can be done for many basements considering the

available time and material sources, and/or may be the most cost-effective shelter when considered against constructing other shelter, such as expedient shelter schemes [5,6], buried conduits/large pipes, etc. This blast resistance level should work out to roughly 10 psi for a sizable number of existing basements or, say, 6-15 psi as a range.

(3) Blast resistance of 30-50 psi peak free field overpressure, the level needed for risk areas as shelter for key workers, again under CRPs (Crisis Relocation Plans): Work to date has indicated a low probability of getting this protection level of shelter in existing buildings, unless a truly high level of manpower and material resources is spent and, even then, serious questions need to be first answered by tests concerning the blast resistance that can be expected from existing exterior basement walls. This shelter level and related problems are discussed further in Chapter 7.

Another general or common technical matter that should be mentioned at this point is that of bearing between existing members and upgraded members, and their loads and supports. Bearing problems are discussed frequently in the following portions of this report, in terms of specific building materials, but two points are worth making at this juncture: first, if two different materials, or two different strength levels of the same material (e.g., wood), must bear on each other, the required common bearing area will, of course, be determined by that required for the weaker material; and, second, bearing damage, even destruction, may be acceptable in *upgrading design*, because of the 1-shot/1-time loading assumed for nuclear shelter design and the energy absorbed in damaging components such as the wood cribbing members or the wood wedges under columns.

For the last general upgrading suggestion, the reader/user of this report is encouraged to review Reference [1] for its eleven upgrading schemes shown in pages 15-33, specifically Figures 1 through 11 of that report. While these figures are not to scale and are thus called upgrading schemes herein, the following chapters and appendices will give the reader/user the means to "size" the members, whether beams, columns, or beam-columns, at least in some of the materials contemplated for use.

Chapter 4

CLOSURES

Closure is used herein to describe the upgrading covering put over all apertures or openings in the existing basements selected as potential shelter, whether the opening is for a window, door, ventilation, utility, or other, whether small or large (for a 2-in. pipe or a double door). In planning for each closure, shelter ventilation should be continually kept in mind, and the closure be planned for its ability to remain open as long as possible if such is needed for ventilation; for example, a vertical opening might use a steel plate, guillotine-type closure, that need only be tripped to fall into place, perhaps the quickest operating of any closure type.

Vertical closures should be avoided to the maximum extent possible, simply because they usually involve dealing with the blast load's peak reflected pressure rather than its side-on pressure on the surface of the ground, the difference being an approximate three times increase in peak pressure level, if one has to deal with reflected pressure created by the blast wave striking a vertical wall, or going down a window-well and striking an end wall. This ratio of reflected to side-on pressure varies considerably with conditions, but to handle the simple situation of a blast wave travelling along the ground and striking a vertical surface head-on, with a reasonable distance to travel to clear around the vertical surface, the ratio is approximately 3 to 1 (at least in the range of free field overpressures of interest herein); more precise ratios may be obtained by referring to the scale shown on page B1-26 of Appendix B1. Perhaps the commonest example that might be used would be that of a window-well, serving the basement of a building from large to single family residence in size; a closure of the guillotine-type might well be mounted on the outside of the exposed basement wall to drop down over the well opening into the building, but such planning would require this closure to bear the multiple-reflected pressure caused by the blast wave entering the window-well. A simpler and far better solution might be to put a closure over the top of the window-well, preferably at ground surface, and cover it with berming material (soil, sand, whatever); the closure might be a plywood panel or a steel plate, among others. In later chapters, where the sizing of the closure, beam, column, etc., is described, the anticipated peak applied blast loading is used, leaving it to the reader/user to make this peak applied blast loading the one that is felt by the closure or structural member, whether it be the peak reflected (head-on) blast loading or a peak side-on blast loading.

An easily overlooked item in the matter of closures is that of the need for examining the blast resistance/strength of the frame around the opening to be protected - in other words, can that frame support the loading to be transferred from the closure to the frame? For example, openings built into reinforced concrete floor slabs usually have extra

reinforcing steel to support their perimeter. Any doubts about the adequacy of the frame to support the closure must usually be handled by some upgrading supports specifically for the frame: For example, there may need to be some timber framing around the door opening, which framing is next supported by some diagonal members blocked on the floor to other walls or to interior footings, etc., (see Figure 8 of Reference [1] for illustration of this diagonal bracing for a wall; it would be similar for a door frame or any other component requiring added support).

Frames that support the closure on two opposite sides only, call for a closure that acts as a one-way bending member that primarily experiences bending stresses along its span between the two support edges. Similarly, a closure supported on four sides is termed a two-way bending member; at the risk of being premature, passing mention is, nonetheless, made here that a two-way bending member gains little in blast resistance over a one-way member spanning the shorter span of the opening, if the longer span is more than 2 to 3 times the shorter span of the opening (more on this later). For a closure supported on three sides, the purposes of this upgrading report are best served by treating such a closure as being supported only on the two opposite sides and ignoring the support given by the third side, at least for sizing (design/analysis) purposes.

Fastening blast closures in place is a matter worth brief mention: If the closure is to resist blast (we do use closures for radiation resistance soil alone, of course), and the blast is expected to apply head-on to the plate as with a reflected blast loading, the situation is good for a closure lying on the blastward side of the frame, because there is little tendency for the closure to move sideways and it needs be held in place only to the extent required for positioning prior to blast loading. If the blast is expected to hit the closure side-on, then the fastenings would probably include cleat-type blocks on some or all sides of the closure to prevent sliding, as well as the fastenings to maintain closure position prior to blast loading. The negative blast wave that usually follows the blast wave's transit has been purposely ignored; if it's considered a problem in a specific instance, then the fastenings must be adequate to meet the negative blast phase.

Chapter 5

STRUCTURE STRENGTHENING NEEDS

This chapter deals briefly with the strengthening needs of members in the existing structure, that is the basement that has been selected for upgrading for nuclear protective shelter.

A. Wood and Steel Existing Bending Members

Wood and steel existing structure bending members have a considerable advantage for upgrading over existing reinforced concrete (R/C) members. For example, because of their material homogeneity, steel or wood beams/girders/floor stringers can have added intermediate supports at any chosen location along their length (assuming they are prismatic), whereas an existing R/C bending member can have added supports only where appropriate after considering whether the principal tensile reinforcing steel is located near the bottom or top face of the member, a serious restriction for the R/C members. One exception should be made for wood bending members, in contrast to steel members: the wood member might have been carefully placed so that a tight wood knot (falling near midspan and located near one edge of the member) is placed so that the tight wood knot is near the top (compressive) face of the member under bending stress, rather than near the bottom (tensile) face of the member under bending load; for such a case (perhaps uncommon), problems to be dealt with are similar to those in upgrading R/C bending members. Assessing the benefits of added (upgrading) interior supports under an existing wood beam, for example, does require consideration of each criterion for sizing (designing/analyzing) the member, because the mode of failure may change (from flexural to horizontal shear) in shortening the effective span of the member. For examples in wood bending member upgrading, see Appendix B2.

B. Reinforced Concrete Existing Bending Members

Reinforced concrete existing members entail much more difficulty in terms of providing added (upgrading) interior supports in beams: Perhaps the foremost problem is that of finding out how much reinforcing steel is imbedded in the concrete and where it is located. Certainly some assumptions can be made but they are an inadequate substitute for seeing a set of as-built plans, which may indeed be available from the building owner or the files of the building department of the city/county building department.

For upgrading R/C beams, assumptions can be made: The center third of a single-span, simply-supported beam will behave at least as strong as a shorter simply-supported span, if supports are added at the third points of the existing beam. This means that the blast loading resist-

ance of the center third of the beam has been increased roughly nine times over its strength prior to upgrading. For a R/C beam continuous over several simple supports, any span might be similarly examined; if a quarter of the span length, centered on the midspan, has added (upgrading) columns, that quarter-span portion of the beam span might have its blast resistance increased roughly 16 times its strength before upgrading.

Once these easy assumptions have been made and utilized, though, things may get sticky for further upgrading, in the absence of details on the as-built reinforcing steel and its placement: However, in the beam portions between the upgraded midspan portion and the original column supports, one might put in equally spaced columns at a spacing at or less than 3 times the overall depth of the R/C beam. This approach amounts to treating each segment of the R/C beam, outside of the midspan portion just discussed, as if each new short span of such ends is a concrete pedestal (lightly reinforced). Conversely, if one has as-built details on the reinforcing steel amount and location, such things as a better approximation of lengths for the midspan portions just described, the possible membrane action of the reinforcing steel based on its imbedment lengths, etc., will allow some refinement of the location of added (upgrading) columns.

Existing structures evaluation (ESE) techniques developed by Wiehle, Bockholt, and Beck [7A,7B,7C,8] are complex and detailed, but do deal with these analysis problems peculiar to all R/C members. Similar thinking to that just described for R/C beams may be used for R/C one-way slabs, and sometimes R/C two-way slabs.

While R/C flat slabs apparently offer an upgrading potential, R/C flat plates were examined⁶ for upgrading, especially against punching shear of columns through the flat plate; the upgrading considered was to put vertical supports around each column, but even at various distances (from close to each column out to one-quarter of their span) the flat plate's predicted blast resistance was increased very little by such upgrading.

The ESE work specifically in connection with 11 selected NSS buildings scattered throughout the United States is described in Chapter 7.

Related testing and analytical work under FEMA support has been, and continues to be, done by others [9A,9B,9C], including not only the R/C member types discussed above, but also including R/C waffle slabs, flat plates, flat slabs, etc.

The following paragraphs were prepared by J. E. Beck who has just completed a report on the "as built" strength of building floor systems [8] and is currently analyzing an additional 25 buildings along with further model verification/upgrading work:

⁶ For this project by C. K. Wiehle using his ESE techniques.

The best (R/C) candidate members for upgrading are those members with spans of 18 ft or greater that can be upgraded by intermediate supports at third or quarter points. Furthermore, analytical results therein indicated that if upgrading is accomplished by putting intermediate supports at about 6-ft spacing, an R/C slab about 6-in. thick can resist an overpressure of approximately 30 psi blast, and a slab about 8-in. thick can resist approximately 50 psi.

"Most R/C slabs are designed to have a span-to-thickness ratio of 24. Therefore, slabs having clear spans greater than 12 ft will normally have a potential resistance greater than 30 psi, and slabs having clear spans greater than 18 ft have a potential resistance greater than 50 psi.

"The predicted upgrading potential of R/C slabs is based on the calculated capacity of one-way simply-supported slabs with reduced spans (about 6 ft). Although it is true that the steel reinforcing in an "as-built" structure may not be at optimum points for upgrading, it is still probable that the calculated simply-supported strength of each upgraded center portion is representative of the strength of the slab. This will also be true even at ends that are designed as fixed and, therefore, may at first glance appear weaker. This statement is based on the following observations:

"(1) The detailing of a fixed joint requires twice the moment capacity of the center of the slab, and compression steel at least equal to 1/3 the central steel is present in this section.

"(2) If the joint is designed as fixed, it will act as a simply-supported beam straddling the top of the support member and will have approximately the strength of a simply-supported element having a free span equal to the clear distance between the supports."

C. Existing Columns

ESE work on R/C columns has indicated that they are adequate for much heavier blast loads than their (normal-use) design loads, and especially in the case of a multistory structure, say, 4 stories or higher, where the superstructure may be expected to allow most of the blast loading to blow through between floors rather than subject the basement columns to higher blast loadings due to such things as overturning considerations. Further, the upgrading just described for beams makes clear the plan to add a large number of columns, meaning that each original/existing column will have its contributory blast loading area on the first floor considerably reduced in size, and thus its individual column loading considerably reduced. The need, therefore, for increasing the strength of reinforced concrete columns for upgrading is small to nonexistent. Similar thinking may be applied to columns of other materials existing in the basement to be upgraded into a nuclear protective shelter, in that the remedy is simply to have existing columns share the anticipated blast loading with additional columns.

Column footings in structures to be upgraded pose no significant problems in that those under existing columns will have their load shared as just described for the columns themselves; for columns added in upgrading work, a recent test [10] at the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, involved loading a square column consisting of four 4x4s (actual 3.5x3.5 in.) strapped together and bearing on a "typical" concrete floor slab over compacted gravel over compacted soil, with the results indicating that this column could carry a static load initially peaking at 50 kips then increasing to a new peak of 84 kips and degrading somewhat, but always above 50 kips for a loading of some 10 minutes (many, many times the duration of a nuclear blast loading). This loading amounts to roughly 2,000 psi on the column and footing. At the peak loading, displacement of the footing was 1.0 in. and its displacement at the time the tests was stopped was about 1.45 in. The test will be reported in a forthcoming WES technical report, of course, but these results were gratefully received by those who have planned extensive upgrading work, where many additional columns have been planned for use supported on a "typical" basement floor slab with nothing more than a couple of wedges under the column (for erection convenience).

D. Existing Exterior Basement Walls

Exterior basement walls, whether plain CMUs, reinforced CMUs, or brick, or certainly R/C construction, have been generally accepted as having sufficient blast resistance capacity for host area shelters, and those shelters contemplated where the inherent strength of the main slab over the basement is to be exploited (say, 8-15 psi air blast peak free field overpressure). However, serious concern over the adequacy of exterior basement walls, including walls of R/C construction, has been expressed when upgrading for higher blast resistance, especially as high as 30-50 psi, is contemplated. As a result, a FEMA research technical project officer, at a recent meeting with structural research contractors at WES, included in the discussions the need for static and dynamic tests of "typical" exterior R/C walls (fully and partially buried). The purpose of the proposed tests is not to see whether design procedures apparently used were adequate (they may be conservative), but more specifically to determine the blast resistant capacity of such walls in terms of absolute ultimate behavior (through collapse), that is, to answer the question of whether or not the soil transmitting the blast loading to the exterior basement wall will indeed follow and force the wall inward beyond an acceptable displacement, closely approaching collapse. Meanwhile, planning for upgrading potential, especially for overpressure levels greater than, say, 15 or 20 psi: must be conservatively evaluated as requiring strengthening of the exterior basement walls; or can be optimistically continued on the basis that the exterior basement walls would not fail because the soil will not move fast enough to push the wall inward further than acceptable. This matter impinges seriously on the direction of Chapter 7, Shelter for Key Workers (30 to 50 psi air blast peak free field overpressure).

Chapter 6

STRUCTURE STRENGTHENING TECHNIQUES AND MATERIALS

This chapter concerns meeting the upgrading needs perceived for a particular shelter candidate basement, as described in the preceding chapter.

The chapter deals with bending members (lateral loads only) such as beams, closures, girders, etc.; columns (axial loads only); and beam-columns (both lateral and axial loads). It deals with these three kinds of structural members by sizing the members to meet the need. The sizing may be done: by determining from the need the size of member required (design), and whatever material; or by assuming a particular member's availability and finding its strength capacity (analysis), then comparing the capacity with the need. The aids available herein, as described below, may be oriented to both design and analysis depending upon which way they are entered for use. One may, for example, know the load to be met on a given floor over a basement and the contributory area to be served by a column, then look at a table of column strengths and find which column(s), if any, will meet or exceed the need, using a table/graph/chart of column strengths (thereby using a little design with a little analysis!). Above all, the reader/user should not let terminology deter him/her from using the material presented (the author hereby invites comments on any simplification needs encountered).

Appendix A has been prepared for the artisan or the professional engineer/architect as an attempt to introduce the terminology basic to the structural problems in handling impact/dynamic loads, such as air blast, meeting loads that are applied in zero to a very short time and that have a short duration (of the order of seconds).

A very useful aid in reading graphs (such as those encountered in the appendices of this report, as well as those used in such civil defense training such as that given for Fallout Shelter Analysis) may be easily prepared and will prove very useful if given a trial: obtain an ordinary drafting (plastic) 45-degree triangle, clear, of about 6 to 8 inches on the two equal (short) sides; use any straight edge, a careful eye, and any sharp pointed instrument to scribe lines parallel to each of these two sides, perhaps a quarter-inch in from the edge although this is not critical; and, use a sharp pencil to darken the bottoms of the scribed lines, then wipe off the excess graphite. The aid then can be used on a design graph where multiple curves must be either read directly or interpolated between (for a particular value described by the curves or spaces between them), in order to read values from the abscissa (bottom) and ordinate (vertical) sides of the graph.

This chapter is organized first by materials to be used (wood and steel in various forms), then by specific strengthening applications, such as the aforementioned beams (bending members), columns, and beam-columns.

Wood Availability and Use

Appendix C, Typical Stockage - Local Lumberyards, Reference [2], is short and is recommended for complete reading by the reader/user.

Specifically, the predesigns reported on below (Appendices A1 and A2) used generally available stress graded lumber in one relatively strong and one relatively weak grade and species for each of two size ranges - one pair of grades for 2x3s⁷ and 2x4s (Light Framing; Construction grade for strong, and Standard grade for weak) and one pair of grades for 2x6s and 2x8s (Joists and Planks; No. 1 grade for strong, and No. 2 grade for weak). These choices are shown in Table 1B, Appendix C [2],⁸ complete with the predesigns' stresses as underlined in that Table and used below (Appendices A1 and A2). Tables 1A and 2, Appendix C [2], show the complete list from which the grade choices were made. (A later complete list is provided by Table B1-2, Appendix B1 herein.)

Lumberyards have shirt-pocket size booklets, published by grading associations (list in Reference [2] of Appendix B2 herein) giving stress grading data by lumber kind, type and size; see also Reference [11].⁹ In any case, try to use construction grades/stress-graded wood members; if stress data are unknown, simply use the tables/figures that follow (e.g., as in Appendices A1 and A2) and choose those calling for "lower strength stringers" or similar notation.

Although specific data have not yet been published/distributed, strong indications are appearing, among the professional engineers/architects in the research community, that lumber grading in too many areas and instances to be ignored, has not been up to standards achieved in the past; there may be well-founded reasons for this (the increasing proportion of second growth timber in the market), and thus not quality of work alone. At any rate, an artisan familiar with wood, even only casually, can use the shirt-pocket size grading booklets and their data, as mentioned above, plus some careful reading, to check or redo the grading work on any wood materials that become available for upgrading use. The regrading may not be as precise as that done by one regularly working in the field, but that is not the point here. Further, the artisan (e.g., one semiskilled in carpentry) need not follow upgrading guidance alone but, for an example, can see to it that, if he/she must use a wood member as a beam and must use one that has a sound knot near an edge, the sound knot is put in the top edge of the (single-span, simply-supported) beam (compressive stress) rather than in the bottom edge (tensile stress).

⁷ Nominal (actual) dimensions are 2x3 (1.5x2.5), 2x4 (1.5x3.5), 2x6 (1.5x5.5), and 2x8 (1.5x7.25) inches.

⁸ Delete "Select Structural" in left column of table.

⁹ Copies of References [11] and [12], one each, are furnished with each copy of this report sent to Distribution List addressees shown herein.

For plywoods, the predesigns (Appendices A1, A2, and A3) used primarily plywood grade Underlayment Interior (American Plywood Association (APA)) in Species Groups (of both face plies) #1 and #3, but the results also cover plywood grades Underlayment Exterior (APA), C-D Interior (APA), and C-C Exterior (APA), all in Species Groups #1 and #3; nominal thicknesses used were 1/2", 5/8", and 3/4". Additionally, plywood grade 2.4.1 Sturd-I-Floor Interior (APA), manufactured only in Species Group #1, was used in 1-1/8" nominal thickness, both alone and in combination with some of the thinner plywood grades just mentioned (the results also cover 2.4.1 Sturd-I-Floor Exterior (APA) used in the same thickness). All plywood grades used in predesigns are the most plentiful and were assumed to be planned for use under dry conditions (equilibrium moisture content less than 16%).¹⁰

Each sheet of plywood should bear an APA (American Plywood Association) Grade-Trademark stamp on one of its faces. Typical markings are shown in the fourth column of the table on page 14 of Reference [12].⁹ The reader/user of this report is urged to look for and use such stamped markings, because lumberyards are prone to apply a local terminology, contrary to the formal terminology and that one stamped on the sheet of plywood; for example, a usage local to the author's home area is to advertise and describe a plywood grade as "CDX" (both orally and printed), even "CD Exterior", a plywood "grade" that is not listed in those of the APA. Instead, the plywood grade of these sheets is actually C-D Interior (APA) with exterior glue, a plywood grade that can be found in the listing of APA grades, meaning that geometric dimensions, allowable stresses, etc., can be obtained (see Reference [12]⁹).

It is recommended that the stress-graded lumber user consider obtaining a copy of Reference [11].⁹ Similarly but for plywood, Reference [12].⁹

A. Plywood Stressed-Skin Panels (Two-Sided)¹¹ as Closures

Appendix A1, Plywood Stress-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication, includes a detailed treatment of the subject, aimed at the designer (engineer or architect); certain sections would be, however, of interest to the artisan reader, who might choose to gain an overview of Appendix A1 through use of its lists of Contents, Tables and Figures (pages A1-iii and -iv).

¹⁰ If dry conditions do not apply, all plywood must have "Exterior Glue" (so stamped); blast resistance shown in tables of Appendices A1, A2, and A3 would be reduced about 50% (conservatively; full redesigns are needed for a better estimate).

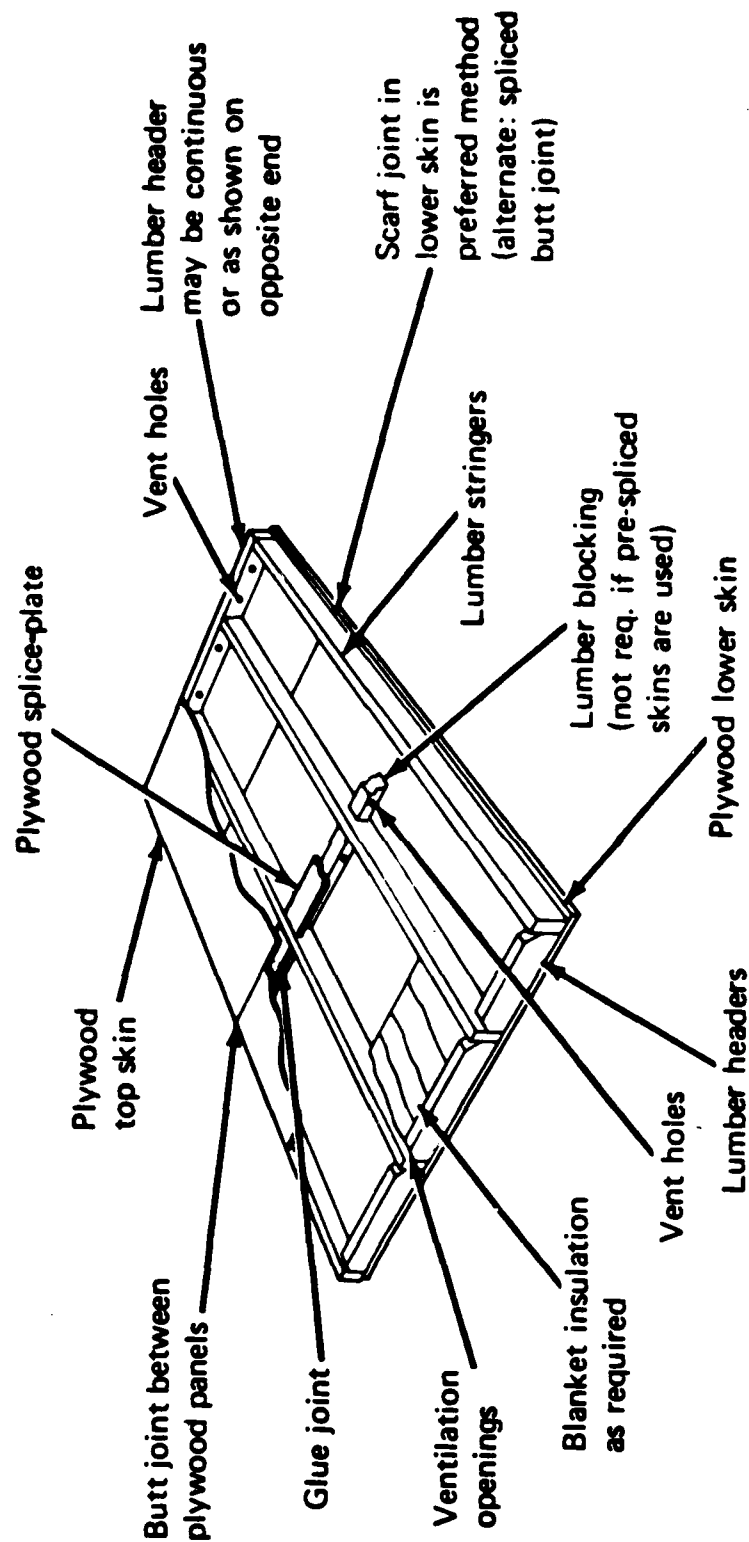
¹¹ Abbreviated as PSSPs herein.

Lumber ("2-by") sizes and grades considered for use in building PSSPs are discussed in the preceding section, as are plywood grades and nominal thicknesses. Figure 1 shows a cutaway perspective view of a 4-stringer PSSP, which also illustrates both continuous and non-continuous headers, as well as splice-plate installation. The latter may have little use herein, because closures are not expected to be needed in lengths longer than the 8-ft (sometimes 12-ft) plywood sheets that are available in plywood stocks of lumberyards. Another view (this one dimensioned) of a similar PSSP is shown by Figure A1-1A of Appendix A1. Both views show PSSPs with one outside stringer inset 1 in. and the other outside stringer projecting 3/4 in. so as to provide tongue-and-groove behavior among side-by-side panels; such detailing is perhaps impractical for the rapid construction of expedient option closures during a one- to three-day warning period, and such detailing is not recommended for other (mostly strength) reasons as well.

To obtain the strength benefits of stressed-skin structural behavior,¹⁰ PSSPs are built with the plywood face plies running parallel to the stringers, which must be joined to the plywood by either nailed-glued or pressure-glued construction (the latter is better so should be used if facilities are available therefor). See the Fabrication section, Appendix A1, for details.

Table A1-1 of Appendix A1 presents 248 designs in a form for direct reading: Columns 1-4 present nominal thickness and Face Ply Species Group for the top and bottom plywood skins of the PSSP designs; columns 5-6 give the nominal size (2x4 to 2x8) and number of stringers (per 48-in. wide PSSP), with the latter showing 4, 5, 7, and 9, but 6 or 8 may be used by taking values (in columns 7 and beyond) between 5-7 and 7-9, respectively; columns 7-8 show the PSSP required bearing length on each end support (in addition to the clear span dimension), in terms of bearing length on the plywood bottom skin and on the stringer bottom edge, respectively (the latter happens to be controlling in all designs); the remaining columns show, for each of the designs, the PSSP's estimated peak air blast overpressure resistance (psi) for clear spans from 2 to 12 ft by half-ft increments, but omitting any value below 5 psi. In a particular application, the clear span for the blast closure will be known, as will availability (sizes, grades, etc.) of plywood and stringers; from this point one might proceed as follows:

(1) Consider that the data presented covers four plywood grades in three nominal thicknesses and two face ply species grades, plus a fifth grade in one nominal thickness and one face ply species, and the plywood combinations are each used in predesigns with two stringer strengths (termed lower and higher in Table A1-1) - all as described in the preceding section, Wood Availability and Use.



Source: Reference 1 of Appendix A1

FIGURE 1 TYPICAL TWO-SIDED PLYWOOD STRESSED-SKIN PANEL

(2) Consider the data for, say, the 6th PSSP design (with 7 stringers), Table A1-1: Top and bottom skins nominal thickness are 3/4" and 1/2", respectively, both in Face Ply Species Group #3, and the 7 (lower strength) stringers are 2x4s (nominal dimensions).

(3) Required bearing length at each end of the PSSP, in addition to the clear span length, would be the larger value of Columns 7-8, or 3.5 in.

(4) For a clear span of 2.5 ft, the estimated peak air blast (side-on) overpressure resistance would be 9 psi (Column 10).

(5) Equivalent free-field air blast peak overpressure to the 9 psi just found, which is for overpressure when applied side-on, would be 4 psi free-field air blast peak overpressure if applied fully reflected (i.e., head-on); the peak blast pressure felt by the PSSP would be 9 psi in either case, if taxed to its estimated design resistance. The graph on page B1-26, Appendix B1, may be used to find such "side-on" versus "head-on" equivalent free-field air blast peak overpressures, in either English or SI (metric) units (it is sufficiently accurate for purposes herein to use $1 \text{ kg/cm}^2 = 100 \text{ kPa} = 100 \text{ kN/m}^2$ in reading the graph).

(6) For PSSP widths less than 48 inches: Convert the planned PSSP width and stringer spacing to a 48-in. wide equivalent PSSP; select the equivalent 48-in. PSSP number of stringers so that its stringer spacing is equal to or wider than that in the planned PSSP. Find the applicable overpressure value for the known clear span, as above; such value may be used without reduction for PSSP widths of 24 in. or more, but it is recommended that it be reduced linearly from 0% to 50% for PSSP widths of 24" to 8", respectively, with the latter being the narrowest width recommended for use (this recommendation is adapted from Reference [2] on page A1-29). NOTE: Panel width, as used throughout this section, is always measured perpendicular to the span direction of the PSSP.

(7) Selection of a particular PSSP design for planned use would probably be by trial-and-check repeated use of Steps 2 through 6 above.

An upgrading example using the above PSSP as a horizontal closure is shown on page A1-22 and illustrates the case of apportioning the PSSP's load capacity between peak air blast loading and soil (for nuclear radiation shielding) loading.

B. Plywood Stressed-Skin Panels (Two-Sided) as Columns & Beam-Columns

Appendix A2 deals with this subject and includes a continuation of the design procedure of Appendix A1, thus requiring frequent reference work between the two appendices. Aside from the continuation of the design procedure, the preceding section on PSSPs as closures, and its Figure 1, should be reviewed by the reader/user, as should be certain descriptive figures and general sections of Appendix A1, perhaps espe-

cially the section on fabrication of PSSPs. Table A2-1 of Appendix A2 is a listing of the computer program developed for the design procedures in both Appendices A1 and A2, thus the program will handle design/analysis of PSSPs, whether for use as beams, columns, or beam-columns. Table A2-2 provides data on PSSP designs similar to that provided by Table A1-1 of Appendix A1, but the former has the design data for columns and beam-columns, and includes only PSSPs that have the same plywood top and bottom skins. Table A2-2 shows, for column use, the total axial dynamic/impact load (in kips, or 1000s of pounds) that each PSSP may carry, the principal (and perhaps only real) use of the data. Also shown, for beam-column use, is data on the beam-column capacity of each PSSP in terms of an axial load equal to 20%, 40%, 60%, and 80% of the pure column use load, with each percentage related to the capacity remaining for lateral loads in terms of psi. All axial loads are related to a 48 in. wide PSSP.

Appendix A2 includes a sheet, PSSP Upgrading Example - Column, showing a sample calculation for a PSSP acting as a column, axially loaded (only) with both air blast and soil for fallout radiation shielding.

C. Plywood Panels as Closures

Appendix A3, Plywood Use for Closures - Design, covers the design of plywood panels as closures over apertures such as those found in basements. The design procedure applies to panels supported on two opposite sides of an aperture, or on all four sides.

The design procedure was used to develop the predesigns shown by Tables A3-1 and -2. An example of use is as follows: Referring to Table A3-1A, assume that 3/4 in. nominal thickness plywood is available in CD-PLUGGED INTERIOR (APA) grade in Face Ply Group 3 (so stamped on each sheet of plywood), and find from the table that one should refer to Table A3-1B, Block Numbers 8 and 16, for 1/2" and 3/4" plywood, respectively, meaning that Block No. 16 applies to this example; further, one should use the second line of that Block for Face Ply Group 3, in which line one finds values of (free field, side-on, long duration) peak air blast overpressure, in psi, for seven values of clear span, in inches, such as 20 psi for an 8-inch clear span. Attention should be given to the footnote of Table A3-1B. So far span conditions dealt with have been one-way simply-supported.

Two-way (supported on all four sides) span conditions are handled by further referring to Table A3-2 where one finds that, for Block Number 16 (of Tables A3-1B), the assumed plywood would have its 20 psi/ 8-inch span strength increased by 19% if supported on all four sides of a square opening/aperture 8"x8" (1:1 ratio of longer to shorter clear spans).

Appendix A3 contains further details if desired by the user. If available plywood is not one of the 10 grades entered in Table A3-1A (including 2 multi-grade entries), recourse to the design procedure starting on page A3-3 will be necessary.

D. Wood Beams and Columns

Appendix B1, Wood Beam and Column Design - Simply Supported, covers sizing (design/analysis) of wood beams - whether used solidly (side-by-side, on edge or flatwise) or spaced as in floor joists and roof rafters, and also whether used single-span or continuous over several spans, all simply supported - as well as wood columns. Further, different treatment is shown for wood beams used solidly, when covered with a thin sheet of plywood (repetitive-member use) or without a plywood cover (single-member use), because the allowable flexural stress f_b is different for these two uses.

Appendix B1 has two major sections, Wood Beams - Simply Supported and Wood Columns - Simple Supports. A design/analysis procedure is provided in each of these two major sections.

An illustrative example is presented in paragraph (5), page B1-13, of Appendix B1, for the use of wood beams solidly side-by-side, with a thin plywood covering. The next paragraph (6) deals with the same problem, but with the thin plywood covering omitted.

Sizing of wood beams used at spacings of 12, 16, and 24 in. is covered by an illustrative example in paragraph b., page B1-14, Appendix B1. Note: The results of this sizing differs from the example shown in Appendix B2, Home Basements Upgrading in Host Areas, because the latter results are based on interior blast limited to 5 psi or less on the floor above the basement, resulting in the use of a gradual rise time; in contrast, the treatment in Appendix B1 considers the blast loading rise time as zero. The results in Appendix B2 can be converted to match those of Appendix B1 by multiplying the former by a factor of 2/3.

For correcting the blast resistance found for a simply supported beam over a single-span to a simply supported beam extending continuously over two, three, four, or five equal spans of the same individual span length, a procedure and tabulation of correction factors is provided on page B1-14, Appendix B1.

For the sizing (design/analysis) of wood columns on simple supports, a design procedure is provided beginning on page B1-15, Appendix B1. A numerical example begins on page B1-22 and uses for its solution the table and graphs of Figure B1-2. A discussion of the adequacy of end bearing in columns follows.

Review, light or heavy, of Appendix B2, Home Basements Upgrading in Host Areas, is recommended for the reader/user concerned with use of

wood members in upgrading, both beams and columns; numerical examples and blast resistance values for lower and higher strength woods are included.

E. Peak Blast Resistance - Side-On versus Head-on

Finally, a conversion graph is given in Appendix B1 (page B1-26) for converting free field overpressures up to 50 psi when applied side-on, to an equal value of free field overpressure applied head-on, or fully reflected; "equal" in this sense means equal in peak blast pressure felt by the member (for example, the peak blast pressure felt by a structural member from 45 psi applied side-on is equal to 16 psi applied head-on (fully reflected)).

F. Steel Plates, Sheets, and Shapes

Appendix E1, Structural Steel Local Availability and Use for Blast Shelter Upgrading, is recommended for complete reading by the reader/user of this report.

Appendix D1, Blast-Resistant Design/Analysis of Steel Members, discusses steel design generally, structural steel material properties and strength/resistance expressions. Tables present the stresses used for sizing members (primarily plates and sheets), based on the ASTM designation for the steel, designations that are regularly used by steel suppliers. An applications section begins on page D1-9, which covers the sizing of steel plates as closures, acting either as one-way or two-way flat plates on simple supports; the section includes numerical examples carried along with the discussion. The example uses a table and graph, figure D1-1, for solving the one-way plate sizing problem, with correction factors for all steels, and correction factors for two-way plates from the one-way plate graphical solution results.

Use of steel plates and shapes may be considerably limited by weight-handling limitations where no equipment is available, other than hand labor. (Figure D1-1 solutions stop at around 500 pounds maximum weight of the steel member.) The use of steel shapes is not covered further in this report.

Chapter 7

SHELTER FOR KEY WORKERS

There is a need for shelters upgraded to the 30 to 50 psi air blast (peak free field overpressure) range. Such shelters will be in the "risk" areas briefly mentioned on page 2 herein; this shelter is needed at locations within 15-minutes travel time from each key worker's place of work. Such shelter has also been mentioned above in paragraph (3) on page 12 where it is stated that "Work to date has indicated a low probability of getting this protection level of shelter in existing buildings, unless a truly high level of manpower and material resources is spent and, even then, serious questions need to be first answered by tests concerning the blast resistance that can be expected from existing exterior basement walls."

Also discussed earlier (paragraph D, page 18), is the very serious problem of lack of knowledge/test results on the ultimate blast resistance capacity (through collapse) of existing exterior basement walls in buildings considered for shelter use.

Table 2 presents the results of existing structures evaluation (ESE) work done by James E. Beck [8] on 11 buildings across the United States; these NSS buildings are part of the 210-building "RTI statistical sample" often used for FEMA studies, both research and operational. Table 2, presenting some results of the ESE work, also has information added under this project, as shown along the right side of each sheet of the table. Attention is particularly invited to the data on "TYPICAL BASEMENT WALL" on the right side of the first sheet of the table; the data point up the discussion of the existing exterior basement wall problem mentioned earlier and cited above. Whether one considers the mean or 1% probability value (of the incipient collapse overpressures) shown in the data, as on the right of Table 1, the overall results show very little promise for upgrading existing basements to the 30 to 50 psi blast range; whether one looks at slabs (RCS), beams (RCB), girders (RCG), or joists (RCJ), of reinforced concrete, the prospects are bleak for their united floor strength exceeding a value considerably below the 30 to 50 psi range. Momentarily ignoring the basement wall problem, extensive upgrading would be required for all structural members (almost without exception) and for each closure's support frame, to reach the desired overpressure range. Cost effectiveness, in terms of expenditures of manpower and materials, could well dictate use of field type, expedient shelters, or other alternatives.

One alternative is the use of conventional large diameter pipe and culvert materials (8 ft i.d. for example); such use has been proven (by nuclear field tests) to provide high degrees of protection in a buried configuration. These include R/C pipe, corrugated steel culvert materials of round and "cattle pass" (Junior underpass) cross sections, and arch shaped (semi-circular) multi-plate corrugated steel (ammunition magazine) structures [4, p. 6-119 to -134].

TABLE 2. RESULTS FROM EXISTING STRUCTURES EVALUATION OF ELEVEN NSS BUILDINGS
11/26/79

Case	Element Type	L (in.)	b (in.)	h (in.)	f'c (psi)	fy (psi)	Support Case	Reinforcing Ratios at Cross Sections						Tensile Membrane Steel Ratio		Incipient Collapse Overpressure			Incipient Collapse Overpr. (Tens. Membr.) 1% Mean Std. Dev. Prob. (psi) (psi) (psi)		
								1	2	3	4	5	6	Short	Long	Mean Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)			
Building 100, U.S. Post Office (RCF)																					
1a	RCB-1	337	1,320	12	12	3,000	53,000	1	0.0172	0.0013	0.0013	0.0013	0.0013	0.0013	5.49	0.53	4.00	6.49	6.51	0.78	4.69
1b	RCB-1	168	1,320	12	12	3,000	53,000	5	0.0172	0.0013	0.0013	0.0013	0.0013	0.0013	23.06	2.76	19.52	26.40	10.29	0.92	8.15
1c	RCB-1	112	1,320	12	12	3,000	53,000	5	0.0172	0.0013	0.0013	0.0013	0.0013	0.0013	51.23	4.36	45.38	57.06	14.06	1.33	10.97
1d	RCB-1	64	1,320	12	12	3,000	53,000	5	0.0172	0.0013	0.0013	0.0013	0.0013	0.0013	91.43	0.63	80.37	102.49			
2a	RCB-2	223	2,320	18	18	3,000	53,000	5	0.0100	0.0000	0.0000	0.0000	0.0000	0.0000	5.16	0.72	4.26	6.08			
2b	RCB-2	111	2,320	18	18	3,000	53,000	5	0.0100	0.0000	0.0000	0.0000	0.0000	0.0000	22.72	2.83	19.89	26.35			
2c	RCB-2	74	2,320	18	18	3,000	53,000	5	0.0100	0.0000	0.0000	0.0000	0.0000	0.0000	58.33	4.60	44.33	56.33			
3a	RCB-3	197	1,900	12	12	3,000	53,000	5	0.0090	0.0105	0.0105	0.0105	0.0105	0.0105	45.71	7.05	49.67	67.76	58.71	7.05	42.30
3b	RCB-3	90	1,900	12	12	3,000	53,000	5	0.0139	0.0000	0.0000	0.0000	0.0000	0.0000	5.03	0.62	5.03	6.62			
3c	RCB-3	209	1,900	12	12	3,000	53,000	5	0.0139	0.0000	0.0000	0.0000	0.0000	0.0000	25.60	2.67	22.18	29.03			
3d	RCB-3	104	1,900	12	12	3,000	53,000	5	0.0139	0.0000	0.0000	0.0000	0.0000	0.0000	59.47	6.82	50.72	60.22			
4a	RCB-4	17	360	3	3	3,000	53,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	40.23	2.91	26.50	33.94			
4b	RCB-4	17	360	3	3	3,000	53,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	143.70	12.46	127.81	159.78			
5a	RCB-5	360	6.23	17	17	3,000	53,000	51	0.0237	0.0003	0.0003	0.0003	0.0003	0.0003	3.31	0.41	2.79	3.83			
5b	RCB-5	180	6.23	17	17	3,000	53,000	51	0.0237	0.0003	0.0003	0.0003	0.0003	0.0003	16.77	1.46	12.89	16.44			
5c	RCB-5	120	6.23	17	17	3,000	53,000	51	0.0237	0.0003	0.0003	0.0003	0.0003	0.0003	35.38	4.26	29.92	40.44			
5d	RCB-5	90	6.23	17	17	3,000	53,000	51	0.0237	0.0003	0.0003	0.0003	0.0003	0.0003	62.62	7.29	53.27	71.96			
Building 110, Navy B. Lendis State Hospital (RCF)																					
1a	RCB-1	169	1,552	7	7	3,300	53,000	7	0.0074	0.0000	0.0000	0.0000	0.0000	0.0000	5.20	0.55	4.49	5.90			
2a	RCB-1	204	1,552	10	10	3,300	53,000	6	0.0127	0.0000	0.0000	0.0000	0.0000	0.0000	3.05	0.44	3.29	4.41			
2b	RCB-1	102	1,552	10	10	3,300	53,000	6	0.0127	0.0000	0.0000	0.0000	0.0000	0.0000	0.60	0.66	7.49	9.70			
2c	RCB-1	60	1,552	10	10	3,300	53,000	7	0.0127	0.0000	0.0000	0.0000	0.0000	0.0000	17.03	2.00	14.46	19.48			
3a	RCB-2	164	254	14	14	3,300	53,000	7	0.0037	0.0000	0.0000	0.0000	0.0000	0.0000	3.31	0.49	2.69	3.94			
3b	RCB-2	82	254	14	14	3,300	53,000	7	0.0037	0.0000	0.0000	0.0000	0.0000	0.0000	6.13	0.83	5.07	7.19			
3c	RCB-2	41	254	14	14	3,300	53,000	5	0.0093	0.0000	0.0000	0.0000	0.0000	0.0000	14.36	1.42	12.55	16.18			
4a	RCB-3	254	254	20	20	3,300	53,000	6	0.0111	0.0000	0.0000	0.0000	0.0000	0.0000	7.79	0.83	6.73	8.86			
4b	RCB-3	128	254	20	20	3,300	53,000	6	0.0291	0.0000	0.0000	0.0000	0.0000	0.0000	4.33	0.53	3.52	5.14			
4c	RCB-3	85	254	20	20	3,300	53,000	5	0.0291	0.0000	0.0000	0.0000	0.0000	0.0000	12.66	1.26	11.04	14.28	14.68	1.46	11.28
4d	RCB-3	53	254	20	20	3,300	53,000	5	0.0291	0.0000	0.0000	0.0000	0.0000	0.0000	19.17	1.42	17.36	20.99	24.64	1.83	20.38
5a	RCB-5	154	20	20	20	3,300	53,000	7	0.0245	0.0000	0.0000	0.0000	0.0000	0.0000	17.06	1.72	14.85	19.26	22.14	2.23	16.95
5b	RCB-5	77	20	20	20	3,300	53,000	6	0.0245	0.0000	0.0000	0.0000	0.0000	0.0000	5.33	0.52	4.66	5.99			
5c	RCB-5	39	20	20	20	3,300	53,000	7	0.0245	0.0000	0.0000	0.0000	0.0000	0.0000	17.94	1.83	15.60	20.28			
Building 111, Grant Building (STF)																					
1a	RCB-1	18.33	--	2.5	2,000	44,000	4	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	14.61	0.59	13.73	15.80			
2a	RCB-1	216	--	5.21	2,000	44,000	71	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	67.59	6.87	50.73	74.48			
2b	RCB-1	108	--	5.21	2,000	44,000	51	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	7.27	0.70	6.37	8.16			
2c	RCB-1	72	--	5.21	2,000	44,000	51	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	15.13	1.44	13.28	16.98			
3a	RCB-1	18.33	--	5.21	2,000	44,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	34.28	3.43	29.89	38.67			
3b	RCB-1	108	--	5.21	2,000	44,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	10.97	1.00	8.92	13.02			
3c	RCB-1	72	--	5.21	2,000	44,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.25	4.37	33.65	44.85			
4a	RCB-2	18.33	--	12.04	16.1	41,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	90.63	12.86	74.35	107.32			
4b	RCB-2	108	--	12.04	16.1	41,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	14.61	0.59	13.73	15.80			
4c	RCB-2	72	--	12.04	16.1	41,000	6	0.0040	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	67.59	6.87	50.73	74.48			

TYPICAL BASEMENT WALL	
Lat/vert soil 1%	100%
press. ratio	6.2
15% (unlikely)	40

16.68	1.46	11.28
24.64	1.83	20.38
22.14	2.23	16.95

21.40	2.04	16.66**
32.34	3.11	24.81**
29.08*		60.90*

* Steel member, thus Mean and Standard Deviation values apply unchanged from regular ESE values (i.e., no problems with beam-action stress reversals, as there are with R/C).

** Based on added girder lines at midspan (.2B) or third-points (.2C), which would increase existing girders as if upgraded (.3B and .3C, respectively). However, added girder lines would require new columns and footings/caissons carried through from five to one basements (depending on which basement(s) are to be upgraded). Even so, basement walls are non-bearing, inadequate, and infeasible to upgrade; some of first basement wall is 100% abovegrade and penetrated by garage doors.

TYPICAL BASEMENT WALL
Lat/vert soil 1%
press. ratio Prob.
100% 6.2
50% 12
15% (unlikely), 40

Incipient Collapse
Overpr (Tens. Membr.)
Mean Std. 1%
Dev. Prob.
(psi) (psi) (psi) (psi)
6.51 0.78 4.69
10.29 0.92 8.15
14.06 1.33 10.97
58.71 7.05 42.30

Table 2 (continued)

Case	Element Type	L (in.)	L (in.)	b (in.)	h (in.)	c (in.)	f (psi)	Support Case	Reinforcing Ratios at Cross Sections								Tensile Membrane Steel Ratio		Incipient Collapse Overpressure				Incipient Collapse Overpr (Tens. Membr.) Mean Std. 1% Dev. Prob. (psi) (psi) (psi) (psi)								
									1				2				3				4				Short	Long	Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	
									p	p'	p	p'	p	p'	p	p'	p	p'	p	p'	p	p'									
Building 136, First Federal Savings & Loan Assn. (RCF)																															
.1a	RCB-1	16	62	2	3,000	56,000	6	6	0.0036	0.0	--	--	0.0036	0.0	--	--	0.0036	--	12.40	1.20	10.06	13.93									
.1b	RCB-1	16	62	2	3,000	56,000	6	6	0.0036	0.0	--	--	0.0036	0.0	--	--	0.0036	--	70.86	6.12	63.01	76.70									
.2A	RCB-1	160	62	6.10	14	3,000	53,000	7T	0.0039	0.0039	--	--	0.0040	0.0	--	--	0.0040	0.0	3.53	0.49	2.90	4.16									
.2B	RCB-1	84	62	6.10	14	3,000	53,000	5T	0.0039	0.0039	--	--	0.0040	0.0	--	--	0.0040	0.0	9.70	1.23	8.13	11.20									
.3A	RCB-1	220	62	6.10	14	3,000	53,000	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	0.0113	0.0113	4.29	0.56	3.50	5.00									
.3B	RCB-1	114	62	6.10	14	3,000	53,000	7	0.0113	0.0113	--	--	0.0113	0.0113	--	--	0.0113	0.0113	14.00	1.65	11.88	16.11									
.3C	RCB-1	76	62	6.10	14	3,000	53,000	5	0.0113	0.0113	--	--	0.0113	0.0113	--	--	0.0113	0.0113	20.70	2.07	18.05	23.36									
.3D	RCB-1	220	62	6.10	14	3,000	53,000	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	0.0113	0.0113	8.50	0.63	7.44	9.57									
Building 167, Lafayette Towers Building 2 (RCF)																															
.1a	RCFP-1	240	240	10	3,000	53,000	11	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0100	0.0070	--	--	6.31	0.76	5.33	7.29									
.1b	RCFP-1	240	240	10	3,000	53,000	11	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0100	0.0070	--	--	1.63	0.01	1.62	1.64									
.1c	RCFP-1	120	120	10	3,000	53,000	11	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0100	0.0070	--	--	30.03	3.96	25.75	35.91									
.1d	RCFP-1	120	120	10	3,000	53,000	11	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0100	0.0070	--	--	8.77	0.02	8.75	8.79									
.1e	RCFP-1	240	240	10	3,000	53,000	2	2	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0100	0.0070	--	--	9.13	0.96	7.87	10.39									
.1f	RCFP-1	120	120	10	3,000	53,000	1	1	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0100	0.0070	--	--	19.90	2.59	16.57	23.22									
Building 168, State Wildlife Conservation Building (RCF)																															
.1a	RCB-1	192	--	6	3,000	44,000	6	6	0.0094	0.0	--	--	0.0140	0.0025	--	--	0.0094	--	9.95	1.07	8.52	11.32	20.81								
.1b	RCB-1	192	--	6	3,000	44,000	7	7	0.0094	0.0	--	--	0.0140	0.0025	--	--	0.0094	--	20.81	2.34	17.81	23.81	2.34								
.2A	RCB-1	157.5	46	15.5	3,000	44,000	5	5	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0086	--	7.15	0.96	5.92	8.39	11.47								
.2B	RCB-1	78.75	46	15.5	3,000	44,000	7	7	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0086	--	13.59	1.71	11.40	15.79	1.44								
.3A	RCB-1	233.5	46	15.5	3,000	44,000	7	7	0.0131	0.0	--	--	0.0126	0.0049	--	--	0.0086	--	3.36	0.58	2.61	4.10	8.11								
.3B	RCB-1	116.75	46	15.5	3,000	44,000	5	5	0.0131	0.0	--	--	0.0126	0.0049	--	--	0.0086	--	8.05	0.95	7.64	10.46	12.97								
.3C	RCB-1	77.83	46	15.5	3,000	44,000	5	5	0.0131	0.0	--	--	0.0126	0.0049	--	--	0.0086	--	21.49	2.53	18.24	24.73	19.84								
Building 206, Fitzsimmons General Hospital (RCF)																															
.1a	RCB-1	76	266	4	3,000	44,000	6	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	0.0073	--	6.54	0.54	5.05	7.22	18.96								
.1b	RCB-1	76	133	4	3,000	44,000	4	4	0.0073	0.0	--	--	0.0073	0.0073	--	--	0.0073	--	4.79	0.56	4.07	5.82	2.28								
.2A	RCB-1	266	133	12	10	3,000	44,000	4	0.0138	0.0	0.0040	0.0	0.0160	0.0084	--	--	0.0138	--	13.86	1.67	11.72	15.99	13.64								
.2B	RCB-1	133	133	12	10	3,000	44,000	7	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	4.92	0.52	4.25	5.80	0.41								
.3A	RCB-1	255	16	28	3,000	44,000	6	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0074	--	17.09	2.15	14.32	19.85	2.29								
.3B	RCB-1	127.5	16	28	3,000	44,000	5	5	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0074	--	28.50	3.46	24.07	32.93	0.63								
.3C	RCB-1	85	16	28	3,000	44,000	7	7	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0074	--	8.55	0.96	7.31	9.78	1.81								
.3D	RCB-1	255	180	16	28	3,000	44,000	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0074	--	3.20	0.30	2.90	3.46	0.28								
.4A	RCB-2	190	106	4	3,000	44,000	3	3	0.0040	0.0	0.0073	0.0	0.0073	0.0085	--	--	0.0040	--	6.06	0.73	5.13	6.99									
.4B	RCB-2	177	106	4	3,000	44,000	6	6	0.0040	0.0	0.0073	0.0	0.0073	0.0085	--	--	0.0040	--	3.94	0.62	3.15	4.73									
.5A	RCB-2	86.5	12	16	3,000	44,000	7	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	11.99	1.16	10.50	13.47	1.41								
.5B	RCB-2	86.5	12	16	3,000	44,000	7	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	18.45	2.23	15.80	21.81	2.60								
.5C	RCB-2	105	12	20	3,000	44,000	6	6	0.0041	0.0029	--	--	0.0029	0.0041	--	--	0.0041	--	12.17	0.74	11.22	13.12									

Table 2 (continued)

Case	Element Type	L (in.)	L (in.)	L (in.)	b (in.)	h (in.)	f' c (psi)	f dy (psi)	Support Case	Reinforcing Ratios at Cross Sections								Tensile Membrane Steel Ratio		Incipient Collapse Overpressure				Incipient Collapse Overpr (Tens. Membr.)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
										1				2				3				4				Short	Long	Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	Mean (psi)	Std. Dev. (psi)	1% Prob. (psi)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
										p	p'	p	p'	p	p'	p	p'	p	p'	p	p'	p	p'																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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1a	RCB-1	95	306	153	4.5	3,000	53,000	6	0.0043	0.0	0.0043	0.0	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043	0.0021	0.0043

Table 2 (concluded)

Case	Element Type	L _s (in.)	L _b (in.)	b _b (in.)	h _b (in.)	h _c (in.)	f' _c (psi)	f _{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections								Tensile Membrane Steel Ratio		Incipient Collapse Overpressure				Incipient Collapse Overpr (Tens. Membr.)						
										1		2		3		4		Short	Long	Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	Mean (psi)	Std. Dev. (psi)	1% Prob. (psi)				
										p	p'	p	p'	p	p'	p	p'													
Building 245, Portland Hilton Hotel																														
1a	RC3-1	24	112	3	3	3	3,000	56,000	6	0.0029	0.0	0.0029	0.0	0.0005	0.0	0.0029	0.0	14.45	0.08	14.34	14.56									
1b	RC3-1	24	112	3	3	3	3,000	55,000	61	0.0029	0.0	0.0029	0.0	0.0005	0.0	0.0029	0.0	8.49	0.66	7.38	9.59									
2a	RCJ-1	278		5.5	5.5	15	3,000	53,000	77	0.0181	0.0103	0.0181	0.0106	0.0005	0.0	0.0275	0.0106	25.79	3.78	21.71	29.86									
2b	RCJ-1	139		5.5	5.5	15	3,000	53,000	57	0.0181	0.0103	0.0181	0.0106	0.0005	0.0	0.0275	0.0106	32.78	4.12	27.50	38.05									
2c	RCJ-1	98		5.5	5.5	15	3,000	53,000	6	0.0221	0.0110	0.0221	0.0104	0.0005	0.0	0.0221	0.0104	11.26	1.56	9.26	13.27									
2d	RCJ-1	448		30	30	76.8	3,000	53,000	7	0.0221	0.0110	0.0221	0.0104	0.0005	0.0	0.0221	0.0104	35.04	3.75	30.23	39.85									
3a	RCB-1	936		30	30	76.8	3,000	53,000	5	0.0221	0.0110	0.0221	0.0104	0.0005	0.0	0.0221	0.0104	53.65	6.50	45.31	61.98									
3b	RCB-1	448		30	30	76.8	3,000	53,000	5	0.0221	0.0110	0.0221	0.0104	0.0005	0.0	0.0221	0.0104	42.98	8.69	81.86	104.12									
3c	RCB-1	312		30	30	76.8	3,000	53,000	5	0.0221	0.0110	0.0221	0.0104	0.0005	0.0	0.0221	0.0104	92.98	8.69	81.86	104.12									
3d	RCB-1	234		30	30	76.8	3,000	53,000	5	0.0221	0.0110	0.0221	0.0104	0.0005	0.0	0.0221	0.0104	92.98	8.69	81.86	104.12									
														</																

The question arises, however, as to how much of these materials could be mobilized and installed in a crisis build-up period of, say, one year.

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Appendix A

BLAST-RESISTANT DESIGN/ANALYSIS GENERAL APPROACH

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Appendix A

BLAST-RESISTANT DESIGN/ANALYSIS GENERAL APPROACH

Introduction

The purpose of this appendix is to discuss several matters that are common to all blast-resistant design/analysis approaches, whatever the construction material. This and the following engineering appendices are aimed at architects and engineers, but generally those weak in, say, indeterminate structures and dynamic structural analysis.

For collateral reading, there are many references: the overall source-book on nuclear weapons effects [1, using its Contents and Index to locate items of interest],¹ an early paper, tightly written, that is now a classic [2] (an errata was later published: delete the denominator "6" in Eq. 2, p. 49; delete the fraction bar in text line below Eq. 7a, p. 56; and, change the "q" subscript to "e" in line 7, p. 58); the first professional society manual on the subject, still current [3]; two bulky manuals [4,5]; a textbook [6]; and, two publications used in the preparation of this and other appendices herein [7,8].

In this and following appendices, design and analysis may be used with little difference in meaning. In most cases dealt with in upgrading, design is accomplished by seeing what materials one has to work with, then determining each potential member's blast resistance (analysis) when used under a particular set of conditions. Most often a design/analysis procedure will be presented followed by a simplifying graph or table, which is used by entering either with the member parameters to find its blast resistance, or with its desired blast resistance to find the needed member parameters (size, useful stresses, span, etc.).

Newmark has clearly stated a simplified approach to blast-resistant design/analysis [8(Ch.7)] from which the following material is quoted (parenthetic connective words or insertions are by this writer):

"The various factors governing structural design for blast resistance . . . include loading, structural resistance, design considerations as affected by the material used, and the structural properties of these materials. . . . Resistance expressions for different materials and support conditions (are presented in following appendices).

¹ Brackets are used herein to indicate sources in the References list at the end of this appendix.

Loading

The simplified loading (Figure A-1) of the various structural elements . . . are . . . assumed (to be) . . . long duration step pulses² with pressures p_m , which are a function of the location of the elements and their orientation with respect to the blast wave. (In Figure A-1:

t = time, measured from arrival of the blast front (sec).)

Structural Yield Resistance

"For the long duration step pulse loading shown in Figure A-1 the relationship (among) the design parameters is:

$$p_m/q_y = 1 - 1/(2\mu) \quad (1)$$

where:

p_m = peak pressure (psi)

p_y = the (idealized) yield resistance of the structural element (psi)

μ = the ductility factor defining the maximum acceptable response of the structure, i.e., the ratio of the maximum deflection to the yield deflection

"Two important design parameters do not appear in the above equation. These parameters are:

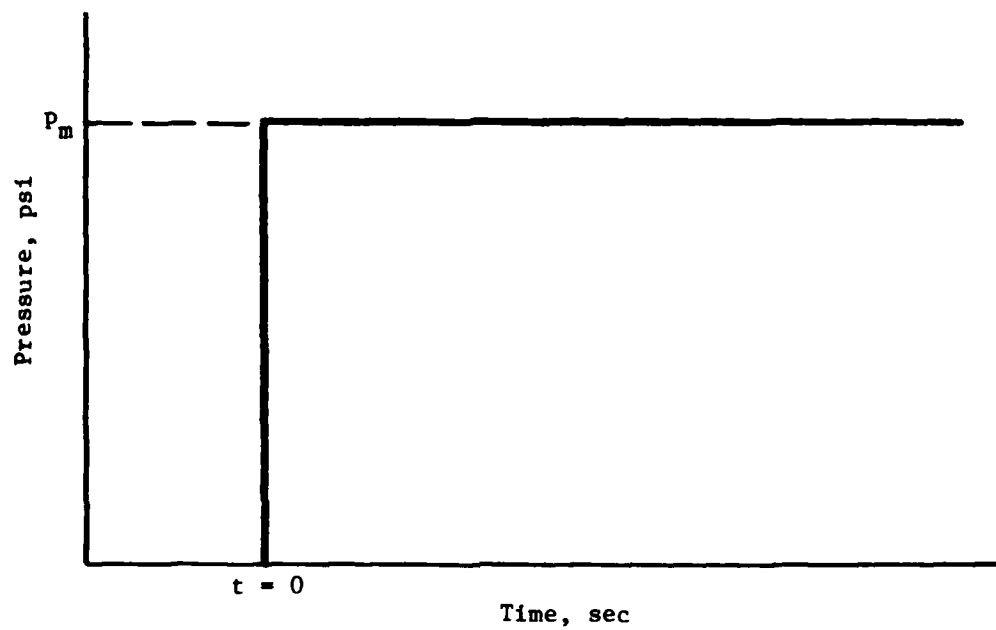
t_d = the duration of the (positive phase of the blast) loading (sec)

T = the (effective natural) period (of vibration) of the structural element (sec)

These parameters do not appear (in Eq. 1) because the ratio t_d/T is considered to be infinite, or effectively greater than about 3. Reference [2 or 3] may be consulted to obtain the relationships replacing the above equation when the ratio t_d/T is small.

"The structural yield resistance q_y and the limit of acceptable structural response μ are determined by considering the resistance-deflection curve for the structural element. The resistance q (is) considered as a static loading distributed spatially in the same manner as the air blast loading;

² A loading whose application is instantaneous (i.e., rise time equals zero) and duration is long.



Source: Reference 8, page 169

Figure A-1 BLAST LOADING

q = structural resistance (psi)

is plotted in Figure A-2, where

q_y = the structural yield resistance (psi)

The deflection is also plotted in Figure A-2, where

x_e = the yield deflection.

The ductility factor

$$\mu = (x_m/x_e)$$

where

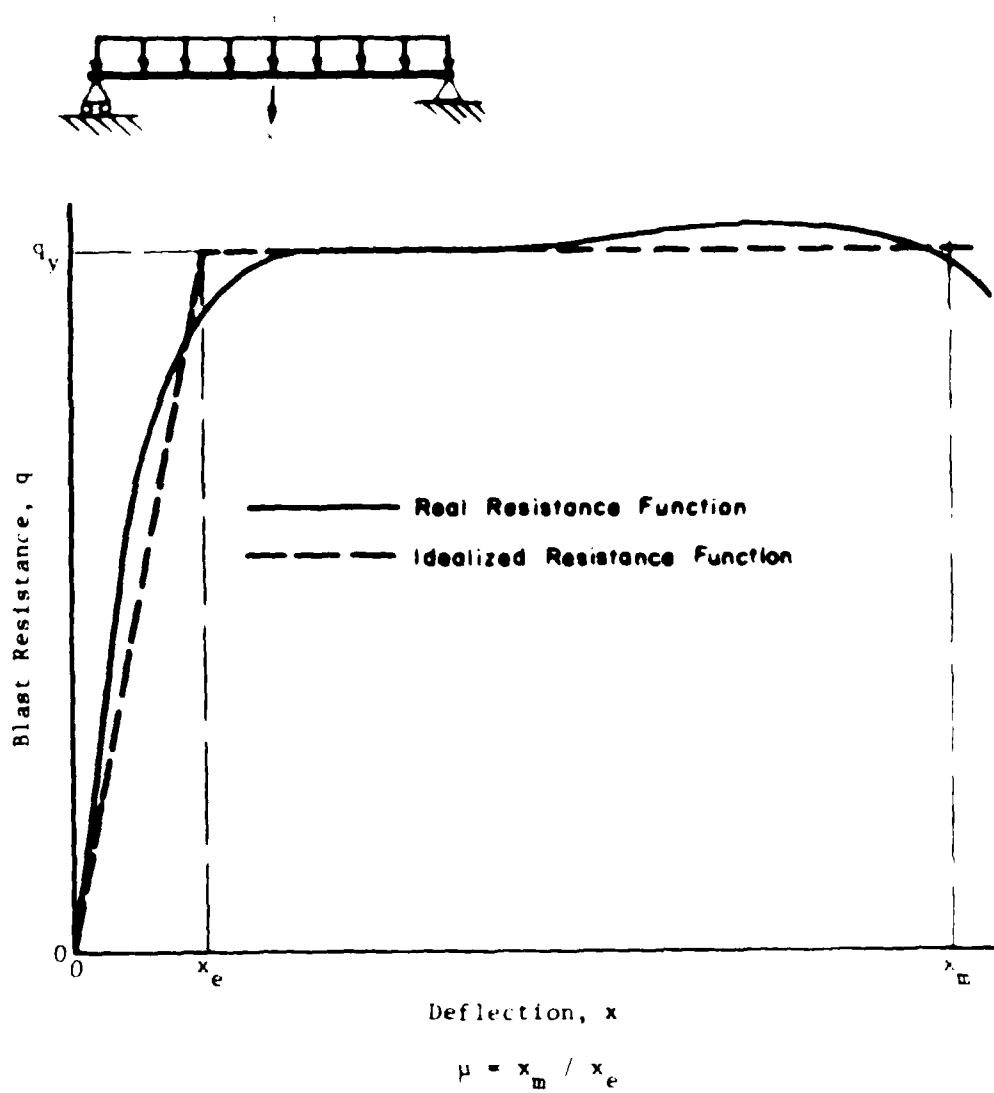
x_m = the maximum acceptable deflection.

In the selection of x_m , both structural integrity and structural function should be considered. The value of x_m should not be greater than the deflection at which the resistance of the structure begins to drop off or fracture occurs. Operational requirements, e.g., avoidance of jamming of a door or its operating mechanism, may set a lower limit on x_m .

"Figure A-2 is a typical resistance-deflection curve with its idealized bi-linear representation as used in design. The idealized resistance function is constructed so that the area under both the real and ideal curves are equal (from zero to yield (x_e) and (from yield to maximum response (x_m)).

"The design involves establishing the required yield resistance for the structural element and then providing this resistance in the structural element (or analysis involves knowing the latter and finding q_y). The peak pressure p_m is evaluated (by considering expected peak overpressure, peak expected pressure and reflection factor, peak room filling pressure [7(p.8-112)], etc., and) the orientation of the structural element with respect to the blast wave. If allowable maximum deflection is set by structural considerations, the ductility factor μ is a function of the type of structural element and materials used. If operational requirements govern, the ductility factor μ will be selected to limit the maximum deflection of the structure to the permissible magnitude.

"In the (following appendices), expressions are presented for yield resistance q_y , yield deflection x_e , and the period T for (structural) elements of various materials and structural types. The maximum recommended value of the ductility factor based on consideration of structural integrity is presented for each material and each structural type considered. These μ values must then be checked to insure that the resulting deformations are operationally acceptable.



Source: Adapted from Reference 8, page 170

Figure A-2 BLAST RESISTANCE
(Typical Real and Idealized Load-Deflection Curves)

Structural Properties of Materials

The following appendices are presented the strength properties of materials considered suitable for (basement protective) structures. In many instances the variety of commercially available materials is too great for complete description of pertinent properties herein. Therefore, expressions for design stresses for protective design (may be) given in lieu of extensive strength tabulations.

Protective structures are designed on the basis of a predicted failure load, failure being defined either by the limit of acceptable deformation or by the (incipient) collapse of the structural element. Normally a ductile, large deformation, failure mode is desired in design since large amounts of energy are absorbed in inelastic deformation.

"The design stresses given herein correspond generally to the probable yield stress of the material under the blast loading conditions. These design stresses represent probable yield stresses for the material, not guaranteed minimum values, since it is desired to estimate the actual yield load for the structural element rather than a lower limit to the yield load."

Design versus Weapons Analyst Overpressure

The most emphasis that the peak blast design overpressure p_m is not a weapon analyst overpressure. The former is a value that should give, say, a 99% probability of not reaching incipient collapse or some other failure definition, whereas the latter should be a median (50% probability) value (Fig. 6-21).

Factor of Safety

Blast resistance design analysis (for, say, 99% probability of not reaching incipient collapse or some other failure definition) is based on a peak blast pressure value that has all factors of safety wrung out of it. In structural design to resist dynamic loads, various structural elements are likely to have different dynamic response characteristics (e.g., effective natural period of elastic vibration T). Thus, if all structural elements are to be designed to the same dynamic strength level, the loading must be in terms of a dynamic load(s). Therefore, if a factor of safety common to all structural elements is to be set, it must be in terms of the dynamic loading, that is, in this case air blast peak pressure. Because protective structures against nuclear air blast are very expensive, the dynamic factor of safety is usually taken as

1) $F = 1.5$ the peak air blast pressure (1) considered most likely to occur, or
2) the one assumed for design use as the pressure that one can afford to protect against. If the latter is to be subject to a further factor of safety, for example two, then this factor is simply used

as a multiplier on the peak pressure arrived at by either of the two methods just described.

NOTATION

- p_m = peak pressure (psi)
- p_y = the (idealized) yield resistance of the structural element (psi)
- q = structural resistance (psi)
- q_y = the structural yield resistance (psi)
- T = the (effective natural) period (of vibration) of the structural element (sec)
- t = time, measured from arrival of the blast front (sec)
- t_d = the duration of the (positive phase of the blast) loading (sec)
- x_e = the yield deflection
- x_m = the maximum acceptable deflection
- μ = (x_m/x_e)
- μ = the ductility factor defining the maximum acceptable response of the structure, i.e., the ratio of the maximum deflection to the yield deflection

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1. Glasstone, S., editor, The Effects of Nuclear Weapons, U. S. Department of Defense and Atomic Energy Commission, February 1964 reprint (with changes) of 1962 edition, Superintendent of Documents, Washington, D.C.; a 1977 edition is now available.
2. N. M. Newmark, "An Engineering Approach to Blast-Resistant Design," Transactions, Volume 121, p. 45-64, American Society of Civil Engineers, 1956.
3. Anderson, F. E., Jr., R. J. Hansen, H. L. Murphy, N. M. Newmark, and M. P. White, Design of Structures to Resist Nuclear Weapons Effects, ASCE Manual of Engineering Practice No. 42, 1961, and Supplement 1963 (combined into a "1964 edition").
4. Newmark, N. M., and J. D. Hiltiwanger, Principles and Practices for Design of Hardened Structures, Air Force Design Manual, published by Air Force Special Weapons Center, Kirtland AFB, New Mexico, Report No. SWC-TDR 62-138, December 1962. (AD-295 408)³
5. Manuals-Corps of Engineers, U. S. Army, Engineering and Design, Design of Structures to Resist the Effects of Atomic Weapons, EM 1110-345-413 (July 1959), -414 to -416 (March 1957), -417 (January 1958), and -418 to -421 (January 1960), U. S. Government Printing Office, Washington, D.C. (AD-419 for -416).⁴
6. Biggs, J. M., Introduction to Structural Dynamics, 1964 (McGraw-Hill).
7. Murphy, H. L., J. R. Rempel, and J. E. Beck, SLANTING IN NEW BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: A Consolidated Printing of Four Technical Reports, Stanford Research Institute⁵ Technical Reports, 3 volumes, for U. S. Defense Civil Preparedness Agency,⁶ October 1975. (AD-A023 237) Omitted, of course, was any material in a later report that replaced material published in an earlier report.

³ Those references for which "AD-" numbers are shown are understood to be available for purchase from NTIS, Springfield, Virginia, 22151.

⁴ Reprinted, without change, and redesignated; e.g., EM 1110-345-413 through EM 1110-345-421 became Headquarters, Department of the Army, TM 5-856-1 through TM 5-856-9.

⁵ Now SRI International

⁶ Now Federal Emergency Management Agency

8. Newmark, N. M., Design of Openings for Buried Shelters, Report 2-67, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1963; Chapter 7.

Appendix A1

PLYWOOD STRESSED-SKIN PANELS (TWO-SIDED ONLY) AS CLOSURES -
DESIGN AND FABRICATION

Extracts (with minor revisions) from the main text and Appendix A1 of

Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects:
Predesigned Expedient Options, Stanford Research Institute* Technical
Report, for U.S. Defense Civil Preparedness Agency, # October 1977.
(AD-A054 409)

* Now SRI International

Now Federal Emergency Management Agency

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Design

A design procedure for plywood stressed-skin panels was developed because plywood and suitable wood members for the necessary stringers are in abundant supply in local lumberyards, and because efficient use of such materials can assist greatly in meeting the existing basement upgrading need for many closures against air blast entry into the basement.

Existing design procedures were studied, used as a basis for developing the procedure that follows, but had to be carefully reviewed/modified/rederived to make them both dimensionally consistent (and thus more readily convertible to metric units, a contract requirement) and usable for panel widths other than 48 in. (a limitation built into the present procedure).^{1,2*}

The developed design procedure is limited to plywood stressed-skin panels with both top and bottom skins, both of which are used with the grain of the outer plies parallel to the stringers. Adequate shear transfer between plywood (flanges) and stringers (webs) is assumed, based on using pressure-glued or nail-glued joining techniques. The normal-use allowable stresses in the procedure are intended for application to panels at least 2 ft wide (measured perpendicular to stringers); narrower panels are subject to reductions in allowable stresses.^{2(p.19)}

Design Procedure. The design procedure (steps) follows:

1. Assume a trial section and clear span (in direction of stringers), and that panel is fully and uniformly loaded. See Figure 1A.
2. Get values for b ("b distance"), both for top b_t and bottom b_b skins (Figure 1B). If clear distance between stringers, Figure 1A, exceeds $2b$ for both skins, this design procedure is inapplicable.^{1(p.5)}
3. Calculate N.A. (neutral axis location) for deflection. Use bottom of panel as reference line for moment arms y applied to areas A_i/E ,

* Superscript numbers are related to the references list at the end of this appendix. Reference 2 must be held by the user, particularly for its Tables, pp. 9, 14-17 and 26; holding Reference 1 is unnecessary but may be desirable. (Reference of this report's main text is later reference and not significantly changed from Reference 2 herein.)

Top Skin - 5/8" UNDERLAYMENT Group 1 INT-APA
(For this thickness and stringer spacing, a 5 ply 5-layer panel should be used for resistance to concentrated floor loads.)

$$A_{||} = 2.728 \text{ in.}^2/\text{ft}$$

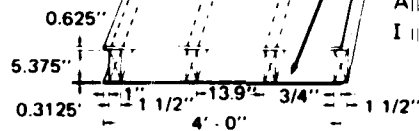
$$I_{||} = 0.141 \text{ in.}^4/\text{ft}$$

$$I_{\perp} = 0.023 \text{ in.}^4/\text{ft}$$

2 x 6 Douglas fir-
Larch No 1
stringers

$$A^* = 8.06 \text{ in.}^2$$

$$I^* = 19.4 \text{ in.}^4$$



Bottom Skin - 5/16"
C-D 20/0 INT-APA

$$A_{||} = 1.914 \text{ in.}^2/\text{ft}$$

$$I_{||} = 0.025 \text{ in.}^4/\text{ft}$$

$$\text{Clear distance between stringers} = \frac{48 - 3 \times 1.5 - 1 - 0.75}{3} = 13.9''$$

$$\text{Total splice plate width} = 3(13.9 - 0.5) = 40.2''$$

*Includes a 1/8" reduction in depth to allow for resurfacing.

A.

Basic Spacing, b , For Various Plywood Thicknesses

(Face grain parallel to stringers*)

Plywood	Basic Spacing b (inches)				
	3	4	5	8	7
		(3 layer)	(5 layer)	(5 layer)	(7 layer)
1/4" Sanded	10				
5/16" Unsanded	12				
3/8" Unsanded	16				
3/8" Sanded	19				
1/2" Unsanded sanded touch sanded	22	22	23		
5/8" Unsanded sanded	27	35	33		
5/8" 19/32" Touch sanded		27	32		
3/4" Unsanded sanded touch sanded		36	38	38	
23/32" Touch sanded		35	34	37	
7/8" Unsanded			48		39
7/8" Sanded					51
1" Unsanded sanded					53
2 1/4"					56

*Where plywood face grain is across stringers, write APA for appropriate "b" distances.

B.

FIGURE A1-1 PLYWOOD STRESSED-SKIN PANEL (Example Trial Section)^{1(p.4)}
AND TABLE ON STRINGER SPACING^{1(p.5)}

counting only plies parallel to stringers (for $A_{//}$) and increasing E values (to correct from effective E to true E in bending), by 10% for skins^{2(p.17),1(p.7)} and 3% for stringers.^{3,1(p.7)} $A_{//}$ values are available from tables^{2(p.16: note units, col. 4: in.²/ft)}. A calculation example is shown in Figure 2A.

4. Calculate panel (EI_g) using N.A. of Step 3. This stiffness factor is for moment deflection only (i.e., excludes shear deflection). Obtain I_o values for skins.^{2(p.16,col.5)} Calculate I_o values for (combined) stringers ($bd^3/12$), including a portion of any stringer that is partially outside the plywood skins, as one stringer is in the calculation example shown in Figure 2B. Same E values and percentage increases are used as in Step 3.

5. Calculate allowable load (TL) - deflection:^{*†}

$$p_d = 1 / [C \ell \ell' (\frac{5}{384} \frac{\ell^2}{EI_g} + \frac{0.15}{AG})] + DL$$

where: p_d = allowable TL - panel deflection (psi)

C = factor for max. allowable deflection*
(often 360 floors, 240 roofs, LL only)

(EI_g) from Step 4 (lb-in.²)

A = (actual) total X-sec. area of all stringers (in.²)

G = modulus of rigidity of stringers (psi)
(taken as 0.06 E plus 3%)

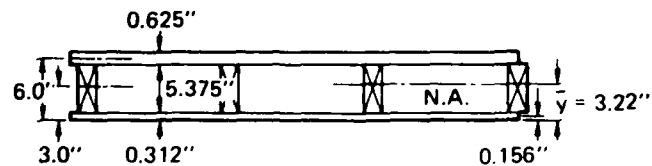
ℓ = clear span of panel (in direction of stringers)(in.)

ℓ' = width of panel (skins only)(perpendicular to ℓ)(in.)

6. Calculate allowable load (TL) - top skin deflection (cross-panel). Usually only the top skin deflection need be checked, but unusual assumed sections may require top skin moment and shear investigations.^{1(p.9)}
Check strip 1 in. wide for allowable total load (TL) and deflection

* If C is based on TL, then p_d will be directly in TL units (psi), without adding the DL term in the equation.^{1(p.9)}

† While (EI_g) excludes shear deflection, the formula for p_d includes it.

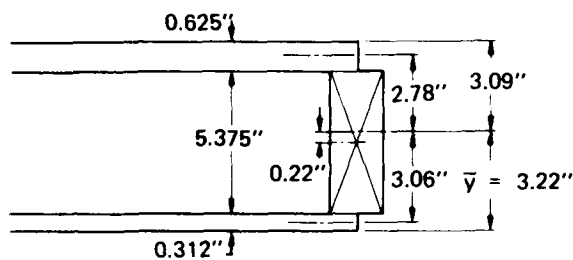


Values of $A_{||}$ of plywood from PDS Table 1.

Item	E	$A_{ }$	$A_{ }E$	y	$A_{ }E y$
Top Skin	$1,800,000 \times 1.1 = 1,980,000$	$4 \times 2.778 = 10.9$	21,600,000	6.000	129,600,000
Stringers	$1,800,000 \times 1.03 = 1,850,000$	$4 \times 8.06 = 32.2$	59,600,000	3.000	178,800,000
Bottom Skin	$1,800,000 \times 1.1 = 1,980,000$	$4 \times 1.914 = 7.66$	15,200,000	0.156	2,370,000
Total		50.8	96,400,000		310,770,000

$$\bar{y} = \frac{\sum A_{||} E y}{\sum A_{||} E} = \frac{310,770,000}{96,400,000} = 3.22''$$

A.



Values of I_o of plywood from PDS Table 1.

Item	E	I_o	$A_{ }$	d	d^2	$A_{ } d^2$	$I_o + A_{ } d^2$	$E(I_o + A_{ } d^2)$
Top Skin	1,980,000	0.564	10.9	2.78	7.73	84.3	84.9	168,000,000
Stringers	1,850,000	77.6	32.2	0.22	0.48	1.55	79.2	147,000,000
Bottom Skin	1,980,000	.100	7.66	3.06	9.36	71.7	71.8	142,000,000
Total						$I_o = 235.9$		457,000,000

$$EI_g = 457,000,000 \text{ lb-in.}^2 \text{ per 4-ft width}$$

B.

FIGURE A1-2 NEUTRAL AXIS FOR DEFLECTION AND (EI_g)
(Calculations Examples)^{1(p.8)}

(FF or fixed ends beam assumption), based on cross-panel top skin deflection behavior, as follows:

$$p_t = 384 EI / [C(l'')^3] + DL$$

where: p_t = allowable TL - top skin deflection (psi)

C = factor for max. allowable deflection*
(often 360 floors, 240 roofs, LL only)

E is for top skin^{2(p.17, no 10% added)} (psi)

I is for stress applied perpendicular to stringers and face grain^{2(p.16, col.9)}; table's in.⁴/ft values must be changed to in.⁴/in.

l'' = clear distance between stringers (Step 2 and Fig 1A) (should be uniform; if not, use longest value)(in.)

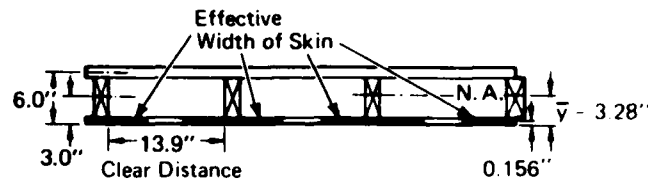
Mid-span cross-panel deflection, of course, then equals l''/C .

7. Calculate N.A. for bending. Effective width of skins (as "flanges" to each stringer) is $b/2$ on each side of stringer, plus the width of the stringer. Get b from Step 2. Make sketch showing effective widths with each stringer, of both top and bottom skins. Calculate N.A. location, using bottom of panel as reference line for moment arms y ; see example, Figure 3A; E values are used plus percentages, as in Step 3. Recall that $A_{//}$ tabular values are in in.²/ft width and must be corrected for effective width of skins (versus total width used in Step 3), as must I_o skin values; moment arms for skins and stringers are the same as in Step 3. NOTE: Non-Stress-Graded stringers are omitted in the calculations of this Step (i.e., valued at zero), even though in Steps 3 and 4 they would be included.

8. Calculate (EI_n) for bending. Use all data from Step 7, plus using I_o for each skin as flanges (from Step 4, but correcting I_o values from full panel width to "effective widths" of Step 7), again correcting for tabular units of in.⁴/ft width, as necessary; use I_o values for stringers, as in Step 4 (omit Non-Stress-Graded stringers, though, as in Step 7). See example calculations, Figure 3B.

* If C is based on TL, then p_t will be directly in TL units (psi) without adding the DL term in the equation.^{1(p.9)}

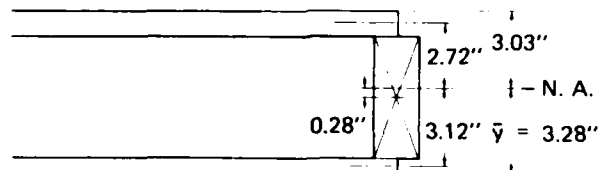
Effective width of top skin = 48"
 Effective width of bottom skin = $48'' - 3(13.9 - 12) = 42.3''$



Item	E	A	A E	y	A Ey
Top Skin	1,980,000	10.9	21,600,000	6.00	129,600,000
Stringers	1,850,000	32.2	59,600,000	3.00	178,800,000
Bottom Skin	1,980,000	$\frac{42.3}{12} \times 1.914 = 6.75$	13,400,000	0.156	2,090,000
			94,600,000		310,490,000

$$\bar{y} = \frac{\sum A_{||} E y}{\sum A_{||} E} = \frac{310,490,000}{94,600,000} = 3.28''$$

A.



Item	E	I _o	A	d	d ²	A d ²	I _o + A d ²	E(I _o + A d ²)
Top Skin	1,980,000	0.564	10.9	2.72	7.40	80.7	81.3	161,000,000
Stringers	1,850,000	77.6	32.2	0.28	0.078	2.51	80.1	148,000,000
Bottom Skin	1,980,000	0.088	6.75	3.12	9.73	65.7	65.8	130,000,000
							I _n = 227.2	439,000,000

$$EI_n = 439,000,000 \text{ lb-in.}^2 \text{ per 4-ft width}$$

B.

FIGURE A1-3 NEUTRAL AXIS FOR BENDING MOMENT AND (EI_n)
 (Calculations Examples)¹(p.10,11)

9. Determine top skin allowable compressive stress. Obtain F_c .^{2(p.17)} correct F_c in same manner as F_t (for F_c , using b_c from Step 6) to F_{c1} . Step 6 is as follows: for ratio ≤ 0.9 , use 100%; for ratio > 1.0 up to 1.1, see parenthetical comment in Step 2), use 67%; and for ratios between 1.1 and 1.2, vary percentage correction linearly between 100% and 67%.^{1(p.11)*}

10. Determine bottom skin allowable tensile stress. Obtain F_t .^{2(p.17)} correct F_t in same manner as Step 9 (for F_t , using b_t from Step 2).^{1(p.11)}

11. Calculate allowable load (H) - bending:

$$p_b = (8 F_c - (c(\ell' \ell')^2) / (EI_n)) / E$$

where: p_b = allowable load (H) - bending (psi)

F = F_c or F_t from Steps 9 and 10, as appropriate (psi)

(EI_n) from Step 8 (lb-in.²)

E for skin under check, top or bottom (as in Step 3, including percentage increase)(psi)

c = distance from N.A. for bending (Step 7) to extreme fibre (of skin under check, top or bottom)(in.)

ℓ and ℓ' are same as in Step 5 (in.)

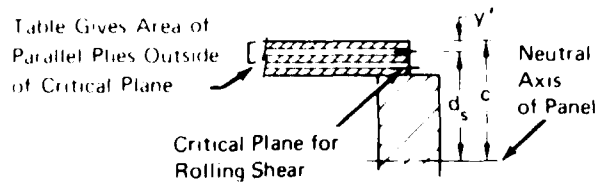
Check p_b for both top (p_{bt}) and bottom (p_{bb}) skins, then use smaller value as the applicable p_b .

12. Calculate allowable load (TL) - rolling shear:

It is generally sufficient to check rolling shear only in the thicker skin (it usually has the larger Q_s of the two skins, which leads to a smaller allowable load).^{1(p.13)} The skin's critical plane for checking (in panels with face plies parallel to stringers, a fundamental limitation in the overall procedure herein) is along the glued plane on the inner side of the inside face ply of the panel; see Figure 4A.

Find area A (in.²) for parallel-grain plies outside the critical plane (note that tabular values are for 48-in. wide panels, so must be corrected proportionately for other panel widths ℓ'), Figure 4B, 2nd or

* Reference 1, p.11, figure erroneously shows 67.5% instead of correct value of 66.7% (as shown in text example and in other sources).



A.

A and y' for Computing Q_s *

Plywood Thickness (in.)	STRUCTURAL I Grades, or any Group 4 Panel				All Other Panels			
	Face Grain to Stringers		Face Grain to Stringers		Face Grain to Stringers		Face Grain to Stringers	
	Area (in ²)	y' (in.)	Area (in ²)	y' (in.)	Area (in ²)	y' (in.)	Area (in ²)	y' (in.)
Unsanded Panels								
5/16	4.75	0.0495	4.75	0.149	3.83	0.0479	2.64	0.149
3/8	4.45	0.0464	5.75	0.180	3.13	0.0461	3.19	0.180
1/2	5.81	0.0606	7.75	0.242	6.25	0.131	4.31	0.242
5/8	9.24	0.176	9.75	0.305	7.18	0.140	5.42	0.305
3/4	7.34	0.0765	11.8	0.367	8.16	0.208	6.53	0.367
Sanded Panels								
1/4	3.36	0.0350	4.91	0.121	3.36	0.0350	2.73	0.121
3/8	3.36	0.0350	8.51	0.184	3.36	0.0350	4.73	0.184
1/2	3.89	0.0406	9.23	0.246	3.89	0.0406	5.13	0.246
5/8	4.56	0.0475	11.7	0.309	4.56	0.0475	6.51	0.309
3/4	12.0	0.277	15.1	0.371	8.18	0.233	8.42	0.371
Touch Sanded Panels								
1/2	4.56	0.0475	8.35	0.224	4.56	0.0475	4.64	0.224
19/32	7.93	0.174	11.6	0.276	5.90	0.139	6.44	0.276
5/8	8.21	0.185	12.3	0.288	6.06	0.148	6.86	0.288
23/32	6.06	0.0632	14.5	0.344	8.14	0.213	8.06	0.344
3/4	6.06	0.0632	15.3	0.356	8.37	0.225	8.50	0.356
2 1/4					12.7	0.359	16.5	0.547

* Area based on 48" wide panel. For other widths, use a proportionate area.

B.

FIGURE A1-4 ROLLING SHEAR CRITICAL PLANE AND Q_s ^{1(p.14)}

6th column. Calculate distance d_s (in.) from N.A. (for deflection, Step 3) to centroid of A, using moment arm $d_s = c - y'$ (all in. units), where c is distance from N.A. to extreme fibre and y' can be taken from table, Figure 4B. Then calculate the statical moment Q_s (in.³):

$$Q_s = A d_s$$

Calculate $\Sigma F_s t$ (psi-in., or lb/in.), the sum of the glueline widths over each stringer, each multiplied by its applicable allowable rolling shear stress F_s (Reference 2, page 17) but with a 50% reduction applied to outer stringer(s) whose clear distance to a panel edge is less than half the clear distance between stringers.^{1(p.15)}

Calculate allowable load (TL) - rolling shear (p_s , psi):

$$p_s = (2(\Sigma F_s t) / (\ell \ell' Q_s)) ((EI_g) / E)$$

where: $(\Sigma F_s t)$ (lb./in.), ℓ and ℓ' (in.), and Q_s (in.³) are defined above

(EI_g) from Step 4 (lb-in.²)

E for skin under check, usually thicker one (tabular value plus 10% if taken from Ref. 2, p. 17)

13. Calculate allowable load (TL) - horizontal shear:

Calculate statical moment Q_v of all parallel-grain plies and stringers in full panel width ℓ' , working either above or below* the N.A. for deflection (Step 3 and Figure 2 can provide numerical data for these $Q_v = A d$ calculations, as an example of course):

$$Q_v = Q_{\text{stringers}} + Q_{\text{skin}} [E_{\text{skin}} / E_{\text{stringers}}]$$

where: Q_v is defined above (in.³)

$Q_{\text{stringers}}$ = x-sec. area of all stringer portions either above or below N.A. (depending on chosen approach)* times its centroidal distance from deflection N.A. (as moment arm) (in.³)

Q_{skin} = $A_{//}$ for chosen skin $\times$ moment arm (in.³)^{2(p.16, col. 4 for $A_{//}$)}
 E 's as before (Step 3, including percentage increases)(psi)

* Calculations "below" are easier, if deflection N.A. calculations (Step 3) were made as stated, i.e., using bottom surface of panel as reference plane.

Calculate:

$$p_v = (2 F_v t / (\ell \ell' Q_v)) ((EI_g) / E_{st})$$

where: p_v = allowable load (TL) - horizontal shear (psi)

F_v = allowable stress in stringer horizontal shear (psi)³(Table 1)

t = sum of stringer widths (including side projecting portions, Figures 1A and 2)(in.)

(EI_g) from Step 4 (lb-in.²)

E_{st} for stringers, as in Step 3 including percentage increase (psi)

ℓ , ℓ' , and Q_v as above (in., in., and in.³)

14. Calculate required end bearing length:

The preceding steps that have led to allowable load (TL) under various criteria have used ℓ = clear span (in.)(Steps 5,6,11,12 and 13), but for end bearing, the full length of the panel will be greater than ℓ , sufficiently to provide for the allowable load (TL) in end bearing. Further, properly installed headers will have to be capable of spreading the end bearing load across the full panel width of the (thin) bottom skin; thus continuous headers crossing (nail-glued or pressure-glued) the stringer ends, and within the cover of both top and bottom skins, are recommended (see Reference 1, page facing page 1, top sketch, far end, for example).

The following approach to calculating ℓ_e (required plywood end bearing length at each end) considers adoption of the continuous-headers recommendation just above, but may be also used, perhaps with less confidence in ultimate strength behavior, for blocking-type headers (see same Reference 1 sketch, near end, for an example).

Let: ℓ_e = required plywood end bearing length at each end of panel (in.)

ℓ = clear span of panel, as before (in.)

ℓ' = full panel width (skins only)(note: entire panel area, including end bearing lengths, are assumed to be under a uniform loading)(in.)

p_m = smallest of the calculated allowable loads (TL), from Steps 5, 6, 11, 12 and 13 (psi)

$F_{c \perp}$ = allowable bearing stress on plywood face, for load perpendicular to plane of outer ply actually in bearing (psi)²(p.17)

Then: applied load must be less than or equal to resisting capacity:

$$p_m l_e \leq 2 l_e F_{c\perp}$$

or l_e (min. at each end of panel) $= p_m / (2 F_{c\perp})$

It is recommended that l_e be at least 1.5 in. (38 mm).

The bearing length of each stringer end (at least 1.5 or 2 inches) (38 or 51 mm) should be sufficient to handle the unit blast load on the plywood panel multiplied by the maximum c-c spacing of stringers and divided by the stringer width, all in accordance with Appendix B (especially Figure 6-12, which may be extended as needed based on last "bullet" paragraphs on page 6-111). See also Figures 9 later herein.

15. Glued plywood end joints (across face grain):^{2(p.25,Sec.5.6)}

15A. Scarf joints: Sketches of end-of-grain joints are available.^{4(p.9-11)} Scarf joints are made by bevelling across the plywood end edges (i.e., perpendicular to stringers and face plies of top and bottom skins), then joining the bevelled ends with an appropriate adhesive.

For the tension skin: 1 in 8 or flatter bevels transmit 100% of full allowable stress; 1 in 5 transmit 75%; use linear proportioning between these two bevels; and steeper than 1 in 5 are not to be used.^{2(p.25)}

For compression skin: 1 in 5 or flatter bevels transmit 100% of allowable stress; steeper than 1 in 5 are not to be used.^{2(p.26)}

(Note: Finger joints are too complicated to form and otherwise unsuitable for further consideration herein.)

15B. Splice-plate design (butt joints): While scarf joints are the recommended technique, this design section is presented for use if needed.^{1(p.12,Sec.2.5.6)}

For a splice-plate illustration, see Figure 1^(2,p.4) or top sketch of page facing page 1 of reference 1.

Splice-plates are to: be 1/4 in. clear of stringers at both plate ends; have skin face grain perpendicular to splice; be of grade and species group equal to the plywood spliced; and be no thinner than the skin being spliced. Tension skins with splice-plates are capable of transmitting 100% of maximum allowable stress.^{2(p.26,table)} If the splice-plate is shorter than required for use of an allowable stress in the referenced table, the allowable stress is to be reduced proportionately.

Calculate splice plate allowable load (TL) - tension:

$$p_p = (8 F / (c \ell' \ell^2)) ((EI_g) / E)$$

where: p_p = allowable load (TL) on tension splice at point of max. moment (psi)

F = allowable splice-plate stress $\times$ proportion of panel width actually spliced^{2(p.26,table)}

c = distance from deflection neutral axis to extreme bottom (tension) fibre (in.)

c and (EI_g) are as in Figure 2A ($\bar{y}$) and Step 4, respectively (in.⁸ and lb-in.²)

E is for tension skin, as used in Step 3 (with the percentage increase)(psi)

ℓ and ℓ' are as before (in.)

Splice plate allowable load (TL) - compression: These plates can be approved by inspection, for 100% transmittal of allowable stress, subject to cited references.^{2(Sec.5.6.1.2 and 5.6.2.2)}

Design Stresses - Blast Protection Use versus Normal Use. The design procedure detailed above is that for normal, day-to-day uses, for which allowable stresses are prescribed.^{2(p.17),3} Such allowable stresses are totally inappropriate for one-time blast loadings, with their extremely short (essentially zero) rise-times and short durations (1 or 2 seconds in our range of interest, even for megaton weapons), inappropriate in that they result in seriously underestimating the ultimate strength of structural members under blast loadings. The reader is referred to Appendix B1 herein, especially the introductory section and the "Design Procedure" section; within the latter, specific attention is

invited to its introductory section and design steps 1 through 4. Such referenced reading covers the very basic structural dynamics, bilinear blast resistance, ductility ratio μ , etc., as well as the increased stresses used in blast-resistant design: for wood beams, the increases are four times for F_b and F_v (extreme fibre stress in bending and horizontal shear stress, respectively) and no increase in $F_{c\perp}$ (compression stress perpendicular to grain, or bearing stress). Authorities are cited.

An examination of literature helpfully furnished by the U.S. Forest Products Laboratory, Madison, Wisconsin, indicated the following: Tests on plywood stressed-skin panels (PSSPs) to destruction were few in the literature furnished, being restricted to tests on PSSPs with narrow, plywood stringers where all (predictably) failed along the stringer glue-lines; the allowable stress increase of 100% for impact loads^{2(Sec.3.3.1.1)} seems to be well supported by a test report⁵ in terms of both short duration loads and fast rate of loading, for both wood and wood-based materials (including plywood).

If one considers that "allowable" stresses in most cases (and materials types) are based on a factor of safety of about two, we can then arrive at a factor of four (including the 100% for impact) for ultimate strength under short, rapidly applied loads - a factor of four for certain stresses, at least. These stresses might include, for PSSPs:^{2(p.17)} F_b , F_t , F_c , F_v and F_s , but not $F_{c\perp}$.

Pending receipt of better information based on sorely needed tests, these dynamic stress increases were tentatively adopted for use herein; test data found were for static loadings, or for loads within severe limits on deflection, or for loadings far short of failure/collapse (as used in typical air blast loadings and design technology).^{5,6}

For a value of the ductility ratio μ , however (see App. B1, Design Step 3), a value of two was similarly and tentatively adopted for PSSPs, with a value of three tentatively continued for wood beams,* again hoping to obtain appropriate test information in the early future. For a

* See Appendix B1 following.

dynamic load simplified to a step pulse (zero rise-time to a constant loading of infinite duration), the relationship is

$$P_{dm} = P_m (1 - 1/(2\mu)) = P_m (3/4), \text{ for } \mu = 2$$

Typical Designs of PSSPs. In order to handle the many sets of design computations required in producing a reasonably adequate catalog of pre-designs, a computer program was prepared (in Dartmouth BASIC), following the above 15-step design procedure (Step 15 on design of plywood end joints was not included in the design output, although it is included in the computer program). A listing of the computer program is used, but as revised for use there, is shown in Appendix A2.

The pre-designs* covered clear span ranges from 24" to 96" for lighter panels and from 24" to 144" for heavier. Stringers included 2x4s, 2x6s and 2x8s of both relatively low and high strengths, thereby covering a considerable range of lumber species among those readily available in local lumberyards. Several plywood types/species/grades were examined, with complete pre-designs using two types/grades throughout; this was coupled with use of face ply species groups #1 and #3, except that #3 was not used for the 1-1/8" plywood because of unavailability.

The pre-designs of Table 1 are limited to two plywoods: Underlayment Interior (APA) in face ply group species #1 and #3, for 1/2", 5/8", and 3/4" thicknesses; and 2.4.1 Sturd-I-Floor Interior (APA), which is only made in #1, for the 1-1/8" thickness. These plywoods have high availability in local lumberyards in the indicated thicknesses.

Also having similar availability are three other plywood grades: Underlayment Exterior (APA), C-D Interior (APA) and C-C Exterior (APA). Pre-designs were prepared using these three plywoods in sufficient number to show that Table 1 may also be used for them with insignificant errors (all on the conservative side).

* All included panel dead load (DL), which was less than 0.1 psi in all cases, thus P_{dm} values are appropriate for laterally loaded panels used horizontally or vertically.

Table A1-1A PSSP DESIGNS FOR LOWER STRENGTH STRINGERS ($F_v = 280$ psi)*
(Beams)

Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQ'D. BEAR. EACH END		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																				
Nom. Th. in.	Face Ply Grp	Nom. Th. in.	Face Ply Grp	Nom. Ply Size in.	Face Ply Grp	Ply. Skin in.	Str- ngs in.	Clear Span, ft																				
								2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
1/2	#1	1/2	#1	2X4	4	1.5	4.5	7	6	5																		
					5	1.5	4.0	9	7	6	5																	
					7	1.5	3.5	11	9	8	6	6	5	5														
					9	1.5	3.5	14	11	9	8	7	6	6	5	5												
1/2	#3	1/2	#3	2X4	4	1.5	4.5	7	5																			
					5	1.5	4.0	8	6	5	5																	
					7	1.5	3.5	10	8	7	6	5	5															
					9	1.5	3.0	12	10	8	7	6	6	5	5													
5/8	#1	1/2	#1	2X4	4	1.5	5.0	7	6	5																		
					5	1.5	4.5	9	7	6	5																	
					7	1.5	4.0	12	9	8	7	6	5	5														
					9	1.5	3.5	14	11	9	8	7	6	6	5	5												
5/8	#3	1/2	#3	2X4	4	1.5	4.5	7	5	5																		
					5	1.5	4.0	8	6	5	5																	
					7	1.5	3.5	11	8	7	6	5	5															
					9	1.5	3.0	13	10	8	7	6	6	5	5													
3/4	#1	1/2	#1	2X4	4	1.5	5.0	8	6	5																		
					5	1.5	4.5	9	7	6	5	5																
					7	1.5	4.0	12	10	8	7	6	5	5														
					9	1.5	3.5	15	12	10	8	7	6	6	5	5												
3/4	#3	1/2	#3	2X4	4	1.5	4.5	7	6	5																		
					5	1.5	4.0	8	7	6	5																	
					7	1.5	3.5	11	9	7	6	6	5															
					9	1.5	3.5	13	11	9	8	7	6	6	5	5												
3/4	#1	3/4	#1	2X4	4	1.5	5.0	8	6	5	5																	
					5	1.5	4.5	10	8	6	5	5																
					7	1.5	4.0	13	10	8	7	6	6	5	5													
					9	1.5	4.0	15	12	10	9	8	7	6	6	5	5											
3/4	#3	3/4	#3	2X4	4	1.5	5.0	7	6	5																		
					5	1.5	4.5	9	7	6	5																	
					7	1.5	4.0	12	9	8	7	6	5	5														
					9	1.5	3.5	14	11	9	8	7	6	6	5	5												
1-1/8	#1	1/2	#1	2X4	4	1.5	5.0	7	6	5																		
					5	1.5	4.5	9	8	6	5	5																
					7	1.5	4.5	13	11	9	8	7	6	5	5													
					9	1.5	4.0	16	13	11	9	8	7	6	6	5	5											
1-1/8	#1	1/2	#3	2X4	4	1.5	4.5	7	6	5																		
					5	1.5	4.5	9	7	6	5	5																
					7	1.5	4.5	13	11	9	8	7	6	5	5													
					9	1.5	4.0	16	13	11	9	8	7	6	6	5	5											

Table A1-1A (continued)

[illegible]

Table A1-1a PSSP DESIGNS FOR LOWER STRENGTH STRINGERS ($F_v = 280 \text{ psi}$)* (concluded)

Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQD. BEAR. EACH END		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																						
Nom. Th. in.	Face Ply Grp	Nom. Th. in.	Face Ply Grp	Nom. Ply Size in.	No.	Ply. Bot. Skin in.	Strngs in.	Clear Span, ft																						
								2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12		
1-1/8 #1		1/2	#1	2X6	4	1.5	7.5	12	9	8	7	6	5	5																
						5	1.5	7.0	15	12	10	8	7	7	6	5	5	5	5											
						7	1.5	7.0	21	17	14	12	11	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5
1-1/8 #1		1/2	#3	2X6	4	1.5	7.0	11	9	7	6	5	5																	
						5	1.5	7.0	15	12	10	8	7	6	6	5	5	5	5											
						7	2.0	7.0	22	17	15	12	11	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5
1-1/8 #1		3/4	#1	2X6	4	1.5	8.0	12	10	8	7	6	5	5																
						5	1.5	7.5	15	12	10	9	8	7	6	6	5	5	5	5	5	5	5	5	5	5	5	5	5	5
						7	1.5	7.0	22	17	14	12	11	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5
1-1/8 #1		3/4	#3	2X6	4	1.5	6.5	27	22	18	15	13	12	11	10															
						5	1.5	7.5	11	9	8	7	6	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
						7	2.0	7.0	22	18	15	13	11	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5
1-1/8 #1		1-1/8	#1	2X6	4	1.5	8.5	13	11	9	8	7	6	5	5															
						5	1.5	8.0	16	13	11	9	8	7	6	6	5	5	5	5	5	5	5	5	5	5	5	5	5	5
						7	1.5	7.0	22	18	15	13	11	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5
1-1/8 #1		1/2	#1	2X8	4	1.5	6.5	27	22	18	16	14	12	11	10															
						5	1.5	10.0	16	12	10	9	8	7	6	6	5	5	5	5	5	5	5	5	5	5	5	5	5	5
						7	1.5	9.5	30	24	20	17	15	13	12	11	10	9	8	7	7	6	6	5	5	5	5	5	5	5
1-1/8 #1		1/2	#3	2X8	4	1.5	9.5	15	12	10	8	7	7	6	5	5														
						5	1.5	10.0	20	16	14	12	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5	5
						7	1.5	9.5	30	24	20	17	15	13	12	11	10	9	8	8	7	7	6	6	5	5	5	5	5	5
1-1/8 #1		1/2	#3	2X8	4	1.5	9.5	39	31	26	22	20	17	16	14	13	12	11	10											
						5	1.5	10.5	16	13	11	9	8	7	7	6	5	5	5	5	5	5	5	5	5	5	5	5	5	5
						7	2.5	10.0	31	25	21	18	16	14	13	11	10	9	8	8	7	7	6	6	5	5	5	5	5	5
1-1/8 #1		3/4	#3	2X8	4	1.5	10.0	15	12	10	9	8	7	6	6	5	5													
						5	1.5	10.0	21	16	14	12	10	9	8	7	7	6	6	5	5	5	5	5	5	5	5	5	5	5
						7	2.5	10.0	31	25	21	18	15	14	12	11	10	10	9	8	8	7	7	6	6	5	5	5	5	5
1-1/8 #1		1-1/8	#1	2X8	4	1.5	9.5	39	31	26	22	19	17	16	14	13	12	11	10											
						5	1.5	11.0	17	14	12	10	9	8	7	6	5	5	5	5	5	5	5	5	5	5	5	5	5	5
						7	1.5	9.5	29	23	20	17	15	13	12	11	10	9	8	8	7	7	6	6	5	5	5	5	5	5

Table A1-1B PSSP DESIGNS FOR HIGHER STRENGTH STRINGERS ($F_v = 380 \text{ psi}$)*
Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQD. BEAR.		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																				
Nom. Th. in.	Face Ply Grp	Nom. Th. in.	Face Ply Grp	Nom. Ply Size in.	Ply. Str- ngs in.	EACH END		Clear Span, ft.																				
						No. in.	in.	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
1/2	#1	1/2	#1	2X4	4	1.5	3.5	9	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	14	11	9	8	7	6	6	5	5												
					9	1.5	2.5	17	14	11	10	8	8	7	6	6	5	5										
1/2	#3	1/2	#3	2X4	4	1.5	3.5	8	7	6	5	5																
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	2.5	13	10	8	7	6	6	5	5													
					9	1.5	2.5	15	12	10	9	8	7	6	6	5	5											
5/8	#1	1/2	#1	2X4	4	1.5	3.5	9	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	14	11	10	8	7	6	6	5	5												
					9	1.5	2.5	17	14	11	10	9	8	7	6	6	5	5										
5/8	#3	1/2	#3	2X4	4	1.5	3.5	8	7	6	5	5																
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	2.5	13	10	9	7	6	6	5	5													
					9	1.5	2.5	15	12	10	9	8	7	6	6	5	5											
3/4	#1	1/2	#1	2X4	4	1.5	3.0	8	6	5	5	5																
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	15	12	10	9	8	7	6	5	5	5											
					9	1.5	3.0	18	14	12	10	9	8	7	6	6	5	5	5									
3/4	#3	1/2	#3	2X4	4	1.5	3.5	8	7	6	5	5																
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	13	11	9	8	7	6	5	5													
					9	1.5	2.5	16	13	11	9	8	7	6	6	5	5	5										
3/4	#1	3/4	#1	2X4	4	1.5	3.5	8	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	16	13	11	9	8	7	6	6	5	5	5										
					9	1.5	3.0	19	15	13	11	10	8	8	7	6	6	5	5	5								
3/4	#3	3/4	#3	2X4	4	1.5	3.5	9	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	14	11	9	8	7	6	6	5	5												
					9	1.5	2.5	17	14	11	10	9	8	7	6	6	5	5										
1-1/8	#1	1/2	#1	2X4	4	1.5	3.0	7	6	5		5																
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	16	12	10	9	8	7	6	6	5	5											
					9	1.5	3.0	20	16	14	12	10	9	8	7	6	6	5	5	5								
1-1/8	#3	1/2	#3	2X4	4	1.5	3.0	7	6	5		5																
					5	1.5	3.0	10	8	6	6	5	5															
					7	1.5	3.0	15	12	10	8	7	7	6	5	5	5											
					9	1.5	3.0	20	16	13	11	10	9	8	7	6	6	5	5	5								

Table A1-1B PSSP DESIGNS FOR HIGHER STRENGTH STRINGERS ($F_v = 380$ psi)* (continued)
 Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQD. BEAR. EACH END		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																				
Nom. Ply Th. in.	Face Ply Grp	Nom. Ply Th. in.	Face Ply Grp	Nom. Ply Size in.	No.	Ply. Bot. Skin in.	Strngs in.	Clear Span, ft																				
								2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
1-1/8 #1		3/4	#1	2X4	4	1.5	3.0	8	6	5	5																	
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.5	17	13	11	10	8	7	7	6	6	5	5										
					9	1.5	3.0	21	17	14	12	11	9	8	8	7	7	6	6	5	5							
1-1/8 #1		3/4	#3	2X4	4	1.5	3.0	7	6	5																		
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	16	12	10	9	8	7	6	6	5	5											
					9	1.5	3.0	20	16	14	12	10	9	8	7	7	6	6	5	5								
1-1/8 #1		1-1/8 #1		X4	7	1.5	3.5	9	7	6	5																	
					7	1.5	3.5	12	10	8	7	6	5	5														
					7	1.5	4.0	19	15	12	11	9	8	7	7	6	6	5	5									
					11	1.5	3.5	23	18	15	13	11	10	9	8	8	7	7	6	6	5	5						
1/2 #1		1/2	#1	X6	4	1.5	6.0	15	12	10	8	7	6	6	5	5												
					5	1.5	5.5	18	14	12	10	9	8	7	6	6	5	5										
					7	1.5	4.5	24	19	16	13	12	10	9	9	8	7	7	6	6	5	5	5					
					9	1.5	4.5	29	23	19	17	15	13	12	11	10	9	8	8	7	7	6	6	6	5	5	5	
1/2 #3		1/2	#3	2X6	4	1.5	5.5	14	11	9	8	7	6	6	5	5												
					5	1.5	5.0	17	14	11	10	8	7	6	6	5	5											
					7	2.0	4.5	22	18	15	13	11	10	9	8	7	7	6	6	6	5	5	5					
					9	2.5	4.0	28	22	18	16	14	12	11	10	9	9	8	7	7	6	6	5	5				
5/8 #1		1/2	#1	2X6	4	1.5	6.0	15	12	10	8	7	7	6	5	5												
					5	1.5	5.5	18	14	12	10	9	8	7	7	6	6	5	5									
					7	1.5	5.0	24	19	16	14	12	11	10	9	8	7	7	6	6	6	5	5	5				
					9	1.5	4.5	30	24	20	17	15	13	12	11	10	9	8	8	7	7	6	6	6	5	5	5	
5/8 #3		1/2	#3	2X6	4	1.5	5.5	14	11	9	8	7	6	6	5	5												
					5	1.5	5.0	17	14	12	10	9	8	7	6	6	5	5										
					7	2.0	4.5	22	18	15	13	11	10	9	8	7	7	6	6	6	5	5	5					
					9	2.5	4.0	28	22	19	16	14	12	11	10	9	9	8	7	7	6	6	6	5	5	5		
3/4 #1		1/2	#1	2X6	4	1.5	5.5	14	11	9	8	7	6	5	5													
					5	1.5	5.0	17	14	12	10	9	8	7	6	6	5	5										
					7	1.5	5.0	25	20	17	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5			
					9	1.5	4.5	31	25	20	18	15	14	12	11	10	9	9	8	8	7	7	6	6	6	5	5	
3/4 #3		1/2	#3	2X6	4	1.5	6.0	15	12	10	8	7	6	6	5	5												
					5	1.5	5.5	18	15	12	10	9	8	7	7	6	6	5	5									
					7	2.0	4.5	23	19	16	13	12	10	9	8	8	7	7	6	6	6	5	5	5				
					9	2.5	4.5	29	23	19	16	14	13	12	10	10	9	8	8	7	7	6	6	6	6	5	5	
3/4 #1		3/4	#1	2X6	4	1.5	5.5	14	12	10	8	7	6	6	5	5												
					5	1.5	5.5	18	14	12	10	9	8	7	7	6	6	5	5									
					7	1.5	5.0	25	20	17	14	13	11	10	9	8	8	7	7	6	6	6	5	5	5	5		
					9	1.5	4.5	31	25	21	18	16	14	12	11	10	10	9	8	8	7	7	6	6	6	6	5	5

Table A1-1B (concluded)

[illegible]

* For dynamic loads (blast) use herein, F_v values were four times the normal use values, as published in Reference 3's Supplement.

PSSP Upgrading Example - Closure

Try one of the PSSP designs as a horiz. closure over an aperture (such as a stairwell), in the floor over a candidate basement shelter.

Assume use of the last PSSP design on page A1-20: $3/4"$ #1 both skins, 9 - 2×6 stringers (higher strength). Assume clear span of 4 ft.

From Table A1-1B: peak blast overpr. = 16 psi

Factors included in the blast design are: 2.0 for impact duration of loading; 2.0 for factor of safety; and 0.75 for $\mu = 2$. See page A1-12.

Basic (static) load strength of PSSP =

$$\frac{16 \text{ psi}}{2.0 \times 2.0 \times 0.75} = 5.33 \text{ psi}$$

Fallout shielding soil:

Assume 18" th. layer of 95 pcf soil.

Allow 15% added stress for load duration less than 2 mos.

$$\text{Static load strength used up for fallout soil} = \frac{1.5 \times 95}{144} \times \frac{1}{1.15} = 0.86 \text{ psi}$$

Static load strength remaining = 4.47 psi

Blast:

Use static load strength remaining (4.47), increased by the 3 factors taken out above:

$\therefore$ Peak blast overpressure strength:

$$4.47 \times 2.0 \times 2.0 \times 0.75 = \underline{13.41 \text{ psi}}$$

Fabrication

Fabrication of plywood stressed-skin panels (PSSPs) is concisely yet thoroughly described in a publication available upon request.⁷ The publication emphasizes the need for adequate gluing in order to develop the composite action of plywood stressed-skins and the stringers. Results from mechanical-pressure gluing have been found to be generally superior to nail-gluing (latter, properly performed, is the basis for the design section herein, however); supplies needed for nailing may have to be estimated in advance, for which the following extract will be useful:^{7(p.6)}

"Nails shall be at least . . . 6d for 1/2" to 7/8" plywood, 8d for 1" to 1-1/8" plywood, . . . spaced not to exceed . . . 4" (along the framing members) for plywood 1/2" and thicker, using one line for lumber 2" thick or less, and two lines for lumber more than 2" and up to 4" thick (wide)."

Glue, recommended for use in accordance with the manufacturers' recommendations, should be one of the two following types: Interior, for use when the equilibrium moisture content of the materials used does not exceed 18%, may be casein type with a mold inhibitor, conforming with ASTM Specification D3024; Exterior, for higher moisture contents, conforming to ASTM Specification D2559.

Nailing without gluing simply does not exploit the strength of PSSPs and the capabilities of their materials - the nails can too easily yield along the grain of the stringers so that they are inadequate as a shear transfer mechanism. The sparse test data found clearly show concern with deflection, not flexural, behavior as the controlling criterion, thus ultimate strength is not considered.

In the absence of some kind of ultimate strength behavior tests, the author has no basis for a recommendation, even heavily qualified, on the relative strength of nailed-only to nail- or pressure-glued PSSPs.

Further Work

As mentioned in the section on "Design Stresses . . ." above, tests for ultimate strength (i.e., through to failure/collapse, recording full load-deflection history including time) under dynamic loadings, or even under static loadings if well into the plastic range, are badly needed as a better basis for design of PSSPs as blast closures. With such information, one might be, for example, justified in design procedure use of numerical integration of the equation of motion, instead of the less rigorous approach of using a step-pulse loading of infinite duration, as has been done in preparing the design procedure above. Further, the wood design stresses would be better known, of course, as would the composite behavior including the primary cause of each test PSSP's failure mode. Some tests have been completed at the Ballistics Research Laboratory, U. S. Army, Aberdeen Proving Grounds, Maryland 21005, and more are planned.

NOTATION

A	total x-section area of all stringers
A	x-section area of parallel-grain plies outside the critical plane for rolling shear
A	total x-section area of all stringers and skins $A_{//}$ (beam-columns)
$A_{//}$	x-section area (finished) of plies // stringers, in each skin
$A_{\perp}$	x-section area (finished) of plies $\perp$ stringers, in each skin
b, b_b, b_t	basic stringer spacing; subscripts are for bottom and top skins, respectively
C	factor for maximum allowable deflection (usually based on LL only)
c	distance from neutral axis (for deflection or bending, as locally defined) to extreme fibre (of skin under check) (see $\bar{y}$)
d	moment arms for various x-sectional areas (subscripted A's), used in I_g and I_n calculations
$d_s = c - y'$	
E	modulus of elasticity
E_{st}	E of stringers
(EI_g)	panel parameter, calculated using neutral axis for deflection
(EI_n)	panel parameter, calculated using neutral axis for bending moment
F	allowable stress, general
F	allowable splice-plate stress multiplied by proportion of panel width actually spliced
F_c	allowable stress, compression in plane of plies // stringers
F_c	allowable stress, compression // grain in stringers
$F_{c\perp}$	allowable stress, bearing on plywood face
F_s	allowable stress, rolling shear
F_t	allowable stress, tension in plane of plies // stringers
F_t	allowable stress, tension // grain in stringers
F_v	allowable stress, horizontal shear, in stringers
G	modulus of rigidity in stringers
I	moment of inertia, total x-sectional area (finished) of all stringers
I	moment of inertia, in direction $\perp$ stringers, of top skin $A_{\perp}$
I_g	gross I of total panel x-section about deflection N.A.

NOTATION (concluded)

I_n	gross I of total panel x-section about bending N.A.
I_o	gross moment of inertia of x-section portion about own centroidal axis
$I_{//}, I_{\perp}$	moment of inertia for plies, corresponding to $A_{//}$ and $A_{\perp}$ areas
ℓ	clear span of panel, in direction of stringers
ℓ_e	plywood end bearing length required at <u>each</u> end of panel
ℓ'	panel width (skins only), perpendicular to ℓ
ℓ''	clear distance between stringers
p	design LL or TL (use load related to assumed C factor)
p_a	allowable axial load (TL) in beam-column
p_b	allowable load (TL) - bending moment
p_d	allowable load (TL) - panel deflection
p_{dm}	same as p_m but specifically for dynamic loads/loadings
p_m	smallest of calculated allowable transverse loads (TL) (in PSSPs for: deflection, bending moment, rolling shear and horizontal shear)
p_p	allowable load (TL) - tension splice-plate
p_s	allowable load (TL) - rolling shear
p_t	allowable load (TL) - top skin deflection
p_v	allowable load (TL) - horizontal shear
Q_s	statical moment, about neutral axis for deflection, of parallel plies outside critical plane for rolling shear (see A above)
Q_v	statical moment, about neutral axis for deflection, of stringers and $A_{//}$ plies x-sectional areas, taken <u>either</u> above <u>or</u> below that axis (used in horizontal shear allowable load calculations)
t	glueline width of each stringer (used in $\Sigma F_s t$)
t	sum of stringer widths, including side projecting portions
t_h	thickness of header (solid across all panel stringers)
y	moment arms used in neutral axes calculations
y'	half-thickness of parallel plies outside critical plane for rolling shear (see Q_s and A above)
$\bar{y}$	distance from neutral axis to bottom extreme fibre (calculated in both deflection and bending moment calculations for neutral axis)
μ	ductility ratio (maximum to elastic deflection, of a selected point, usual at mid-span or mid-height)

REFERENCES

1. "Design of Plywood Stressed-Skin Panels," Supplement 3 to Plywood Design Specification (PDS), American Plywood Association, 1119 A Street, Tacoma, Washington 98401, revised December 1976.
2. Plywood Design Specification (PDS), American Plywood Association, revised December 1976.
3. National Design Specification for Stress-Grade Lumber and Its Fastenings, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D.C. 20036, 1973 edition, with Table 1 Supplement (allowable unit stresses, published separately), April 1973, revised November 1974.
4. Wood Handbook: Wood as an Engineering Material, Forest Products Laboratory, Forest Service, U.S. Department of Agriculture, Agriculture Handbook No. 72 (Government Printing Office, Washington, D.C.), revised August 1974.
5. Gerhards, C. C., Effect of Duration and Rate of Loading on Strength of Wood and Wood-Based Materials, USDA Forest Service Research Paper FPL 283, 1977, U.S. Forest Products Laboratory, Madison, Wisconsin 53705.
6. Drawsky, R. H., and J. M. Carney (editor), Stressed Skin Panel Tests, Laboratory Report No. 82, Douglas Fir Plywood Association, Tacoma, Washington, April 1960.
7. Fabrication of Plywood Stressed-Skin Panels, Plywood Fabrication Specification SS-8, American Plywood Association, Tacoma, Washington 98401, 1974.

Appendix A2

PLYWOOD STRESSED-SKIN PANELS (TWO-SIDED) AS BEAM-COLUMNS

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Appendix A2

PLYWOOD STRESSED-SKIN PANELS (TWO-SIDED) AS BEAM-COLUMNS

Design

In the preceding Appendix A1, a design procedure, useful stresses, and typical designs were developed for plywood stressed-skin panels (PSSPs) and their estimated ultimate/collapse strength capacity for lateral/transverse (blast) loads. Such panels are of considerable interest to the overall purposes of the project work because abundant supplies of wood for stringers and plywood for skins are available in local lumberyards. Thus their potential is high for use in expedient upgrading of existing basements for shelter against the combined effects of a nuclear weapon detonation. These panels are treated in Appendix A1 in terms of their usefulness as closures, that is to resist transverse blast loads. The purpose of this appendix is to develop procedures for use of such panels as beam-columns, that is to resist axial (blast) loads, without or combined with transverse/lateral (blast) loads.

The basic references of Appendix A1 also contain information pertinent to beam-column design, or simple column design alone [1(sec.3),2].¹ The formula provided for the latter is

$$P_a = 3.619 (EI_g) / \ell^2 \quad (1)$$

or

$$P_a = F_c A \quad (2)$$

whichever value is less, where²

P_a = allowable axial load (lbs), if axial load only exists

(EI_g) = stiffness factor for moment deflection [1(Sec.2.4.3)]
(lbs-in.² for full panel)(from Step 4, Appendix A1)

ℓ = clear span of member (simply-supported/pin-ended)(in.)

¹ Brackets are used herein to indicate sources in the References list at the end of this appendix.

² Variables are defined herein at point of first use and in Notation at end of appendix.

F_c = allowable compressive stress (parallel to grain) for plywood skins (psi) [2(p.17)], corrected for buckling [1(Sec.2.5.4)]

A = total x-sectional area of longitudinal grain material in both plywood skins and stringers (in.²)

The interaction formula provided for beam-columns is

$$P/P_a + (M/S) / F_c \leq 1 \quad (3)$$

where

P = allowable axial load (lbs), under combined loading

M = allowable bending moment (in.-lb), under combined loading

S = I_n / c

in which

S = section modulus of full panel (in.³)

I_n = bending moment of inertia of full panel (in.⁴)

c = distance from N.A. (bending) to extreme fiber in compression (in.)

Calculations of I_n and c (or $\bar{y}$) are shown in Appendix A1, Figure 3, and Steps 7 and 8.

Assuming that the authors of References [1 and 2] used theory including a solid, rectangular cross-section column, then $I = bd^3/12$. Using this for I_g and solving Equations 1 and 2 for F_c (also recalling that $r^2 = I/A$) leads to

$$F_c = (0.3016 E) / (\ell/d)^2 = (\pi^2 E) / (2.727(\ell/r)^2)$$

where

I_g = gross moment of inertia of cross section (in.⁴)

b = least dimension of solid rectangular cross section (in.)

d = greater dimension of solid rectangular cross section (in.)

r = radius of gyration (in.)

E = modulus of elasticity (psi)

The equation's left form is found in Reference [3(p.15 and 65)], under simple solid-column design, indicating that the assumption above is cor-

rect. The equation's right form is Euler's equation [4(Eq.3 and 14)] in one of its many forms. Euler's equation is suitable for simply-supported/pin-ended long columns at ultimate (not allowable) load; it is non-conservative [4] when applied to columns with ℓ/r less than about 150,³ a value much too high for the uses contemplated herein; and the above constant, 2.727, is a factor of safety.

The serious concern with using the foregoing for blast loads is that various approximations have been introduced that can be collectively tolerated because of the allowable/working stress approach for normal uses. Where one is dealing with collapse strength of a column or beam-column, the design approach must take dynamic buckling directly into account and must consider deflection, usually at mid-height, caused by all loads, plus initial eccentricity if it is known or can be estimated. Thus it was concluded that a beam-column design approach should include iteration toward an estimated total deflection from all sources, i.e., initial eccentricity if any, as well as deflection from moments due to transverse and axial loads.⁴ The following design approach includes such iteration; it comes from Reference [4(p.5-42, Eq.18)], and is converted to PSSP Notation (Appendix A1 and herein). Henceforth

$$F_c = P/A + (M + P_y)(c/I_n) \quad (4)$$

where

M_{max} = maximum moment caused by transverse loads only (in.-lb)

y = deflection of column at M (in.)

The referenced source suggests iteration toward a final value for y , using for a first trial value that from M alone⁵ in the right-side second term of Equation 4. An approach to performing the suggested iteration follows, using the simply-supported/pin-ended member assumption stated earlier.

From Reference [6], for transverse loads⁵ (and modified to Notation herein):

$$M_{mid-ht} = p_m \ell' \ell^2 / 8 \quad (5)$$

$$\bar{y}_{mid-ht} = 5 p_m \ell' \ell^4 / (384(EI_n)) \quad (6)$$

³ Meaning, for a rectangular column cross-section, ℓ/d less than about 43 (but it's about 24 per Ref.[3],p.65).

⁴ An iterative numerical method for analyzing a beam-column is available [5(App.A,p.6-160)].

⁵ That is, with $P = 0$.

where

$\bar{y}_{\text{mid-ht}}$ = deflection of column at M_{max} (transverse loads only)(in.)

Thus, for combined transverse and axial loads:

$$y_{\text{mid-ht}} = 5 \ell^2 (M_{\text{mid-ht}} + P_a y) / (48(EI_n)) \quad (7)$$

where

p_m = smallest of calculated allowable transverse (only) loads
(in PSSPs, calculated loads for: deflection, bending moment,
rolling shear and horizontal shear) (psi) (from Appendix A1
design of PSSPs)

$y_{\text{mid-ht}}$ = deflection of column at M (in.)

ℓ' = width of PSSP skins (perpendicular to stringers) (in.)

(EI_n) = stiffness factor for bending moment [1(Sec.2.5.3)] (lb-in.²
for full panel) (from Step 8, Appendix A1)

y = trial y at mid-height

For examining locations other than at mid-height of the (prismatic)
beam-column, similar equations to Equations 5 to 7 would then be [6]:

$$M_x = p_m \ell' x (\ell - x) / 2 \quad (8)$$

$$\bar{y}_x = p_m \ell' x (\ell^3 - 2\ell x^2 + x^3) / (24(EI_n)) \quad (9)$$

$$y_x = (M_x + P_a y) (\ell^2 + \ell x - x^2) / (12(EI_n)) \quad (10)$$

where

x = location being examined (length along member) (in.)

The overall design approach just described should be applied with
due regard to variation in units: some of the parameters are for full
panel width, some would usually be applied to design of a one-inch wide
strip of panel. All units, therefore, should be checked for values
appropriate to one width or the other. All formulas herein are dimen-
sionally consistent; there are no dimensions hidden in constants.

A. Design Procedures

Steps in the design procedure follow.

1. Assume a trial section and clear span/height (in direction of stringers); see Figure 1A, Appendix A1. Use only stress-graded stringers, with face grain of both plywood skins parallel to the stringers. Plan connections to PSSP such that loads are only axial, on pinned ends, with or without uniformly distributed transverse/lateral loads.

2. Same as Step 2, Appendix A1.

3. Calculate A as in Step 3 and Figure 2A, Appendix A1.

4-6. Same as Steps 7 through 9, respectively, of Appendix A1. The c value needed later comes from Step 4 (either $\bar{y}$ in Figure 3A, Appendix A1, or the actual PSSP thickness minus $\bar{y}$, probably the latter but certainly whichever value is for the compression side).

At this point, values for the following variables used in this appendix are known: A (from Step 3), c (4), (EI_N) (5), F_c (6), I_N (5), ℓ (1), and ℓ' (1).

7. Set $M = 0$ and Equation 4 becomes:

$$F_c = P_a/A + P_a(cy/I_N) \quad \text{or} \quad P_a = F_c / (1/A + cy/I_N) \quad (11)$$

From Equation 2:

$$F_c = P_a / A \quad (12)$$

Whether $y \geq 0$ (with trial design F_c held constant), the P_a of Equation 12 will be $\geq$ the P_a of Equation 11, thus Equation 11 is used below. This P_a is the maximum allowable axial load, applied when $M = 0$ (i.e., transverse/lateral load is zero).

8. Set $M = 0$, then iterate on trial y , using as a minimum value/ eccentricity (in.)

$$y = 0.01 \ell / d'$$

where

$$d' = \text{depth of PSSP stringers (in.)}$$

then solving Equation 11 for P_a and Equation 7 with trial y value on right; repeat until y values on right and left sides of Equation 7 are equal or acceptably close (say 1% to 5%).

9. For combined transverse and axial loads, the PSSP must first be investigated, using Steps 1-14, Appendix A1, to find p_m , the peak tran-

verse load capability with $P_a = 0$; if a dynamic loading p_{dm} value was found, it must be corrected to an equivalent static load p_m :

If the PSSP is one of those pre-designed and shown in Appendix A1, its p_{dm} value may be read from Figures 5-7 there, as the air blast peak overpressure (psi). However, such p_{dm} is based on the Design Stresses-Blast . . . section⁶ of the appendix, which includes use of $\mu = 2$ and a step pulse, meaning that the static equivalent p_m is $4/3$ the chart value p_{dm} (still with design stresses greatly increased over those for normal, not blast-resistant, use). A value for μ and blast design stresses in beam-columns, in contrast to normal-use design stresses, are discussed in the next section.

Subscripts for mid-height will be dropped from here on, for convenience; the PSSP should be prismatic and with negligible initial eccentricity,⁷ therefore all M and y values will be for mid-height (mid-length) for a vertical (horizontal) beam-column.

10. Solve Equations 5 and 6 for M and related y , when $P = 0$.

From Step 8, values of P_a and related y , when $M = 0$, are known. Thus the two extreme values of transverse or axial load capacity, with their related mid-height/mid-length deflections, are known at this point. These unique values will be identified as M_{max} (or its related p_m of Step 9) and P_a in the steps below.

11. Assume: a value for P between P_a and zero, and a first trial value⁷ for y proportional to those found in Steps 8 and 10 (for P_a and for $P = 0$, respectively).

12. Solve Equation 4 for M :

$$M = (I_n / c) (F_c - P / A) - Py, \quad \text{but } \leq (M_{max} - Py) \quad (13)$$

13. Solve Equation 7 using the trial y on the right side. Compare the left-side y , found from solving Equation 7, with the trial y used. If the two y values are not in acceptable agreement (say, 1% to 5%), use the left-side value as the new trial value⁷ of y and repeat Steps 12 and 13; otherwise, proceed with the next design step.

14. Find allowable p'_m related to the final M of Step 12:

$$p'_m = (M_{(step\ 12)} / M_{max}) p_{m(step\ 9)} \quad (14)$$

⁶ Includes, at the end of that section, a definition of step pulse and the basic relationship $p_{dm} = p_m (1 - 1/(2\mu))$. It follows that $P_{da} = P_a (1 - 1/(2\mu))$, but μ will usually be different in the two uses: (1) lateral loads only; or (2) axial loads only, or axial and lateral loads.

⁷ But using not less than the minimum eccentricity value of Step 8.

(This allowable p'_m could also be found using Equation 5 with the final M of Step 12, or Equation 6 with the final left-side y of Step 13.)

15. With allowable P (Step 11) and p'_m (Step 14) known, one pair of pertinent values for the assumed trial section PSSP has been found, besides the two pairs of extreme values (Step 10). Other pairs of values are found by repeating Steps 11-14. To complete the design, a new trial section(s) may have to be assumed, repeating Steps 1-14.

B. Design Stresses (Blast versus Normal Loads) and Ductility

The user of this appendix is referred to a section with the same title, appearing in Appendix A1; the information there is applicable to this appendix except for the last paragraph, which deals with a value for the ductility ratio μ .

For a value of the ductility ratio μ for beam-columns, $\mu = 1$ is recommended for use because of buckling considerations. Increases of normal-use stresses are those already recommended for adoption.⁸ If a step pulse (defined in the Appendix A1 section) is appropriate, then the footnote to design Step 9 applies, thus $P_{da} = P_a/2$ ($P_d = P/2$) and $P_{dm} = p_m/2$ ($p'_{dm} = p'_m/2$); p_{dm} or p'_{dm} is the allowable load from peak exterior blast incident overpressure on the PSSP, and p_m or p'_m is the pseudostatic uniform load capacity, respectively.

It is possible that a significant rise time should be applied to the axial blast load but probably not. However, the transverse blast load occurring inside a basement shelter is very likely to have a significant rise time as well as a significant reduction in peak value from the blast peak exterior incident overpressure, due to room filling.⁹ If a rough approximation must be suggested it would be that $p_{dm} = p_m$ ($p'_{dm} = p'_m$) where only human-size doorways and typical basement windows constitute the apertures; large openings would indicate use of $p_{dm} = p_m$ times 3/4, even approaching 1/2. This suggested approach attempts to consider both lengthened rise time and reduced peak value of overpressure in terms of that incident on the basement's exterior.

⁸ See Appendix A1 section with same title as this one: multiply normal-use F_b , F_v , and F_c values by four, but not F_{c1} or E values.

⁹ See published guidance on design of combined nuclear weapons effects shelter in planned (new) basements, References [5 and 7], especially the latter's Appendix E appearing in Volume 3; the same Appendix E, written by J. R. Rempel, a colleague, was published in an earlier report, Reference [8]. The Appendix E technique was used to produce a short section and two design graphs [7 and 8 (p.8-112 to 8-114)] giving maximum interior pressure and time to reach such pressure, both in terms of V/A (room volume/total aperture area).

Applications

The computer program used (for PSSP design of "beam/slab" type members) in the earlier publication of Appendix A1, was updated to add beam-column and column PSSP design. Table A2-1 shows a listing of the program, as well as a sample problem. The program listed does not automatically increase normal-use design stresses⁸ (they must be entered with multiples, if any, already included by the user), and it does not include the dynamic factors for blast loads (section B above, second paragraph).

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F/G 13/13

UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDE--ETC(U)

JUL 80 H L MURPHY

DCPA01-77-C-0227

NL

UNCLASSIFIED

EX-9
2-11-1

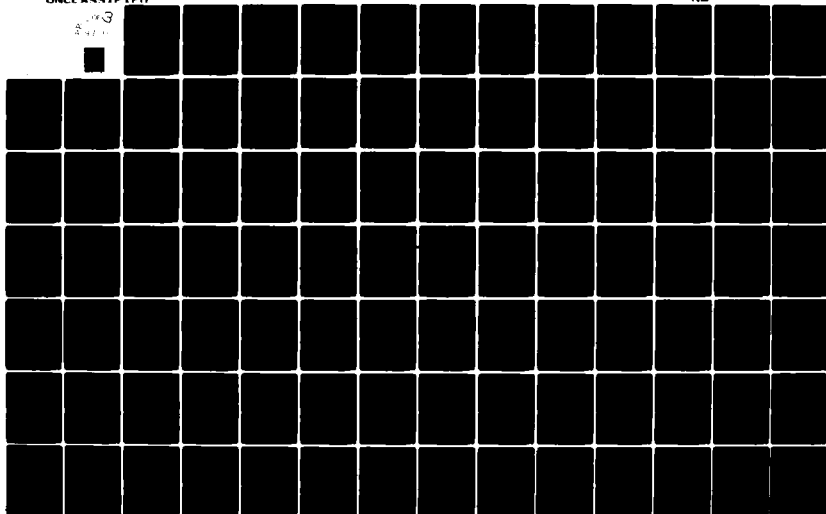


Table A2-1 PSSP DESIGN: COMPUTER PROGRAM (HLMSP) LISTING AND SAMPLE PROBLEM*

```

LIST
HLMSP
0050 REM
0052 REM
0054 REM
0100 REM
0110 REM
0120 REM
0130 REM
0140 REM
0150 REM
0160 REM
0170 REM
0180 REM
0185 REM
0190 REM
0192 DIM M(9),P(9),C(10)
0192 DIM Q(9),Y(9)
0200 GOSUB 540
0210 GOSUB 730
0220 GOSUB 1050
0230 GOSUB 1330
0240 GOSUB 1720
0250 GOSUB 1910
0260 GOSUB 2270
0270 GOSUB 2610
0280 GOSUB 2830
0290 GOSUB 3400
0300 GOSUB 3540
0310 GOSUB 3990
0320 GOSUB 5000
0330 PRINT "ANY CHANGES DESIRED (YES OR NO) "
0340 INPUT B$
0350 IF B$="NO" THEN 430
0360 PRINT "GIVE ONE STEP NUMBER WHERE CHANGE(S) DESIRED "
0370 INPUT N2
0380 IF INT(N2)<1 THEN 360
0390 IF INT(N2)>15 THEN 360
0400 GOSUB N2 OF 540,540,730,1050,1330,1720,1910,2270,2610,2830,3400,3540,3990
0410 GOSUB N2-10 OF 2610,2830,3400,3540,3990
0420 GOTO 330
0430 PRINT
0440 PRINT "UNIT TABULATION OF INPUT VALUES (YES OR NO) "
0450 INPUT B$
0460 IF B$="NO" THEN 490
0470 GOSUB 4240
0480 GOTO 320
0490 GOSUB 5000
0500 REM
0510 REM
0520 REM
0530 REM
0540 GOTO 640
0550 PRINT "STEP 2:"
0560 PRINT "ENTER B, DISTANCES FOR TOP & BOT. SKINS, RESP-Y(IN.) "
0570 PRINT "HLMSP: FIG.1B1 ALSO PDS3, P.5(TABLE)"
0580 INPUT B1,B2
0590 PRINT
0600 PRINT "ENTER CLEAR DISTANCE BETWEEN STRINGERS (FIG.1A,HLMSP) (IN.) "
0610 PRINT "HLMSP: FIG.1B1 ALSO PDS3, P.5(TABLE)"
0620 INPUT L4
0630 PRINT
0640 IF L4 (= 249) THEN 720
0650 IF L4 (= 249) THEN 720
0660 PRINT "WARNING: PROGRAM DESIGN PROCEDURE NOT APPLICABLE"

THIS PROGRAM IS A MODIFICATION AND EXTENSION OF HLMSP.
TO ADD PROVISION FOR COLUMN AND BEAM-COLUMN DESIGN.
PROGRAM FOR DESIGN OF PLYWOOD STRESSED-SKIN PANELS
PDS = PLYWOOD DESIGN SPEC., SHEAR, PLYWOOD ASSN (REV. 1-76)
PDS3= SUPPLEMENT 3 (1974) TO PDS
NDS = NAIL DES. SPEC. FOR STRESS-GRADE LUMBERS AND ITS FASTENINGS, TABLE 1 (SUPPLEMENT, REV. 11-74)
WOODBK = WOOD HANDBOOK REV. 8-74, U.S. FOREST PRODUCTS LAB
HLMSP1 = H.L. MURPHY REPORT DESCRIPTION OF DESIGN PROCEDURE
PROGRAM MODIFIED TO INCLUDE MIN. EXCENTRICITY 11/11/80 - JEB
N120),R8(110),B8(110)
0190 DIM M(9),P(9),C(10)
0192 DIM Q(9),Y(9)
0200 GOSUB 540
0210 GOSUB 730
0220 GOSUB 1050
0230 GOSUB 1330
0240 GOSUB 1720
0250 GOSUB 1910
0260 GOSUB 2270
0270 GOSUB 2610
0280 GOSUB 2830
0290 GOSUB 3400
0300 GOSUB 3540
0310 GOSUB 3990
0320 GOSUB 5000
0330 PRINT "ANY CHANGES DESIRED (YES OR NO) "
0340 INPUT B$
0350 IF B$="NO" THEN 430
0360 PRINT "GIVE ONE STEP NUMBER WHERE CHANGE(S) DESIRED "
0370 INPUT N2
0380 IF INT(N2)<1 THEN 360
0390 IF INT(N2)>15 THEN 360
0400 GOSUB N2 OF 540,540,730,1050,1330,1720,1910,2270,2610,2830,3400,3540,3990
0410 GOSUB N2-10 OF 2610,2830,3400,3540,3990
0420 GOTO 330
0430 PRINT
0440 PRINT "UNIT TABULATION OF INPUT VALUES (YES OR NO) "
0450 INPUT B$
0460 IF B$="NO" THEN 490
0470 GOSUB 4240
0480 GOTO 320
0490 GOSUB 5000
0500 REM
0510 REM
0520 REM
0530 REM
0540 GOTO 640
0550 PRINT "STEP 2:"
0560 PRINT "ENTER B, DISTANCES FOR TOP & BOT. SKINS, RESP-Y(IN.) "
0570 PRINT "HLMSP: FIG.1B1 ALSO PDS3, P.5(TABLE)"
0580 INPUT B1,B2
0590 PRINT
0600 PRINT "ENTER CLEAR DISTANCE BETWEEN STRINGERS (FIG.1A,HLMSP) (IN.) "
0610 PRINT "HLMSP: FIG.1B1 ALSO PDS3, P.5(TABLE)"
0620 INPUT L4
0630 PRINT
0640 IF L4 (= 249) THEN 720
0650 IF L4 (= 249) THEN 720
0660 PRINT "WARNING: PROGRAM DESIGN PROCEDURE NOT APPLICABLE"

0670 PRINT "HLMSP1: STEP 2: ALSO PDS3, SEC. 2.1.1, P. 5"
0680 PRINT
0690 REM
0700 REM
0710 REM
0720 GOTO 430
0730 PRINT "STEP 3:"
0740 PRINT "ENTER E VALUES (PSI) FOR TOP, BOT. SKINS & STRINGERS, RESP-Y"
0750 PRINT "HLMSP: FIG.1P, 17 & NDS"
0760 INPUT E1,E2,E3
0770 PRINT
0780 PRINT "ENTER A// VALUES FOR TOP, BOT. SKINS (50 IN-FT WIDTH)"
0790 PRINT "HLMSP: FIG.1P, 16, COL. 4"
0800 INPUT A1,A2
0810 PRINT
0820 PRINT "ENTER TOTAL X-SECT. AREA OF ALL STRINGERS (50 IN)"
0830 INPUT A3
0840 PRINT
0850 PRINT "ENTER MOMENT ARMS FOR TOP, BOT. SKINS & STRINGERS, RESP-Y (IN.)"
0860 PRINT "FROM BOTTOM SURFACE OF PANEL"
0870 INPUT Y1,Y2,Y3
0880 PRINT
0890 PRINT "ENTER WIDTH (PLYWOOD) OF PANEL (IN.)"
0900 INPUT L2
0910 PRINT
0920 RETURN
0930 LET N11=A1/12+R2/12+L2+R3
0940 LET N12=A1/12+L2+E1*1.1+R2/12+L2+E2*1.1+R3+E3*1.03
0950 LET N13=A1/12+L2+E1*1.1+Y1+R2/12+L2+E2*1.1+Y2+R3+E3*1.03+Y3
0960 LET Y7=N13/N12
0970 PRINT "STEP 3:"
0980 PRINT "V-BAR (DEFLECTION N.A.) = "Y7" IN."
0990 PRINT "HLMSP: FIG.2A TOTALS: "N11,N12,N13
1000 PRINT
1010 REM
1020 REM
1030 REM
1040 GOTO 1140
1050 PRINT "STEP 4:"
1060 PRINT "ENTER J OF TOP, BOT. SKINS ABOUT OWN AXES (IN. 4-FT WIDTH)"
1070 PRINT "HLMSP: FIG.1P, 16, COL. 5"
1080 INPUT J1,J2
1090 PRINT
1100 PRINT "ENTER TOTAL WIDTH OF STRINGERS (IN.) "
1110 INPUT T5
1120 PRINT
1130 RETURN
1140 LET T3=T5*(R3/T5)^3/12
1150 LET D1=Y1-Y2
1160 LET D2=Y7-Y2
1170 LET D3=ABS(Y7-Y3)
1180 LET N1=11/12+R2+R1/12+L2+D1^2
1190 LET N2=12/12+R2+R2/12+L2+D2^2
1200 LET N3=13+R3+D3^2
1210 LET T7=N1+N2+N3
1220 LET N4=E1*1.1+N1
1230 LET N5=E2*1.1+N2
1240 LET N6=E3*1.03+N3
1250 LET E4=N4+N5+N6
1260 PRINT "STEP 4:"
1270 PRINT "E1-SUB G = "E4" LB-IN.2"
1280 PRINT "HLMSP: FIG.2B TOTALS 1",17,E4
1290 REM
1300 REM
1310 REM
1320 GOTO 1380

```

* Sample problem (on last three sheets of this table is the one used in Reference 1.

Table A2-1 (continued)

```

1330 PRINT "STEP 5:"
1340 INPUT L1
1350 PRINT
1360 PRINT
1370 GOTO 1430
1380 IF L1=INT(L1) THEN 1410
1390 LET L1=INT(L1)+1
1400 GOTO 1420
1410 LET L1=INT(L1)
1420 GOTO 1530
1430 PRINT "ENTER C FACTOR IN MAX ALLOWABLE DEF. L/C"
1440 PRINT " (HLMSP1, STEP 6: PDS3, P. 9)"
1450 INPUT C1
1460 PRINT "IS C FACTOR IN TERMS OF LL OR TL?"
1470 INPUT AS
1480 PRINT
1490 IF AS="TL" THEN 1570
1500 IF AS="LL" THEN 1570
1510 GOTO 1470
1520 LET P1=1/C1+1.2+5*1.2/384/E4+.15/A2/1.06+E3*1.03
1530 IF AS="TL" THEN 1620
1540 LET N1=P1
1550 GOTO 1610
1560 PRINT "ENTER DESIGN DL (PSI)"
1570 INPUT P9
1580 PRINT
1590 PRINT
1600 RETURN
1610 LET P1=P1+P9
1620 PRINT "STEP 5:"
1630 PRINT "P-SUB D = "P1" PSI"
1640 IF AS="TL" THEN 1670
1650 PRINT "MADE UP OF LL = "N1" AND DL = "P9" PSI"
1660 PRINT "PANEL DEF. (MID-SPAN, SS) = "L1/C1" IN."
1670 REM *****
1680 REM ***** STEP 6 *****
1690 REM ***** (HLMSP1) *****
1700 REM *****
1710 GOTO 1780
1720 PRINT "STEP 6:"
1730 PRINT "ENTER 1 FOR TOP SKIN (STRESS PERPENDICULAR TO STRINGERS)"
1740 PRINT " (PDS-P. 16, CDL. 9) (IN. 4/ET)"
1750 INPUT I8
1760 PRINT
1770 RETURN
1780 IF AS="TL" THEN 1810
1790 LET P7=384*E1*18/12/C1/L4*3*P9
1800 GOTO 1820
1810 LET P7=384*E1*18/12/C1/L4*3
1820 PRINT "STEP 6:"
1830 PRINT "P-SUB I = "P7" PSI"
1840 PRINT "TOP SKIN DEF. BETW. STRINGERS = "L4/C1" IN."
1850 PRINT
1860 PRINT
1870 REM *****
1880 REM ***** STEP 7 *****
1890 REM ***** (HLMSP1) *****
1900 REM *****
1910 GOTO 1970
1920 PRINT "STEP 7:"
1930 PRINT "ENTER EFFECTIVE WIDTHS (AS FLANGES) OF TOP, BOT. SKINS"
1940 PRINT " (HLMSP1, FIG. 3A: PDS3, P. 10, PT. FIG.)"
1950 INPUT W1, W2
1960 PRINT
1970 RETURN
1980 LET N15J=A1/12+M1+A2/12+M2+A3
1990 LET N16J=A1/12+M1+E1*1.1+A2/12+M2+E2*1.1+A3+E3*1.03
2000
2010 PRINT "STEP 7:"
2020 PRINT "BENDING M.M./IN. = "N15J" IN."
2030 PRINT "HLMSP1 FIG. 3A: TOTAL C (PLUS TOTAL OF AREAS)"
2040 PRINT " (N15J+N16J+N17)"
2050 PRINT
2060 REM *****
2070 REM ***** STEP 8 *****
2080 REM ***** (HLMSP1) *****
2090 REM *****
2100 LET J4=Y4-Y2
2110 LET J4=J4-Y2
2120 LET D6=J4*(Y4-Y2)
2130 LET N1=11/12+M1+A1/12+M1+D4*2
2140 LET N2=12/12+M2+A2/12+M2+D5*2
2150 LET N3=13/12+M3+D6*2
2160 LET N4=E1*1.1+N1
2170 LET N5=E2*1.1+N2
2180 LET N6=E3*1.03+N3
2190 LET N5=N4+N5+N6
2200 PRINT "STEP 8:"
2210 PRINT "E1-SUB N1 = "N1" LB-IN. 2"
2220 PRINT "HLMSP1 FIG. 3B: TOTAL S1 = "N1+E5
2230 REM *****
2240 REM ***** STEP 9 *****
2250 REM ***** (HLMSP1) *****
2260 REM *****
2270 GOTO 2330
2280 PRINT "STEPS 9 & 10:"
2290 PRINT "ENTER TOP SKIN F-SUB C & BOT. SKIN F-SUB T (PSI)"
2300 PRINT " (PDS-P. 17)"
2310 INPUT F1, F2
2320 PRINT
2330 RETURN
2340 IF L4/B1 < .5 THEN 2450
2350 IF L4/B1 < 2 THEN 2360
2360 PRINT "WARNING: STRINGER SPACING SHOULD BE <= 2 B (SUB T OR C)"
2370 PRINT " (B VALUES FROM STEP 2)"
2380 IF L4/B1 < 1 THEN 2410
2390 LET F1=F1*1-.333*(L4/B1-.5)
2400 GOTO 2450
2410 LET F1=F1*.667
2420 REM *****
2430 REM ***** STEP 10 *****
2440 REM ***** (HLMSP1) *****
2450 REM *****
2460 IF L4/B2 < .5 THEN 2570
2470 IF L4/B2 < 2 THEN 2580
2480 PRINT "WARNING: STRINGER SPACING SHOULD BE <= 2 B (SUB T OR C)"
2490 PRINT " (B VALUES FROM STEP 2)"
2500 IF L4/B2 < 1 THEN 2530
2510 LET F2=F2*1-.333*(L4/B2-.5)
2520 GOTO 2570
2530 LET F2=F2*.667
2540 REM *****
2550 REM ***** STEP 11 *****
2560 REM ***** (HLMSP1) *****
2570 REM *****
2580 PRINT "STEPS 9 & 10:"
2590 PRINT "F-SUB C AND T-RESP Y = "F1" AND "F2" PSI"
2600 GOTO 2660
2610 PRINT "STEP 11:"
2620 PRINT "ENTER OVERALL PANEL THICKNESS (IN.)"
2630 INPUT T1
2640 PRINT
2650 RETURN

```

Table A2-1 (continued)

```

2600 LET N1=8-YA/L2-L1/2+E5*(E1+1)
2610 LET N2=8-YA/L2-L1/2+E5*(E2+1)
2620 LET N3=F1+M1
2630 LET N10=F2+M2
2640 IF N1=N10 THEN 2730
2650 LET P2=M1/10
2660 LET P2=M2/10
2670 LET P2=M3/10
2680 LET P2=M4/10
2690 LET P2=M5/10
2700 LET P2=M6/10
2710 LET P2=M7/10
2720 LET P2=M8/10
2730 LET P2=M9/10
2740 LET P2=M10/10
2750 LET P2=M11/10
2760 LET P2=M12/10
2770 LET P2=M13/10
2780 LET P2=M14/10
2790 LET P2=M15/10
2800 LET P2=M16/10
2810 LET P2=M17/10
2820 LET P2=M18/10
2830 LET P2=M19/10
2840 LET P2=M20/10
2850 LET P2=M21/10
2860 LET P2=M22/10
2870 LET P2=M23/10
2880 LET P2=M24/10
2890 LET P2=M25/10
2900 LET P2=M26/10
2910 LET P2=M27/10
2920 LET P2=M28/10
2930 LET P2=M29/10
2940 LET P2=M30/10
2950 LET P2=M31/10
2960 LET P2=M32/10
2970 LET P2=M33/10
2980 LET P2=M34/10
2990 LET P2=M35/10
3000 LET P2=M36/10
3010 LET P2=M37/10
3020 LET P2=M38/10
3030 LET P2=M39/10
3040 LET P2=M40/10
3050 LET P2=M41/10
3060 LET P2=M42/10
3070 LET P2=M43/10
3080 LET P2=M44/10
3090 LET P2=M45/10
3100 LET P2=M46/10
3110 LET P2=M47/10
3120 LET P2=M48/10
3130 LET P2=M49/10
3140 LET P2=M50/10
3150 LET P2=M51/10
3160 LET P2=M52/10
3170 LET P2=M53/10
3180 LET P2=M54/10
3190 LET P2=M55/10
3200 LET P2=M56/10
3210 LET P2=M57/10
3220 LET P2=M58/10
3230 LET P2=M59/10
3240 LET P2=M60/10
3250 LET P2=M61/10
3260 LET P2=M62/10
3270 LET P2=M63/10
3280 LET P2=M64/10
3290 LET P2=M65/10
3300 LET P2=M66/10
3310 LET P2=M67/10
3320 LET P2=M68/10
3330 LET P2=M69/10
3340 LET P2=M70/10
3350 LET P2=M71/10
3360 LET P2=M72/10
3370 LET P2=M73/10
3380 LET P2=M74/10
3390 LET P2=M75/10
3400 LET P2=M76/10
3410 LET P2=M77/10
3420 LET P2=M78/10
3430 LET P2=M79/10
3440 LET P2=M80/10
3450 LET P2=M81/10
3460 LET P2=M82/10
3470 LET P2=M83/10
3480 LET P2=M84/10
3490 LET P2=M85/10
3500 LET P2=M86/10
3510 LET P2=M87/10
3520 LET P2=M88/10
3530 LET P2=M89/10
3540 LET P2=M90/10
3550 LET P2=M91/10
3560 LET P2=M92/10
3570 LET P2=M93/10
3580 LET P2=M94/10
3590 LET P2=M95/10
3600 LET P2=M96/10
3610 LET P2=M97/10
3620 LET P2=M98/10
3630 LET P2=M99/10
3640 LET P2=M100/10
3650 LET P2=M101/10
3660 LET P2=M102/10
3670 LET P2=M103/10
3680 LET P2=M104/10
3690 LET P2=M105/10
3700 LET P2=M106/10
3710 LET P2=M107/10
3720 LET P2=M108/10
3730 LET P2=M109/10
3740 LET P2=M110/10
3750 LET P2=M111/10
3760 LET P2=M112/10
3770 LET P2=M113/10
3780 LET P2=M114/10
3790 LET P2=M115/10
3800 LET P2=M116/10
3810 LET P2=M117/10
3820 LET P2=M118/10
3830 LET P2=M119/10
3840 LET P2=M120/10
3850 LET P2=M121/10
3860 LET P2=M122/10
3870 LET P2=M123/10
3880 LET P2=M124/10
3890 LET P2=M125/10
3900 LET P2=M126/10
3910 LET P2=M127/10
3920 LET P2=M128/10
3930 LET P2=M129/10
3940 LET P2=M130/10
3950 LET P2=M131/10
3960 LET P2=M132/10
3970 LET P2=M133/10
3980 LET P2=M134/10
3990 LET P2=M135/10
4000 LET P2=M136/10
4010 LET P2=M137/10
4020 LET P2=M138/10
4030 LET P2=M139/10
4040 LET P2=M140/10
4050 LET P2=M141/10

```

Table A2-1 (continued)

```

4060 INPUT L5
4070 PRINT
4080 RETURN
4090 IF L5<L2 THEN 4130
4100 PRINT "SPICE-PLATE LENGTH MUST BE SHORTER THAN PANEL WIDTH"
4110 PRINT
4120 GOSUB 4040
4130 LET P=80*(L5/L2)*Y7/L2/L1*E4/(E2*1.1)
4140 PRINT "P-SUB P = "P61" PSI"
4150 PRINT "IF P-SUB P < "P61" PSI" OR DESIGN TL, WHICHEVER"
4160 PRINT "CRITERION IS USED, SPICE-PLATE SHOULD BE REDESIGNED"
4170 PRINT "OR REDUCED (P-SUB P CALC'D ON MID-SPAN LOCATION)"
4180 GOSUB 5000
4190 PRINT "RUN A BEAM-COLUMN DESIGN (YES OR NO)"
4200 INPUT B5
4210 IF B5="NO" THEN 4190
4220 IF B5="NO" THEN 4190
4230 INPUT B5
4240 PRINT "ARE TRANSVERSE (LATERAL) LOADS ZERO (YES OR NO)"
4250 INPUT C5
4260 GOSUB 5000
4270 PRINT "YES"
4280 GOSUB 5000
4290 PRINT "NO"
4300 PRINT "TOP SKIN"
4310 PRINT "AREA STRINGS"
4320 PRINT "MOD. ARM STR"
4330 PRINT "PANEL WIDTH (PLYWOOD)"
4340 PRINT
4350 PRINT "4" "I TOP SK" "I BOT. SK" "TOTAL WIDTH STRING"
4360 PRINT "11.12*15"
4370 PRINT
4380 PRINT "5" "PANEL CL.SP." "C FACTOR" "C IN LL OR TL"
4390 PRINT "11.12*15"
4400 PRINT
4410 PRINT "6" "DESIGN DL" "I TOP SK (STRESS PERPEND. TO STR)"
4420 PRINT "1.49*18"
4430 PRINT
4440 GOTO 4500
4450 PRINT "6" "I TOP SK (STRESS PERPEND. TO STRINGERS)"
4460 PRINT "1.18"
4470 PRINT
4480 PRINT "7" "TOP SKIN" "BOT. SKIN" "EFFECTIVE WIDTHS"
4490 PRINT "4.14*2"
4500 PRINT
4510 PRINT "8" "I TOP SK-F-SUB C" "BOT. SK-F-SUB I"
4520 PRINT "1.18"
4530 PRINT
4540 PRINT "9" "OVERALL PANEL THICKNESS"
4550 PRINT "1.1"
4560 PRINT
4570 PRINT "10" "TOP SK-A" "BOT. SK-A" "BOTH OUTSIDE CRITICAL PL."
4580 PRINT "1.18*17"
4590 PRINT
4600 PRINT "11" "RELATED Y-PRIME VALUES"
4610 PRINT "1.18*17"
4620 PRINT
4630 PRINT "12" "UNGLUED" "EXT. GLUED" "INT. GLUED"

```

SUBROUTINE TO ADD BEAM-COLUMN (BC) DESIGN CAPABILITY
 SEE HLMSP1 REPORT OF 10/77 PP. A2-4 TO -6 FOR BC DESIGN STEPS.
 BC DESIGN STEPS 1-6 ARE COVERED BY HLMSP DESIGN STEPS 1-9.
 BC STEP 6 CLOSED, REQUIRING VALUES FOR 7 VARIABLES FIRST 2
 ARE PROVIDED BY STATEMENTS FOLLOWING THESE REM'S, AND OTHER 5
 ARE L2 (L-PRIME), ALL FROM PRECEDING HLMSP STEPS.

PANEL DEFLECTION AND DEFLECTION BETWEEN STRINGERS HAVE BEEN
 EXCLUDED AS TRANSVERSE LOAD CAPACITY CRITERIA BY LINE 3640-3720
 OF THIS MODIFICATION! DELETE THEM IF SUCH CRITERIA EXCLUSION IS
 UNDESIRABLE.

WHERE P-SUB DM = P-SUB M * (1 - 1/(2*U2))
 BUT, DEPENDING ON INTERIOR RISE-TIME, PERHAPS REDUCED PEARL BLAST
 PRESSURE EFFECT U2 = 1.1000 AS SUCH THINGS REDUCE THE BLAST
 LOADING TOWARDS A STATIC LOAD, I.E., WITH UNITY DYNAMIC
 MAGNIFICATION FACTOR.

SEE REPORT TEXT (REVISED APP. A2 IN NEW REPORT).
 REM * STEP 10 BC (HLMSP1) *

Table A2-1 (continued)

```

4460 REM *****
4480 LET M(9)=P54.24112/8
4490 LET Y(9)=54.24112/384/E5
4500 LET Q(9)=P5
4510 LET M(11)=Q(11)+P(9)=0
4520 REM *****
4530 REM * STEPS 11-15 BC (HMSRI) *
4540 REM *****
4550 FOR I=8 TO 2 STEP -1
4560 LET P(11)=P(11)+(9-I)/8
4570 LET M(11)=Y(9)+(1-I)/8
4580 FOR J=1 TO 50
4590 LET M(11)=9/2*(F1-P(11)/M)-P(11)+M(11)
4600 IF M(11) < M(9)-P(11)+M(11) THEN 6620
4610 LET M(11)=M(9)-P(11)+M(11)
4620 LET M(11)=54.112*(M(11)+P(11)+M(11))/48/E5
4630 IF ABS(M(11)-M(11)) < .01 THEN 6680
4640 LET M(11)=M(11)
4650 IF J=50 THEN 6670
4660 PRINT "J=50 IN STEP 13 THIS TRIAL/ERROR SOLUTION EQ. A2-7 UNSATISFACTORY"
4670 NEXT J
4680 LET Y(11)=M(11)
4690 LET Q(11)=M(11)-M(9)+Q(9)
4700 NEXT I
4710 REM * OUTPUT PORTION
4720 PRINT "STEPS 7-8 BC:"
4730 PRINT "MAX. AXIAL LOAD (W/ ZERO LATERAL LOAD), P-SUB A= "
4740 PRINT INT(P(11)*1/2)*1" LBS."
4750 IF CS="YES" THEN 4189
4760 PRINT "RELATED MID-LENGTH DEF., Y="11*INT(Y(11)*10000*1/2)/10000" IN."
4770 PRINT
4780 PRINT "STEPS 9-10 BC:"
4790 PRINT "MAX. MID-LENGTH MOMENT FROM LATERAL LOADS (WITH ZERO AXIAL LOAD)"
4800 PRINT "11*INT(M(9)/100*1/2)/101
4810 PRINT "IN-KIPS (KIP=KIL-POUND=1,000 LBS)"
4820 PRINT "RELATED DEFLECTION,"11*INT(Y(9)*100*1/2)/1001" IN., MID-LENGTH"
4830 PRINT "RELATED LATERAL LOADING, P-SUB A= "11*INT(Q(9)*1000*1/2)/1000
4840 PRINT "PSI"
4850 PRINT "STEPS 11-15 BC:"
4860 PRINT "AXIAL LOAD (PSI) (IN-KIPS) (IN-KIPS) DEFLECTION"
4870 PRINT " (LBS.) (IN-KIPS) (IN-KIPS) (IN.)"
4880 PRINT
4890 FOR I=1 TO 9
4900 PRINT USING "8.7D"11*INT(P(11)*1/2)
4910 PRINT USING "8.2X7D.D"11*INT(Q(11)*10*1/2)/10
4920 PRINT USING "8.2X7D.D"11*INT(M(11)/100*1/2)/10
4930 PRINT USING "8.2X9D.D"11*INT(P(11)+Y(11)/100*1/2)/10
4940 PRINT USING "2X7D.DD"11*INT(Y(11)*100*1/2)/100
4950 NEXT I
4960 GOSUB 5000
4970 LET T=.00274112*(144+P9*2/E5)^.5
4980 PRINT "SIMPLY SUPPORTED, EFFECTIVE NATURAL PERIOD OF (LATERAL) "
4990 PRINT "VIBRATION."
7000 PRINT "T="11*INT(T*1000000.*1/2)/100000.1" SEC."
7010 PRINT
7020 RETURN
9999 END

```

A2-13

SAMPLE PROBLEM (as used in Ref. 1)

STEP 21
ENTER 'B' DISTANCES FOR TOP & BOT. SKINS, RESP Y (IN.)
(HMSRI, F16.1B1 ALSO PDS3, P.5 (TABLE))
?38.12

ENTER CLEAR DISTANCE BETWEEN STRINGERS (F16.1B, HMSRI) (IN.)
(SHOULD BE UNIFORM IF NOT, USE LARGEST VALUE)
?13.9

STEP 31
ENTER E VALUES (PSI) FOR TOP, BOT. SKINS & STRINGERS, RESP Y
(PDS, P.17 & NDS)
?1800000, 1800000, 1800000

ENTER A// VALUES FOR TOP, BOT. SKINS (SQ IN/FT WIDTH)
(PDS, P.16, COL. 4)
?2.728, 1.914

ENTER TOTAL X-SECT. AREA OF ALL STRINGERS (SQ IN)
?32.2

ENTER MOMENT ARMS FOR TOP, BOT. SKINS & STRINGERS, RESP Y (IN.)
(FROM BOTTOM SURFACE OF PANEL)
?6.156, 3

ENTER WIDTH (PLYWOOD) OF PANEL (IN.) ?48

STEP 41
ENTER 1 IF TOP, BOT. SKINS ABOUT OWN AXES (IN. 4/FT WIDTH)
(PDS, P.16, COL. 5)
?141.025

ENTER TOTAL WIDTH OF STRINGERS (IN.) ?6

STEP 51
ENTER CLEAR SPAN L OF PANEL (IN.) ?168

ENTER C FACTOR, IN MAX. ALLOWABLE DEF., L/C
(HMSRI, STEP 61 PDS3, P.9)
?360

IS C FACTOR IN TERMS OF LL OR TL ?LL

ENTER DESIGN DL (PSI) ?0.694444

STEP 61
ENTER 1 FOR TOP SKIN (STRESS PERPENDICULAR TO STRINGERS)
(PDS, P.16, COL. 9) (IN. 4/FT)
?0.23

STEP 71
ENTER EFFECTIVE WIDTHS (AS FLANGES) OF TOP, BOT. SKINS
(HMSRI, F16.3B1 PDS3, P.10, RT. F16.)
?48.42, 3

STEPS 9 & 101
ENTER TOP SKIN F-SUB C & BOT. SKIN F-SUB T (PSI)
(PDS, P.17)
?1540, 1650

STEP 111
ENTER OVERALL PANEL THICKNESS (IN.) ?6.312

Table A2-1 (continued)

STEP 121 ENTER AREA // PLIES OUTSIDE CRITICAL PLANES, TOP & BOT. SKINS, RESP. Y (IN. 2 / 48 IN.) (ENTER 6TH COLS. HLMSP: STEP 158: P. 14)	TOP B 32	BOTTOM B 12	CL. DIST. BETW. STRINGERS 13.9
2.148, .0479	E TOP SKIN 1.8000E+06 A// BOT. SKIN 1.8000E+06 1.914 32.2 3	E STRINGERS 1.8000E+06 AREA STRINGERS 7P. SK. NOM. AREA 6 PANEL WIDTH (PLYWOOD) 48	A// TOP 2.728 BOT. DITTO .156
ENTER Y-PRIME RELATED VALUES (SAME SOURCE, 3RD & 7TH COLS. IN.) 2.148, .0479	I TOP SK. .141	I BOT. SK. .025	TOTAL WIDTH STRINGERS 6
ENTER 3 STRINGER WIDTH TOTALS (IN.) IN FOLLOWING ORDER: INCLUDED (PROTRUDING) STRINGER(S) WIDTH (TOTAL IN.), WHOSE CLEAR DIST. TO PANEL EDGE IS LESS THAN HALF GLUELINE (TOTAL IN.) WIDTH OF ALL OTHER STRINGERS. 2.75, 2.25, 3	PANEL CL. SP. 168	C FACTOR 360	C IN LL OR TL? LL
ENTER F-SUB S FOR TOP, BOT. SKINS, RESP. Y (PSI) (PDS, P. 17) 248.48	DESIGN DL 6.9444E-02	I TOP SK. (STRESS PERPEND. TO STR'S) .023	
STEP 131 ENTER ALLOWABLE HORIZ. SHEAR STRESS IN STRINGERS (PSI) (MDS) 795	TOP SKIN 48	I BOT. SKIN 42.3	EFFECTIVE WIDTHS
STEP 141 ENTER ALLOWABLE STRESS IN BEARING ON PLYWOOD BOT. FACE (PSI) (PDS, P. 17) 7340	TOP SK. F-SUB C BOT. SK. F-SUB Y 1540 1650		
STEP 151 ENTER ALLOWABLE SPLICE-PLATE MAX. STRESS (PSI) (PDS, P. 26 TABLE) 21200	OVERALL PANEL THICKNESS 6.312		
ENTER TOTAL LENGTH OF SPLICE-PLATE ACROSS PANEL (IN.) (HLMSP: STEP 158: P. 14 FIG.) 240.2	TOP SK. A// 6.06 3.83 RELATED Y-PRIME VALUES .148 UNGLUED .75 TOP SK. F-SUB S BOT. SK. F-SUB S 48 48	I BOT. SK. A// 3.83 VALUES .0479 I EXT. GLUED 2.25 I INT. GLUED 3	BOTH OUTSIDE CRITICAL PL. BOT. OUTSIDE CRITICAL PL. I INT. GLUED STRAIN WIDTHS
ANY CHANGES DESIRED (YES OR NO) 2 NO	ALLOW. HORIZ. SHEAR STRESS, STRINGERS 95		
WANT TABULATION OF INPUT VALUES (YES OR NO) 2 YES	PLYM. BOT. FACE ALLOW. BEAR. STRESS 340		
	ALLOW. SPL-PL. MAX. STRESS 1200	TOTAL X-PANEL SPL-PL. LENGTH 40.2	

Table A2-1 (concluded)

STEP 31 Y-BAR (DEFLECTION N.A.) = 3.22501 WLMSP1 FIG.2A TOTALS:1	IN.	50.768	9.64634E+07	3.11096E+08	STEPS 7-8 BC1 MAX. AXIAL LOAD (W/ ZERO LATERAL LOAD), P-SUB A= 49172. RELATED MID-LENGTH DEFL. Y= .313 IN.	LBS.
STEP 41 (E1-SUB 6) = 4.56779E+08 WLMSP1 FIG.2B TOTALS:1	LB-IN.2	235.716	4.56779E+08		STEPS 9-10 BC1 MAX. MID-LENGTH MOMENT FROM LATERAL LOADS (WITH ZERO AXIAL LOAD) RELATED DEFLECTION= .5 IN. MID-LENGTH RELATED LATERAL LOADING, P-SUB M= .439 PSI	PSI
STEP 51 P-SUB D = .476408 MADE UP OF LL = .406964 PANEL DEFL. (MID-SPAN, SS) = .466667	PSI	AND DL = 6.94444E-02	IN.		STEPS 11-15 BC1 AXIAL LOAD LATERAL LOAD/MOMENT P-DEL (A NOM. DEFLECTION (LBS.) (IN.-KIPS) (IN.-KIPS) (IN.)	
STEP 61 P-SUB T = 1.43971 TOP-SKIN DEFL. BETW. STRINGERS = 3.86111E-02	PSI	IN.				
STEP 71 Y-BAR (BENDING N.A.) = 3.28337 WLMSP1 FIG.3A TOTALS:1 (PLUS TOTAL OF AREAS:1 49.8588	IN.	3.10819E+08				
STEP 81 (E1-SUB M) = 4.39474E+08 WLMSP1 FIG.3B TOTALS:1	LB-IN.2	227.039	4.39474E+08			
STEPS 9 & 101 P-SUB C AND T, RESP. Y = 1540	AND	1100.55	PSI			
STEP 111 P-SUB B = .439327 TOP-SKIN = .666459	PSI	& BOT. SKIN = .439327	PSI			
STEP 121 TOP SKIN CONTROL:1 P-SUB S = .63600 (BOT. SKIN P-SUB S = .931003	PSI)					
STEP 131 P-SUB V = .678801 Q-SUB V = 51.3103	PSI	IN.3				
STEP 141 P-SUB M = .439327	PSI					
1-SUB E = 1.5	IN.					

A2-15

STAMP Y SUPPORTED, EFFECTIVE NATURAL PERIOD OF (LATERAL) VIBRATION:
T = .07964 SEC.

RUN ANOTHER PROBLEM (YES OR NO)

?NO

DONE

BYE

STEP 151
RESERVE PRESCRIBED MIN. LENGTHS FOR TENSION SPLICE-
PLATES (PDS1 P.26 TABLE) & OTHER LIMITATIONS (WLMSP1)
P-SUB P = .424524 PSI
IF P-SUB P < P-SUB M (.439327) OR DESIGN TL, WHICHEVER
CRITERION IS USED, SPLICE-PLATE SHOULD BE REDESIGNED
OR RELOCATED (P-SUB P CALC'D ON MID-SPAN LOCATION)

RUN A BEAM-COLUMN DESIGN (YES OR NO)

?YES

ARE TRANSVERSE (LATERAL) LOADS ZERO (YES OR NO)

?NO

The PSSP designs in Table A2-2* show the results of using the computer program (Table A2-1) for columns and beam-columns. These results include use of the recommended increases in normal-use design stresses⁸ and use of the recommended dynamic factors for blast loads (section B above, second paragraph).

An illustrative example is shown on the page following Table A2-2.

* Prepared by J. E. Beck of James E. Beck and Associates (subcontractor).

Table A2-2A PSSP DESIGNS FOR LOWER STRENGTH STRINGERS ($F_v = 280$ psi)
(Columns and Beam-Columns)

Panel Width: 48 in.

Top & Bottom Skins		Stringers		Req'd* Bearing Length Ea. End	Panel Height = 7 ft					Panel Height = 8 ft					Panel Height = 9 ft				
					P _{da} (kips)	P _{dm} (psi) for P _d /P _{da} =				P _{da} (kips)	P _{dm} (psi) for P _d /P _{da} =				P _{da} (kips)	P _{dm} (psi) for P _d /P _{da} =			
						0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2
1/2	#1	4	2x4	4.5	130.7	0.6	0.9	1.0	1.2	128.5	0.4	0.7	0.9	1.0	126.5	0.2	0.5	0.7	0.8
		5		4.0	169.5	0.6	1.0	1.2	1.4	166.4	0.4	0.7	0.9	1.1	163.4	0.2	0.5	0.7	1.0
		7		3.5	219.5	0.6	1.1	1.4	1.7	214.8	0.3	0.7	1.1	1.4	191.1	0.2	0.5	0.9	1.2
		9		3.5	268.0	0.5	1.1	1.6	2.0	261.5	0.2	0.7	1.2	1.7	203.1	0.3	0.7	1.1	1.5
1/2	#3	4	2x4	4.5	84.0	0.4	0.7	1.0	1.1	82.6	0.3	0.5	0.8	0.9	81.3	0.2	0.4	0.6	0.8
		5		4.0	109.0	0.4	0.8	1.1	1.3	107.0	0.3	0.5	0.9	1.1	105.0	0.1	0.3	0.6	0.9
		7		3.5	141.1	0.4	0.8	1.3	1.6	138.1	0.2	0.5	0.9	1.4	135.1	0.1	0.3	0.6	1.0
		9		3.0	172.3	0.4	0.8	1.3	1.9	168.1	0.2	0.5	0.9	1.5	148.5	0.1	0.4	0.7	1.2
3/4	#1	4	2x4	5.0	152.3	0.6	1.1	1.2	1.3	150.0	0.5	0.9	1.0	1.1	147.9	0.4	0.7	0.8	1.0
		5		4.5	176.5	0.6	1.2	1.4	1.6	175.5	0.5	1.0	1.1	1.3	172.7	0.3	0.7	0.9	1.1
		7		4.0	229.3	0.8	1.5	1.7	2.0	224.8	0.5	1.0	1.4	1.7	220.5	0.3	0.6	1.1	1.4
		9		4.0	276.5	0.8	1.5	2.0	2.3	272.2	0.4	0.9	1.6	2.0	264.2	0.2	0.6	1.2	1.7
3/4	#3	4	2x4	5.0	97.9	0.5	0.9	1.2	1.3	96.4	0.4	0.7	1.0	1.1	95.1	0.2	0.5	0.8	0.9
		5		4.5	114.7	0.5	1.0	1.3	1.5	112.9	0.4	0.7	1.1	1.3	111.0	0.2	0.5	0.8	1.1
		7		4.0	147.4	0.5	1.0	1.5	1.9	144.5	0.3	0.7	1.1	1.6	141.7	0.2	0.5	0.8	1.2
		9		3.5	179.0	0.5	1.0	1.6	2.2	175.0	0.3	0.7	1.1	1.7	171.2	0.2	0.4	0.8	1.3
1-1/8	#1	4	2x4	5.5	170.1	1.2	1.4	1.5	1.6	168.0	0.8	1.1	1.2	1.4	166.0	0.6	0.9	1.1	1.2
		5		5.0	197.2	1.2	1.6	1.7	1.9	194.5	0.8	1.3	1.5	1.6	191.9	0.6	1.1	1.2	1.4
		7		5.0	249.8	1.2	2.0	2.2	2.4	245.7	0.8	1.5	1.8	2.1	241.8	0.6	1.1	1.5	1.8
		9		4.5	300.8	1.2	2.2	2.6	2.9	295.1	0.8	1.5	2.1	2.5	289.6	0.5	1.1	1.8	2.1

* For full lateral load and zero axial load; use 20% axial load (i.e., $P_d = 0.2P_{da}$). Reduce required bearing length (each end) with larger axial loads, varying in accordance with change in p_{dm} values; use minimum of 2 in. each end, however. For example, using data for 1/2 in. #3, 2x4s, 4-stringer, 7-ft long PSSP, and axial load = $0.6P_{da}$: required bearing length each end is $4.5(0.7/1.1) = 2.9$ in. each end. The bearing length values were obtained from Table A1-1A.

P_{da} is the maximum axial load, that is, when lateral load, $p_{dm} = 0$.

Table A2-2A (Concluded)

Panel Width: 48 in.

Top & Bottom Skins			Stringers	Req'd* Bearing Length Ea. End	Panel Height = 7 ft				Panel Height = 8 ft				Panel Height = 9 ft							
					Nom. Thick.	Face Ply	Nom. Size	No.	P _{da} (kips)	P _{dm} (psi) for P/P _{da} =			P _{da} (kips)	P _{dm} (psi) for P/P _{da} =			P _{da} (kips)	P _{dm} (psi) for P/P _{da} =		
0.8	0.6	0.4	0.2	0.8						0.6	0.4	0.2		0.8	0.6	0.4		0.2		
1/2	#1	2x6	4	7.5	201.2	0.7	1.3	1.8	2.3	2.0	199.4	0.8	1.4	1.6	1.7	197.6	0.6	1.1	1.3	1.5
			5	6.5	266.7	1.2	2.0	2.2	2.4	2.4	264.0	0.8	1.6	1.8	2.0	261.3	0.6	1.2	1.5	1.8
			7	6.0	356.2	1.3	2.5	2.8	3.1	3.1	352.0	0.9	1.8	2.3	2.7	347.8	0.6	1.2	1.9	2.3
			9	5.5	444.6	1.4	2.8	3.4	3.8	3.8	438.7	0.9	1.9	2.8	3.2	433.0	0.6	1.3	2.3	2.8
1/2	#3	2x6	4	7.0	129.4	0.7	1.3	1.8	2.3	2.0	128.2	0.5	1.0	1.5	1.7	127.0	0.4	0.7	1.2	1.4
			5	6.5	171.4	0.8	1.5	2.1	2.5	2.3	169.7	0.6	1.1	1.7	2.0	168.0	0.4	0.8	1.3	1.7
			7	5.5	229.0	0.9	1.7	2.6	3.0	3.0	226.3	0.6	1.2	1.9	2.6	223.6	0.4	0.9	1.4	2.1
			9	5.5	285.8	1.0	1.9	2.9	3.7	3.7	282.0	0.7	1.3	2.1	3.1	278.4	0.4	0.9	1.6	2.4
3/4	#1	2x6	4	8.0	229.7	1.4	1.9	2.0	2.1	2.1	227.8	1.0	1.6	1.7	1.8	225.9	0.7	1.3	1.4	1.6
			5	7.0	275.3	1.4	2.2	2.4	2.6	2.6	272.8	1.0	1.8	2.0	2.2	270.3	0.7	1.4	1.7	1.9
			7	6.5	365.3	1.6	2.8	3.1	3.4	3.4	361.3	1.1	2.1	2.6	2.9	357.3	0.7	1.5	2.2	2.5
			9	6.0	454.1	1.7	3.1	3.8	4.1	4.1	448.4	1.1	2.2	3.1	3.5	442.9	0.7	1.6	2.6	3.0
3/4	#3	2x6	4	7.5	147.7	0.9	1.7	2.0	2.1	2.1	146.5	0.7	1.3	1.7	1.8	145.3	0.5	0.9	1.4	1.6
			5	7.0	177.0	1.0	1.8	2.3	2.5	2.5	175.4	0.7	1.3	2.0	2.1	173.7	0.5	1.0	1.5	1.9
			7	6.0	234.9	1.1	1.9	3.0	3.3	3.3	232.3	0.7	1.4	2.2	2.8	229.7	0.5	1.0	1.7	2.4
			9	5.5	291.9	1.1	2.1	3.2	4.0	4.0	288.3	0.8	1.5	2.4	3.4	284.7	0.5	1.1	1.8	2.6
1-1/8	#1	2x6	4	8.5	247.3	1.9	2.2	2.3	2.4	2.4	245.6	1.4	1.8	1.9	2.1	243.9	1.0	1.6	1.7	1.8
			5	8.0	293.6	1.9	2.6	2.7	2.9	2.9	291.3	1.4	2.2	2.3	2.5	289.0	1.1	1.8	2.0	2.2
			7	7.0	384.8	2.1	3.4	3.6	3.8	3.8	381.2	1.5	2.8	3.0	3.3	377.6	1.1	2.1	2.6	2.9
			9	6.5	474.6	2.2	4.0	4.4	4.7	4.7	469.5	1.5	2.9	3.7	4.0	464.4	1.1	2.1	3.1	3.5
1-1/8	#1	2x8	4	11.0	313.7	2.6	3.0	3.1	3.2	3.2	312.2	1.9	2.5	2.6	2.7	310.7	1.5	2.2	2.3	2.4
			5	10.5	376.7	2.7	3.6	3.7	3.9	3.9	374.6	2.0	3.0	3.2	3.4	372.6	1.5	2.6	2.8	2.9
			7	9.5	501.5	3.0	4.8	5.0	5.2	5.2	498.3	2.2	4.0	4.3	4.5	495.1	1.6	3.2	3.7	4.0
			9	9.0	625.3	3.2	5.9	6.2	6.6	6.6	620.7	2.4	4.5	5.3	5.7	616.2	1.7	3.4	4.5	4.9

Table A2-2B PSSP DESIGNS FOR HIGHER STRENGTH STRINGERS ($F_v = 380$ psi)
(Columns and Beam-Columns)

Panel Width: 48 in.

Top & Bottom Skins		Stringers		Req'd* Bearing Length Ea. End	Panel Height = 7 ft				Panel Height = 8 ft				Panel Height = 9 ft										
					P _{dm} (psi) for P _d /P _{da} =				P _{dm} (psi) for P _d /P _{da} =				P _{dm} (psi) for P _d /P _{da} =										
					P _{da} (kips)	0.8	0.6	0.4	0.2	P _{da} (kips)	0.8	0.6	0.4	0.2	P _{da} (kips)	0.8	0.6	0.4	0.2				
1/2	#1	2x4	4	3.5	130.7	0.6	1.1	1.4	1.6	0.4	0.7	1.2	1.3	126.5	0.4	0.7	1.2	1.3	126.5	0.2	0.5	0.9	1.2
			5	3.5	109.5	0.6	1.1	1.6	1.8	0.4	0.8	1.3	1.5	166.4	0.4	0.8	1.3	1.5	163.4	0.2	0.5	0.9	1.3
			7	3.0	219.5	0.6	1.1	1.9	2.3	0.3	0.7	1.3	1.9	214.8	0.3	0.7	1.3	1.9	191.1	0.2	0.5	1.0	1.7
			9	2.5	268.0	0.5	1.1	2.0	2.7	0.2	0.7	1.3	2.2	261.5	0.2	0.7	1.3	2.2	203.1	0.3	0.7	1.2	1.8
1/2	#3	2x4	4	3.5	84.0	0.4	0.7	1.1	1.5	0.3	0.5	0.8	1.2	82.6	0.3	0.5	0.8	1.2	81.3	0.2	0.4	0.6	0.9
			5	3.0	109.0	0.4	0.8	1.2	1.7	0.3	0.5	0.9	1.3	107.0	0.3	0.5	0.9	1.3	105.0	0.1	0.3	0.6	1.0
			7	2.5	141.1	0.4	0.8	1.3	1.9	0.2	0.5	0.9	1.4	138.1	0.2	0.5	0.9	1.4	135.1	0.1	0.3	0.6	1.0
			9	2.5	172.3	0.4	0.8	1.3	2.0	0.2	0.5	0.9	1.5	168.1	0.2	0.5	0.9	1.5	148.5	0.1	0.4	0.7	1.2
3/4	#1	2x4	4	3.5	152.3	0.6	1.2	1.3	1.5	0.5	1.0	1.1	1.3	150.0	0.5	1.0	1.1	1.3	147.9	0.4	0.7	0.9	1.1
			5	3.5	176.5	0.6	1.5	1.8	2.0	0.5	1.0	1.5	1.7	175.5	0.5	1.0	1.5	1.7	172.7	0.3	0.7	1.2	1.5
			7	3.0	229.3	0.8	1.5	2.3	2.7	0.5	1.0	1.7	2.3	224.8	0.5	1.0	1.7	2.3	220.5	0.3	0.6	1.2	1.9
			9	2.5	278.5	0.8	1.5	2.4	3.2	0.4	0.9	1.7	2.6	272.2	0.4	0.9	1.7	2.6	264.2	0.2	0.6	1.2	2.0
3/4	#3	2x4	4	3.5	97.9	0.5	0.9	1.4	1.5	0.4	0.7	1.1	1.3	96.4	0.4	0.7	1.1	1.3	95.1	0.2	0.5	0.8	1.1
			5	3.5	114.7	0.5	1.0	1.5	2.0	0.4	0.7	1.1	1.6	112.9	0.4	0.7	1.1	1.6	111.0	0.2	0.5	0.8	1.2
			7	3.0	147.4	0.5	1.0	1.5	2.2	0.3	0.7	1.1	1.6	144.5	0.3	0.7	1.1	1.6	141.7	0.2	0.5	0.8	1.2
			9	2.5	179.0	0.5	1.0	1.6	2.3	0.3	0.7	1.1	1.7	175.0	0.3	0.7	1.1	1.7	171.2	0.2	0.4	0.8	1.3
1-1/8	#1	2x4	4	3.5	170.1	1.2	1.4	1.5	1.6	0.8	1.1	1.2	1.4	168.0	0.8	1.1	1.2	1.4	166.0	0.6	0.9	1.1	1.2
			5	3.5	197.2	1.2	1.8	2.0	2.1	0.8	1.5	1.7	1.8	194.5	0.8	1.5	1.7	1.8	191.9	0.6	1.1	1.4	1.6
			7	4.0	249.8	1.2	2.2	3.0	3.2	0.8	1.5	2.4	2.8	245.7	0.8	1.5	2.4	2.8	241.8	0.6	1.1	1.8	2.4
			9	3.5	300.8	1.2	2.2	3.4	4.0	0.8	1.5	2.4	3.4	295.1	0.8	1.5	2.4	3.4	289.6	0.5	1.1	1.8	2.7

Table A2-2B (Concluded)

Panel Width: 48 in.

Top & Bottom Skins		Stringers	Req'd* Bearing Length Ea. End	Panel Height = 7 ft				Panel Height = 8 ft				Panel Height = 9 ft						
				P _{dm} (psi) for P _d /P _{da} =				P _{dm} (psi) for P _d /P _{da} =				P _{dm} (psi) for P _d /P _{da} =						
Nom. Thick.	Face Ply	Nom. Size	No.	P _{da} (kips)	0.8	0.6	0.4	0.2	P _{da} (kip.)	0.8	0.6	0.4	0.2	P _{da} (kips)	0.8	0.6	0.4	0.2
1/2	#1	2x6	4	201.2	1.1	2.1	2.5	2.7	199.4	0.8	1.5	2.1	2.3	197.6	0.6	1.1	1.8	2.0
			5	266.7	1.2	2.3	3.0	3.2	264.0	0.8	1.7	2.5	2.8	261.3	0.6	1.2	2.1	2.4
			7	356.2	1.3	2.5	3.8	4.2	352.0	0.9	1.8	2.9	3.6	347.8	0.6	1.2	2.1	3.1
			9	444.6	1.4	2.8	4.4	5.2	438.7	0.9	1.9	3.1	4.4	433.0	0.6	1.3	2.1	3.6
1/2	#3	2x6	4	129.4	0.7	1.3	2.0	2.6	128.2	0.5	1.0	1.5	2.1	127.0	0.4	0.7	1.2	1.6
			5	171.4	0.8	1.5	2.4	3.1	169.7	0.6	1.1	1.7	2.5	168.0	0.4	0.8	1.3	1.9
			7	229.0	0.9	1.7	2.6	3.7	226.3	0.6	1.2	1.9	2.8	223.6	0.4	0.9	1.4	2.1
			9	285.8	1.0	1.9	2.9	4.1	282.0	0.7	1.3	2.1	3.1	278.4	0.4	0.9	1.6	2.4
3/4	#1	2x6	4	229.7	1.4	2.2	2.3	2.5	227.8	1.0	1.8	2.0	2.1	225.9	0.7	1.4	1.7	1.9
			5	275.3	1.4	2.7	3.2	3.4	272.8	1.0	2.0	2.7	3.0	270.3	0.7	1.4	2.3	2.6
			7	365.3	1.6	2.9	4.2	4.6	361.3	1.1	2.1	3.3	3.9	357.3	0.7	1.5	2.5	3.4
			9	454.1	1.7	3.1	4.9	5.6	448.4	1.1	2.2	3.6	4.8	442.9	0.7	1.6	2.6	4.0
3/4	#3	2x6	4	147.7	0.9	1.7	2.5	2.6	146.5	0.7	1.3	1.9	2.3	145.3	0.5	0.9	1.5	2.0
			5	177.0	1.0	1.8	2.7	3.4	175.4	0.7	1.3	2.0	2.8	173.7	0.5	1.0	1.7	2.2
			7	234.5	1.1	1.9	3.0	4.1	232.5	0.7	1.4	2.2	3.1	229.7	0.5	1.0	1.7	2.1
			9	291.9	1.1	2.1	3.2	4.5	288.3	0.8	1.5	2.4	3.4	284.7	0.5	1.1	1.8	2.1
1-1/8	#1	2x6	4	247.3	1.9	2.5	2.4	2.5	245.6	1.4	1.9	2.0	2.2	243.9	1.0	1.8	1.8	1.9
			5	293.0	1.9	3.1	3.2	3.4	291.3	1.4	2.6	2.8	2.9	289.0	1.1	2.0	2.4	2.6
			7	384.8	2.1	3.8	4.9	5.2	381.2	1.5	2.8	4.1	4.5	377.0	1.1	2.1	3.3	3.9
			9	474.6	2.2	4.0	5.9	6.4	469.5	1.5	2.9	4.5	5.5	464.4	1.1	2.1	3.4	4.7
1-1/8	#1	2x8	4	313.7	2.6	3.4	3.3	3.4	312.2	1.9	2.7	2.8	2.9	310.7	1.5	2.3	2.5	2.6
			5	376.7	2.7	4.4	4.5	4.7	374.6	2.0	3.7	3.9	4.1	372.6	1.5	2.9	3.4	3.6
			7	501.5	3.0	5.0	6.8	7.1	498.3	2.2	4.2	5.8	6.2	495.1	1.6	3.2	4.9	5.4
			9	625.3	3.2	6.1	8.5	8.9	620.7	2.4	4.5	6.9	7.7	616.2	1.7	3.4	5.5	6.7

PSSP Upgrading Example - Column

Try one of the PSSP designs as an added column in upgrading a basement for shelter. Assume a closed shelter, therefore no blast or other lateral load on the PSSP.

Assume use of a PSSP design on p. A2-18 that is, the one with $\frac{1}{2}$ " th. #3 top and bottom skins with 4 lower strength 2x4 stringers. Table A2-2A shows a max. column load P of 84 kips, which includes an increase of normal use design by 4x and use of $\mu=1$ (see p. A2-7, 4th paragraph).

Thus, for example, the 84^k column, supporting a floor contributory area of 5' by 6', or 30 sq', will take a blast loading on the floor over the basement of:

$$p_s = \frac{84000}{30} \times \frac{1}{144} = 19 \text{ psi}$$

A more precise figure, used below for demonstration and not to imply a degree of accuracy, would be:

$$p_s = 19.44 \text{ psi (blast loading)}$$

If soil loads are added on the floor for fallout shielding, say 200 psf:

$$P(\text{static loads}) = P_s \times \frac{1}{4} \times 2 = \frac{19.44}{4} \times 2 = 9.72 \text{ psi}$$

where $\frac{1}{4}$ represents removing the multiple of 4 taken (in the table) for blast loads, and 2 represents removing the effect of $\mu=1$ in preparing the table.

The fallout soil, if left on the floor for less than 2 mos. merits a 15% increase in allowable stress, thus, the remaining static load capacity

$$P(\text{static loads}) = 9.72 - \left[\frac{200 \times \frac{1}{144}}{1.15} \right] \\ = 9.72 - 1.21 = 8.51 \text{ psi}$$

Reconverted to blast resistance capacity (but with 200 psf soil added to the floor load):

$$p_s = 8.51 \times 4 \times \frac{1}{2} = 17 \text{ psi (blast loading)}$$

NOTATION

- A = total x-sectional area of longitudinal grain material in both plywood skins and stringers (in.²)
- b = least dimension of solid rectangular cross-section (in.)
- c = distance from N.A. (bending) to extreme fiber in compression (in.)
- d = greater dimension of solid rectangular cross-section (in.)
- d' = depth of PSSP stringers (in.)
- E = modulus of elasticity (psi)
- (EI_g) = stiffness factor for moment deflection [1(Sec.2.4.3)] (lbs-in.² for full panel) (from Step 4, Appendix A1)
- (EI_n) = stiffness factor for bending moment [1(Sec.2.5.3)] 1lb-in.² for full panel) (from Step 8, Appendix A1)
- F_c = allowable compressive stress (parallel to grain) for plywood skins (psi) [2(p.17)], corrected for buckling [1(Sec.2.5.4)]
- I_g = gross moment of inertia of cross-section (in.⁴)
- I_n = bending moment of inertia of full panel (in.⁴)
- ℓ = clear span of member (simply-supported/pin-ended) (in.)
- ℓ' = width of PSSP skins (perpendicular to stringers) (in.)
- M = allowable bending moment (in.-lb), under combined loading
- M_{max} = maximum moment caused by transverse loads only (in.-lb)
- P = allowable axial load (lbs), under combined loading
- P_a = allowable axial load (lbs), if axial load only exists
- P_d = dynamic (blast) load value related to P (lbs)
- P_{da} = dynamic (blast) load value related to P_a (lbs)
- P_m = smallest of calculated allowable transverse (only) loads (in PSSPs, calculated loads for: deflection, bending moment, rolling shear and horizontal shear) (psi) (from Appendix A1 design of PSSPs)
- P_{dm} = dynamic (blast) load value related to p_m (psi)

NOTATION (concluded)

- p'_m = an intermediate value between p_m and zero (psi)
 p'_{dm} = dynamic (blast) load value corresponding to p'_m (psi)
 r = radius of gyration (in.)
 S = section modulus of full panel (in.³)
 x = location being examined (length along member) (in.)
 y = deflection of column at M (in.)
 $\bar{y}$ = deflection of column at M_{max} (transverse loads only) (in.)
 μ = ductility ratio = x_m / x_e (see Appendix A)

REFERENCES

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9. Newmark, N. M., Design of Openings for Buried Shelters, Report 2-67, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1963; p. 151-182.

¹⁰ Now SRI International

¹¹ Now Federal Emergency Management Agency

Appendix A3

PLYWOOD USE FOR CLOSURES - DESIGN

Extracts (with minor revisions) from the main text and Appendix A1 of

Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects:
Predesigned Expedient Options, Stanford Research Institute* Technical
Report, for U.S. Defense Civil Preparedness Agency, # October 1977.
(AD-A054 409)

* Now SRI International

Now Federal Emergency Management Agency

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Background

Appendices A1 and A2, in their early paragraphs, describe plywood uses toward meeting the need for expedient aperture closures and added overhead floor system supports, respectively, in the upgrading of existing basements for shelter use against the combined effects of a nuclear weapons detonation. This Appendix A3 closes the area of plywood applications by describing a design approach for simple use of plywood for closures, especially over those many shelter openings having a rather short span in at least one of its two directions.

Approach

Use was made of two publications and telephone discussions¹⁻³ in developing a design procedure for use of plywood to close apertures in existing basements. The tables of the simplified publication² could not be reproduced through use of the design manual¹ procedures; requested clarification brought the recommendation that the latter be used for the purposes contemplated herein.³

As before, in Appendices A1 and A2, design formulas¹(pp.22-3,Sec.4) were converted to the Notation herein and made dimensionally consistent. The revised formulas follow; plywood weight is ignored as dead load, and single spans, uniform loads, and simple supports are assumed.

The user is cautioned to apply care in units used in entering all values in the equations below; all equations are dimensionally consistent, i.e., there are no units hidden in the constants.

A. For uniform loads based on allowable bending stress:

$$p_b = 8 F_b S / \ell^2 \quad (\text{Eq.A3-1})^\dagger$$

where*

p_b = allowable load - bending moment (psi)

F_b = allowable bending stress (psi)

* Variables are defined herein at point of first use and in Notation at end of appendix.

† In Eq. A3-1, clear span can be used (per Reference 3 fonecon of 1/6/78).

S = effective section modulus ($\text{in.}^3/\text{in. width}$)

ℓ = clear span (in.)

- B. For uniform loads based on allowable rolling shear stress:

$$p_s = 2 F_s (Ib/Q) / \ell \quad (\text{Eq. A3-2})$$

where

p_s = allowable load - rolling shear stress (psi)

F_s = allowable rolling shear stress (psi)

(Ib/Q) = rolling shear constant ($\text{in.}^2/\text{in. width}$)

ℓ = clear span (in.)

The useful allowable load p_m then becomes:

$$p_m = p_b \text{ or } p_s \text{ whichever is smaller (psi)} \quad (\text{Eq. A3-3})$$

- C. For bending deflection (elastic) under uniform load:

$$y_b = p_m \ell^4 / (76.8 I (1.1 E)) \quad (\text{Eq. A3-4})^*$$

where

y_b = bending deflection (elastic) under uniform load (in.)

I = effective moment of inertia ($\text{in.}^4/\text{in. width}$)

E = modulus of elasticity (psi)

- D. For shear deflection (elastic) under uniform load:

$$y_s = p_m C t^2 \ell^2 / (106 EI) \quad (\text{Eq. A3-5})$$

where

y_s = shear deflection (elastic) under uniform load (in.)

C = 120 or 60, for panels applied with face grain perpendicular to or parallel to supports, respectively.

t = nominal panel thickness (in.)

- E. For combined bending and shear deflection (elastic) under uniform load: either (a) add y_b and y_s from Equations 4 and 5; or (b) use Equation 4 only, but with the constant 1.1 dropped from the equation.

* Modified very slightly as to ℓ from Ref. 1, for simplification and because of negligible effect on the uses made of deflection calculations herein.

F. For plywood face bearing under uniform load (at ends over simple supports):

$$l_e = l / (2(F_{c\perp}/p_m - 1)) \quad (\text{Eq.A3-6})$$

where

l_e = required plywood (face) end bearing length at each end of panel (in.)

$F_{c\perp}$ = allowable bearing stress on plywood face, for load perpendicular to plane of outer ply actually in bearing (psi)

It is recommended that l_e be at least 1.5 in.

Design Procedure

The suggested design procedure consists of the following Steps:

1. Assume use of a particular plywood type, grade, nominal thickness t , and face ply(ies) species group (pp. 9, 14 and 15)[†] except that the latter must not be #5. Also assume that panel is uniformly loaded and simply supported,[‡] and assume value for span l (in.).

Neglect the plywood weight as a DL.

2. Determine values (p. 16)[†] for I , S (=KS), and (Ib/Q) , taking care to correct the units to in.^4 , in.^3 , and in.^2 (all per in. width), respectively. Take care to use proper values for plywood used with the face grain running parallel to the span (cols. 5-7)[†] or perpendicular to the span (col. 9-11),[†] as well as the appropriate table (1 or 2)[†] and section (Unsanded, Sanded, or Touch-Sanded Panels).[†] If permitted by available supplies, plywood panels are used with the face grain running parallel to the span, which takes advantage of the stronger direction of the plywood.

[†] See Reference 1; it is necessary that the designer hold this reference.

[‡] On two opposite sides; but Step 7 extends the procedure to plywood panels supported on four sides.

3. Study the plywood data (p.14)* and select appropriate use condition (Wet or Dry) and grade stress level (S-1, -2, or -3). Determine values (p. 17)* for F_b , F_s , $F_{c\perp}$, and E (all psi).
4. Solve Equations 1-3 for p_b , p_s , and p_m , respectively.
5. Solve Equation 6 for l_e .
6. If deflections are needed or desired, either:
 - (a) Solve Equations 4 and 5 for y_b and y_s , respectively ; then $y = y_b + y_s$; or
 - (b) Solve Equation 4 with the value 1.1 deleted on right side; then $y = y_b$.
7. For plywood panels supported on four sides, the procedure is as follows:^{2,3}
 - (a) Complete Steps 1-4 and 6 for each span direction, finding p_m and y for each direction;
 - (b) Reduce the p_m value associated with the larger y , by multiplying that p_m by the ratio of the smaller y to the larger y . The two y values will then be equal, and the total capacity p_m of the panel supported on four sides will be the sum of the p_m just reduced and the unchanged p_m associated with the smaller y of Step 7a; use the latter two p_m values to find l_e in each direction (Step 5).

Design Stresses - Blast Protection Use versus Normal Use

An Appendix A1 section with the same title applies fully herein, excepting that $\mu = 3$ is recommended for this appendix; thus, $p_{dm} = (5/6) p_m$, and F_b and F_s (but not $F_{c\perp}$ and E) are multiplied by four.

Typical Designs of Plywood Panels as Closures

Data in the preceding sections have been used to prepare the typical designs of plywood panels as closures shown in Tables A3-1A, -1B, -1C and -2. Computer programs used are listed in Table A3-3 of the original publication but not extracted herein.

* See Reference 1; it is necessary that the designer hold this reference.

Table A3-1A PLYWOOD PANELS AS CLOSURES (ONE-WAY)

Plywood panels considered herein are each stamped with American Plywood Association (APA) Type (Interior or Exterior), Grade and, in most cases, with Face Ply Species Group(s) (the latter exception is discussed further below), as follows:

<u>Plywood Type and Grade</u>	<u>Table A3-1B&C Block Nos.</u>
C-D INTERIOR (APA), * usual:	3,11
If "interior with exterior glue" is specified:	2,10
UNDERLAYMENT INTERIOR (APA), usual:	8,16
If "interior with exterior glue" is specified:	7,15
C-D PLUGGED INTERIOR (APA), usual:	8,16
If "interior with exterior glue" is specified:	7,15
2.4.1 STURD-I-FLOOR INTERIOR (APA), with veneer inner plies only	17
APPEARANCE GRADES (Interior) (APA), † usual:	6,14
If "interior with exterior glue" is specified:	5,13
C-C EXTERIOR (APA)*	1,9
UNDERLAYMENT EXTERIOR (APA)	7,15
C-C PLUGGED EXTERIOR (APA)	7,15
2.4.1 STURD-I-FLOOR EXTERIOR (APA), with veneer inner plies only	17
APPEARANCE GRADES (Exterior) (APA), ‡ with Surface A or C, face & back:	4,12
With Surface B face or back:	5,13

* Face Ply Species Groups are as follows: When stamped 24/0 on 1/2 in. (13 mm) thick plywood, Group 4; 32/16, Group 1; on 3/4 in. (19 mm): 42/20, Group 3; 48/24, Group 1.

† Generally applied where a high quality surface is required; includes N-N, N-A, N-B, N-D, A-A, A-B, A-D, B-B and B-D INTERIOR (APA) Grades.

‡ Generally applied where a high quality surface is required; includes A-A, A-B, A-C, B-B, B-C, HDO and MDO EXTERIOR (APA) Grades.

Table A3-1B PLYWOOD PANELS AS CLOSURES (ONE-WAY)*

Block No.	PLYWOOD				(SIDE-ON) PEAK AIR BLAST OVERPRESSURE VS. CL. SPAN												
	Nom. Th. in.	Surface Finish	Grade Str. Level	Face Ply Grp	Clear Span, in.												
					4	6	8	10	12	14	16	18	20	22	24	26	28
1.	1/2	UNSANDED	S-1	1 2,3 4	31 31 31	21 21 20	15 12 11	11 8 7	8 5 5	6 psi							
2.	1/2	UNSANDED	S-2	1 2,3 4	31 31 31	21 18 17	14 10 10	9 7 6	6 5 5	5							
3.	1/2	UNSANDED	S-3	1 2,3 4	28 28 28	19 18 17	14 10 10	9 7 6	6 5 5	5							
4.	1/2	SANDED	S-1	1 2,3 4	36 36 36	24 23 22	18 13 12	12 8 8	8 6 5	6 5							
5.	1/2	SANDED	S-2	1 2,3 4	36 36 36	24 20 18	15 11 10	10 7 7	7 5 5	5							
6.	1/2	SANDED	S-3	1 2,3 4	32 32 32	21 20 18	15 11 10	10 7 7	7 5 5	5							
7.	1/2	TOUCH-S.	S-2	1 2,3 4	31 31 31	21 20 19	16 11 10	10 7 7	7 5 5	5							
8.	1/2	TOUCH-S.	S-3	1 2,3 4	28 28 28	19 19 19	14 11 10	10 7 7	7 5 5	5							
9.	3/4	UNSANDED	S-1	1 2,3 4	50 50 50	33 33 33	25 24 23	20 16 15	15 11 10	11 8 8	9 6 6	7 5 5	6 5				
10.	3/4	UNSANDED	S-2	1 2,3 4	50 50 50	33 33 33	25 21 19	18 13 12	13 9 9	9 7 6	7 5 5	6 5	5				
11.	3/4	UNSANDED	S-3	1 2,3 4	45 45 45	30 30 30	23 21 19	18 13 12	13 9 9	9 7 6	7 5 5	6 5	5				
12.	3/4	SANDED	S-1	1 2,3 4	58 58 58	39 39 37	29 22 21	20 14 13	14 10 9	10 7 7	8 5 5	6 5	5				
13.	3/4	SANDED	S-2	1 2,3 4	58 58 58	39 33 31	26 19 17	17 12 11	12 8 8	8 6 6	6 5	5					
14.	3/4	SANDED	S-3	1 2,3 4	53 53 53	35 33 31	26 19 17	17 12 11	12 8 8	8 6 6	6 5	5					
15.	3/4	TOUCH-S.	S-2	1 2,3 4	51 51 51	34 34 33	25 20 18	17 13 12	12 9 8	9 6 6	7 5 5	5					
16.	3/4	TOUCH-S.	S-3	1 2,3 4	46 46 46	31 31 31	23 20 18	17 13 12	12 9 8	9 6 6	7 5 5	5					
17.	1-1/8	TOUCH-S.	S-2	1-3	52	39	31	25	19	14	11	9	8	6	5	5	

* Face ply grain running in span direction (i.e., perpendicular to the two supports).
Required bearing length at each end (beyond clear span) is 1½ in. (38 mm) in all cases.

Table A3-2 PLYWOOD PANELS AS CLOSURES (TWO-WAY)

The purpose of this Table is to provide conversion percentages (increases) so that the user can use the data of Table A3-1B to obtain overpressure versus clear span data for two-way plywood panels (that is, supported on all four sides of the opening/aperture to be closed).

This Table is based on using plywood panels with their face ply grain running in the direction of the shorter of the aperture's two clear spans. Its results are expressed in terms of the ratio of the longer to the shorter of the two clear spans; such results are expressed as percentage increases in overpressure resistance values applied to the values in Table A3-1B, with such increases related to the BLOCK NUMBERS of the table.

Recommended support bearing length on all four sides is $1\frac{1}{2}$ in.

TABLES A3-1B&C <u>BLOCK NUMBERS</u>	RATIO OF LONGER TO SHORTER CLEAR SPANS			
	<u>1:1</u>	<u>1.25:1</u>	<u>1.5:1</u>	<u>2:1</u>
1 - 3	6%	2%	1%	*
4 - 6	23	10	5	1%
7, 8	7	3	1	*
9 - 11	15	6	3	1
12 - 14	47	19	9	3
15, 16	19	8	4	1
17	43	18	9	3

* Less than 1/2%

NOTATION

C	120 <u>or</u> 60, for panels applied with face grain perpendicular to <u>or</u> parallel to supports, respectively
E	modulus of elasticity (psi)
F_b	allowable bending stress (psi)
$F_{c\perp}$	allowable bearing stress on plywood face, for load perpendicular to plane of outer ply actually in bearing (psi)
F_s	allowable rolling shear stress (psi)
I	<u>effective</u> moment of inertia ($\text{in.}^4/\text{in. width}$)
(Ib/Q)	rolling shear constant ($\text{in.}^2/\text{in. width}$)
L	span center-to-center of supports (in.)
t	clear span (in.)
ℓ_e	required plywood (face) end bearing length at <u>each</u> end of panel (in.)
p_b	allowable load - bending moment (psi)
p_{dm}	dynamic (blast) uniform load capacity (psi)
p_m	smaller of p_b or p_s = static uniform load capacity (psi)
p_s	allowable load - rolling shear stress (psi)
S	<u>effective</u> section modulus ($\text{in.}^3/\text{in. width}$)
t	nominal panel thickness (in.)
y	deflection (elastic) under uniform load (in.)
y_b	bending deflection (elastic) under uniform load (in.)
y_s	shear deflection (elastic) under uniform load (in.)

REFERENCES

1. Plywood Design Specifications (PDS), American Plywood Association, 1119 A Street, Tacoma, Washington 98401, Revised December 1976.*
2. Plywood Design Manual - Shelving, American Plywood Association, 1119 A Street, Tacoma, Washington 98401, 1975.
3. Personal communications: Author with Wm. A. Baker, P.E., Head, Engineering Service, Applied Research Service, American Plywood Association, 1119 A Street, Tacoma, Washington 98401, July 22, 1977; and with James Elliott of Mr. Baker's staff, August 1, 1977.

* An April 1978 revision is now available and can be used; it was checked to determine the need for any recalculations for this extract from the Appendix A3 as previously published.

Appendix B1

WOOD BEAM AND COLUMN DESIGN - SIMPLY SUPPORTED

Revision of Appendix B, Design of Wood Beams, Simply Supported, as published in:

Murphy, H. L., and J. E. Beck, Slanting for Combined Nuclear Weapons Effects: BLAST-RESISTANT DESIGN/ANALYSIS WITH EXAMPLES, Stanford Research Institute¹ Final Report, for Defense Civil Preparedness Agency,² December 1974. (AD-A016 631)

Murphy, H. L., J. R. Rempel, and J. E. Beck, SLANTING IN NEW BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: A Consolidated Printing of Four Technical Reports, 3 Vols., Stanford Research Institute² Technical Reports, for Defense Civil Preparedness Agency,³ October 1975. (AD-A023 237)

Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects: Predesigned Expedient Options, Stanford Research Institute² Technical Report, for U.S. Defense Civil Preparedness Agency³, October 1977. (AD-A054 409)

¹ Now SRI International

² Now Federal Emergency Management Agency

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Appendix B1

WOOD BEAM AND COLUMN DESIGN - SIMPLY SUPPORTED

This appendix deals with solid wood used in beams and columns.³

Wood Beams - Simply Supported

The wood contemplated for use under the design procedures described herein is structural or stress-graded lumber, which has been carefully graded in accordance with the standard grading rules for the appropriate trade association. A complete list of such associations is available; see Reference [1]⁴ Supplement, page 19. It is urged that all lumber contemplated for shelter use - specifically, lumber in structural components or members whose stress-resisting capability is important to the survival of shelterees (in contrast to such things as a door cross-brace that simply holds together the structurally significant members) - be reinspected and regraded by even poorly qualified personnel using the appropriate association's grading rules.

Other items for the designer's general consideration are:

- The lack of homogeneity in wood members dictates that every effort be made to design wood structural members so that they interact in such a manner as to transfer load from a weaker, below-standard member to the better members. Examples are: really good blocking between floor joists; and use of tongue-and-groove planking as members used flat in a blast door. This is "repetitive-member use," see Reference [1] Supplement, Table 4A, 4th column.
- Only very tight knots should be accepted in a situation such as that of an unclad wood shelter blast door where an air blast loading could make a missile or bullet out of a knot that is even slightly loose.

³ Beam design chart solutions herein are for only single-span simply-supported (SS) beams; correction factors cover multi-span beams (section C, page B1-14). The design procedure also covers propped cantilever (PC) and fixed-fixed (FF) support conditions.

⁴ Brackets are used herein to indicate sources in the References list at the end of this appendix.

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

Because this appendix is intended for use by engineers and architects, some technical competence in the usual design of wood structural members is assumed [2,3], and only those design considerations peculiar to nuclear blast effects loading will be treated in some detail in this appendix.

Reading/study of Appendix B2 may serve to increase the reader/user's understanding of wood design and use in upgrading.

A. Design Procedure

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

It is recommended that the beam design procedures and graphs that follow be used only for L / d values equal to or greater than five (5), because of doubt that they apply to "deep beams" (L / d less than 5).

The design steps are as follows:

(1) A design air blast peak overpressure is specified, also whether its loading geometry will provide: a side-on overpressure (as in a wood door mounted flush with the earth's surface); a fully reflected overpressure (as in the front wall of a rectangular building); or a peak value of the average loading caused by a combination of side-on and drag pressure (as in the side-wall or roof of a rectangular building) [4,Sec.4.38]. Related variables, in the same order of loading geometries, look like this:

$$P_{dm} = P_{so} \text{ or } P_r \text{ or } [(P_{so} + C_d q) L / 2U] \quad (1)$$

where q is the dynamic (wind) blast pressure (unlike the q for structural resistance used in the remainder of this section) [4,Table 4.40].

(2) A trial size of wood beam (actual depth d , measured in direction of load, and thickness or width b) and kind of structural or stress-graded lumber are selected, then the grading association's design stresses are determined from their publications [1, Supplement, Table 4A].⁵ Need for the latter may be limited to F_b (extreme fiber stress in bending), F_v (horizontal shear stress), and $F_{c\perp}$ (compression stress perpendicular to grain, or bearing stress as used for beams only herein). For the short duration loadings furnished by nuclear air blast, dynamic values of the above three design stresses are recommended [5] as follows:

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

Some grading rules allow increases in design stress values for such things as: repetitive-member uses [1, Supplement, Table 4A, fourth column]; and, members used flatwise [1, Supplement, page 20, paragraph 6].

(3) A design ductility ratio μ is selected (see discussion in earlier Appendix A, General Comments on Blast-Resistant Design/Analysis General Approach). A value of 3 is recommended [5], certainly as an upper limit, and with 1.3 or 2 even better [6].

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

$$p_{dm}/q = 1 - (1 / (2\mu))$$

where q is the ultimate resistance to blast loading of the wood beam. Using the recommended value of $\mu = 3$, the equation becomes: $p_{dm} = (5/6) q$.

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

⁵ But if a Table B1-1 value for F_v is higher (than that given in Ref. [1]), use it.

⁶ Alternatives to this use of a step pulse are chart solutions and the Newmark β Method [7] or (better) the Modified Newmark β Method (by J. E. Beck) [8, p. 6-162].

Table B1-1

ALLOWABLE HORIZONTAL SHEAR VALUES F_v (psi)

	Maximum Moisture Content		
	Unseasoned	19 percent	15 percent
Aspen	85	90	95
Balsam Fir	85	95	95
Black Cottonwood	70	75	80
California Redwood	115	120	130
Coast Sitka Spruce	90	95	100
Coast Species	90	95	100
Douglas Fir-Larch	130	140	145
Douglas Fir-South	130	140	145
Eastern Hemlock-Tamarack	120	130	135
Eastern Spruce	95	105	110
Eastern White Pine	90	95	100
Eastern Woods	85	95	95
Engelmann Spruce/Alpine Fir	95	105	110
Hem-Fir	105	110	115
Idaho White Pine	95	100	105
Lodgepole Pine	95	105	110
Mountain Hemlock	130	140	150
Northern Aspen	90	95	100
Northern Pine	100	105	110
Northern Species	90	95	100
Northern White Cedar	85	95	100
Ponderosa Pine-Sugar Pine	100	105	110
Red Pine	100	110	115
Sitka Spruce	105	115	120
Southern Pine	125	135	145
Spruce-Pine-Fir	95	105	110
Western Cedar	100	105	110
Western Hemlock	125	135	145
Western White Pine	90	100	105
White Woods (Western Woods, West Coast Woods, Mixed Species)	95	100	105

Source: National Design Specification for Wood Construction, 1977 Edition,
National Forest Products Association, 1619 Massachusetts Avenue, N.W.,
Washington, D. C. 20036; art. 3.4.4.2

Author Comments: Use column of above table that specifies "19 percent"
moisture content. For wood species not shown, consult "Design Values
for Wood Construction," 1980 Supplement to above Source.

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

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

$$q_b = F_{db}(d/L)^2 / (6c) = 2F_b(d/L)^2 / (3c) \quad (2)$$

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

(7) Horizontal shear resistance q_v (in terms of load/unit area) is also calculated for the trial member, with horizontal shear equal to vertical shear and taken at a distance d in from each end of the member: [2(p.4-12), 5(p.161)]

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

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

where $c' = 1/2$ (SS) and (FF), $5/8$ (PC), the latter value being approximate but close enough for the purposes herein. NOTE: EQ. 2 AND 3 INCLUDE THE MULTIPLE 4 OF STEP 2.

(8) Wood beam resistance q is then equal to the lesser value between q_b and q_v ; q is converted to peak air blast pressure by using:

$$p_{dm} = q (1 - (1 / (2\mu))) \quad (4)$$

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

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

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

$$V = q_b Lc' = F_{c1} bL'$$

$$L' = qLc' / F_{c1} \quad (5)$$

where the values of c' are the same as in step 7 above. It is recommended that L' be at least 2 inches.

Another presentation of wood beam formulas is given in Appendix B2 (page B2-20).

Limitations on the above design procedure are stated in the two applications that follow.

B. Application to a Closure (Shelter Door) Design

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

- An estimate, calculated or judgmental, is made of the blast resistance of the wall adjacent to an aperture (door or window opening) for which a wood blast door is needed. The only designed structural element will be a wood beam, or series of wood beams side-by-side and preferably tongue-and-groove,⁷ simply supported on the two sides of the door frame (that has been either strengthened or found adequate to take the load from the door onto the wall).
- Stress-graded wood of various kinds (species and grades) in standard thicknesses⁸ (2, 3, 4 inches, nominal; 1.5, 2.5, 3.5 inches, actual) are checked for availability. See Table B1-2 for several popular woods and their allowable stresses.

a. The pre-design or chart approach developed for simplified handling of this problem is as follows, using solid, multiple wood beams, side-by-side:

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

⁷ If not t&g (tongue-and-groove), use a light plywood sheet covering on the blast side; if either t&g or plywood-covered, "repetitive-member use" in F_b design values is appropriate [1, Supplement, Table 4A, 4th column].

⁸ Thickness and width are the terms applied to the smaller and larger cross-section dimensions, respectively, in the industry [1, Supplement, page 20, paragraph 6]. Engineers use thickness and width the same way in columns, but in beams they use width and depth as the cross-section dimensions perpendicular and parallel to the direction of loading, respectively (for example, the width of a beam 1.5"x3.5" in cross-section would be 1.5" if used edgewise, 3.5" if used flatwise (to the load direction)).

Table B1-2
SELECTED DIMENSION LUMBER SPECIES, SIZES AND GRADES *
(CONSTRUCTION GRADES)

Size and Grade	Species					
	Douglas Fir-Larch	Western Hemlock	Western Pines (Ponderosa, Lodgepole, Sugar, and Idaho White)	Southern Pine	Northern Pine	
Allowable Unit Stresses in Normal Use, psi†						
Structural Light Framing (2" to 4" th., 2" to 4" wide)						
Stud (2x4s only)	F _b	925	800	600	900	725
	F _v	95	90	70	90	70
	F _{c⊥}	385	280	190	405	280
	E (x 10 ⁶)	1.5	1.3	1.2	1.4	1.1
	F _c	600	550	425	775	475
Light Framing (2" to 4" th., 4" wide)						
Construction	F _b	1200	1050	775	1150	950
	F _v	95	90	70	100	70
	F _{c⊥}	385	280	190	405	280
	E	1.5	1.3	1.2	1.4	1.1
	F _c	1150	1050	775	1100	875
Standard	F _b	675	600	425	675	525
	F _v	95	90	70	90	70
	F _{c⊥}	385	280	190	405	280
	E	1.5	1.3	1.2	1.4	1.1
	F _c	925	850	650	900	725
Joists and Planks (2" to 4" th., 5" and wider)						
No. 1	F _b	1750	1550	1100	1700	1400
	F _v	95	90	70	90	70
	F _{c⊥}	385	280	190	405	280
	E	1.8	1.6	1.4	1.7	1.4
	F _c	1250	1150	875	1250	975
No. 2	F _b	1450	1250	925	1400	1100
	F _v	95	90	70	90	70
	F _{c⊥}	385	280	190	405	280
	E	1.7	1.4	1.3	1.6	1.3
	F _c	1050	975	725	1000	825
No. 3	F _b	850	750	550	800	650
	F _v	95	90	70	90	70
	F _{c⊥}	385	280	190	405	280
	E	1.5	1.3	1.2	1.4	1.1
	F _c	675	625	450	625	525

* Table is for visually stress graded lumber only, used at 19% maximum moisture content.

† Notation used is: F_b for extreme fiber in bending (repetitive member use); F_c for compression parallel to grain; F_v for horizontal shear; $F_{c\perp}$ for compression perpendicular to grain; and E for modulus of elasticity. For dynamic uses in this report, F_b , F_c , and F_v stresses are multiplied by 2.0 to 4.

‡ F_c values in italics are for bending members only; not recommended for PSSP stringers. See SOURCE below, main report not supplement, article 4.2.2.
SOURCE: "Design Values for Wood Construction," 1980 Supplement to the 1977 Edition of National Design Specification for Wood Construction, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D. C. 20006.

- Bending design stress (in extreme fiber), in repetitive-member use, F_b
- Horizontal shear design stress, F_v
- Compression perpendicular to grain design stress, F_{c1}

(2) Conversion of design stresses to dynamic values (step 2 above) is unnecessary hereunder; the chart p_{dm} includes this conversion and the chart is therefore entered directly with the design stresses for normal loading. Similarly included is the factor (5/6) for $p_{dm} = (5/6) q$ (per design steps 4 and 8 above). Increases for "flatwise" use are not included in the chart p_{dm} (see design step 2 above).

(3) For each wood and thickness, determine the blast resistance in terms of free-field overpressure:

Enter the table on the page facing Figure B1-1 and read values A and B, for each pair of values of span L (in.) and (actual, not nominal) beam depth d (in.).

Enter the left portion of Figure B1-1 with the A and B values as follows: Use A (at top of figure) with the "curves" for horizontal shear F_v values and read q_n (or p_{dm}) on left (ordinate) scale; use B (at bottom of figure) with the "curves" for bending stress F_b values and read q_n (or p_{dm}) on left scale; use only the smaller value of q_n (or p_{dm}) read!

(4) For each wood and thickness still of interest, determine the required bearing length at each end of the wood beam:

Use the F_{c1} value found above and the p_{dm} value just found, and calculate the required bearing length L' (in.) at each end of the beam, from the following modification of Eq. 5:

$$L' = 0.6 p_{dm} L / F_{c1} \quad (6)$$

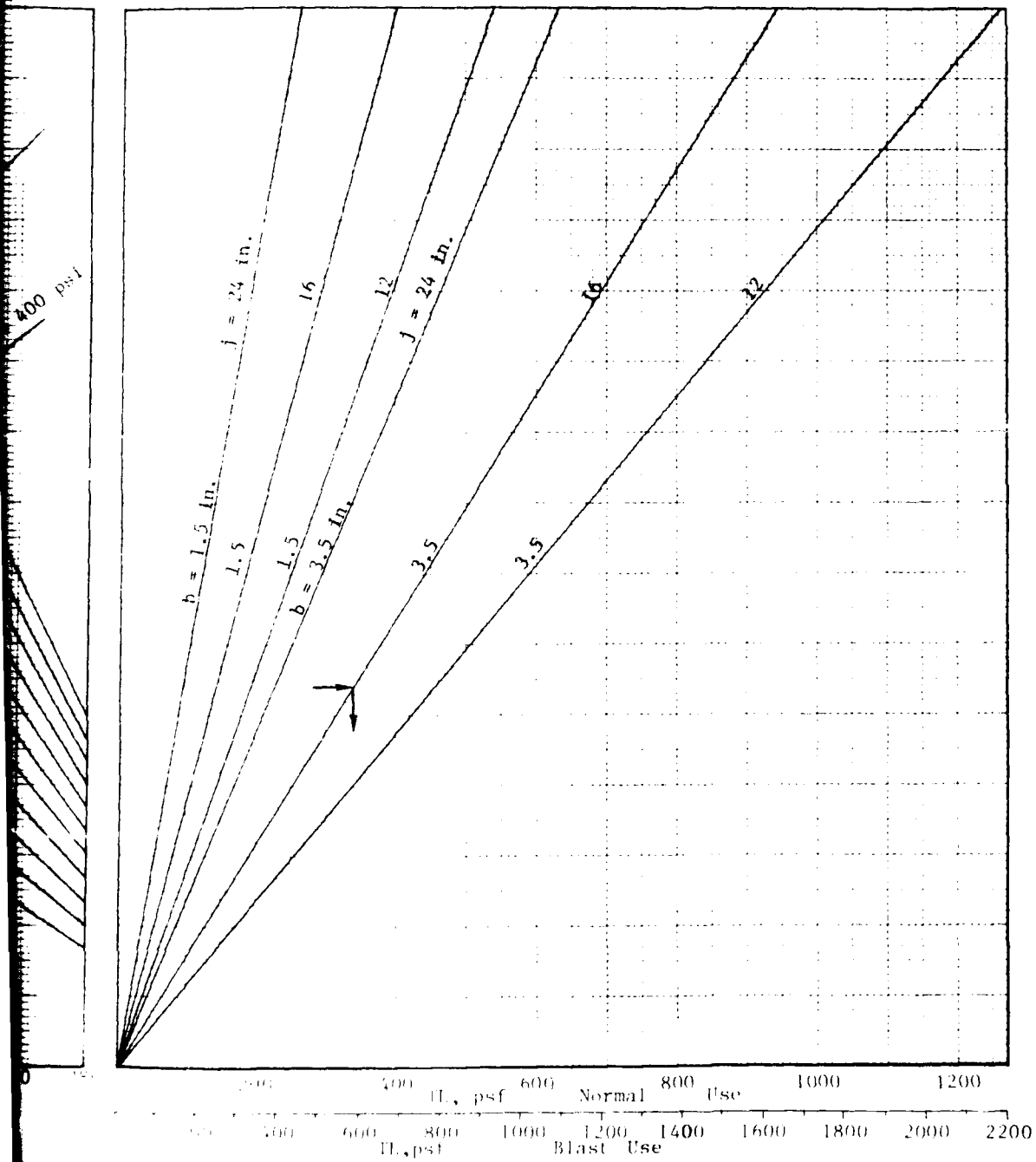
but not less than 2 in. Strictly speaking, if the beam-ends in bearing are exposed to the air blast, Eq. 6 should be:

$$L' = 0.6 p_{dm} L / (F_{c1} - p_{dm}).$$

VALUES OF A AND B FOR FIGURE B1-1

Span L (in.)	d=1.5 in.		=2.5 in.		=3.5 in.		=4.5 in.		=5.5 in.		=7.25 in.		=9.25 in.		=11.25 in.	
	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
12	167	104	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	136	77	278	213	0	0	0	0	0	0	0	0	0	0	0	0
16	115	59	227	163	0	0	0	0	0	0	0	0	0	0	0	0
18	100	46	192	129	318	252	0	0	0	0	0	0	0	0	0	0
20	88	38	167	104	269	204	0	0	0	0	0	0	0	0	0	0
22	79	31	147	86	233	169	0	0	0	0	0	0	0	0	0	0
24	71	26	132	72	206	142	300	234	0	0	0	0	0	0	0	0
26	65	22	119	62	184	121	265	200	0	0	0	0	0	0	0	0
28	60	19	109	53	167	104	237	172	324	257	0	0	0	0	0	0
30	56	17	100	46	152	91	214	150	289	224	0	0	0	0	0	0
32	52	15	93	41	140	80	195	132	262	197	0	0	0	0	0	0
34	48	13	86	36	130	71	180	117	239	174	0	0	0	0	0	0
36	45	12	81	32	121	63	167	104	220	156	0	0	0	0	0	0
38	43	10	76	29	113	57	155	93	204	140	309	243	0	0	0	0
40	41	9	71	26	106	51	145	84	190	126	284	219	0	0	0	0
42	38	9	68	24	100	46	136	77	177	114	264	199	0	0	0	0
44	37	8	64	22	95	42	129	70	167	104	246	181	0	0	0	0
46	35	7	61	20	90	39	122	64	157	95	230	166	0	0	0	0
48	33	7	58	18	85	35	115	59	149	88	216	152	314	248	0	0
50	32	6	56	17	81	33	110	54	141	81	204	140	294	228	0	0
52	31	6	53	15	78	30	105	50	134	75	193	130	276	211	0	0
54	29	5	51	14	74	28	100	46	128	69	184	120	261	196	0	0
56	28	5	49	13	71	25	96	43	122	64	175	112	247	182	0	0
58	27	4	47	12	69	24	92	40	117	60	167	104	234	170	315	251
60	26	4	45	12	66	23	88	38	112	56	159	97	223	158	300	234
62	25	4	44	11	64	21	85	35	108	52	153	91	213	146	285	219
64	25	4	42	10	61	20	82	33	104	49	146	86	203	139	271	206
66	24	3	41	10	59	19	79	31	100	46	141	80	195	131	259	194
68	23	3	40	9	57	18	76	29	95	44	136	76	187	123	247	182
70	22	3	38	9	56	17	74	28	93	41	131	72	180	116	237	172
72	22	3	37	8	54	16	71	26	90	39	126	68	173	110	227	163
74	21	3	36	8	52	15	68	25	87	37	122	64	167	104	218	154
76	21	3	35	7	51	14	67	23	85	35	118	61	161	99	210	146
78	20	2	34	7	49	13	65	22	82	33	114	58	155	94	203	139
80	19	2	33	7	48	13	63	21	80	32	111	55	150	89	196	132
82	19	2	32	6	47	12	62	20	77	30	107	52	146	85	189	125
84	19	2	32	6	45	12	60	19	75	28	104	50	141	81	183	120
86	18	2	31	6	44	11	58	18	73	27	101	47	137	77	177	114
88	18	2	30	5	43	11	57	17	71	26	99	45	133	74	172	109
90	17	2	29	5	42	10	56	17	70	25	96	43	129	70	167	104
92	17	2	29	5	41	10	54	16	68	24	94	41	126	67	162	100
94	16	2	28	5	40	9	53	15	66	23	91	40	123	65	157	95
96	16	2	27	5	39	9	52	15	65	22	89	38	119	62	153	92
98	16	2	27	4	38	9	51	14	63	21	87	36	116	59	149	88
100	15	2	26	4	38	8	49	13	62	20	85	35	113	57	145	84
102	15	1	26	4	37	8	48	13	60	19	83	34	111	55	142	81
104	15	1	25	4	36	8	47	12	59	19	81	32	108	53	138	78
106	15	1	25	4	35	7	46	12	58	18	79	31	106	51	135	75
108	14	1	24	4	35	7	45	12	57	17	78	30	103	49	132	72
110	14	1	24	3	34	7	45	11	56	17	76	29	101	47	129	70
112	14	1	23	3	33	7	44	11	54	16	74	28	99	45	126	67
114	14	1	23	3	33	6	43	10	53	16	73	27	97	44	123	65
116	13	1	23	3	32	6	42	10	52	15	71	26	95	42	120	63
118	13	1	22	3	32	6	41	10	51	14	70	25	93	41	116	61
120	13	1	22	3	31	6	41	9	50	14	69	24	91	40	115	59
122	13	1	21	3	30	5	40	9	50	14	67	24	89	38	113	57
124	12	1	21	3	30	5	39	9	49	13	66	23	88	37	111	55
126	12	1	21	3	29	5	38	9	48	13	65	22	86	36	109	53
128	12	1	20	3	29	5	38	8	47	12	64	21	84	35	107	51
130	12	1	20	2	28	5	37	8	46	12	63	21	83	34	105	50
132	12	1	20	2	28	5	37	8	45	12	62	20	81	33	103	48
134	11	1	19	2	28	5	36	8	45	11	61	20	80	32	101	47
136	11	1	19	2	27	4	35	7	44	11	60	19	79	31	99	46
138	11	1	19	2	27	4	35	7	43	11	59	18	77	30	97	44
140	11	1	19	2	26	4	34	7	43	10	58	18	76	29	96	43
142	11	1	18	2	26	4	34	7	42	10	57	17	75	28	94	42
144	11	1	18	2	26	4	33	7	41	10	56	17	74	28	93	41

In above, A = d / (L-2d) x 1000, and B = (d/L)² x (2/3) x 10000



(5) As an illustrative example: Using Douglas Fir-Larch 2x4s (on edge, solidly side-by-side), Structural Light Framing, Stud grade, read from Tabl B1-1: F_b (repetitive-member use) is 925 psi, F_v is 140 psi, and $F_{c\perp}$ is 385 psi. Span L is 40 in. center-center of supports. Enter table facing Figure B1-1 and read: A is 106, and B is 51. Enter Figure B1-1 with A and B and read: for A of 106 and F_v of 140, q_n is 19.8 psi; for B of 51 and F_b of 925, q_n is about 9.4 psi. Thus, the q_n of 9.4⁹ psi is the smaller, and p_{dm} of 31.5 psi is the peak air blast (free field) overpressure design resistance of this solid door of 2x4s on edge (assuming that the door frame has been checked and found adequate).⁹

Required bearing length on each end of each 2x4 is calculated using Eq. 6 and $F_{c\perp}$ of 385 minus p_{dm} (assuming the blast hits the supporting ends of the 2x4s, as it usually would), or an $F_{c\perp}$ of, say, 350 psi: $L' = 0.6 \times 31.5 \times 40 / 350 = 2\text{-}1/4$ in. bearing length on each end.

(6) The above example assumes that the closure is covered with plywood (e.g., 1/4 in.). If not: use F_b for single-member use [1, Table 4A, third column]; use a stress multiple of 2.6 (not 4),¹⁰ use a μ of 1.5 (with same step pulse); and, therefore, read q_n (not p_{dm}) when using Figure B1-1, and calculate $p_{dm} = 2.6 (2/3) q_n = 1.73 q_n$. Thus, the single-member use applied to the above illustrative example changes the resulting p_{dm} from 31.5 psi to $p_{dm} = 1.73 q_n = 1.73 \times 9.4 = 16.3$ psi.

b. The chart approach is as follows when using beams spaced as in floor stringers (chart is only for beam widths b of 1.5 and 3.5 in., each spaced at 12, 16 and 24 in. center-center):

Using the same wood beams, span L (40 in.), F_v of 140 psi, A of 106, B of 51, and $F_{c\perp}$ of 385 psi as in the illustrative example above, find F_b (single-member use) of 800 psi [1, Supplement, Table 4A]; enter Figure B1-1 as before and read q_n values (q_n of 19.8 from A of 106 and F_v of 140; q_n of 8.2 from B of 51 and F_b of 800) of which 8.2 psi is the lower. The q_n value is stated only for illustration; the reader/user should move to the right from finding the intersection of B and F_b values - finding that the total load TL in pounds per square foot of covering (in normal use) over the spaced wood beams is: 150 psf TL for beam width b of 1.5 in. and spacing j of 12 in.; 113 psf for 16 in. spacing; and 75 psf for 24 in. spacing.

⁹ The increase factor from q_n to p_{dm} is 3.333. It represents a 100% increase for short duration (impact) loading [1, sec. 2.2.5.3], another 100% increase to use up the assumed factor of safety for repetitive-member use (reader/user may want to reduce this to 30% by hand calculation, if no regrading has been done), and a reduction by (5/6) for using $\mu = 3$ (reader/user may also want to reduce this by hand calculation); thus, the overall factor is $2 \times 2 (5/6) = 3.333 \dots$, or $p_{dm} = 3.333 q_n$.

¹⁰ Representing increases of 100% for short duration (impact) loading and 30% for factor of safety in single-member use.

These normal-use design TL values can be corrected for air blast loading by use of a multiple of 1.73 (as in paragraph (C) just above), giving 260, 195, and 130 psf for spacings of 12, 16, and 24 in., respectively, as calculated, or such results may be read directly from the outer (top and bottom) scales of the right graph, Figure B1-1.

C. Support Conditions Other Than Single-Span Simply Supported (SS)

The foregoing deals with single-span beams, on simple (non-moment carrying) supports (SS). For two-equal-span beams on simple supports (PC), formulas for one span (of the two) are on pages B2-20 and -21, Appendix B2, using constants for PC. (The formulas for FF conditions are of little interest to the upgrading sizing herein.)

If the contemplated wood beam use involves one beam extending over more than a single span: find q_{nv} and q_{nb} for a single-span beam as before (i.e., from A-F_v and B-F_b, respectively); find related p_{dmv} and p_{dmb} ; and find related TL_v and TL_b. Correct these values for multiple-span beams (with all simple supports) as follows [9,p.5-41]:

	<u>2-span</u>	<u>3-span</u>	<u>4-span</u>	<u>5-span</u>
q_n , p_{dm} , and TL with subscript b:	1.000	1.250	1.168	1.190
q_n , p_{dm} , and TL with subscript v:	0.800	0.833	0.824	0.836

Use the lower value of each: q_n , p_{dm} , and TL.

Wood Columns - Simple Supports

The wood columns contemplated in this design section have square ends and receive/deliver loads through well-fitted joints (not fixed). (Such conditions are sometimes called pin-ended.) The columns may or may not have intermediate supports.

Much of the guidance about wood and the wood beam design approach (both as presented in the preceding section on wood beams) apply to column design. Accordingly, the column design procedure that follows is considerably abbreviated, as compared to the beam design section, and depends a lot on an illustrative example.

A. Design Procedure

a. Column (normal-use) design formulas are shown in Table B1-3. Note the sketch and use of two different L/d ratios where there is an intermediate support for the weak direction; per Eq. 7, only the larger L/d value is used further. Use the table facing Fig. B1-2A to get each needed L/d value; enter the table with span L and depth d (both d_1 and d_2 of the sketch in Table B1-3 are termed "depth" - call them larger and smaller d 's, and similar for the L 's); read the L/d value. The sketch of Table B1-3 actually shows a part-column with two sets of intermediate supports; if only the weaker direction has intermediate support, L_2 is the end-to-end column length. If the (larger) design L/d value is greater than 50, the selected member and/or its lateral supports is/are inadequate; change member or supports.

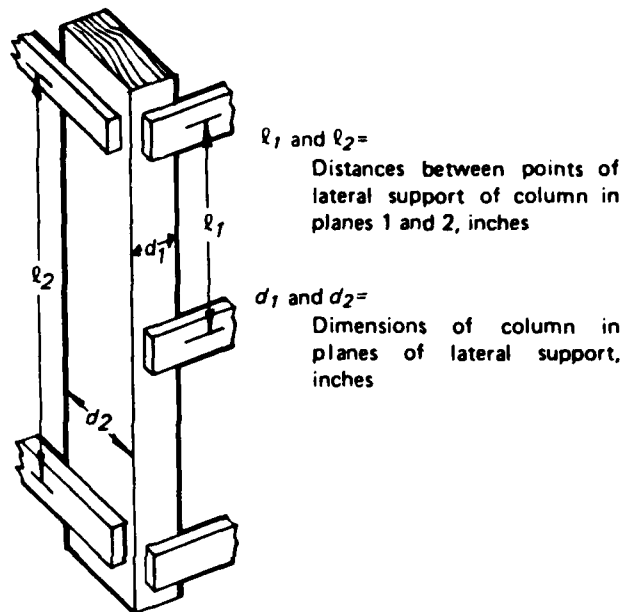
Determine the modulus of elasticity E and the design value for compression stress parallel to grain F_c for the wood member selected (1, Supplement, Table 4A); regrading is urged (time permitting), just as it was for wood beams.

Using Figure B1-2A, enter graph 1 with the F_c value; go up to the appropriate E value (or interpolate vertically between the two curves bracketing the E value), then right to read a value for K (on scale between graphs 1 and 2). If the design L/d value is less than or equal to K , continue to the right until hitting the appropriate L/d curve (or interpolate horizontally between L/d curves); using the same F_c (as used with graph 1) go vertically to the F_c line of graph 3 (or interpolate vertically); then go horizontally to the graph 4 line representing the nominal (actual) column dimensions; finally, go vertically to read P and P_d .

The final P found comes from use of Eq. 13. For P_d , multiples of 2 on F_c for short duration (impact) loading, and 1.3 for a 30% factor of safety, are reduced by a factor of 0.5 from using a μ of 1; thus P_d equals $(2 \times 1.3 \times 0.5)$ or 1.3 times P .

Table B1-3

COLUMN FORMULAS - SIMPLE SOLID COLUMN DESIGN [3]
(Pin-ended conditions assumed)



For conditions shown in the sketch:

Use larger of L_1 / d_1 and L_2 / d_2 as the L / d for design (7)

Slenderness ratio L / d must be ≤ 50 (8)

For $L / d \leq 11$: $F'_c = F_c$ (9)

For $L / d > 11$, but $\leq K$:

$$K = 0.671 \sqrt{E / F_c} \quad (10)$$

$$F'_c = F_c (1 - 1/3 (L / dK)^4) \quad (11)$$

$$\text{For } L / d \geq K: F'_c = 0.3E / (L / d)^2 \quad (12)$$

$$P = AF'_c = bdF'_c \quad (13)$$

L / d VALUES FOR FIGURE B1-2

L (in.)	d (in.)							
	1.5	2.5	3.5	4.5	5.5	7.25	9.25	11.25
18	12.0	7.2	5.1	4.0	3.3	2.5	1.9	1.6
20	13.3	8.0	5.7	4.4	3.6	2.8	2.2	1.8
22	14.7	8.8	6.3	4.9	4.0	3.0	2.4	2.0
24	16.0	9.6	6.9	5.3	4.4	3.3	2.6	2.1
26	17.3	10.4	7.4	5.8	4.7	3.6	2.8	2.3
28	18.7	11.2	8.0	6.2	5.1	3.9	3.0	2.5
30	20.0	12.0	8.6	6.7	5.5	4.1	3.2	2.7
32	21.3	12.8	9.1	7.1	5.8	4.4	3.5	2.8
34	22.7	13.6	9.7	7.6	6.2	4.7	3.7	3.0
36	24.0	14.4	10.3	8.0	6.5	5.0	3.9	3.2
38	25.3	15.2	10.9	8.4	6.9	5.2	4.1	3.4
40	26.7	16.0	11.4	8.9	7.3	5.5	4.3	3.6
42	28.0	16.8	12.0	9.3	7.6	5.8	4.5	3.7
44	29.3	17.6	12.6	9.8	8.0	6.1	4.8	3.9
46	30.7	18.4	13.1	10.2	8.4	6.3	5.0	4.1
48	32.0	19.2	13.7	10.7	8.7	6.6	5.2	4.3
50	33.3	20.0	14.3	11.1	9.1	6.9	5.4	4.4
52	34.7	20.8	14.9	11.6	9.5	7.2	5.6	4.6
54	36.0	21.6	15.4	12.0	9.8	7.4	5.8	4.8
56	37.3	22.4	16.0	12.4	10.2	7.7	6.1	5.0
58	38.7	23.2	16.6	12.9	10.5	8.0	6.3	5.2
60	40.0	24.0	17.1	13.3	10.9	8.3	6.5	5.3
62	41.3	24.8	17.7	13.8	11.3	8.6	6.7	5.5
64	42.7	25.6	18.3	14.2	11.6	8.8	6.9	5.7
66	44.0	26.4	18.9	14.7	12.0	9.1	7.1	5.9
68	45.3	27.2	19.4	15.1	12.4	9.4	7.4	6.0
70	46.7	28.0	20.0	15.6	12.7	9.7	7.6	6.2
72	48.0	28.8	20.6	16.0	13.1	9.9	7.8	6.4
74	49.3	29.6	21.1	16.4	13.5	10.2	8.0	6.6
76	50.7	30.4	21.7	16.9	13.8	10.5	8.2	6.8
78	52.0	31.2	22.3	17.3	14.2	10.8	8.4	6.9
80	53.3	32.0	22.9	17.8	14.5	11.0	8.6	7.1
82	54.7	32.8	23.4	18.2	14.9	11.3	8.9	7.3
84	56.0	33.6	24.0	18.7	15.3	11.6	9.1	7.5
86	57.3	34.4	24.6	19.1	15.6	11.9	9.3	7.6
88	58.7	35.2	25.1	19.6	16.0	12.1	9.5	7.8
90	60.0	36.0	25.7	20.0	16.4	12.4	9.7	8.0
92	61.3	36.8	26.3	20.4	16.7	12.7	9.9	8.2
94	62.7	37.6	26.9	20.9	17.1	13.0	10.2	8.4
96	64.0	38.4	27.4	21.3	17.5	13.2	10.4	8.5
98	65.3	39.2	28.0	21.8	17.8	13.5	10.6	8.7
100	66.7	40.0	28.6	22.2	18.2	13.8	10.8	8.9
102	68.0	40.8	29.1	22.7	18.5	14.1	11.0	9.1
104	69.3	41.6	29.7	23.1	18.9	14.3	11.2	9.2
106	70.7	42.4	30.3	23.6	19.3	14.6	11.5	9.4
108	72.0	43.2	30.9	24.0	19.6	14.9	11.7	9.6
110	73.3	44.0	31.4	24.4	20.0	15.2	11.9	9.8

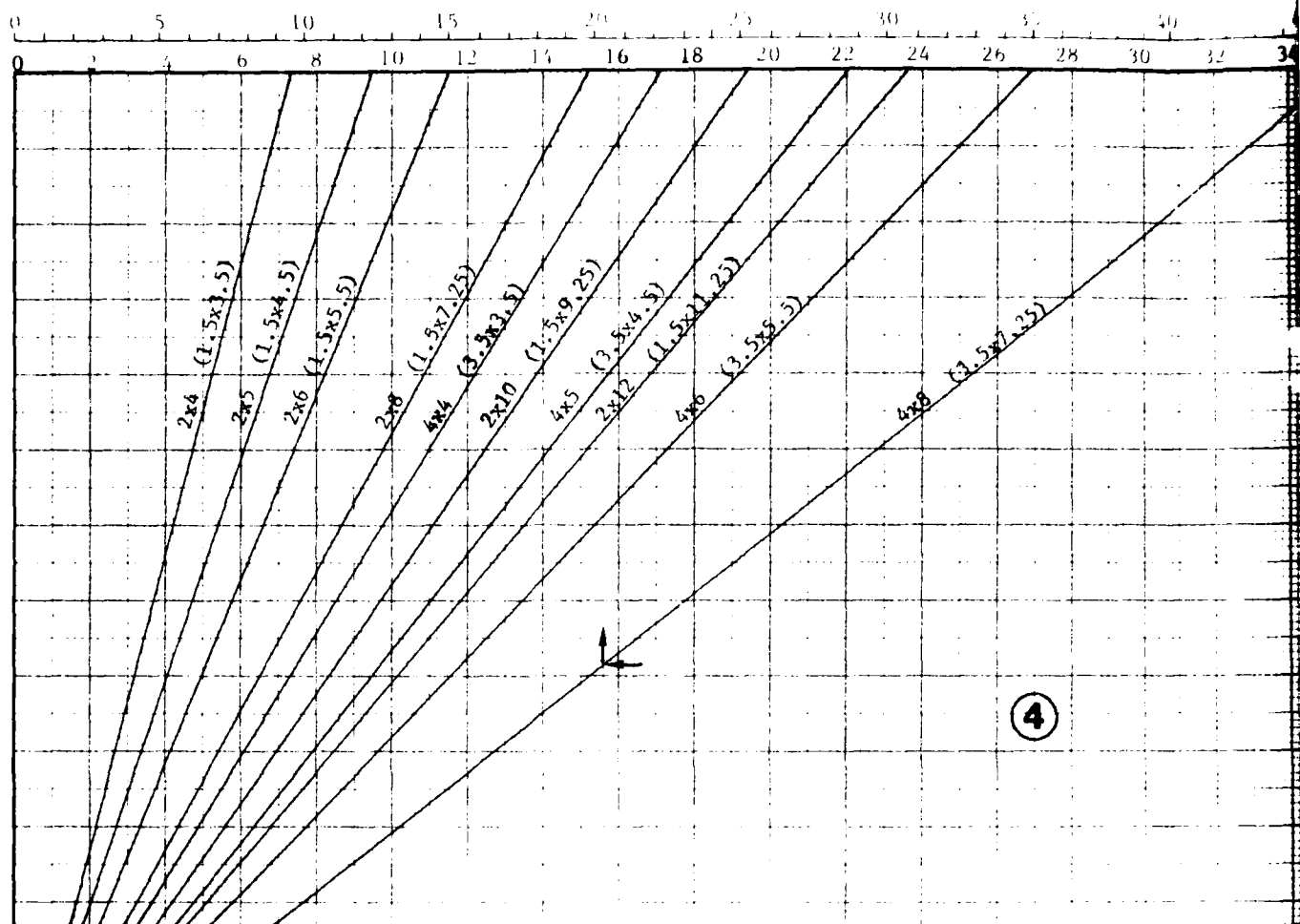
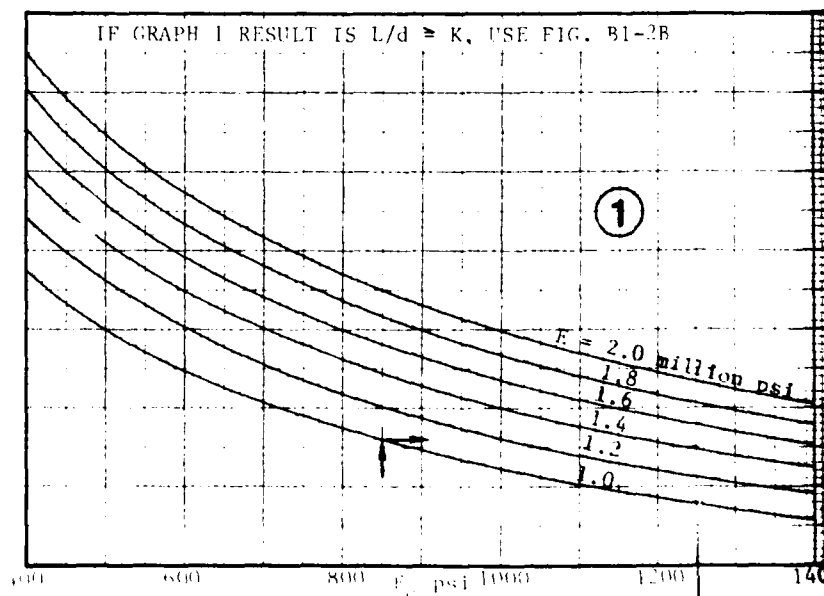
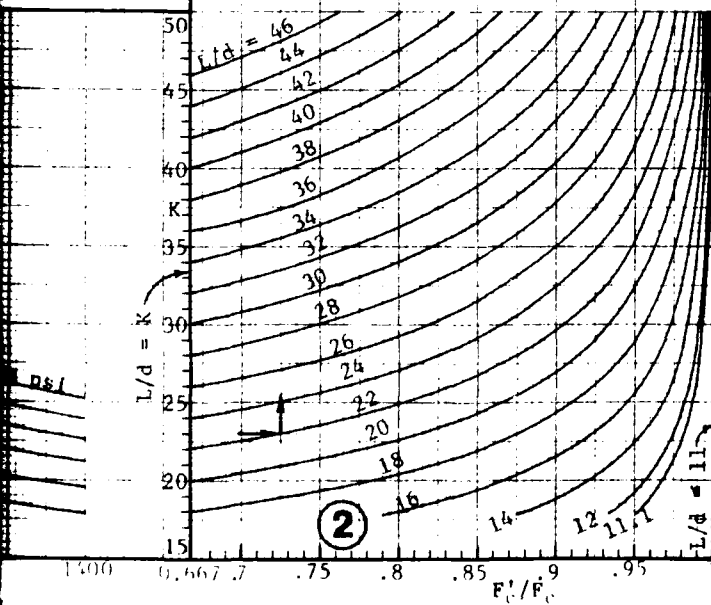
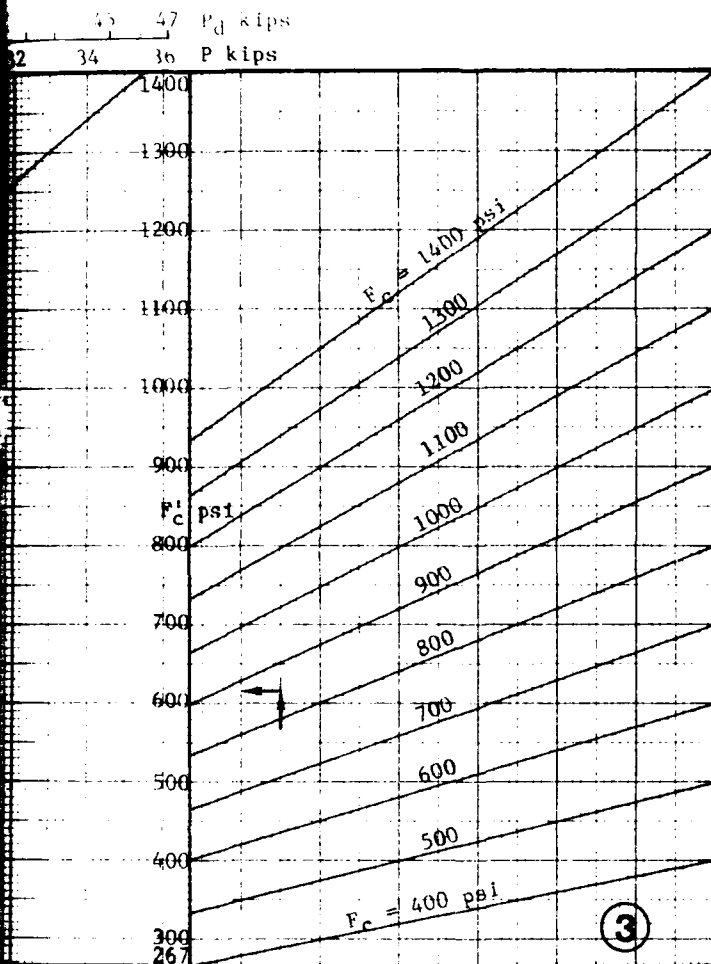


Figure B1-2A
COLUMN DESIGN, WOOD





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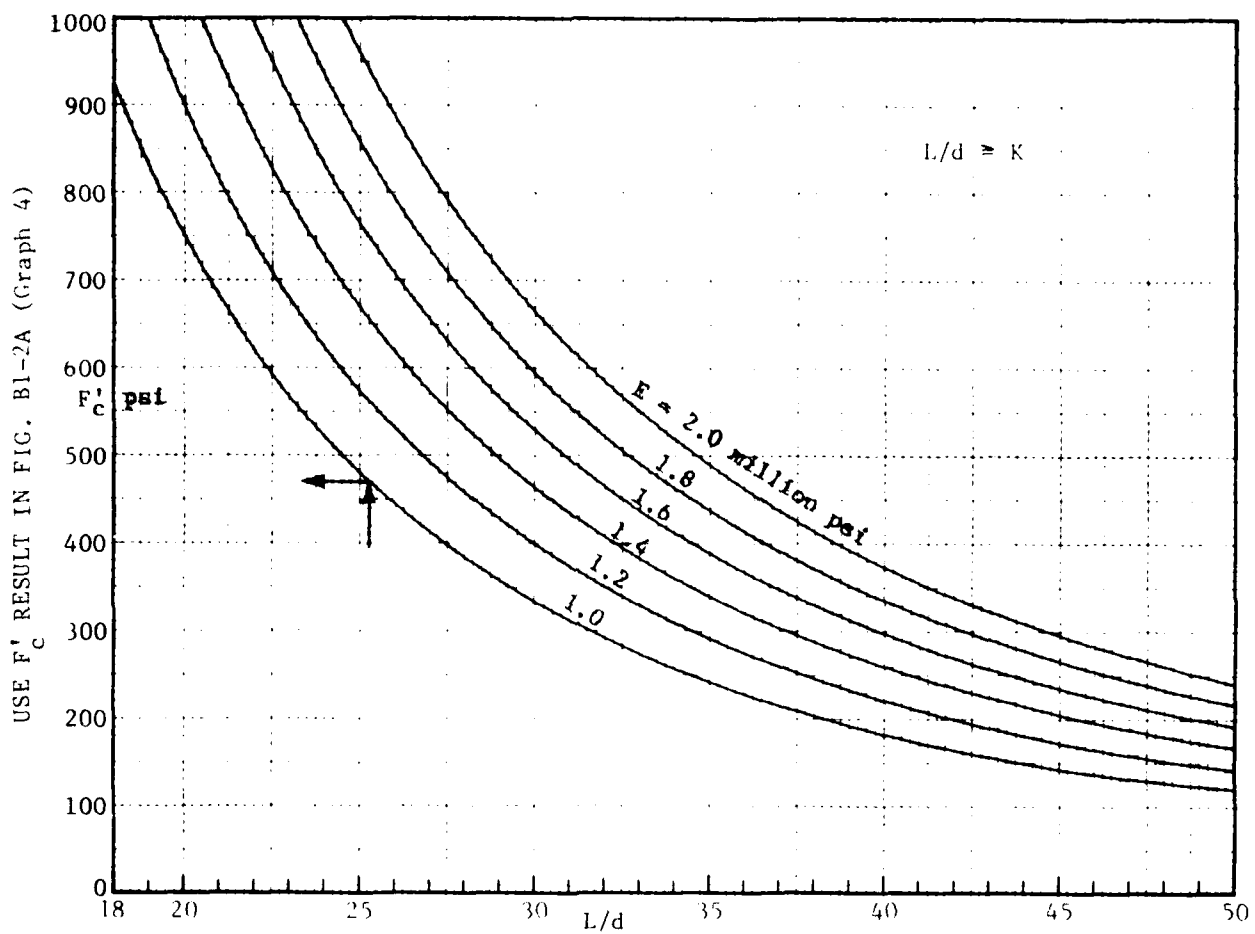


Figure B1-2B

B1-21

b. Use of the 1.5 multiple is recommended only if regrading is done. If not, it is recommended that P_d be taken as equal to P , thus leaving intact the 30% factor of safety to cover grading errors and/or buckling hazards.

See page B2-29, Appendix B2, for a calculations example of handling combined (fallout shielding) soil loads and blast loads. (There are deliberate but minor differences in some of the details, between Appendices B1 and B2, which are attributable to the general nature of B1 versus the more specific nature of B2 (here basement shelter for fallout and only low blast levels.)

c. For checking adequacy of designed columns in terms of their end bearing, and also for some comments on sill or bottom plate or blocking "design," see section "C. End Bearing and Sill/Bottom Plate Design," below.

B. Numerical Example - Design Using Figure B1-2

As an illustrative example: assume 2x4s are to be checked for their blast capacity in use as upgrading columns; Idaho White Pine is available, for which F_c is 650 psi and E is 1.2 million psi in Standard grade [1, Supplement, p. 11; or table on p. B2-16, Appendix B2]; column height L (L is also used in this section) is 96 in. Using table facing Figure B1-2A, L/d values are 64 for narrow (weaker) direction (d is 1.5 in.) and 27.4 for wide (stronger) direction (d is 3.5 in.); the larger L/d of 64 controls (Eq. 7). According to Eq. 8, this column is unusable (buckling hazard is too great).

The 2x4s can be used if supported at mid-height, in the weak direction, which is one alternative; however, let's take the other, that is, use a larger column by assuming that 4x4s are to be checked instead (normally, such a great increase would not be called for unless the designer is way off in his first guess, but poetic licence is taken here in order to demonstrate the two things that can happen in using Figure B1-2). Values for F_c and E are unchanged; L/d is 27.4 as before. Entering B1-2A with F_c of 650 and E of 1.2 million, read K as 29; since L/d is less than K , continue horizontally, interpolating horizontally between curves for L/d of 28 and 26, that is about 3/4 of the way from the 26 to the 28 curve ($1.4/2 \approx 3/4$); go vertically from this interpolated point (about on 0.73 of bottom scale) to F_c of 650 in graph 3; then horizontally (along about the F'_c of 470 line) to the specific line for 4x4 (3.5x3.5) in graph 4 (no interpolating in this area!); then vertically to read P as 5.8 kips (5,800 pounds) and P_d as about 7.5 kips (7,500 pounds). (From calculations on page B2-28, Appendix B2: L/d is 27.43; K is 28.83; F'_c is 472, thus F'_c/F_c for bottom scale is 0.726; P is 5,782, from which P_d is 7,517.)¹¹

¹¹ Numerical results are shown to illustrate the work, not to imply a degree of accuracy in design/analysis.

Now, assuming that the wood and grade are Douglas Fir-Larch and Construction, find F_c of 1,150 psi and E of 1.5 million psi from same sources as before. Using Figure B1-2A, graph 1 (interpolate vertically between E curves), find K as 24, meaning that L/d of 27.4 is greater than K , and Figure B1-2B must be used (instead of graphs 2 and 3 of B1-2A). Using B1-2B, enter with L/d of 27.4 and E of 1.5 million, read F'_c as 595; using graph 4 of B1-2A, enter with F'_c of 595 and read (for 4x4s) P of 7.3 kips (7,300 pounds) and P_d of 9.5 kips (9,500 pounds). (Again, from calculations on page B2-28, Appendix B2: L/d is 27.43 as before; K is 24.23; F'_c is 598; P is 7,327, from which P_d is 9,525.)

C. End Bearing and Sill/Bottom Plate Design

a. Adequacy of columns for end grain in bearing (top and bottom) should be checked. The method can be shown by example, using the two columns in the numerical example of the preceding section: Table B1-4 (fourth column) shows the design value for end bearing F_g for 4x4s as 1,390 psi for the Idaho White Pine and as 2,020 psi for the Douglas Fir-Larch. With the 4x4 (3.5x3.5) end area of 12.25 in.², the two example columns have end bearing capacities of 17,028 and 24,745 pounds if bearing is through metal plates, and 12,771 and 18,559 pounds if on less rigid materials (wood, concrete, etc.); see the comment along the bottom of Table B1-4. With the two example columns blast capacities P_d of 7,500 and 9,500 pounds, there is ample end bearing capacity even under the 75% limitation. Further, the design values F_g of Table B1-4 are subject to duration of loading factors [1, Sec. 2.2.5.3]: increase the F_g values by 15% for loads of 2 months or less (as with fallout-shielding soil), and 100% for impact loads (such as air blast)! Thus, column adequacy for end grain in bearing might be concluded to be generally of no concern; nonetheless the checks should be made.

b. Sill or bottom plate "design" could be complex, involving as it does a beam with concentrated loads (the columns, plus foot plates if any) and supported by an elastic/plastic/fracturing foundation (the usual light, concrete floor slab). Such design is considered unwarranted for the purposes herein. It should be accepted that the floor slab will crack/break up and will be pushed down, under and adjacent to the bottom plate, should the upgraded floor system over the basement receive an air blast overpressure loading; such action absorbs energy. Recent tests indicate a very high floor slab resistance to column punching from blast duration loads (see page 18 of main text).

For the added loading of soil placed on the first floor for fallout shielding upgrading, it is doubted that the concrete basement floor will experience anything more than localized cracking, if that, under the following approach:

Table B1-4

END GRAIN IN BEARING (psi)

Design values for end grain not bearing parallel to grain on a rigid surface F_g
in pounds per square inch

Species	Wet service conditions ¹	Dry service conditions ¹		Glued laminated timber
		Sawn lumber ²		
		More than 4" thick	Not more than 4" thick	
Ash - White	1370	1510	2060	2400
Alder	400	820	1110	1300
Balsam Poplar	890	980	1330	1560
Birch	1340	1310	1780	2080
Brown Sweetgum	1350	1260	1720	2010
Black Cherry	620	690	930	1090
Black Gum	560	720	2270	2620
Black Hickory	1310	1270	1670	1940
Black Locust	950	1040	1420	1660
Black Spruce	950	1040	1420	1660
Black Walnut	765	840	1150	1340
Chinquapin - Green ⁴	1570	1730	2360	2750
Chinquapin - Yellow ⁴	1340	1480	2020	2350
Chinquapin - Striped	1220	1340	1820	2130
Eastern Hemlock - Taniguchi ⁴	1150	1270	1730	2020
Eastern Redwood	970	1070	1460	1700
Eastern White Pine ⁴	900	1000	1360	1580
Eastern White Pine ⁴	820	900	1230	1440
Engelmann Spruce - Alpine for	810	890	1220	1420
Engelmann ⁴	1110	1220	1670	1940
Hickory - Hard	1370	1510	2050	2400
Hickory - White Pine	930	1020	1390	1630
Hickory - Red Pine	970	1060	1450	1690
Major - Black and Square	1140	1250	1710	2000
Major - Square	1070	1170	1600	1870
Norfolk - Alder	740	810	1110	1290
Norfolk - Birch	1040	1150	1570	1830
Norfolk - Spruce	880	970	1320	1540
Norfolk - White Pine	740	810	1110	1290
Norfolk - White Pine	1050	1160	1590	1850
Redwood - Western	910	1000	1370	1600
Redwood	880	970	1320	1540
Redwood	940	1090	1480	1730
Redwood	1110	1260	1690	2000
Redwood	1040	1190	1600	1870
Redwood	1120	1270	1700	2000
Redwood	1140	1290	1750	2070
Redwood	1160	1310	1800	2140
Redwood	1180	1330	1850	2210
Redwood	1200	1350	1900	2280
Redwood	1220	1370	1950	2350
Redwood	1240	1390	2000	2420
Redwood	1260	1410	2050	2490
Redwood	1280	1430	2100	2560
Redwood	1300	1450	2150	2630
Redwood	1320	1470	2200	2700
Redwood	1340	1490	2250	2770
Redwood	1360	1510	2300	2840
Redwood	1380	1530	2350	2910
Redwood	1400	1550	2400	2980
Redwood	1420	1570	2450	3050
Redwood	1440	1590	2500	3120
Redwood	1460	1610	2550	3190
Redwood	1480	1630	2600	3260
Redwood	1500	1650	2650	3330
Redwood	1520	1670	2700	3400
Redwood	1540	1690	2750	3470
Redwood	1560	1710	2800	3540
Redwood	1580	1730	2850	3610
Redwood	1600	1750	2900	3680
Redwood	1620	1770	2950	3750
Redwood	1640	1790	3000	3820
Redwood	1660	1810	3050	3890
Redwood	1680	1830	3100	3960
Redwood	1700	1850	3150	4030
Redwood	1720	1870	3200	4100
Redwood	1740	1890	3250	4170
Redwood	1760	1910	3300	4240
Redwood	1780	1930	3350	4310
Redwood	1800	1950	3400	4380
Redwood	1820	1970	3450	4450
Redwood	1840	1990	3500	4520
Redwood	1860	2010	3550	4590
Redwood	1880	2030	3600	4660
Redwood	1900	2050	3650	4730
Redwood	1920	2070	3700	4800
Redwood	1940	2090	3750	4870
Redwood	1960	2110	3800	4940
Redwood	1980	2130	3850	5010
Redwood	2000	2150	3900	5080
Redwood	2020	2170	3950	5150
Redwood	2040	2190	4000	5220
Redwood	2060	2210	4050	5290
Redwood	2080	2230	4100	5360
Redwood	2100	2250	4150	5430
Redwood	2120	2270	4200	5500
Redwood	2140	2290	4250	5570
Redwood	2160	2310	4300	5640
Redwood	2180	2330	4350	5710
Redwood	2200	2350	4400	5780
Redwood	2220	2370	4450	5850
Redwood	2240	2390	4500	5920
Redwood	2260	2410	4550	5990
Redwood	2280	2430	4600	6060
Redwood	2300	2450	4650	6130
Redwood	2320	2470	4700	6200
Redwood	2340	2490	4750	6270
Redwood	2360	2510	4800	6340
Redwood	2380	2530	4850	6410
Redwood	2400	2550	4900	6480
Redwood	2420	2570	4950	6550
Redwood	2440	2590	5000	6620
Redwood	2460	2610	5050	6690
Redwood	2480	2630	5100	6760
Redwood	2500	2650	5150	6830
Redwood	2520	2670	5200	6900
Redwood	2540	2690	5250	6970
Redwood	2560	2710	5300	7040
Redwood	2580	2730	5350	7110
Redwood	2600	2750	5400	7180
Redwood	2620	2770	5450	7250
Redwood	2640	2790	5500	7320
Redwood	2660	2810	5550	7390
Redwood	2680	2830	5600	7460
Redwood	2700	2850	5650	7530
Redwood	2720	2870	5700	7600
Redwood	2740	2890	5750	7670
Redwood	2760	2910	5800	7740
Redwood	2780	2930	5850	7810
Redwood	2800	2950	5900	7880
Redwood	2820	2970	5950	7950
Redwood	2840	2990	6000	8020
Redwood	2860	3010	6050	8090
Redwood	2880	3030	6100	8160
Redwood	2900	3050	6150	8230
Redwood	2920	3070	6200	8300
Redwood	2940	3090	6250	8370
Redwood	2960	3110	6300	8440
Redwood	2980	3130	6350	8510
Redwood	3000	3150	6400	8580
Redwood	3020	3170	6450	8650
Redwood	3040	3190	6500	8720
Redwood	3060	3210	6550	8790
Redwood	3080	3230	6600	8860
Redwood	3100	3250	6650	8930
Redwood	3120	3270	6700	9000
Redwood	3140	3290	6750	9070
Redwood	3160	3310	6800	9140
Redwood	3180	3330	6850	9210
Redwood	3200	3350	6900	9280
Redwood	3220	3370	6950	9350
Redwood	3240	3390	7000	9420
Redwood	3260	3410	7050	9490
Redwood	3280	3430	7100	9560
Redwood	3300	3450	7150	9630
Redwood	3320	3470	7200	9700
Redwood	3340	3490	7250	9770
Redwood	3360	3510	7300	9840
Redwood	3380	3530	7350	9910
Redwood	3400	3550	7400	9980
Redwood	3420	3570	7450	10050
Redwood	3440	3590	7500	10120
Redwood	3460	3610	7550	10190
Redwood	3480	3630	7600	10260
Redwood	3500	3650	7650	10330
Redwood	3520	3670	7700	10400
Redwood	3540	3690	7750	10470
Redwood	3560	3710	7800	10540
Redwood	3580	3730	7850	10610
Redwood	3600	3750	7900	10680
Redwood	3620	3770	7950	10750
Redwood	3640	3790	8000	10820
Redwood	3660	3810	8050	10890
Redwood	3680	3830	8100	10960
Redwood	3700	3850	8150	11030
Redwood	3720	3870	8200	11100
Redwood	3740	3890	8250	11170
Redwood	3760	3910	8300	11240
Redwood	3780	3930	8350	11310
Redwood	3800	3950	8400	11380
Redwood	3820	3970	8450	11450
Redwood	3840	3990	8500	11520
Redwood	3860	4010	8550	11590
Redwood	3880	4030	8600	11660
Redwood	3900	4050	8650	11730
Redwood	3920	4070	8700	11800
Redwood	3940	4090	8750	11870
Redwood	3960	4110	8800	11940
Redwood	3980	4130	8850	12010
Redwood	4000	4150	8900	12080
Redwood	4020	4170	8950	12150
Redwood	4040	4190	9000	12220
Redwood	4060	4210	9050	12290
Redwood	4080	4230	9100	12360
Redwood	4100	4250	9150	12430
Redwood	4120	4270	9200	12500
Redwood	4140	4290	9250	12570
Redwood	4160	4310	9300	12640
Redwood	4180	4330	9350	12710
Redwood	4200	4350	9400	12780
Redwood	4220	4370	9450	12850
Redwood	4240	4390	9500	12920
Redwood	4260	4410	9550	12990
Redwood	4280	4430	9600	13060
Redwood	4300	4450	9650	13130
Redwood	4320	4470	9700	13200
Redwood	4340	4490	9750	13270
Redwood	4360	4510	9800	13340
Redwood	4380	4530	9850	13410
Redwood	4400	4550	9900	13480
Redwood	4420	4570	9950	13550
Redwood	4440	4590	10000	13620
Redwood	4460	4610	10050	13690
Redwood	4480	4630	10100	13760
Redwood	4500	4650	10150	13830
Redwood	4520	4670	10200	13900
Redwood	4540	4690	10250	13970
Redwood	4560	4710	10300	14040
Redwood	4580	4730	10350	14110
Redwood	4600	4750	10400	14180
Redwood	4620	4770	10450	14250
Redwood	4640	4790	10500	14320
Redwood	4660	4810	10550	14390
Redwood	4680	4830	10600	14460
Redwood	4700	4850	10650	14530
Redwood	4720	4870	10700	14600
Redwood	4740	4890	10750	14670
Redwood	4760	4910	10800	14740
Redwood	4780	4930	10850	14810
Redwood	4800	4950	10900	14880
Redwood	4820	4970	10950	14950
Redwood	4840	4990	11000	15020
Redwood	4860	5010	11050	15090
Redwood	4880	5030	11100	15160
Redwood	4900	5050	11150	15230
Redwood	4920	5070	11200	15300
Redwood	4940	5090	11250	15370
Redwood	4960	5110	11300	15440
Redwood	4980	5130	11350	15510
Redwood	5000	5150	11400	15580
Redwood	5020	5170	11450	15650
Redwood	5040	5190	11500	15720
Redwood	5060	5210	11550	15790
Redwood	5080	5230	11600	15860
Redwood	5100	5250	11650	15930
Redwood	5120	5270	11700	16000
Redwood	5140	5290	11750	16070
Redwood	5160	5310	11800	16140
Redwood	5180	5330	11850	16210
Redwood	5200	5350	11900	16280
Redwood	5220	5370	11950	16350
Redwood	5240	5390	12000	16420
Redwood	5260	5410	12050	16490
Redwood	5280	5430	12100	16560
Redwood	5300	5450	12150	16630
Redwood	5320	5470	12200	16700
Redwood	5340	5490	12250	16770
Redwood	5360	5510	12300	16840
Redwood	5380	5530	12350	16910
Redwood	5400	5550	12400	16980
Redwood	5420	5570	12450	17050
Redwood	5440	5590	12500	17120
Redwood	5460	5610	12550	17190
Redwood	5480	5630	12600	17260
Redwood	5500	5650	12650	17330
Redwood	5520	5670	12700	17400
Redwood	5			

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Table 1. The mean number of correct responses for each age group on the 10-item test. The mean number of correct responses for each age group is shown in parentheses. The mean number of correct responses for each age group is shown in parentheses.

[illegible][illegible]

Source: National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N. W., Washington, D. C. 20005, art. 2.4, Table 2A.

Author comments: When stress in end-grain bearing exceeds 75% of above values, bearing shall be on a metal plate or strap, or on other durable, rigid, homogeneous material of adequate strength, per Source. (This criterion should be applied to both top and bottom ends of each stud or post.)

Use a bottom plate consisting of stress-graded selected dimension lumber (better than, say, utility grade), used flatwise, "2x" and at least as wide as the greater dimension of the columns (preferably wider) for a column spacing of 12 in. to 24 in.

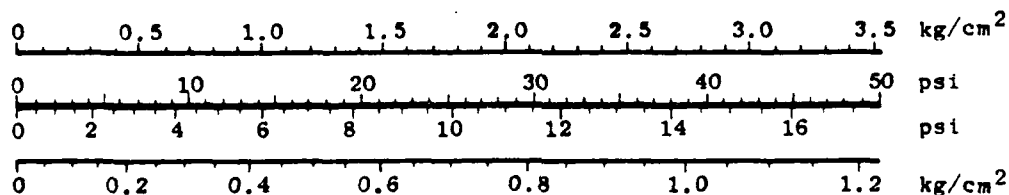
For greater column spacings, use any blocking available under each column flatwise and perhaps stacked, preferably with the bottom block wider than just prescribed for a single member bottom plate.

For column spacings of, say, 4 ft or more, one should leave off thinking of a continuous bottom plate as the column spacing grows larger and take recourse in use of a grillage under each column; the basic premise is to spread the load on the floor slab and underlying soil.

Peak Air Blast Resistance Capacity - Side-on versus Head-on

Peak applied air blast pressure resistance capacity (psi), determined above for the particular applied situation (with its design stresses from the grading association), was just that, as felt by the structure member. If the member is used so that the blast wave strikes it head-on, e.g., as if the member is part of the front wall of a building struck by the blast wave, then the blast wave is fully reflected, making it equivalent in loading force to a much stronger wave applied only side-on. To relate these two situations by putting both in terms of air blast peak free-field overpressure resistance (that is, out in the open, unaffected by structures), use the scales below:

FREE-FIELD OVERPRESSURE (WHEN APPLIED SIDE-ON)



FREE-FIELD OVERPRESSURE (WHEN APPLIED HEAD-ON OR FULLY REFLECTED)

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

NOTATION

- A = area of beam cross-section (in.²)
- b = width of beam (in.)
- C_d = drag coefficient (= ratio of drag pressure on object to dynamic/wind pressure in free field)
- c, c' = dimensionless coefficients
- d = depth of wood beam (in.)
- d = for columns, see figure in Table B1-3 (in.)
- F_b = extreme fiber stress in bending (psi)
- F_{db} = dynamic F_b (psi)
- F_c = compression stress parallel to grain (psi)
- F_c' = F_c corrected for buckling hazard (psi)
- F_{c1} = compression stress perpendicular to grain, or bearing stress (psi)
- F_{dc1} = dynamic F_{c1} (psi)
- F_v = horizontal shear stress (in wood) (psi)
- F_{dv} = dynamic F_v (psi)
- K = column slenderness ratio; see Table B1-3
- L = span length of beam (center-center of supports unless otherwise indicated) (in.)
- L = column overall length or unsupported length (see Table B1-3) (in.)
- L' = bearing length at each end of wood beam (in.)
- M = bending moment (in.-lb)
- P = total load (TL) column load capacity (normal use) (lb)
- P_d = total load (TL) column load blast capacity (lb)
- P_{dm} = peak (unit) value of applied (air blast) loading (psi)
- P_r = peak reflected (air blast) overpressure (psi)

NOTATION (concluded)

p_{so} = peak side-on (air blast) overpressure (psi)

q = resistance of member, ultimate (psi)

q_b = bending q (psi)

q_n = resistance under normal use design (psi)

q_v = horizontal shear q (psi)

S = section modulus (in.³)

U = shock velocity (air blast) (fps)

V = vertical shear (lb)

w = load per unit length of beam (psf)

μ = ductility ratio (of maximum deflection to yield deflection)

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¹² Now SRI International.

¹³ Now Federal Emergency Management Agency

Appendix B2

HOME BASEMENTS UPGRADING IN HOST AREAS

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Appendix B2

HOME BASEMENTS UPGRADING IN HOST AREAS¹

Introduction

With the advent of CRP (Crisis Relocation Planning) there is considerable interest in upgrading the floor over host area basements, in order that mass may be added to that floor (commonly soil over a sheet of plastic film) for fallout protection, perhaps even upgraded enough for say, 2 or 3 psi peak air blast overpressure protection, in addition to the added soil for fallout protection.

Commonly such floors are of wood subflooring (and often with wood finish flooring); carried by wood joists (with cross-bridging or blocking); in turn carried on the outer basement walls and a steel beam on a pipe column(s) running along the long-direction centerline of the basement. The pipe column(s) are supported on some kind of footing to spread the load, or infrequently on piling.

The prescribed floor live loading for a residential dwelling first floor is, and has been, 40 psf (pounds per square foot) [1(p.5-222)]² under usual allowable design stresses [1,2].

All wood members that are depended upon (designed) to carry specified or calculated loads must be stress-graded by species and type of member, which data can in turn be used to look up allowable design stresses [2].

The discussion herein is for a "typical" basement, that is, one containing as many commonly-used features as possible; variations are many!

To discuss each of the structural elements just mentioned:

(1) The flooring must be not only strong enough for the 40 psf design live load plus the dead load of the flooring, both uniformly distributed loads, but also strong enough to carry safely (to the joists) the intermittent concentrated loads, such as that from heavy household

¹ Support for preparation of this appendix was provided under FEMA contracts with the Center for Planning and Research, Inc., Palo Alto, California and SRI International, Menlo Park, California.

² Brackets are used herein to indicate sources in the References list at the end of this appendix.

appliances. The result is that the flooring, considered by itself, is stronger than enough to handle the uniformly distributed dead load and 40 psf design live load.

(2) The floor joists - say, 2x6 or deeper,³ and blocked or cross-bridged to prevent twisting along their bottom edges - are supported on an outer basement wall (along the long side of the basement, so that joist spans are in the shorter dimension of a rectangular house or of a rectangular portion of a larger house). Each floor joist is supported also on the steel beam discussed further below, and seldom if ever does one continuous wood joist run from one outer basement wall to the other but most often consists of two joists nailed together (for erection purposes, not flexural continuity) over the steel beam. Floor joists are used at intervals of 12 in. or 16 in. (rarely at 24 in.), measured center-to-center. Joist design must also consider (as prescribed by local building code) concentrated live loads, but the uniformly distributed 40 psf live load (also prescribed) usually controls design because of the ability of the joist blocking or cross-bridging to distribute concentrated and other loads to adjoining joists. In fact, the joists (and the flooring materials directly supporting loads) may be totally prescribed by the local building code, thus requiring no design work except perhaps for prudent checking.

(3) The supporting steel beam⁴ for the floor joists, and its pipe⁵ column(s) and footing(s):

(a) May carry only its share of the first floor live (40 psf) and dead (weight of materials built into the building) loads;

(b) More likely, however, the steel beam will carry directly above it an interior bearing partition that in turn carries half of the total ceiling load and attic load (latter, if habitable, 30 psf live load; uninhabitable, 20 psf [1]); and,

(c) Half the roof dead and live loads (latter is 20 to 100 or more psf, per local building code) are sometimes carried by an attic bearing partition (or row of columns) through the first floor bearing partition to the supporting steel beam under the first floor.

³ Nominal dimensions; actual (finished) dimensions are smaller, i.e., a 2x6 is 1½ in. x 5½ in. (current) or 1-5/8 in. x 5½ in. (formerly).

⁴ What is described herein as a supporting steel beam may be, instead, a built-up or solid wood beam, but such substitution has little effect on the discussion in and purpose of this report.

⁵ The most common support; however, another steel shape or a wood member(s) may be used.

Returning to the general discussion: A floor system, especially one with wood flooring and joists, is designed to meet both deflection and strength limits, with deflection often the controlling criterion (one dislikes walking on a springy floor, regardless of repeated assurances that it is safe). What this means, however, is that the floor is actually stronger than it needs to be TO CARRY LOADS, that is, to carry more than the prescribed design live and dead loads and still remain within safe design stresses. Further, the allowable stresses have built-in factors of safety that are reduced for unusual loads such as those from wind and earthquake, so can certainly be exploited for such an emergency need as supplementing the building's fallout and low level air blast protection. Finally, actual live loads expected to occur are usually less than prescribed (code) live loads described earlier; for example, a residential basement converted to a fallout shelter by adding soil a foot or so deep on the first floor is not apt also to have its usual live loads (people and furniture) to consider, certainly not to normal use levels.

Design/Analysis Annex Work

The Annex shows an analysis of an assumed floor joist system (2x8s on 16 in. centers) in its usual and an upgraded situation (support added at mid-span). Other supporting work included the design of top beams to transfer loads from the joists to columns spaced to carry $1\frac{1}{2}$ joists per column, and the design/analysis of several potential column sizes. The work and results are described in more detail for each member type in the following paragraphs.

(1) Annex sheets provide:

(a) Notation used.

(b) A tabulation of design stresses for one lower and one higher strength, widely available, wood species of two grades each, the latter to cover Dimension Lumber of two sizes: 2 in. to 4 in. thick, 4 in. wide; and 2 in. to 4 in. thick, 5 in. and wider.

(c) A set of wood beam design formulas for uniformly distributed loads: flexure; horizontal shear; end bearing length required; and, deflection limits as prescribed by building codes.

(d) A set of column design formulas.

(2) Another sheet (Usual Floor Joist Design) uses the above design formulas and allowable stresses to develop the allowable clear span for 2x8s on 16-in. centers, when using the lower and higher strength woods described above. Allowable spans (center-center of supports) proved to be 11 ft and 13 ft 4 in. for lower and higher strengths, respectively. The work uses a live load of 40 psf and a dead load of 10 psf.

But concern here is for host area shelter (see lower portion of the Annex sheet):

First, the deflection criteria may be ignored without hazard to the structure, which in this example increases the allowable live load for the higher strength wood (keeping the span unchanged);

Second, fallout shielding soil loads might be assumed as applied for two months or less, for which the design code allows an increase of 15 percent in live load;

Third, air blast loads only are impact live loads, for which the code allows a 100 percent increase (not additive to the above 15 percent!) in live load, to which one might add a 30 percent increase representing the current estimated factor of safety in wood grading (at 5 percent risk probability); and,

Fourth, combined shielding soil and air blast live loads may be handled as shown on the Annex sheet.

The approach leaves the residential basement undamaged by fallout protection, but risks damage if an attack occurs. The last admonition on the sheet, "re-check adequacy of end bearings," may be met by using Equations 10-12 on the Annex sheet, Wood Beam Design Formulas.

Dynamic factors for air blast are ignored in this appendix, because it is felt that a home getting less than, say, 5 psi air blast peak free field loading will feel an interior air blast pulse with a rise time long enough to make the effect no worse than that from a softly applied static load of short duration.

(3) The next Annex sheet begins the design work by assuming that these 2x8 joists on 16-in. centers have added mid-span supports installed for upgrading; thus the same lower and higher strength joists have their spans changed to 5 ft 6 in. and 6 ft 8 in. (both center to center), respectively. Deflection criteria are NOT used for the emergency purposes contemplated by upgrading work.

The sheet shows the design calculations and results: fallout soil (assuming 100 pcf) shielding capacity alone of soil 26.4 in. thick and 29.4 in. thick, for lower and higher strength woods, respectively; and, an example combined protection capacity of 12 in. deep soil (assuming 95 pcf), plus 1.96 psi and 2.36 psi blast overpressure (long duration), with the technique shown so that other combinations of fallout and blast protection may be calculated. Required bearing lengths are also shown on the sheet.

The adequacy of the flooring materials, to support these new total loads, was assumed to be sufficient for the reason stated in paragraph 1 of the Introduction. Such adequacy should be checked, of course; the approach for the flooring materials is the same as for any wood beam examined with the use of the Annex sheet on beam formulas.

(4) The work included the design of top beams (3 sheets), showing those with capacities of interest for joist loads of the range encountered above. Results⁶ of the top beam designs are shown in Table B2-1. Top beam capacities shown exclude the allowable stress increases (15 and 100 percent) for short duration loads discussed above under floor joist design; similar increases apply to top beams.

(5) The work also covered examination of various sizes of columns (3 sheets) for their load capacity, with the rectangular, non-square sizes also checked for capacity where mid-height supports (and at third-points in some cases) in the weak direction have been added. Results are shown in Table B2-2.

Figure B2-1 shows schemes for floor joists upgrading: one with wood posts at a spacing of $1\frac{1}{2}$ floor joists per post, with top beams to transfer joist loads to the posts; the other with a stud directly under each joist and using a top (or tie) plate to prevent twisting of the bottom edges of the joists. The top (or tie) plate may be unneeded if the joists have adequate blocking or cross-bridging (and each joist bottom edge has enough bearing area for the top of a stud).

Applications

Obviously, if one could go into a house and identify fully the wood floor joist material used (species, size and grade of wood) and happened to have a copy of the applicable grading rules (shirt-pocket size for the particular grading association OR Reference [2] with back editions), design stresses could be applied as they have been in the preceding section and the Annex, leading to analysis of the joists and design of joist upgrading and of top beams and columns. Such a situation is unlikely to apply.

An alternate approach might be as described in the following paragraphs:

(1) Assume that the floor over the basement was built according to a building code calling for 40 psf live load on the first floor and allowing 10 psf for dead load (weight of flooring materials and joists). (Many buildings were not built according to code or the code was not enforced, but many are stronger (especially if owner built) and probably a lot are weaker (e.g., as in speculative housing built to cut corners).)

⁶ Numerical results are shown to illustrate the work, not to imply a degree of accuracy in design/analysis.

Table B2-5

TOP BEAM CAPACITIES

1st Member Design
 Spacing Lower Upper

Top Beam Capacities of 2 Beams per Column

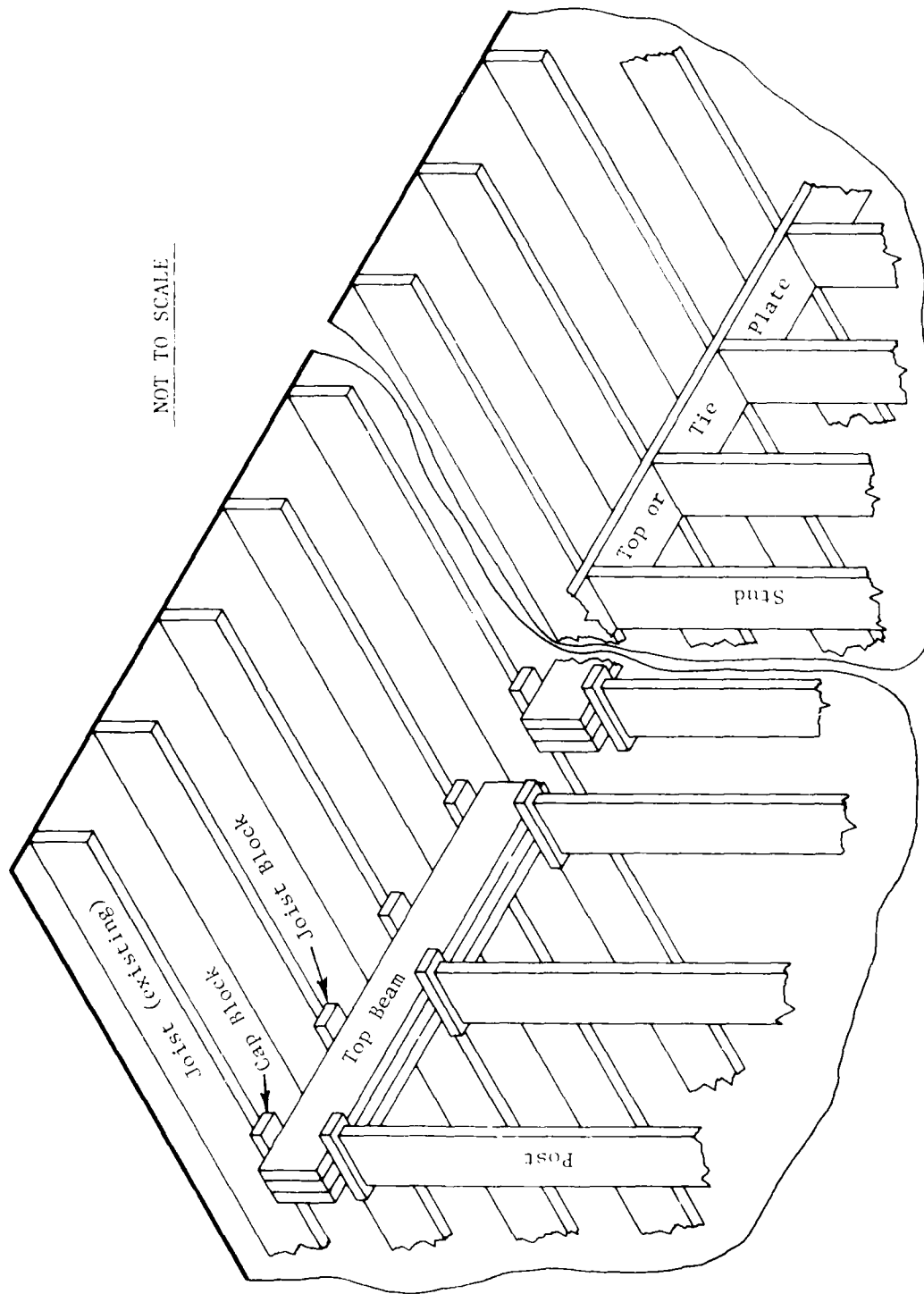
Triple 2x8s, edgewise	12"	}	2509	1081	11.31st
	16"				
	24"				
Quadruple 2x6s, edgewise	12"	}	2588	1024	11.31st
	16"				
	24"				

NOTE: See Annex sheets (3), Top Beams, for design method including required bearing lengths on top and bottom surfaces of these two top beam schemes.

Table B2-7
COLUMN CAPACITIES

		Member Strengths	
		Lower	Higher
Column Capacities (8 ft long) in pounds:			
2x4 column (too long and slim, per code)			
a.	2x4 column, with mid-ht support: (meaning laterally, in weak direction)	1848	2305
b.	4x4 column:	5782	7327
2x6 column (too long and slim, per code)			
c.	2x6 column, with mid-ht support:	3143	4348
d.	2x6 column, with 3rd-point supports:	5346	8621
2x8 column (too long and slim, per code)			
e.	2x8 column, with mid-ht support:	4143	5731
f.	2x8 column, with 3rd-point supports:	7047	11364
g.	4x6 column:	9914	13822
h.	4x6 column, with mid-ht support:	13302	22292

NOTE: Above capacities should be reduced for the column's own weight and may be increased for various live load durations (see Annex sheet 2 of Columns for Upgrading).



NOT TO SCALE

Figure B2-1A FLOOR JOISTS UPGRADING

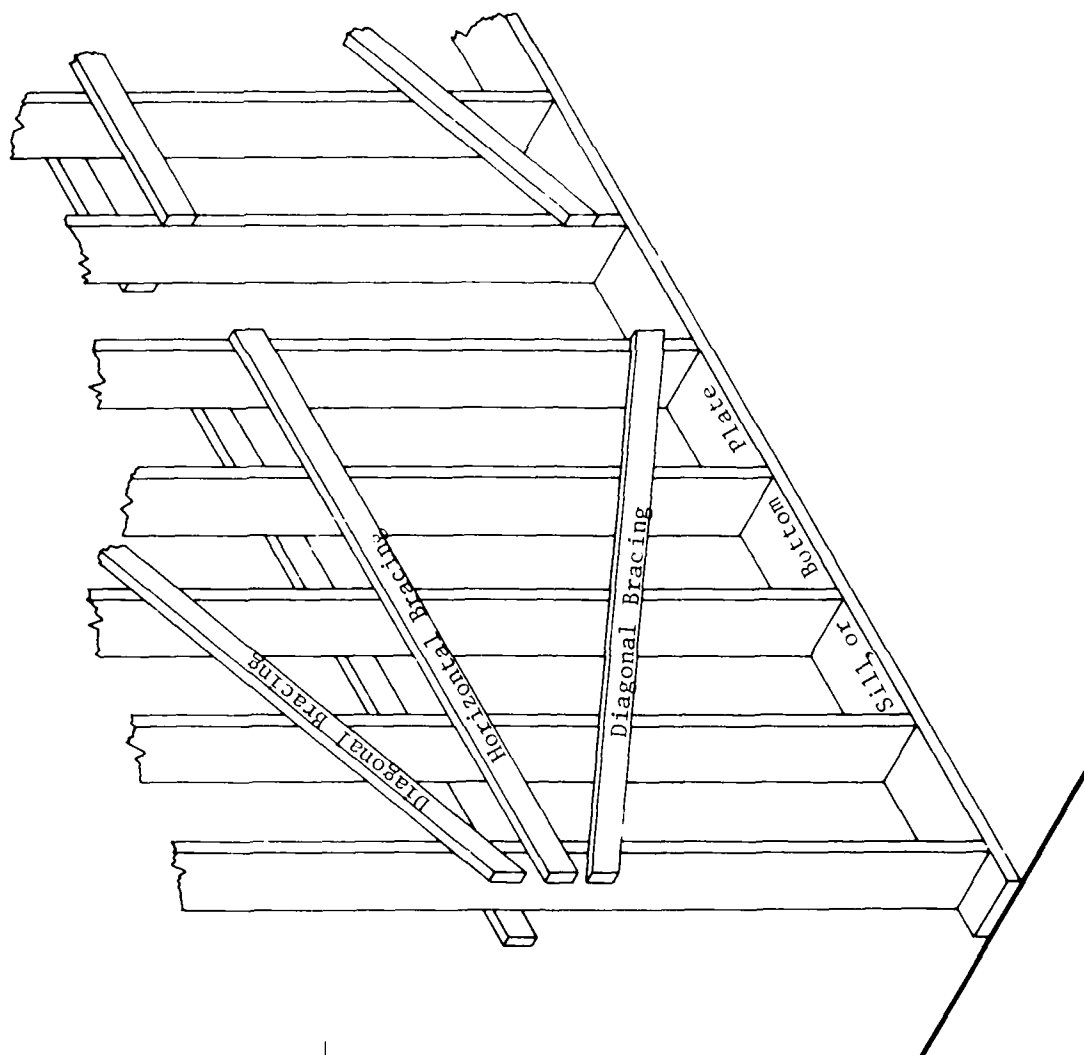


Figure B2-1B

NOT TO SCALE

(2) Measure the actual joist width b , depth d , and center-to-center span⁷ L of the joist being checked. Also measure the c-c. spacing j between joists. Use all four dimensions in inches, unless otherwise indicated below.

(3) Enter Table B2-3, using the nominal b and d (see Table B2-4), and actual j and L dimensions (all in inches, except L in feet and inches here); read the values for E and F_b . (For example: 2x8s on 16 in. centers, span 11 ft 8 in., read E as 1,200,000 psi and F_b as 1,040 psi.) Interpolate as necessary if the table doesn't include the specific field measurements obtained. If the actual b and d measurements agree with the column of Table B2-4 headed "Actual (older)," multiply the E value read in Table B2-3 by the "Corr. Factor" shown in Table B2-4.

As an alternative to the Table B2-3 approach, use the following formulas (no "Corr. Factor"; all actual dimensions, in inches):

$$E = 15.625 (j / b) (L / d)^3 \quad (\text{psi})$$

$$F_b = 0.26042 (j / b) (L / d)^2 \quad (\text{psi})$$

Both table and formula approaches assume floor designs based on 40 psf live load, 10 psf dead load, and live load deflection limited to 1/360th of the span.

(4) Values for F_v and F_{c1} (both in psi) can be calculated from the following:⁸

$$F_v = 22.6736 + 71.3296 (E / 1,000,000)$$

$$F_{c1} = -135.76 + 313.498 (E / 1,000,000)$$

⁷ Joist span is measured between the centers of its supports, usually a steel beam down the basement center and an outer basement wall.

⁸ Least squares regression equations shown were obtained by use of elasticity moduli and allowable stress values [2] for six grades each of eight species of widely available construction woods: Douglas Fir-Larch; Western Hemlock; Ponderosa, Sugar, Idaho White, and Lodgepole Pines; Southern Pine; and Northern Pine; all when used at 19 percent or less moisture content. (Number of significant digits shown does not imply a degree of accuracy in the results.) Range of E values used was 1.0 to 1.8 million psi; of F_v , 100-140 psi; of F_{c1} , 190-405 psi. With 36 pairs of values used for each of the two equations, the range of percentage differences between each actual value and value calculated with the applicable equation was: for F_v , 16.9 percent to -18.3 percent; for F_{c1} , 33.9 percent to -37.3 percent.

Table B2-3

ALLOWABLE SPANS FOR FLOOR JOISTS
(40 psf LL and 10 psf DL.)

DESIGN CRITERIA: Deflection - For 40 lbs. per sq. ft. live load. Limited to span in inches divided by 360. Strength - Live load of 40 lbs. per sq. ft. plus dead load of 10 lbs. per sq. ft. determines the required fiber stress value.

JOIST SIZE (IN.)		SPACING (IN.)				Modulus of Elasticity, "E", in 1,000,000 psi											
		08	09	10	11	12	13	14	15	16	17	18	19	20	22		
2x6	12 0	8-6 720	8-10 780	9-2 830	9-6 890	9-9 940	10-0 990	10-3 1040	10-6 1090	10-9 1140	10-11 1190	11-2 1230	11-4 1280	11-7 1320	11-11 1410		
	16 0	7-9 790	8-0 860	8-4 920	8-7 980	8-10 1040	9-1 1090	9-4 1150	9-6 1200	9-9 1250	9-11 1310	10-2 1360	10-4 1410	10-6 1460	10-10 1550		
	24 0	6-9 900	7-0 980	7-3 1050	7-6 1120	7-9 1190	7-11 1250	8-2 1310	8-4 1380	8-6 1440	8-8 1500	8-10 1550	9-0 1610	9-2 1670	9-6 1780		
2x8	12 0	11-3 720	11-8 780	12-1 830	12-6 890	12-10 940	13-2 990	13-6 1040	13-10 1090	14-2 1140	14-5 1190	14-8 1230	15-0 1280	15-3 1320	15-9 1410		
	16 0	10-2 790	10-7 850	11-0 920	11-4 980	11-8 1040	12-0 1090	12-3 1150	12-7 1200	12-10 1250	13-1 1310	13-4 1360	13-7 1410	13-10 1460	14-3 1550		
	24 0	8-11 900	9-3 980	9-7 1050	9-11 1120	10-2 1190	10-6 1250	10-9 1310	11-0 1380	11-3 1440	11-5 1500	11-8 1550	11-11 1610	12-1 1670	12-6 1780		
2x10	12 0	14-4 720	14-11 780	15-5 830	15-11 890	16-5 940	16-10 990	17-3 1040	17-8 1090	18-0 1140	18-5 1190	18-9 1230	19-1 1280	19-5 1320	20-1 1410		
	16 0	13-0 790	13-6 850	14-0 920	14-6 980	14-11 1040	15-3 1090	15-8 1150	16-0 1200	16-5 1250	16-9 1310	17-0 1360	17-4 1410	17-8 1460	18-3 1550		
	24 0	11-4 900	11-10 980	12-3 1050	12-8 1120	13-0 1190	13-4 1250	13-8 1310	14-0 1380	14-4 1440	14-7 1500	14-11 1550	15-2 1610	15-5 1670	15-11 1780		
2x12	12 0	17-5 720	18-1 780	18-9 830	19-4 890	19-11 940	20-6 990	21-0 1040	21-6 1090	21-11 1140	22-5 1190	22-10 1230	23-3 1280	23-7 1320	24-5 1410		
	16 0	15-10 790	16-5 860	17-0 920	17-7 980	18-1 1040	18-7 1090	19-1 1150	19-6 1200	19-11 1250	20-4 1310	20-9 1360	21-1 1410	21-6 1460	22-2 1550		
	24 0	13-10 900	14-4 980	14-11 1050	15-4 1120	15-10 1190	16-3 1250	16-8 1310	17-0 1380	17-5 1440	17-9 1500	18-1 1550	18-5 1610	18-9 1670	19-4 1780		

¹The required extreme fiber stress in bending, F_b , in pounds per square inch is shown below each span.

²Use single or repetitive member bending stress values (F_b) and modulus of elasticity values (E).

³For more comprehensive tables covering a broader range of bending stress values (F_b) and Modulus of Elasticity values (E), other spacing of members and other conditions of loading, see the Uniform Building Code.

⁴The spans in these tables are intended for use in covered structures or where moisture content in use does not exceed 19 percent.

Source: Dwelling Construction Under the Uniform Building Code, 1976 Edition,
International Conference of Building Officials, Whittier, California
90601

Table B2-4

JOIST DIMENSIONS - WIDTH x DEPTH (in.)

Nominal	Actual (current)	Actual (older) & E Corr. Factor
2x6	1.5x5.5	1-5/8 x 5-1/2 0.923
2x8	1.5x7.25	1-5/8 x 7-1/2 0.834
2x10	1.5x9.25	1-5/8 x 9-1/2 0.852
2x12	1.5x11.25	1-5/8 x 11-1/2 0.864

(5) Plan to install an additional support line under the mid-span points of all joists. Identify the measurements made under paragraph 2 as b for actual width of joist, d for actual depth, j for joist spacing center-to-center (c-c.), and change L to HALF the c-c. span measured for the full, original joist (paragraph 2), all in inches. Calculate the allowable unit total load TL on the joists with new mid-span supports as follows (TL in pounds per square foot (psf)):

$$TL = 154 \text{ bdF}_v / (j (L-2d))$$

$$\text{or} = 192 \text{ bd}^2\text{F}_b / (jL^2)$$

whichever is less.

About 10 psf of this TL will roughly cover the flooring materials, joists, top beam if any, and posts or studs, so that the dead load DL of 10 psf (approximately) can be subtracted from the TL to obtain the allowable live load LL on the floor. The equations exclude any allowed stress increases for load duration of two months or less (15 percent) or impact (100 percent), or for removing the design factor of safety (30 percent). Such increases are handled as described in paragraph 2 of the preceding section, Design/Analysis Annex Work; the comments there about the adequacy of the flooring to carry the above TL between floor joists, also apply here.

(6) If p is the total load (pounds) per joist, as transmitted to the top beam if any, or directly to a stud, it equals the TL (as calculated, but plus any allowed percentage increases, both as just described) times the contributory (floor) area, jL; that is:

$$p = (TL) jL / 144.$$

(7) The required joist bearing area, bL' (actual width b; bearing length L' along joist; both in inches) on the bottom of the joist at mid-span can be calculated from the following:

$$bL' = p / F_{c1}$$

with L' in inches, using F_{c1} (psi) as that for the joist (paragraphs 2-4 above).

If the bL' required by the joist is not provided by the stud or top beam, a wood block may be needed under the joist (even if the block crushes under blast loading, it will be absorbing energy), or recourse to a steel plate may be necessary.

The required bearing area on the top and underside of a top beam can be similarly calculated, but using the appropriate F_{c1} value for the top beam wood; calculation sheets in the Annex also show the methods.

It is assumed that upgrading materials used, presumably obtained from lumberyards, will be known as to species, size and grade of wood, and thus that allowable design values - F_b (both single- and repetitive-member use), F_c , F_{c1} , F_v and E - are obtainable from Reference [2] or the applicable association grading rules. Reference [2] includes associations' addresses, and it is urged that the rules be used, time permitting, to verify the grading of each piece of lumber.

(8) Top beams may be omitted by putting a stud under each joist, using a scab board on each side of the stud-joist junction if both members are the same width, or using a top (or tie) plate (Figure B2-1).

If top beams are to be used, posts should be located under the two end joists to be served by each top beam. If the post spacing is to be other than $1\frac{1}{2}$ times the joist spacing, a new top beam design should be prepared by a competent civil engineer with wood design experience. For post spacing at $1\frac{1}{2}$ times the joist spacing, design work has been done, as reported in the preceding section and in Table B2-1. The following calculations may be used to generalize, based on the column spacing at $1\frac{1}{2}$ times the joist spacing ONLY.⁹

Four top beam designs (2 sizes, each at two strengths) are shown in Table B2-1; they may be used as follows:

Top Beam Capacities ($1\frac{1}{2}$ floor joists per column):

Triple 2x8s, edgewise:⁹

$$p = 25.6 F_v$$

Quadruple 2x6s, edgewise:⁹

$$p = 25.9 F_v$$

with the load-per-joist p in pounds, excluding all add-on percentages related to load duration and factor of safety.

(9) Eight column designs (all 8 ft long and using "2x" or "4x" members) are identified as "a." through "h." in Table B2-2; some may be generalized as follows (P is allowable axial load in pounds):

a. $P = 0.00154 E$

e. $P = 0.00319 E$

c. $P = 0.00242 E$

b., d., f., g., and h.: Design column, using "Column Formulas" of Annex.

⁹ Checked for joist spacings of 24 in. or less.

Design values for E and F_c should be obtained as described in paragraph 7 above.

The column capacities in Table B2-2, including those from formulas just above, include no duration of load increases - see NOTE on Table B2-2.

Adequacy of these columns for end grain in bearing (Idaho White Pine = 1390 psi and Douglas Fir-Larch = 2020 psi applied to the lower and higher member strengths, respectively, see Table B2-5 fourth column) has been checked and all are adequate. Note that the F_g design values of Table B2-5 are subject to the same duration of loading increases (15% and 100% for example), as are the column capacities of Table B2-2.

(10) Sill or bottom plate "design" could be complex, involving a beam with concentrated loads (the columns, plus foot plates if any) and supported by an elastic/plastic/fracturing foundation (the usual light, concrete floor slab). Such design is considered unwarranted for the purposes herein. It should be accepted that the floor slab will crack/break up and will be pushed down, under and adjacent to the bottom plate, should the upgraded floor system over the basement receive an air blast overpressure loading; such action absorbs energy. Recent tests show a very high floor slab resistance to column punching from blast duration loads (see page 18 of main text).

For the added loading of soil placed on the first floor for fallout shielding upgrading, it is doubted that the concrete basement floor will experience anything more than localized cracking, if that, under the following approach:

Use a bottom plate consisting of stress-graded selected dimension lumber (better than, say, utility grade), used flatwise, "2x" and at least as wide as the greater dimension of the columns (preferably wider) for a column spacing of 12 in. to 24 in.

For greater column spacings, use any blocking available under each column flatwise and perhaps stacked, preferably with the bottom block wider than just prescribed for a single member bottom plate.

For column spacings of, say, 4 ft or more, one should leave off thinking of a continuous bottom plate as the column spacing grows larger and take recourse in use of a grillage under each column; the basic premise is to spread the load on the floor slab and underlying soil.

Design graphs for wood beams and columns are included in Appendix B1 herein.

Table B2-5

END GRAIN IN BEARING (psi)

Design values for end grain in bearing parallel to grain on a rigid surface F_g
in pounds per square inch

Species	Wet service conditions ¹	Dry service conditions ¹		
		Sawn lumber ²		Glued laminated timber
		More than 4" thick	Not more than 4" thick ³	
Ash, Commercial White	1370	1510	2060	2400
Aspen	740	820	1110	1300
Balsam Fir	890	980	1330	1560
Beech	1190	1310	1780	2080
Birch, Sweet and Yellow	1150	1260	1720	2010
Black Cottonwood	620	690	930	1090
California Redwood (Close grain)	1560	1720	2270	2620
California Redwood (Open grain)	1150	1270	1670	1940
Coast Sitka Spruce	950	1040	1420	1660
Coast Species	950	1040	1420	1660
Cottonwood, Eastern	765	840	1150	1340
Douglas Fir - Larch (Dense) ⁴	1570	1730	2360	2750
Douglas Fir - Larch ⁴	1340	1480	2020	2350
Douglas Fir South	1220	1340	1820	2130
Eastern Hemlock - Tamarack ⁴	1150	1270	1730	2020
Eastern Spruce	970	1070	1460	1700
Eastern White Pine ⁴	900	1000	1360	1580
Eastern Woods	820	900	1230	1440
Engelmann Spruce - Alpine Fir	810	890	1220	1420
Hem. Fir ⁴	1110	1220	1670	1940
Hickory and Pecan	1370	1510	2050	2400
Idaho White Pine	930	1020	1390	1630
Lodgepole Pine	970	1060	1450	1690
Maple, Black and Sugar	1140	1260	1710	2000
Mountain Hemlock	1070	1170	1600	1870
Northern Aspen	740	810	1110	1290
Northern Pine	1040	1150	1570	1830
Northern Species	880	970	1320	1540
Northern White Cedar	740	810	1110	1290
Oak, Red and White	1060	1160	1590	1850
Ponderosa Pine - Sugar Pine	910	1000	1370	1600
Red Pine	880	970	1320	1540
Sitka Spruce	990	1090	1480	1730
Southern Cypress	1330	1460	1990	2320
Southern Pine (Dense)	1540	1690	2310	2690
Southern Pine	1320	1450	1970	2300
Spruce - Pine - Fir	940	1040	1410	1650
Sweetgum and Tupelo	1020	1120	1530	1780
Western Cedars ⁴	1040	1140	1520	1750
Western Hemlock	1240	1360	1860	2170
Western White Pine	930	1030	1400	1630
White Woods (Western Woods)	810	890	1220	1420
West Coast Woods (Mixed Species)	810	890	1220	1420
Yellow Poplar	890	980	1340	1560

1. Wet and dry service conditions are defined in 4.1.4 for sawn lumber and 5.1.5 for glued laminated timber.

2. Applies to sawn lumber members which are at a moisture content of 19 percent or less when full design load is applied, regardless of moisture content at time of manufacture.

3. When 4-inch or thinner sawn lumber is surfaced at a moisture content of 15 percent or less and is used under dry service conditions, the values listed for glued laminated timber may be applied.

4. Values also apply when species name includes the designation "North".

Source: National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N. W., Washington, D. C. 20036; art. 2.3, Table 2A.

Author Comments: When stress in end-grain bearing exceeds 75% of above values, bearing shall be on a metal plate or strap, or on other durable, rigid, homogeneous material of adequate strength, per Source. (This criterion should be applied to both top and bottom ends of each stud or post.)

ANNEX

Stress-graded (visually only) lumber species-sizes-grades ¹ commonly available in local lumberyards were narrowed down to the following, categorized as "higher strength" and "lower strength" for use in an earlier study [4,p.C-3&5]:

Allowable Design Stresses in Normal Use (psi)

<u>Size & Grade</u>	<u>F_b</u> ²	<u>F_v</u> ³	<u>F_{c⊥}</u> ⁴	<u>F_c</u>	<u>E</u>	<u>F_g</u> ⁵
<u>2 in. to 4 in. th., 4 in. wide</u>						
Construction ⁶	1200 (1050)	140	385	1150	1500000	2020
Standard ⁷	450 (400)	100	190	650	1200000	1390
<u>2 in. to 4 in. th., 5 in. & wider ⁸</u>						
No. 1 ⁶	1750 (1500)	140	385	1250	1800000	2020
No. 2 ⁷	925 (850)	100	190	725	1300000	1390

¹ Reference [2] contains a full listing of wood species, sizes, grades, design values, etc. However, it lacks two needed tables from Reference [3], which are included herein as Tables B2-5 and B2-6.

All of above is Dimension Lumber; surfaced dry or surfaced green; used at 19 percent maximum moisture content.

² Repetitive-member use; single-member use values in parentheses [2,p.20]. Both uses are edgewise; for flatwise-use (increase) factors, see Reference [2,p.16].

³ Values taken from Reference [3,art.3.4.4.2]; see Table B2-6.

⁴ For bearing lengths L' less than 6 in. long and not nearer than 3 in. to end of member, correct F_{c⊥} by (1 + 0.375 / L')

⁵ Values taken from Reference [3,Table 2A,p.7]; see Table B2-5.

⁶ "Higher strength"; stresses for Douglas Fir-Larch [2].

⁷ "Lower strength"; stresses for Idaho White Pine, but useful (practically) for other Western Pines (Ponderosa, Sugar and Lodgepole) [2].

⁸ But not more than 12 in. [5,p.16-17].

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Table B2-6

ALLOWABLE HORIZONTAL SHEAR VALUES F_v (psi)

	Maximum Moisture Content		
	Unseasoned	19 percent	15 percent
Aspen	85	90	95
Balsam Fir	85	95	95
Black Cottonwood	70	75	80
California Redwood	115	120	130
Coast Sitka Spruce	90	95	100
Coast Species	90	95	100
Douglas Fir-Larch	130	140	145
Douglas Fir-South	130	140	145
Eastern Hemlock-Tamarack	120	130	135
Eastern Spruce	95	105	110
Eastern White Pine	90	95	100
Eastern Woods	85	95	95
Engelmann Spruce/Alpine Fir	95	105	110
Hem-Fir	105	110	115
Idaho White Pine	95	100	105
Lodgepole Pine	95	105	110
Mountain Hemlock	130	140	150
Northern Aspen	90	95	100
Northern Pine	100	105	110
Northern Species	90	95	100
Northern White Cedar	85	95	100
Ponderosa Pine-Sugar Pine	100	105	110
Red Pine	100	110	115
Sitka Spruce	105	115	120
Southern Pine	125	135	145
Spruce-Pine-Fir	95	105	110
Western Cedar	100	105	110
Western Hemlock	125	135	145
Western White Pine	90	100	105
White Woods (Western Woods, West Coast Woods, Mixed Species)	95	100	105

Source: National Design Specification for Wood Construction, 1977 Edition,
National Forest Products Association, 1619 Massachusetts Avenue, N.W.,
Washington, D. C. 20036; art. 3.4.4.2

Author Comments: Use column of above table that specifies "19 percent"
moisture content. For wood species not shown, consult "Design Values
for Wood Construction," 1980 Supplement to above Source.

WOOD BEAM DESIGN FORMULAS - UNIFORMLY DISTRIBUTED LOADS

With simple end supports and with continuous supports, beam span L for horizontal shear design is measured center-to-center (c-c.) of supports. L is the same with continuous supports for flexure and deflection design; but with simple end supports, beam span L for flexure and deflection design should be the beam length between support faces plus half the REQUIRED bearing length L' at each end [3,art.3.2.1,p.8]. for simplification, however, L will be taken as c-c. of supports for all purposes herein.

Constant C is 360 under LL (live loads) only, and is 240 under LL plus DL (live plus dead loads). Check both conditions.

Constant C' varies with support conditions as shown: SS is simple supports; PC is propped cantilever; and FF is fixed-fixed.

	<u>C' Constants</u>		
	<u>SS</u>	<u>PC</u>	<u>FF</u>
	8	8	12
<u>FLEXURE:</u>			
$M = wL^2 / C' = F_b I / c = F_b b d^2 / 6$			(1)
			($I = b d^3 / 12$)
			($c = d / 2$)
$L^2 = C' M / w = C' F_b b d^2 / (6w)$			(2)
$w = C' M / L^2 = C' F_b b d^2 / (6L^2)$			(3)
<u>HORIZONTAL SHEAR:</u>	2	1.6	2
$V = w (L - 2d) / C' = 2 b d F_v / 3$			(4)
$L = (C' V / w) + 2d = (2C' b d F_v / (3w)) + 2d$			(5)
$w = C' V / (L - 2d) = 2C' b d F_v / (3 (L - 2d))$			(6)
<u>DEFLECTION:</u>	5/384	1/185	1/384
$y = L / C = C' w L^4 / EI = 12C' w L^4 / (E b d^3)$			(7)
$L^3 = E b d^3 / (12C' w)$			(8)
$w = (d / L)^3 E b / (12C' C)$			(9)

BEAM BEARING LENGTH L' (at each end):⁹

2 (10) 2

If $L' < 6$ in. AND bearing area is > 3 in. from end:

$$R = V = wL / C' = bF_{c1} (L' + 0.375) \quad (10)$$

$$L' = (wL / (bC'F_{c1})) - 0.375 \quad (11)$$

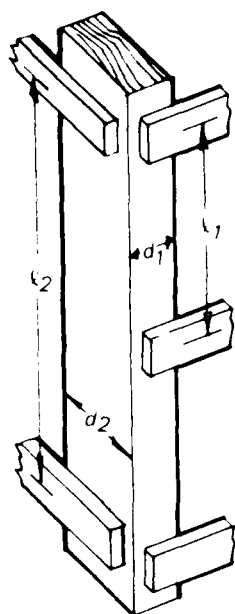
$$w = (bC'F_{c1} / L) (L' + 0.375) \quad (12)$$

If $L' \geq 6$ in. OR bearing area is < 3 in. from end: Drop 0.375 from Eq. 10-12. (For simplification, consider dropping the 0.375 in all cases.)

⁹ It is recommended that L' be not less than 2 in.

¹⁰ $C' = 1.6$ at fixed end; $C' = 2.67$ at simple support or "propped" end.

COLUMN FORMULAS - SIMPLE SOLID COLUMN DESIGN [3]
(Pin-ended conditions assumed)



l_1 and l_2 =
Distances between points of
lateral support of column in
planes 1 and 2, inches

d_1 and d_2
Dimensions of column in
planes of lateral support,
inches

For conditions shown in the sketch:

Use larger of L_1 / d_1 and L_2 / d_2 as the L / d for design (13A)

Slenderness ratio L / d must be ≤ 50 (13B)

For $L / d \leq 11$: $F'_c = F_c$ (14)

For $L / d > 11$, but $\leq K$:

$$K = 0.671 \sqrt{E / F_c} \quad (15)$$

$$F'_c = F_c (1 - 1/3 (L / dK)^4) \quad (16)$$

For $L / d \geq K$: $F'_c = 0.3E / (L/d)^2$ (17)

$$P = AF'_c = bdF'_c \quad (18)$$

Usual Floor Joist Design

Try 2x8 (1.5" x 7.25") floor joist, on 16" centers. Use:
 floor live load, LL = 40 psf; floor dead load, DL = 10 psf
 Thus $DL + LL = (10 + 40) \times \frac{16}{12} \times \frac{1}{12} = \frac{50}{9}$ pli = w.

Find usable c-c spans, for "lower" and "higher" member strengths — see Annex 1st page (above).

Flexure:
 Eq. 2: $L^2 = 8 F_b \times 1.5 \times 7.25^2 \times \frac{6 \times 50}{9} = \frac{132}{182}$ (Strength Wood)

[925]
 [1750]

Horiz. Shear:

Eq. 5: $L = 2 \times 2 \times 1.5 \times 7.25 \times F_v / \frac{3 \times 50}{9} + 2 \times 7.25 = \frac{276}{380}$

[100]
 [140]

Deflection:

Eq. 8: $L^3 = E \times 1.5 \times 7.25^3 / \left(\frac{12 \times 5}{384} \times Cw \right) = \frac{153}{170} \quad C=240$

[1.3 x 10⁶]
 [1.8 x 10⁶]

[240] [50/9] 1+4 (160) C=360
 [360] [40/9]

End Bearing:

Eq. 11: $L' = \frac{50 \times L}{9} / (1.5 \times 2 \times F_{cL})$
 [132] [190] [385]
 129 0.77
 USE 2" USE 2"
 (min.) (min.)

Least L for lower, and least L for higher are
 132" (flexure controlling) and 160" (deflection,
 with C=360, controlling), respectively.
 These 2, L values are used for End Bearing calc's.

Note: (a) For upgrading design, ignore deflection as a design criterion.

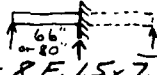
(b) Above two L values agree exactly with Table B1-3.

HOST WELTER USE: L (c-c. supports) = 132 160 in.
 But, if deflection, see (Eq. 8) $w = 8 F_b \times 1.5 \times 7.25^2 / \frac{6 L^2}{9} = \frac{50}{9} \times \frac{16}{12} \times \frac{1}{12} = \frac{50}{9}$ pli TL
 [925] [1750] [132] [160] 1+4/16
 50 65
 40 35
 46 63
 or 5.8 8.0
 38 38
 8 25
 A. Fallout only (assuming soil @ 9.5 pcf) + 15% for loads < 2 mos. in th. soil
 B. Combined: Use $\leq$ soil LL in A; say, 4.5 in. @ 100 pcf used: psf LL (soil)
 46-38 = 8 63-38 = 25 psf LL (soil)
 Blast capacity (remove 15% of A; instead use capacity remaining)
 + 100% for impact loads and + 30% factor of safety given up = $\frac{2.0 \times 1.3 \times 8}{1.15} = \frac{18}{25}$ psf LL (last)
 Re-check end bearing adequacy: $\div 144 = 0.13 \quad 0.39$ psi " "

Upgrading Floor Joist Design

Try same 2x8 (1.5 x 7.25) floor joist, on 16" centers, but with an added mid-span support — same 2 strengths — spans now (132 ÷ 2) 66" and (160 ÷ 2) 80" for "lower" and "higher" strengths, respectively.

Find w . Use $DL = 10$ psf, as before. PC supports.

Flexure: 

Eq. 3: $w = 8 F_b 1.5 \times 7.25 / (6 L^2) =$

Lower		Higher		
w		w		
22.3		28.7		pli TL
201	$\times 1.15, TL =$	259		psf TL
-10	$\frac{16}{16} DL =$	-10		" DL
191	$LL =$	249		" LL

Horiz. shear:

Eq. 6: $w = 2 \times 1.6 \times 1.5 \times 7.25 F_v / (3 \times (L - 2 \times 7.25)) =$

Lower		Higher		
w		w		
22.5		24.8		pli TL
223	$TL =$	223		psf TL
-10	$DL =$	-10		" DL
213	$LL =$	213		" LL

UPGRADING:

A. Fallout shielding only (assume $LL = 191$ 213 psf LL soil @ 100 pcf; add 15% for loads $\times 1.15$)

62 mos. duration (Ref. 3 art. 2.2.5.3) $LL = 220$ 245 psf (soil) LL

Soil th. = 26.4 29.4 in. soil

$[w = (LL + DL) \times 16/144 = 25.6$ 28.3 pli TL]

$\frac{191}{220}$ 213 psf LL

B. Combined: Fallout: Use $\leq$ soil LL in A. $\times 1.15$ 220 245 psf (soil) LL

Assuming 12" th. 95 pcf soil; +15% as before: -95 -95 " " "

125 150 psf (soil) LL

Blast capacity (remove 15% of A; instead use +100% for impact loads and +30% factor of safety given up) $= \frac{2.0 \times 1.3}{1.15} \times \left[\frac{125}{150} \right] =$ 283 339 psf (blast) LL

$\div 1.44$ 1.96 2.36 psi " "

$[w = (LL_{soil} + LL_{blast} + DL) \times 16/144 = 43.1$ 49.3 pli TL]

$\left[\frac{95}{95} \right] \left[\frac{283}{339} \right] \left[\frac{10}{10} \right]$

Bearing Length: (of floor joist on top beam or column or wall)

Eq. 11

At each end of 2-span joist:

$L' = wL / (1.5 \times 2.67 F_c) =$

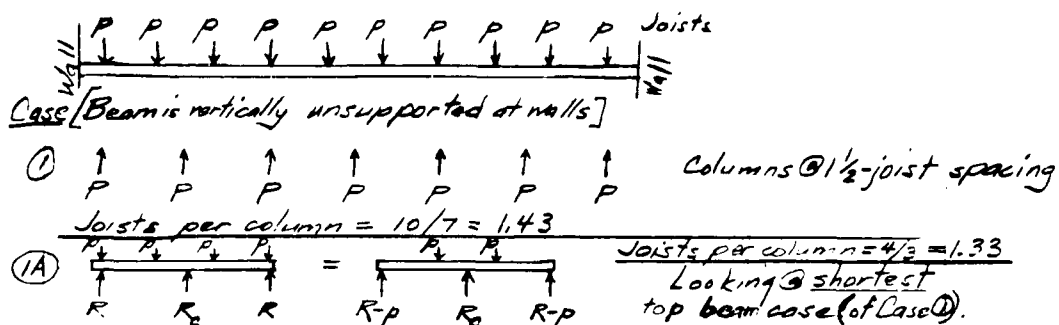
A. Fallout		B. Combined		
25.6	66	43.1	66	
28.3	80	49.3	80	
190		190		
385		385		
2.22"	7.47"	3.74"	2.56"	USE NEXT HIGHER 1/2 in.

At midspan: $2 \times L' = 2(wL) / (1.5 \times 1.6 F_c) =$ 7.41" 4.90"

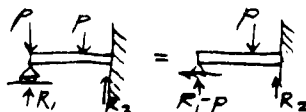
$\left[\begin{smallmatrix} 190 \\ 385 \end{smallmatrix} \right] \left[\begin{smallmatrix} 190 \\ 385 \end{smallmatrix} \right]$ Same as above $\left[\begin{smallmatrix} 190 \\ 385 \end{smallmatrix} \right] \left[\begin{smallmatrix} 190 \\ 385 \end{smallmatrix} \right]$ 12.48" 8.54"

Assume that in each column line, there is a column under each of the 2 end joists, ignoring joists resting ^{on their full length} on the basement end walls.

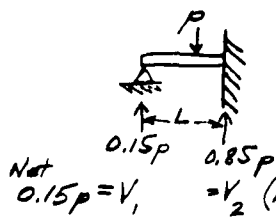
Also, assume that column spacing is either at some whole number (integer) multiple of the joist spacing (usually 12, 16 or 24 in. c-c), or at an integer-divided-by-two multiple of the same. This means that all columns will be under joists; or that half or more will be, with the others located midway between joists. (An occasional irregularity in joist spacing should be treated so that the columns-per-joist ratio is not increased, but is decreased, over the predominating ratio.) If top beam must be discontinuous, break it between 2 joists, and put a column under each of the 2 joists, with top beam ends between joists and columns.



Use either half as a propped cantilever and design member;



Ref. 1;
A2-202
#14



Net $0.15p = V_1 = V_2 \text{ (max. V)}$

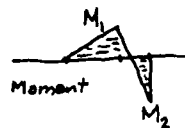
$$(net) V_1 = \frac{P(\frac{2L}{3})^2}{2L^3} (\frac{2L}{3} + 2L) = \frac{4}{27} P = 0.15 P$$

$$R_2 = V_2 = \frac{P(\frac{2L}{3})}{2L^3} (3L^2 - (\frac{2L}{3})^2) = \frac{23}{27} P = 0.85 P$$

Ref: p 2-20-14



$$R_1 - P = V_1 \text{ (in actual case)} \therefore R_1 = 1.15 P$$



$$M_1 = \frac{4}{27} P (\frac{2L}{3}) = \frac{8}{81} PL = 0.099 PL$$

$$M_2 = P (\frac{2L}{3}) (\frac{L}{3}) \times \frac{(\frac{2L}{3} + L)}{2L^2}$$

$$M_2 = PL \times \frac{2}{3} \times \frac{1}{3} \times \frac{1}{2} \times \frac{5}{3} = \frac{5}{27} PL = 0.185 PL$$

FLEXURE:

EQ. NO.

$$M_{max.} = 0.185 PL = F_b \times \frac{bd^2}{6} \quad [L = \text{c-c. of supports}] \quad 22$$

HORIZONTAL SHEAR:

$$V_{max.} = 0.85 P = \frac{2}{3} F_v bd \quad 23$$

V for design is taken at d distance from support center (no effect in this case, however).

BEARING LENGTH (on top and bottom of top beam):

Top of top beam: $L' = \frac{P}{b F_{c\perp}} \quad 24$

Bottom of top beam:

Propped end: $R_1 = 1.15 P = b L' F_{c\perp}$ or $L' = \frac{1.15 P}{b F_{c\perp}} \quad 25$

Fixed end: $R_2 = 0.85 P = b L' F_{c\perp}$

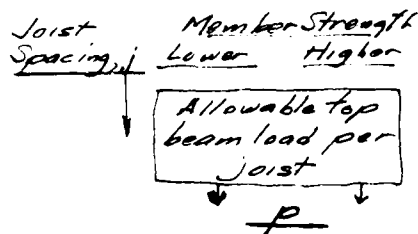
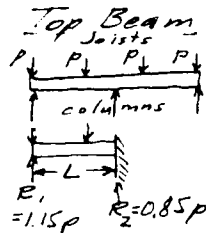
This end is continuous, so total bearing length at midspan is $2 \times L'$ (that is, L' each way from column support R_2)

$$2 \times L' = \frac{2 \times 0.85 P}{b F_{c\perp}} \quad 26$$

Top Beams

(Sheet 3 of 3)

Case 1A



Using
Eq.

Triple 2x8's (1.5x7.25) edgewise, top beam

22 Flexure: $p = \frac{4.5 \times 7.25 \times F_b}{6 \times 0.185 L}$

Joist Spacing	Member Strength Lower	Member Strength Higher	
12"	10951	20717	16.
16"	8213	15538	"
24"	5475	10359	"

L = 1.5x joist spacing

23 Horizontal shear: $p = \frac{2 \times 4.5 \times 7.25 \times F_v}{3 \times 0.85}$

Joist Spacing	Member Strength Lower	Member Strength Higher	
12"	2559	3582	16.
16"	190	385	"
24"	140	100	"

Controlling.

Note: It would be expected that horiz. shear would be controlling over flexure in such short, heavily loaded members.

Bearing length: (On top and bottom of top beam)*

25 Propped end: $L' = \frac{1.15 p}{4.5 F_{c\perp}}$

Joist Spacing	Member Strength Lower	Member Strength Higher	
12"	3.44"	2.38"	
16"	2.99"	2.07"	
24"	5.09"	3.51"	

Top of top beam
L' = $\frac{p}{4.5 F_{c\perp}}$

26 Overall at continuous support: $2L' = \frac{2 \times 0.85 p}{4.5 \times F_{c\perp}}$

Round to next higher 1/2 in.

Quadruple 2x6's (1.5x5.5) edgewise, top beam

(Flexure checked as not controlling)

23 Horiz. shear: $p = \frac{2 \times 6 \times 5.5 \times F_v}{3 \times 0.85}$

Joist Spacing	Member Strength Lower	Member Strength Higher	
All jo's	2588	3624	

Bearing length: (As above)*

25 Propped end: $L' = \frac{1.15 p}{6 F_{c\perp}}$

Joist Spacing	Member Strength Lower	Member Strength Higher	
12"	2.61"	USE 2" min.	
16"	2.27"	USE 2" min.	
24"	3.86"	2.67"	

Top of top beam
L' = $\frac{p}{6 F_{c\perp}}$

26 Overall at continuous end: $2L' = \frac{2 \times 0.85 p}{6 F_{c\perp}}$

Round to next higher 1/2 in.

* Use joist blocks and cap blocks (2 in. or thicker), as necessary, to provide required bearing lengths.

Columns for Upgrading

(Sheet 2 of 3)

Ex: 2x8 column, supported @ midht:

$$13 \quad L_2/d_2 = 96/7.25 = 13.2 \quad L_1/d_1 = 48/1.5 = 32$$

(controlling)

$$15 \quad K = \text{same as } 2 \times 6$$

$$17 \quad F'_c = \text{same as } 2 \times 6 \text{ (midht supp.)} =$$

$$18 \quad P = 1.5 \times 7.25 \times F'_c =$$

Member Strength

	Lower	Higher	
$E =$	1.3	1.8×10^6	psi
$F_c =$	725	1250	"
	$L/d > K$		

	381	527	psi
	4143	5731	lb.

2x8 column, supported @ 1/3 & 2/3 ht:

Same as 2x6 w/ same supports, down to:

$$16 \quad F'_c = \text{same}$$

$$18 \quad P = 1.5 \times 7.25 \times F'_c$$

E, F_c & K 's as above.

	$L/d < K$		
	648	1045	psi
	7047	11364	lb.

4x6 column:

$$13 \quad L_1/d_1 = 96/3.5 = 27.43 \text{ (as above)}$$

$$16 \quad F'_c = F_c \left[1 - \frac{1}{3} \left(\frac{L/d}{K} \right)^4 \right] =$$

$$17 \quad F'_c = 0.3E / 27.43^2$$

$$18 \quad P = 3.5 \times 5.5 \times F'_c$$

E, F_c & K 's as above.

	$L/d < K$	$L/d > K$	
	515	—	psi
	—	718	"
	9914	13822	lb.

4x6 column, supported @ midht:

$$13 \quad L_1/d_1 = 48/3.5 = 13.71 \quad L_2/d_2 = 96/5.5 = 17.45$$

(controlling)

$$16 \quad F'_c = F_c \left[1 - \frac{1}{3} \left(\frac{L/d}{K} \right)^4 \right] =$$

$$18 \quad P = 3.5 \times 5.5 \times F'_c =$$

E, F_c & K 's as above.

	$L/d < K$		
	691	1158	psi
	13302	22292	lb.

Above are "normal" design values for column axial loads;
for UPGRADING in haz areas (blast ≤ 5 psi):

A. Fallout shielding only — soil live loads, for example: *
Increase axial load P values above by 15%,
assuming load duration will be less than or equal to 2 mas.

B. Combined: * Fallout: Use soil load < A; subtract actual
soil load from 1.15 times column's normal, axial
live load capacity, leaving a residual capacity
for blast.

Blast: Using this residual capacity for blast,
remove the 15% put in for soil loads (2 mas. or
less), and substitute a 100% increase for
impact load (of blast) — that is:

$$\left[\frac{\text{Residual capacity for blast}}{\text{Blast}} \right] \times \frac{2}{1.15} = \text{Blast capacity (lb.)}$$

Divide Blast capacity (lb.) by column's contributory
floor area served (Sheet 1), to get blast capacity
(psf) — then divide by 144 to get it in psi.

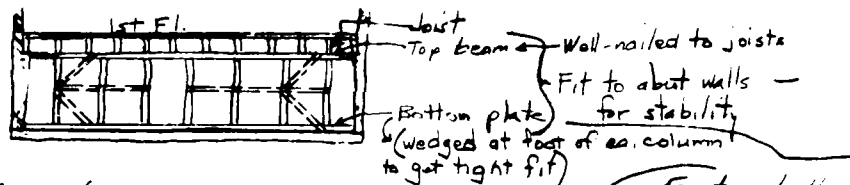
(*) — Saving the factor of safety, (FS) of about 30% (or 1.30)
for buckling failure insurance.

Columns for Upgrading (Sheet 3 of 3)

Note that "2x" columns cannot be longer (See Eq. 13) than 75" [$L/d = 75 / 1.5 \leq 50$], unless they have lateral support in the weak ($1\frac{1}{2}$ ") direction.

Note that lateral supports can be provided by 1"x4" or 1"x6" boards, well-nailed, flat on both sides of columns, OR by blocking between columns* — either one carried to wedged contact with end basement walls, OR by 1"x4" or 1"x6" boards, well-nailed, as diagonals from top to bottom plates, counter-directional with one on each side of the column. Latter (diagonals) technique can be used to allow unobstructed opening between columns, wherever desired.

NOTE: Get a copy of latest (shirt-pocket) grading rules, and re-grade all wood members to be used in stress-graded situations.



Example:

Joists per column = $10/7 = 1.43$

Always put column under last joist at each end (excluding one, or double, resting on each foundation wall)

Run diagonals on one side of column, at angle approx. 90° to those on other side of column line. Lateral supports for columns may be blocking, or face boards on each side of column line made stable by diagonals, or by continuous blocking through entire column line and wedge-fitted to both end walls

(*) Use of steel strapping is another alternative.

NOTATION ¹¹

- A = area of cross-section (in.²)
- b = breadth (edge width) of rectangular member (in.)
- b' = width of (floor) area supported (in.)
- C = deflection coefficient (= L / maximum deflection)
- C' = beam formula constant
- c = distance from neutral axis to extreme fiber (in.)
- DL = dead load (psf)
- d = depth of rectangular member (in.)
- E = modulus of elasticity (psi)
- F_b = design value for extreme fiber in bending (flexure) (psi)
- F_c = design value for compression parallel to grain (psi)
- F_c' = F_c, adjusted for L / d ratio (psi)
- F_{c⊥} = design value for compression perpendicular to grain (psi)
- F_g = design value for end grain in bearing parallel to grain (psi)
- F_v = design value in horizontal shear (psi)
- I = moment of inertia (in.⁴)
- j = joist spacing (in.)
- K = for columns, largest slenderness ratio, L / d, where intermediate column formula applies (see Eq. 15 and 16)
- L = span length of horizontal member (usually center-to-center of supports); or distance (c-c.) between column lateral support points (in.)

¹¹ Generally follows Reference [3].

NOTATION (concluded)

L' = required bearing length in compression perpendicular to grain (in.)

LL = live load (psf)

M = bending or resisting moment, maximum (in.-lb)

P = total concentrated load or axial load (lb)

p = load (live and dead) per joist on top beam, post/stud, etc. (lb)

pcf = pounds per cubic foot

pli = pounds per linear inch

psf = pounds per square foot

psi = pounds per square inch

q = design strength (beam) for static loads (psi)

S = section modulus (= I / c) (in.³)

TL = total load (= DL + LL) (psf)

t = thickness (in.)

V = vertical design shear (= horizontal) in beams (lb)

W = total uniformly distributed load (lb)

w = uniformly distributed load per unit length (pli)

y = deflection of member, usually at mid-length (in.)

REFERENCES

1. Manual of Steel Construction, American Institute of Steel Construction, Inc., Wrigley Building (8th floor), 400 North Michigan Avenue, Chicago, Illinois 60611, 7th ed., 1970. (Eighth edition, 1980, was received October 17, 1980.)
2. "Design Values for Wood Construction," 1980 Supplement to National Design Specification for Wood Construction, 1977 edition [Reference 3]. Shirt-pocket size grading rules may be obtained from the following grading associations:
 - NELMA Northeastern Lumber Manufacturers Association, Inc.
4 Fundy Road
Falmouth, Maine 04105
 - NHPMA Northern Hardwood and Pine Manufacturers Association, Inc.
Northern Building
Green Bay, Wisconsin 54301
 - NLCA National Lumber Grades Authority (Canada)
P. O. Box 97
Ganges, B.C., Canada VDS 1E0
 - RIS Redwood Inspection Service
1 Lombard Street
San Francisco, California 94111
 - SPIB Southern Pine Inspection Bureau
4709 Scenic Highway
Pensacola, Florida 32504
 - WCLIB West Coast Lumber Inspection Bureau
6980 S.W. Varnes Road
P.O. Box 23145
Portland, Oregon 97223
 - WWPA Western Wood Products Association
1500 Yeon Building
Portland, Oregon 97204
3. National Design Specification for Wood Construction, 1977 edition (Structural Lumber, Glued Laminated Timber, Timber Pilings, Fastenings), National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D.C. 20036.

4. Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects: Predesigned Expedient Options, Stanford Research Institute¹² Technical Report for Defense Civil Preparedness Agency,¹³ October 1977.
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¹² Now SRI International

¹³ Now Federal Emergency Management Agency

Appendix D1

BLAST-RESISTANT DESIGN/ANALYSIS OF STEEL MEMBERS

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UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDE--ETC(U)

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Appendix D1

BLAST-RESISTANT DESIGN/ANALYSIS OF STEEL MEMBERS¹

Introduction

It is expected that the reader/user of this appendix will have first read Appendix A containing information applicable to blast-resistant design/analysis generally. This appendix provides information specific to such design/analysis using steel plates and rolled shapes.

As before (Appendix A), useful material developed by Newmark [1]² is included in quotes below (parenthetical insertions or connective words are by this writer).

Design in Steel

"(Blast-resistant) designs in structural steel . . . are based on plastic design principles. For members responding primarily in flexure, if the sections are sufficiently ductile to permit redistribution of moment after the first inelastic action begins, the yield moment is taken as the fully plastic moment of the cross-section.

"Structural carbon steel, i.e., ASTM A-7, A-36, or A-373, possesses a high degree of ductility and strain hardens markedly. Therefore, the zones of inelastic behavior in structural elements formed of these alloys will be widespread. A ductility factor (μ) in flexure of 10 has been used for design in these alloys.

"Higher strength steels . . . have less ductility and strain harden less than the above mild steels. The zones of inelastic behavior will be more limited in extent with limited plastic hinge rotational capacity. Therefore, a ductility factor of no more than 3 is recommended for designs of flexural members in these materials.

"Compression members should be designed for a ductility factor of 1.3. Tension members may be designed with the ductility factor appropriate for the material in flexure.

¹ Flat Plate/Sheet, Corrugated, and Builtup Sections.

² Brackets are used herein to indicate sources in the References list at the end of this appendix.

Stability Limitations

"In order that the yielded cross-section of a member continue to transmit the fully plastic moment through large rotations of the plastic hinge, limitations are placed on the cross-sectional dimensions to insure stability against buckling. Two types of instability are important for flexural members: (1) local buckling of elements of the section, and (2) lateral buckling of the compression flange. Lateral buckling is not commonly a problem for (floor or) door elements since the compressive flanges will ordinarily have continuous lateral support. However, in instances where compression flanges are (laterally) supported, the stability of the member should be evaluated following the procedures given in Reference [2]. . . . It is noted that tests have shown a lessened tendency towards buckling for rapid load application; therefore, the usual specified minimum values for yield stress may be used as the required stress level in equations for stability. . . .

"The following limitations on the dimensions of ASTM A-7, A-36, or A-373 steel sections are taken from Reference [2]. The relations are based on an axial yield stress level of 33 ksi, but tests have shown that sections meeting these requirements perform satisfactorily under rapid loading in spite of the increased yield point.

"Stability Requirement for A-7, A-36 and A-373 Steel Sections

Compression flange of WF, I or H section $b/t_f \leq 17$

Web in shear $d/w \leq 43$

where:

b = flange width (in.)

d = depth of section (in.)

t_f = flange thickness (in.)

w = web thickness (in.)

"Interaction expression for more complex stress states are given in Reference [2] which also contains more general expressions which must be used to set stability limitations on the dimensions of sections fabricated of higher strength steel.

"The structural resistance is defined by the yield level and the ductility. The product of these quantities is a measure of the energy absorption capacity of the structure. Continuity of structural elements provides an increased energy absorption capacity and should be provided where practicable in metal blast-resistant structures. To attain continuity, joints should be designed to develop the full flexural or axial capacity of the member; otherwise, deformations will be concentrated at the joints and the overall ductility of the element will be reduced.

"Welded construction can readily provide structural continuity. However, approved welding procedures, good weld and fabrication details, properly selected welding rods, and weldable base metal are essential if brittle response is to be avoided. Riveted or bolted joint details should be free of sheared edges and punched holes, and adequate edge distances should be provided.

Structural Steel Material Properties

"The (dynamic) design stresses (Table D1-1) are based on the yield strengths for the loading rate range expected in protective construction (References [1,3]).

Resistance Expressions

"The equations involved in the evaluation of the resistance of structural (steel) elements . . . are presented in . . . Table D1-2. These expressions define the flexural resistance, shearing resistance, (effective) natural period (of vibration) in flexure, and (centerline) yield deflection. The design stresses to be used with these expressions are in Table D1-1."

Table D1-1A

DESIGN STRESSES FOR STEEL

Steel	Axial Stress f_{dy} , ksi	Shearing Stress v_{dy} , ksi	Allowable Bearing Stress	
			Single Shear f_{by} , ksi	Double Shear f_{by} , ksi
Structural Carbon, ¹ ASTM A-7, A-36, or A-373	42	25		
Corrugated Iron ²	34	20		
Welds	42	29		
Rivets ASTM A-141	40	30	60	80
ASTM A-195	60	40	80	80
Bolts ASTM A-307	32	19	40	40
ASTM A-325	50	30	60	60

¹For higher strength structural steels, use an axial design stress, f_{dy} , equal to the smaller of 1.10 times the specified minimum yield or 0.90 times the specified minimum ultimate strength. For design shearing stress, v_{dy} , use 0.60 f_{dy} . (See facing page.)

²The value of f_{dy} has been selected to be used with a plastic modulus, Z , of 1.5 times the section modulus, S .

Source: Ref. 1, p. 153

Table D1-1B

Steel	Yield Tens.* ksi	Ult. Tens.* ksi	Axial Stress f _{dy} , ksi	Shearing Stress v _{dy} , ksi
<u>Carbon:</u>				
A529	42	60-85	50	25
<u>High-Strength Low-Alloy:</u>				
A242 (to 3/4" th. incl.)	50	70	55	33
A440 "	"	"	"	"
A441 "	"	"	"	"
A572 (to & incl., th.):				
Grade 42 (4")	42	60	46	28
" 45 (1½")	45	60	50	30
" 50 (1½")	50	65	55	33
" 55 (1½")	55	70	61	36
" 60 (1")	60	75	66	40
" 65 (¾")	65	80	72	43
A588 (to 4" th. incl.)	50	70	55	33

*Source: Manual of Steel Construction, 7th edition, 1970, American Institute of Steel Construction, Chicago; p. 5-212 thru -216.

NOTATION FOR TABLE D1-2

α = L_s/L_l (design as 1-way in short direction for $\alpha < \frac{1}{2}$)

A = total area of element, in²

A/b = A per inch width, in.

b = width of element of section, in.

d = depth of structural element, in.

E = elastic Youngs modulus, psi

f_{dy} = dynamic tensile yield stress, psi

I = moment of inertia of element, in⁴

I/b = I per inch width, in³

$k_1 = (77 + 180 \alpha^3) \frac{Et^3}{12L_s^4}$, psi/in

$k_2 = (307 + 500 \alpha^3) \frac{Et^3}{12L_s^4}$, psi/in

$K_\alpha^2 = 3 - 2\alpha\sqrt{\alpha^2 + 3} + 2\alpha^2$

L = 1-way plate span, in.

L_l = 2-way long span, in.

L_s = 2-way short span, in.

S = section modulus of element, in³

S/b = S per inch width, in²

t = plate thickness, in.

t_w = total web thickness of element, in.

v_{dy} = dynamic shearing yield stress, psi

W = plate weight, psf

Z = plastic modulus of section, in³

$Z \approx 1.5 S$ for corrugated plate
1.15 S for I or WF section

Z/b = Z per inch width, in²

Source: Ref. 1, p. 158

Table D1-2

RESISTANCE EXPRESSIONS FOR STEEL STRUCTURAL ELEMENTS

	Flexure q_y , psi	Shear q_y , psi	Period in Flexure T , sec.	Yield Defl. at d_y , x_y in
FLAT PLATE SECTION				
<u>1-way</u> Simple Support	$2f_{dy} \left(\frac{t}{L}\right)^2$	$2v_{dy} \left(\frac{t}{L}\right)$	$9.4 \times 10^{-3} L^2 \sqrt{\frac{W}{Et^3}}$	$\frac{5}{32} \frac{q_y L^4}{Et^3}$
Fixed Support	$4f_{dy} \left(\frac{t}{L}\right)^2$	$2v_{dy} \left(\frac{t}{L}\right)$	$4.1 \times 10^{-3} L^2 \sqrt{\frac{W}{Et^3}}$	$\frac{q_y L^4}{25.6 Et^3}$
<u>2-way</u> Simple Support	$6f_{dy} \left(\frac{t}{K_\alpha L_s}\right)^2$	$2v_{dy} \frac{t}{L_s} \left[\frac{2}{3}(1+\alpha)\right]$	$2.2 \times 10^{-2} \sqrt{\frac{W}{k_1}}$	$\frac{q_y}{k_1}$
Fixed Support	$12f_{dy} \left(\frac{t}{K_\alpha L_s}\right)^2$	$2v_{dy} \frac{t}{L_s} \left[\frac{2}{3}(1+\alpha)\right]$	$1.9 \times 10^{-2} \sqrt{\frac{W}{k_2}}$	$\frac{q_y}{k_2}$
CORRUGATED SECTION				
<u>1-way</u> Simple Support	$\frac{8f_{dy}}{L^2} \frac{Z}{b}$	$\frac{2v_{dy}}{L} \frac{A}{b}$	$2.7 \times 10^{-3} L^2 \sqrt{\frac{W}{E} \frac{b}{I}}$	$\frac{5}{384} \frac{q_y L^4}{E} \frac{b}{I}$
Fixed Support	$\frac{16f_{dy}}{L^2} \frac{Z}{b}$	$\frac{2v_{dy}}{L} \frac{A}{b}$	$1.2 \times 10^{-3} L^2 \sqrt{\frac{W}{E} \frac{b}{I}}$	$\frac{q_y L^4}{207 E} \frac{b}{I}$
BUILT-UP SECTION				
<u>1-way</u> Simple Support	$\frac{8f_{dy}}{L^2} \frac{Z}{b}$	$2v_{dy} \frac{dt_w}{bL}$	$2.7 \times 10^{-3} L^2 \sqrt{\frac{W}{E} \frac{b}{I}}$	$\frac{5}{384} \frac{q_y L^4}{E} \frac{b}{I}$
Fixed Support	$\frac{16f_{dy}}{L^2} \frac{Z}{b}$	$2v_{dy} \frac{dt_w}{bL}$	$1.2 \times 10^{-3} L^2 \sqrt{\frac{W}{E} \frac{b}{I}}$	$\frac{q_y L^4}{307 E} \frac{b}{I}$

Source: Ref. 1, p. 159

Applications

This section is limited to use of steel sheets and plates as barriers, especially as closures to prevent exterior air blast from entering the basement shelter. However, data useful for other steel applications, such as use of corrugated steel sheets, builtup sections, welds, rivets and bolts, are included elsewhere herein.

It is assumed that the reader/user has reviewed Appendix A herein.

Local availability of steel sheets, plates, and shapes is discussed below in Appendix E1.

A. Allowable Stresses

Allowable stresses for blast-resistant design are shown in Table D1-1. For design of sheet/plate closures, one needs a value for f_{dy} for the particular available steel planned for use, based on its specific ASTM specification number. The f_{dy} value should be in psi (ksi value times 1000 equals the psi value), and is used for both the axial and flexural allowable stresses. (Allowable shear stress v_{dy} is not needed for the applications contemplated herein.)

For an illustrative example: assume A36 steel for use; f_{dy} is 42 ksi (Table D1-1A) or 42,000 psi.

B. One-way Flat Plate Closures, Simple Supports

This design situation applies when the closure sheet or plate is simply supported (i.e., not clamped) on two opposite edges only. The applicable basic formula is the first one of Table D1-2; q_y is found, using known values of plate thickness t (in.) and the clear span L (in.) between supports. Also used is the formula relating p_{dm} (peak value of the blast pressure that can be resisted by the steel plate closure, psi) to q_y and the ductility factor μ .³

Graphic solutions for these combined (two) formulas, for A7/A36/A373 steels in thicknesses of 16 gage (0.06 in.) and 1/8" to 3/4", are shown in Figure D1-1, which is accompanied by a table of correction factors that is discussed further below.

Continuing our example: for 3/4" th. plate and clear span L of 50 in., read $p_{dm} = 18$ psi in Figure D1-1. (Curves of Figure D1-1 are ended arbitrarily at peak p_{dm} value of 50 psi and peak plate weight of about 500 pounds (for square plate of length L on each side).)

³ See third paragraph of earlier section on "Design in Steel" where $\mu = 10$ is recommended for carbon steels and $\mu = 3$ for higher strength steels.

CORRECTION FACTORS FOR FIGURE D1-1

For Carbon Steels

<u>ASTM-</u>	<u>Factor</u>
A7	1.000
A36	1.000
A373	1.000
A529	1.190

For High-Strength Steels

<u>ASTM-</u>	<u>Factor</u>
A242	1.149
A440	1.149
A441	1.149
A572	
Grade 42	0.961
" 45	1.044
" 50	1.149
" 55	1.274
" 60	1.378
" 65	1.504
A588	1.149

For Two-way Plates

Ratio of longer to shorter clear spans, L_1 / L_s	<u>Factor</u>
1.0	3.000
1.1	2.736
1.2	2.531
1.3	2.366
1.4	2.232
1.5	2.121
1.6	2.028
1.7	1.948
1.8	1.879
1.9	1.820
2.0	1.768
> 2.0	1.000

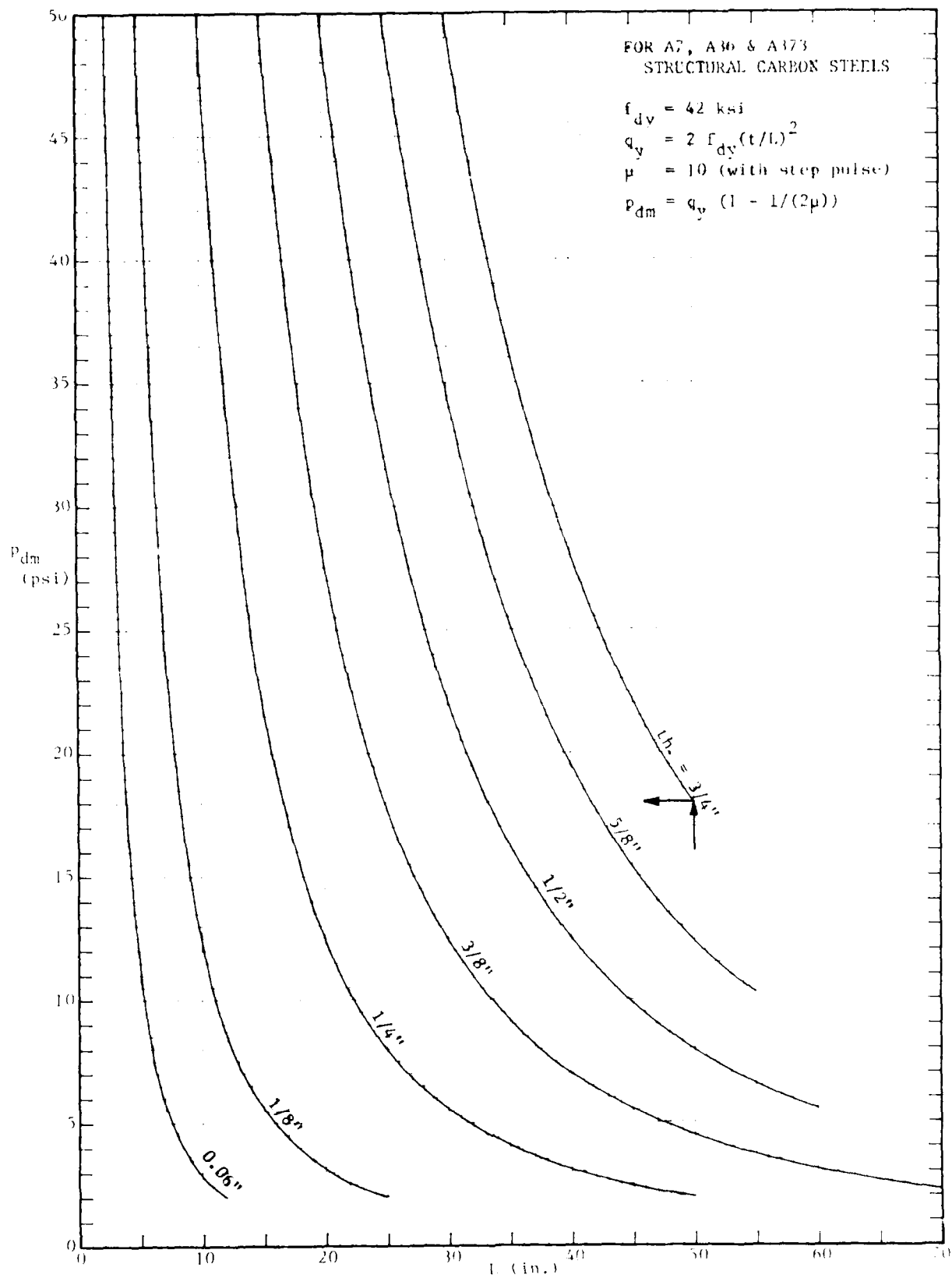


Figure D1-1 STEEL PLATE/SHEET DESIGNS (One-way, Simple Supports)

C. Correction Factors for Other Steels

Correction factors for use with Figure D1-1 are shown in a table accompanying the figure; they allow use of all other steels listed in Table D1-1, each with the appropriate ductility factor μ as discussed above. The figure is used as described just above, then the p_{dm} value obtained is multiplied by the correction factor for the steel used.

Continuing our example: for the same plate and span, but this time with A529 steel: p_{dm} of 18 psi is multiplied by the correction factor of 1.190 for A529 steel, giving a corrected $p_{dm} = 18 \times 1.190 = 21.4$ psi.

D. Correction Factors for Two-way Plates

Correction factors for use with Figure D1-1 are shown in the same accompanying table, and cover use of a two-way plate simply supported on all four edges, instead of the one-way simply supported plate that is the basis for the Figure. These correction factors vary with the ratio of longer clear span to shorter clear span, or L_L/L_S .

Continuing our example: using a two-way simply supported plate instead of one-way, shorter and longer clear spans are 50 and 60 in., respectively; use the shorter span with Figure D1-1 and read the same 18 psi for p_{dm} ; A529 steel dictates using a correction factor of 1.190 (as above); correcting for two-way versus one-way plates inserts another correction factor, that for $L_L/L_S = 60/50 = 1.2$, or a factor of 2.531 from the table accompanying the Figure. Putting this together:
 $p_{dm} = 18 \times 1.190 \times 2.531 = 54.2$ psi.

NOTATION

(Excluding notation defined there and used in Tables D1-1 and D1-2)

b = flange width (in.)

d = depth of section (in.)

ksi = kips (kilo-pounds) per square inch (1 kip = 1,000 psi)

psi = pounds per square inch

p_{dm} = peak air blast pressure applied to member, psi

t_f = flange thickness (in.)

w = web thickness (in.)

μ = x_m/x_e = the ductility factor (see Appendix A)

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1. Newmark, N. M., Design of Openings for Buried Shelters, Report 2-67, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1963; Chapter 7.
2. Commentary on Plastic Design in Steel, Manual of Engineering Practice No. 41, American Society of Civil Engineers (ASCE), New York, New York, 1961.
3. Anderson, F. E., Jr., R. J. Hansen, H. L. Murphy, N. M. Newmark, and M. P. White, Design of Structures to Resist Nuclear Weapons Effects, ASCE Manual of Engineering Practice No. 42, 1961, and Supplement 1963 (combined into a "1964 Edition").

Appendix E1

STRUCTURAL STEEL LOCAL AVAILABILITY AND USE
FOR BLAST SHELTER UPGRADING

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Appendix E1

STRUCTURAL STEEL LOCAL AVAILABILITY AND USE FOR BLAST SHELTER UPGRADING

Introduction and Purpose

Among the various materials useful for construction of closure systems for blast shelters and for strengthening the floor over such shelters, are the standard structural steel products including plates, structural shapes including "I" beams and channels, structural tubing, and pipe.

These materials are those normally used in the structural framing for buildings, bridges, industrial plants, and for other heavy construction purposes. Structural steel materials are stocked rather widely on a regional and local basis.

Structural steel materials may be used for "expedient" or "engineered" upgrading of potential blast shelters. Usually such uses will involve provision of closure or strengthening of floor systems, or most likely both. Structural steel materials will be particularly useful, and in many cases required, for provision of the higher levels of blast protection, or where closure or floor spans are long.

Design guidance for the use of structural steel products in both expedient and engineered blast shelter upgrading is given elsewhere in this report (main text and Appendix D1). The purpose of this appendix is to describe the general characteristics and uses of these materials and to identify sources of supply, together with typical regional and local availability.

The reader desiring more detailed information on structural steel should refer to standard references [e.g., 1],¹ or consult with a Professional Civil or Structural Engineer. Actual designs should be prepared by professional engineers if time is available.

Shapes, Sizes, and Designations

A. Introduction

Structural steel materials considered include plate, rolled shapes, pipe and structural tubing, sheet piling, and railroad rail. Standard shapes, sizes, and designations are discussed for each in the following paragraphs.

¹ Brackets are used herein to indicate sources in the References list at the end of this appendix.

B. Structural Steel Plate

Structural steel plate is manufactured in a variety of thicknesses, widths, and lengths. Materials are designated as strip, bar, or plate depending on the relationship of thickness to width. For the purposes of the present applications, the useful materials are limited to the bar and plate designations of 1/4" thickness or more, which are grouped in thickness and width classifications as follows: Nominal thicknesses (in.) are 1/4, 5/16, 3/8, 7/16, 1/2, 5/8, 3/4, and 1, within which widths of 3-1/2" to 8" are bars and over 8" are plates.

C. Rolled Structural Shapes

Hot rolled structural steel shapes include products formerly referred to as "I" Beams, "Wide Flange" or "WF" Beams, "H" Beams, Channels, and Tees. The present standard designation system (together with the former system) and size ranges are given in Table E1-1.

The designation system shown in Table E1-1 is a method of identification for shapes and sizes of rolled structural steel shapes, employing a standard nomenclature. Its primary use is as a form of abbreviation for identification on drawings, but it is also recognized throughout the trade including sources of supply. The present designation includes three items: (1) type, shape, or group symbol; (2) nominal shape, depth

and (3) the weight in pounds per linear foot (except for angles, where the thickness of the metal is given instead).

The designation system was changed in 1970 [1]. Both the current and former designation systems are given in Table E1-1 since older tables and drawings are still in use.

² Actual dimensions differ significantly from nominal dimensions for some shapes. See the last section of this appendix for further explanation.

Table E1-1

ROLLED STEEL SHAPES - DESIGNATIONS AND SIZES

Generic Type or Group	Example Designation		Size Ranges	
	Present	Former	From	To
<u>Beams and Columns</u>				
Wide Flange Beams and Columns	W8x40	8WF40	W8x20	W36x300
Standard "I" Beams	S8x23	8I23	S3x7.5	S24x120
Light Wide Flange Columns	W6x20	6WF20	W4x13	W6x25
Light Beams	W10x15	10B15	W6x16	W16x31
Miscellaneous Columns	M6x25	6M25	M4x13	M8x34.3
Junior Beams	M10x9	10Jr9	M6x4.4	M12x11.8
Miscellaneous Shapes	M8x20	8M20	M8x22.5	M10x29.1
<u>"H" Shaped Bearing Piles</u>	HP12x74	12BP74	HP12x63	HP14x117
<u>Structural Channels</u>				
Standard Channels	C10x30	10C30	C3x6	C15x50
Miscellaneous Channels	MC12x45	12x4C45	MC8x18.7	MC18x58
Junior Channels	MC10x8.4	10JrC8.4	MC10x6.5	MC12x10.6
<u>Structural Angles</u>				
Equal Leg Angles	L5x5x3/4	45x5x3/4	L3x3x3/16	L8x8x1-1/8
Unequal Leg Angles	L6x4x3/4	46x4x3/4	L3x2x3/16	L9x4x1
<u>Structural Tees</u>				
Cut from Wide Flange Shapes	WT8x25	ST8WF25	WT4x8.5	WT18x150
Cut from Standard "I" Beams	ST6x25	ST6I25	ST3x6.3	ST12x60
Cut from Miscellaneous Shapes	MT5x10.5	ST5M10.5	MT4x8.5	MT5x14.6
Cut from Light Beams	WT7x13	ST7B13	WT3x4.3	WT8x15.5
Cut from Junior Beams	MT6x5.9	ST6Jr5.9	MT3x2.2	MT6x5.9

Examples of the full description of products under the present and former systems follow:

W8x40 - A Wide Flange Beam or column (W) of 8" depth and weighing 40 pounds per foot.

S8x23 - A standard "I" Beam (S) of 8" depth and weighing 23 pounds per foot.

C10x30 - A standard Channel (C) of 10" depth and weighing 30 pounds per foot.

L5x5x3/4 - An equal leg Angle (L) of 5"x5" dimensions (outer faces) and 3/4" thickness.

ST6x25 - A structural Tee cut from a standard "I" Beam (S) of 6" depth and weighing 25 pounds per foot.

Certain special structural shapes have not been included in Table E1-1 because of their limited availability. These include special car building and ship building Channels, bulb Angles, Zees, and small rolled Tee Beams. Sizes and shapes of the car building and ship building Channels are given in Reference [1]. Details on other special shapes can be found in manufacturers' catalogs.

D. Pipe and Structural Tubing

Steel pipe (circular section) is manufactured in standard sizes ranging from 1/8" to 12" nominal diameter in standard, extra strong, and double extra strong weights. The nominal diameter is an approximation of the inside diameter of the pipe in the larger sizes of standard weight. The extra strong and double extra strong weights have reduced inside diameters corresponding to the increased wall thicknesses.

Dimensions of selected sizes and weights of steel pipes are given in Table E1-2.

Structural tubing (square or rectangular section) is manufactured in sizes up to 12" in major dimension. Selected sizes and weights are given in Table E1-3.

Table E1-2
SELECTED PIPE SIZES - DIMENSIONS AND WEIGHTS

Nominal Diameter (in.)	Outside Diameter (in.)	Inside Diameter (in.)			Wall Thickness (in.)			Weight Per Foot (lb)		
		S*	ES	DES	S	ES	DES	S	ES	DES
3	3.500	3.068	2.900	2.300	0.216	0.300	0.600	7.58	10.25	18.58
4	4.500	4.026	3.826	3.152	0.237	0.337	0.674	10.79	14.98	27.54
5	5.563	5.047	4.813	4.063	0.258	0.375	0.750	14.62	20.78	38.55
6	6.625	6.065	5.761	4.897	0.280	0.432	0.864	18.97	28.57	53.16
8	8.625	7.981	7.625	6.875	0.322	0.500	0.875	28.55	43.39	72.42
10	10.750	10.020	9.750	8.750	0.365	0.500	1.000	40.48	54.74	104.13
12	12.750	12.000	11.750	10.750	0.375	0.500	1.000	49.56	65.42	125.49

* Designation: S = Standard Weight
ES = Extra Strong Weight
DES = Double Extra Strong Weight

Table E1-3
STRUCTURAL TUBING SIZES - DIMENSIONS AND WEIGHTS

Outside Dimensions (in.)	Square Section					Outside Dimensions (in.)	Rectangular Section				
	Weight Per Ft (lb) For Various Wall Thicknesses (in.)						Weight Per Ft (lb) For Various Wall Thicknesses (in.)				
	1/2	3/8	5/16	1/4	3/16		1/2	3/8	5/16	1/4	3/16
12x12	74.54	57.23	48.24	39.03	N/A*	12x6	54.15	41.93	35.49	28.83	N/A
10x10	60.95	47.03	39.74	32.23	N/A	10x6	47.35	36.83	31.24	25.44	N/A
8x8	47.35	36.83	31.24	25.44	N/A	8x6	40.55	31.73	26.99	22.04	16.85
7x7	40.55	31.73	26.99	22.04	16.85	8x4	34.48	27.04	23.02	18.82	14.41
6x6	34.48	27.04	23.02	18.82	14.41	7x5	34.48	27.04	23.02	18.82	14.41
5x5	27.68	21.94	18.77	15.42	11.86	6x3	N/A	19.39	16.65	13.72	10.58
4x4	N/A	16.84	14.52	12.02	9.31	5x3	N/A	16.84	14.52	12.02	9.31

* N/A = Not available

E. Other Structural Steel Materials

The materials listed in the previous paragraphs are those generally used in steel building framing, bridges, and industrial applications. In addition to these materials, other structural steel materials may be found to be useful. These include railroad rails and sheet steel piling.

Railroad rails in either a new or used condition offer considerable strength. Most rail is rolled to an American Society of Civil Engineers (ASCE) or an American Railway Engineering Association (AREA) pattern and is classified on both a pattern and weight per yard basis. Table E1-4 lists the properties of selected rail sections.

Sheet steel piling may be available near the coasts and in inland areas near lakes and rivers. It is rolled in several patterns with interlocking edges. Patterns include straight web, arch web, and Zee. Properties of typical sections are given in Table E1-5.

Table E1-4

TYPICAL RR RAIL SECTIONS - DIMENSIONS AND WEIGHTS*

Weight Per Yard (lb)	Height (in.)	Base Width (in.)	Head Width (in.)#
155	8	6-3/4	3
140	7-5/16	6	3
132	7-1/8	6	3
115	6-5/8	5-1/2	2-23/32
100	6	5-3/8	2-11/16
75	4-13/16	4-13/64	2-15/32

* New Condition

Maximum at bottom of head

Table E1-5

TYPICAL SHEET PILE SECTIONS - DIMENSIONS AND WEIGHTS

Section	Nominal Width (in.)	Depth (in.)	Thickness Web (in.)	Flange (in.)	Weight Per Ft (lb)
Straight Web	15	~	1/2	-	40
	15	~	3/8	-	35
Arch Web	19-5/8	3-1/4	3/8	3/8	36
	16	1-11/32	1/2	1/2	37.3
	16	1-11/32	3/8	3/8	30.7
Deep Arch Web	16	6	31/64	3/8	42.7
	16	5	3/8	3/8	36
Zee	21	11-1/2	3/8	1/2	56
	18	12	3/8	1/2	57
	18	12	3/8	3/8	40.5

Material Strength Grades

Structural steel is produced in various grades related to strength and weldability as prescribed in American Society for Testing and Materials (ASTM) specifications. Applicable specifications include:

ASTM A7	Steel for Bridges and Building
ASTM 36	Structural Steel
ASTM A242	High Strength Low Alloy Structural Steel
ASTM A373	Structural Steel for Welding
ASTM A440	High Strength Structural Steel
ASTM A441	High Strength, Low Alloy Structural Manganese Vanadium Steel

The principal variation among the types relates to strength as indicated by yield point. All are weldable except A440, which is not recommended for welded use. The yield point specification values (psi) are:

A7	33,000
A36	36,000
A242	42,000
A373	33,000
A440	42,000
A441	42,000
Rail Steel	39,000

The various standard products are generally produced in grades as follows:

Bar and Plate: A7, A36, A242, A373, A440, A441

Pipe: A7, A36

Railroad Rail: AREA Specification

Rolled Structural Shapes: A7, A36, A242, A373, A440, A441

Sheet Pile: A7, A36

Structural Tubing: A7, A36

Principles of Use and Application

A. General Principles

Structural steel materials are capable of providing relatively high degrees of blast-resistant upgrading as compared to wood; however, they present more difficult problems in fabrication and installation. The principle problems are cutting to size, plus fastening of individual members to each other and to the the basic shelter space structure. The methods of fabrication and installation will vary somewhat between expedient and engineered upgrading applications because of available time and skills.

a. Expedient Applications

The expedient upgrading process will be limited by available time and the skills of the work force. In this process the following general principles should be applied:

(1) The structure (closure, beam support, etc.) should be designed around the material readily available locally.

(2) All cutting to size should be accomplished at the point of supply, where flame cutting, sawing, or shearing may be readily accomplished. On the job cutting would present extreme difficulty, although rough oxyacetylene cutting may be possible if tools and skills are available.

(3) Fastenings should be restricted to simple bolting wherever possible. Bolt holes may be drilled locally although it also would be more efficient to have this done at the point of supply. Fastenings should be designed so as not to be stressed by the blast wave, i.e., the blast wave should add no tension or shear load to the fastenings.

(4) The size and weight of individual structural members should be limited to that which can be handled and erected by a few men without the use of heavy materials-handling equipment.

(5) Advantage should be taken of crisis build-up time to procure pre-cut and pre-drilled material, and to store it for immediate use in the proposed shelter space.

(6) Material strength grades should be limited to A7 or A36 grades wherever possible because of the greater ease in cutting, drilling, or welding.

b. Engineered Applications

The engineered application process permits the utilization of a normal time-frame for design, fabrication, and installation (where desired). The design should be accomplished by a Professional Civil or Structural Engineer and the conventional procurement, fabrication, and installation process should be followed. In this case:

(1) Advantage may be taken of higher strength materials.

(2) More sophisticated fastening systems may be employed to obtain material economy through the use of moment-resisting joints and other economic design features.

(3) Structural welding of subassemblies may be employed, and final erection may also utilize welding, if the structure is to be installed immediately. If the parts of subassemblies are to be stockpiled, however, certain principles listed above for expedient applications - i.e., those related to size, weight, and bolting - should be followed. Such stockpiling is applicable where immediate installation might interfere with the normal use of the proposed shelter space.

B. Potential Applications

Potential applications of structural steel materials in blast shelter upgrading are listed in Table E1-6. For specific design data, member sizing, and blast resistance, see Appendix D1.

Local Availability

A. Manufacturing and Distribution System

Structural steel products are manufactured on a regional basis with major production centers (mills) located in the Ohio Valley, Great Lakes region, Eastern Seaboard (Baltimore), Southeast (Alabama), Rocky Mountain region (Utah and Colorado), and California. In addition, imports are available generally at seaboard locations. The distribution system is from mill to regional warehouses, or to combination warehouse-fabrication yards, with either type owned by the major structural steel producers or large independents. The mill inventory also serves as a regional stock.

The regional network is supplemented by a system of local warehouses and small fabricators, usually of an independent nature. The local facilities obtain their supplies from regional warehouses or direct from mills for large orders.

Table E1-6
POTENTIAL APPLICATIONS OF STRUCTURAL STEEL MATERIALS
TO BLAST SHELTER UPGRADING *

<u>Material</u>	<u>Potential Application</u>	<u>Application Notes</u>
<u>Structural Steel Plate</u>	Closure of small openings such as doors, windows, ventilation shafts, utility penetrations in floors, area ways, etc. Covering for structural steel frame closure for large openings.	Provide hold-down by bolting, loading with sand bags, bracing, etc.
<u>Rolled Structural Steel Shapes</u> Light-Weight Channels and "I" Beams	Closure of openings in flat-wise position Covering for main structural steel frame closure for large openings Framing to support plate covering for small openings	Tie together by tackwelding, transverse bolting through flanges, etc. Provide hold-down by bolting, loading with sand bags, bracing, etc.

* Special attention must be given to attachment of closure and support systems to the structural framework of the building under consideration. In many cases the framework will require upgrading.

Table E1-6 (concluded)

Wide-Flange Beams, "H" Piles, "I" Beams, Channels, Angles, and Tee Beams	Structural frame to support closures for large openings such as vehicle doors, stairwells, and elevator shafts. Covering may include steel plate, wood planks, stressed skin plywood panels, etc.	Fastenings can be made by welding, drilling, and bolting, etc. Rather precise cutting is required
	Columns to strengthen existing columns; or added columns to reduce spans in beams, girders, and slabs.	Requires rather precise cutting to length, or wedging. Top and bottom bearing plates are required.
Pipe and Structural Tubing	Columns (see above). Rectangular tubing can be used in flat-wise position for direct closure of openings or for covering structural steel frames for large openings. Rectangular tubing may be used for fabrication of a structural frame for closure of large openings.	(See above). Tie together by tackwelding. Provide hold-down by bolting, covering with sand bags, etc.
Other Structural Steel Materials	Closure of openings in base-to-base position. Columns	See notes for flat-wise application.
Railroad Rails	Direct closure of openings Covering for structural steel frame for large openings.	See column notes above. Lock edges together. Provide hold-down by tackwelding, bolting, loading with sand bags, etc.
Sheet Pile		

Imported materials go to the independent regional establishments or to large volume local dealers.

Except in the least populated areas of the country, a regional or local warehouse may be expected to be found within one-day truck distance (say 100 miles) of any population center.

All establishments have capability of some fabrication, the minimum being drilling and cutting to size (flame cutting, shearing or sawing). Most also have welding and grinding facilities for prefabrication to some degree. These capabilities should be used to the maximum to reduce on-site problems.

Many salvage yards also maintain some sizes of both new and used structural shapes in limited quantities.

Railroad rails are usually procured by the railroads in large quantity direct from mills and are stockpiled at division points and other central locations. Both new and re-rail (used) materials may be procured from these sources under emergency conditions. Re-rail from main lines is used by the railroads for the construction of sidings, yards, etc. In some cases re-rail materials are handled by specialty salvage firms to be resold or converted to scrap.

Limited amounts of sheet pile will be found in regional warehouses; however, specialty firms deal in both new and used materials. Sheet pile will be generally found to be more available near the seaboard than elsewhere.

Local sources of structural steel products are easily identified from the yellow pages of telephone directories under the headings of "Steel Distributors and Warehouses," "Steel Fabricators," "Steel Mills," "Steel, Used," etc.

B. Typical Availability

Because of relatively high costs, neither regional nor local sources maintain inventories of all potential sizes and shapes of structural steel products. Large and specialty orders must usually be procured directly on mill order. The stockage will consist of those products and shapes used in relatively large quantities in the area served. These products will generally be limited to those used in the general building work of the area served, usually on medium-sized and smaller buildings, or the equivalent. In a large urban area, however, a fairly wide range of products and sizes would be found among all of the available suppliers. Typical availabilities are indicated in Table E1-7 for "regional warehouses" and for "local suppliers". In all cases, however, availability should be determined before proceeding with design.

Table E1-7

TYPICAL AVAILABILITY OF STRUCTURAL STEEL MATERIALS
AT REGIONAL WAREHOUSES AND LOCAL SUPPLIERS

Material	Local Suppliers	Regional Warehouses
<u>Structural Steel Plate</u>	Usually carry a fairly complete stock of thicknesses from 1/4 in. to 3/4 in. and maybe 1 in. Maximum width of 8 ft, lengths to 20 ft. Any size width and length is easily flame cut. Usually limited to A36 steel.	Will carry same as local suppliers in a larger variety of lengths and widths. Thicknesses up to 4 in. may be available. Equipped for production cutting to size. Other grades available.
<u>Rolled Structural Steel Shapes</u> Wide Flange Beams and Columns	Will carry a range of depths in 2 in. increments from about 4 in. up to 16 or 18 in. Weights will be limited to one or two in each size range. Lengths to about 40 ft available in the larger sizes. Any length can be cut. Usually limited to A36 steel.	Carry same as local suppliers plus the larger depths to 36 in. in some quantity. Fairly complete range of weights available in each size range. Some high strength grades (A440, A441, and A242) available in most common sizes. Lengths to 60 ft available in most sizes and weights.
Standard "I" Beams	Generally carry a range of depths from 6 in. to 12 in. to 15 in. Usually only one weight carried. Lengths to about 40 ft available in the larger sizes. Any length can be cut. Usually carry only A36 grade.	All of the local supplies plus the larger sizes to 24 in. and smaller to 3 in. All weights usually available. Lengths to 60 ft. Higher strength grades usually available.

Table E1-7 (concluded)

Light Wide-Flange Columns and Beams; Junior and Miscellaneous Shapes	Fairly large quantities of the smaller sizes, 8 in. depth and less, will be found because of use in light building construction. Weights will be limited to one per size usually. Lengths available to about 30 ft. Usually only A36 grade available.	Will carry a rather complete range of sizes and weights because of heavy use in building construction. Usually not carried in other than A36 grade.
"H" Piles	Usually not carried.	Usually carry complete range of sizes and weights. Longer lengths to 100 ft available in limited quantity. Not carried in other than A36 grade.
Standard Channels	Limited quantity of sizes and weights to about 15 in. carried. Lengths to about 40 ft. Any length can be cut. Usually stocked in A36 grade only.	Carry a fairly complete range of depths and weights to 60 ft length. May have higher strength grades.
Junior Channels	Usually not carried.	Ordinarily carried in all three sizes.
Angles	Fairly complete range of sizes available in the medium and smaller thicknesses.	Will carry same as local supplier plus heavier thicknesses.
Tees	Usually not stocked. Cut on order.	Usually not stocked. Cut on order.
<u>Pipe and Structural Tubing</u>	Usually not carried by local suppliers.	Carry a range of pipe in Standard (Schedule 40) and Extra Strong (Schedule 80) weights. Lengths to 20 ft. Large size (8 to 12 in.) Double Extra Strong possible. Range of square and rectangular tubing is carried.

Dimensioning Practice For Rolled Structural Shapes

Dimensions of rolled structural shapes are generally given in nominal form for depth and width. As a result of a variety of factors including economy in the use of rolls, roll wear, roll re-dressing, etc., a substantial variation of actual dimension from the nominal is produced. The greatest variations result from the process of obtaining heavier weights per foot by "spreading the rolls," thus obtaining greater flange and web thicknesses. This practice permits the rolling of a variety of shapes with the same nominal dimensions on the same set of rolls. Any design or use of structural steel shapes for blast shelter upgrading purposes requires attention to the actual dimensions of the section as opposed to the nominal dimensions. Both nominal and actual (within rolling tolerances) dimensions of the available rolled shapes in the various weights are given in Reference [1], as well as in American Society of Civil Engineers and mill literature. Dimensioning practice varies with the type of shape and is summarized in the following paragraphs.

A. Wide-Flange Beams and Columns (W and HP Shapes)

For a given nominal depth (outside-to-outside of flanges dimension), there is a constant (inside-to-inside) dimension between flanges. The latter remains constant while the former varies with the weight of the section. For example, a W12 section has a constant actual inside flange dimension of about 10-7/8" (actual 10.908") for weights of 40 lb/ft and heavier, while the actual outside depth ranges from 11-7/8" to 14-3/8" with increasing weights per foot. The flange width also varies with the weight, being 8" for a 40 lb/ft section (W12x45) and 12-5/8" for a 190 lb/ft section (W12x190) [1(p.1-17 and 1-38)].

B. Standard "I" Beams (S Shapes)

Differing from wide-flange beam practice, standard "I" Beams are rolled with the actual depth (outside-to-outside of flanges dimension) being equal (within mill tolerances) to the nominal depth. Thus the nominal outside-to-outside depth is the constant, rather than the inside-to-inside depth as in the case of the wide-flange sections. Weight per foot variations result in changes in web thickness and flange width. Thus an 8x4 "I" Beam of 18.4 lb/ft (S8x18.4) has a web thickness of 1/4", a flange (maximum) thickness of 7/16" and a flange width of 4". Increasing the weight to 23 lb/ft (S8x23) results in a web thickness of 7/16" and a flange width of 4-1/8" while the flange thickness remains at 7/16" and the actual depth at 8".

C. Channels (C and MC Shapes)

Standard Channels are rolled and dimensioned in a similar manner to standard "I" Beams with the actual depth (outside-to-outside of flanges dimension) being constant and equal to the nominal depth. For each depth, increased weights result in increased web thickness and flange width while the flange thickness remains constant.

D. Angles (L Shapes)

Variation in weight per foot for Angles is obtained by uniform increase of thickness of legs.

E. Tees (MT, ST, and WT Shapes)

Since Tees are cut from the various beam shapes indicated, dimensioning practice follows that of the basic shape.

REFERENCES

1. Manual of Steel Construction, American Institute of Steel Construction, Inc., Wrigley Building (8th Floor), 400 North Michigan Avenue, Chicago, Illinois 60611; 7th edition, 1970. Note: A later edition is now available.

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UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS:
PREDESIGNED EXPEDIENT OPTIONS II

(UNCLASSIFIED)

By: H. L. Murphy*

SRI International (formerly Stanford Research Institute)
Menlo Park, California 94025, July 1980, 272 pages
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This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. The purpose of upgrading such basement spaces is to provide shelter when needed by persons in: host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using selected target aiming points and Mt-range bursts; and, risk areas, where shelter is needed that is within 15-minutes travel time of each key worker's place of work and provides potential shelter for 30- to 50-psi air blast ranges, in terms of peak free field overpressure.

Chapters of this report's main text are devoted to discussions of:

background; general principles applicable to upgrading basements; closures for all basement shelter openings/apertures, in terms of principles for providing them; needs to be met in strengthening the structure over shelter candidate basements; some techniques and materials that can be used for such structure strengthening; and, shelters for key workers. In general, the main text of this report is intended for the artisan, the appendices having the more extensive, technical data and discussions.

The titles of the appendices are: Blast-Resistant Design/Analysis General Approach; Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication; Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; Plywood Use for Closures - Design; Wood Beam and Column Design - Simply Supported; Home Basements Upgrading in Host Areas; Blast-Resistant Design/Analysis of Steel Members; and, Structural Steel Local Availability and Use for Blast Shelter Upgrading.

The suggestions, guidance and technical help of M. A. Pachuta, G. N. Sisson, and D. W. Bensen, FEMA, are gratefully acknowledged, as are the contributions of former colleagues C. K. Wiehle, E. E. Pickering and J. E. Beck. #

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