The following report is reprinted from
the
Proceedings
of the
Conference on
Building in the Atomic Age

Massachusetts Institute of Technology

June 16 and 17, 1952

This work was carried out at the Civil Engineering Department of the University of Michigan and received partial support from the Rackham School of Graduate Studies and the Engineering Research Institute of the University of Michigan at Ann Arbor.

STEEL FRAMES FOR INDUSTRIAL BUILDINGS

BRUCE G. JOHNSTON, Professor of Structural Engineering
Civil Engineering Department, University of Michigan
With the Assistance of
R. C. BYCE, L. S. HU, C. MUKTABHANT, AND H. H. TUNG

I. INTRODUCTION

This report presents results of dynamic analyses of simple steel frame industrial buildings under assumed conditions of exposure to atomic bomb explosion. Studies are made of relative behavior as a function of type of roof and wall panel and type of frame. Modifications such as steel strand sway bracing that might serve to increase the resistance of the building to atomic blast loads are also considered. Only one story structures are considered herein under the category "industrial buildings" and typical frame types are shown in Figure 1.

The well-known publication, "Effects of Atomic Weapons" (1*) has been used as a principal basis for the studies. The fringe area where partial survival might be expected is of particular interest and studies were made at 4,500, 5,500, and 7,500 feet distance from ground zero for an air burst at 2,000 feet altitude of a "nominal" atomic bomb. Although the results indicate slightly more theoretical damage, they are in general agreement with those reported as typical of Hiroshima and Nagasaki (1, 14). The shielding of adjacent buildings, ground unevenness, and other factors create differences between the actual and the theoretical.

It is emphasized that the purpose of this study is to make relative comparisons, not quantitative predictions as to survival at any given distance from any given bomb. The obvious great uncertainties as to location of burst and intensity of any bomb that might be used during a war render precise design for complete protection both uneconomical and impracticable. Nevertheless, low cost improvement of survival chance in fringe areas should be a worthwhile objective.

The typical blast pressure-time functions do not represent accurately the force that is actually experienced by the usual industrial building structure. The interrelation between load and reaction as it is successively transmitted from panels to panel supports and thence to frames results in time delay and alteration in shape of the force-time function experienced by the main frames. The studies reported herein also show that standard types of asbestos or metal wall panels contribute a relatively negligible impulse to the structural frame compared to the drag on the bare frame after the panels are ripped off.

Although precise dynamic analyses for "design" purposes are hardly warranted, it was nevertheless necessary in the present studies to make reasonably accurate analyses for comparative purposes. Both the elastic and plastic ranges of behavior were considered and deflection for complete collapse was estimated. Anything short of complete collapse may leave a building in a condition where (as shown by experience during the last war) restoration to normal operation is feasible.

The problem of dynamic analysis may be stated briefly as follows for any building or portion of a building. A point on the structure is selected which will significantly represent the load-deflection behavior of the structure or portion of structure under consideration. Considering next the probable geometric pattern of deflection to be expected during dynamic behavior together with an application of principles of work and energy, an "effective" mass and an "effective" force are computed by which the behavior of the distributed structure and distributed force are reduced to equivalent effects at a point or points. In addition, the resistance of the structure to deflection or to relative deflection between the various representative points must be calculated or obtained by experiment. At each point, the difference between the effective force and resistance, divided by the effective mass, is equal to the acceleration. Numerical procedures such as developed by Dr. N. M. Newmark (2) and others may be used for calculation of deflection under dynamic load in both elastic and plastic ranges. Although the effective resistance of a structure is itself a function of rate of deflection and damping, the static load-deflection curve may be used as a first approximation in dynamic analyses to give approximate comparative results.

In calculating the resistance functions of steel panels, beams, and frames, a slight increase in the normal yield strength level of structural steel has been assumed. The average yield strength in the flange of rolled beam sections may be assumed as approximately 35 kips per square inch and a moderate increase in this to allow for the rate of strain that might be experienced in a shock loaded structural member led to the adoption of 45 kips per square inch as the yield stress level to be used in these studies.† Methods for calculating the plastic ultimate resistance of continuous steel beams

* Numbers in parentheses refer to references listed at the end of this report.

† Further study of the problem of strain rate is presented in another paper (3).

have long been available and some of the early work is quoted by Timoshenko (4) in his "Strength of Materials." The analytical studies and tests by J. F. Baker (5) and his associates in Great Britain to support the utilization of plastic strength in structural design are well known. Recent progress in plastic methods of structural analysis have been reviewed by Symonds and Neal (6), and the work of the Lehigh University group during the past few years has furnished valuable test results and analyses relative to the plastic behavior of components of continuous steel frames, including beams (7), columns (8), and connections (9) tested separately, as well as tests now in progress on complete frames (10) of approximately full scale. The applicability of plastic analysis procedures to the study of shock loaded structures is illustrated by such use in England in the prediction of strength and behavior of buildings and shelters subjected to blast and shock of high explosive bombs (11).

Figure 2 illustrates two distinctly different conditions that may arise as the blast wave envelops a building. If the walls and roof remain intact (Fig. 2A) considerable pressure will impinge on all outer surfaces during the positive phase of the blast wave. Walls made of a very easily fractured or "frangible" material (Fig. 2B) would offer little or no impediment to the progress of the blast, thus tending to equalize the pressure on the top of the roof by a similar though retarded pressure on the under-side. Of additional great importance is the fact that after passage of the shock front the drag on the skeleton framework is greatly reduced in comparison with that offered by the over-all structure with walls and roof intact. Complete protection as shown in Figure 2A may be desirable in some cases but is, of course, a much more costly type of construction in comparison with 2B. Type (A) protection would be desirable if the contents of the building were extremely important and easily damaged, whereas type (B) protection might be expedient for heavy manufacturing industries where much of the equipment might be relatively undamaged during the passage of the blast wave through the building. In connection with type (B) protection it is obvious that personnel should be sheltered in adequate blast proof rooms or external shelters during the period of emergency and that important electrical machinery or other key apparatus likewise be protected by individual blast walls and/or individual roof structures as was common during the last war. The matter of machinery and personnel protection based on experience during the past war, as well as preliminary studies regarding the effects of atomic weapons, has been reviewed by the Nationa. Industrial Conference Board (12, 13). The possibility of using frangible wall construction so as to equalize pressure has been discussed by Bowman (14) and is currently recommended by the Federal Civil Defense Administration (15).

Buildings with reinforced concrete walls having window areas would represent a condition intermediate between the two extremes shown in Figure 2. Attention in this report, however, is especially directed toward a comparison of structural steel frame behavior when frangible wall panels are used throughout.

Two basic frame types considered in this program are shown in Figure 3. In the proposed continuation of the work other types (Fig. 1) would be considered, including multi-bay saw-tooth roof construction, gable frames, and buildings in which heavy duty cranes introduce relatively heavy columns.

II. DEFLECTION RESISTANCE OF PANELS, GIRTS, AND FRAMES

The interrelation between loads and reactions that are successively transmitted from panels to purlins or girts and thence to frames is illustrated in Figure 4. No consideration was given to possible reduction in pressure resulting from movement of a structural panel during failure. In the case of light panels with rapid accelerations this would introduce a probable error but the load transmitted by such a panel to the frame is primarily dependent on the load deflection characteristics of the panel itself rather than on the exact shape of the pressure-time curve that is applied. The impulse produced by a low strength frangible panel has little effect on the over-all behavior of the frame since the principal load is that of drag after panel failure. In the case of reinforced concrete panels that might survive an atomic blast, recent studies indicate that at any given time there would be little reduction in air pressure due to movement of such panels (16). Hence, in those cases where reduction would be an important matter it appears that no reduction need be considered.

The types of roof and wall panel shown in Figure 5 are representative of those used in commercial practice and present a range of structural behavior from the frangible corrugated asbestos to the highly resistant reinforced concrete. Of these, three types were selected for use in the studies reported herein. These were corrugated asbestos, steel deck, and reinforced concrete.

Corrugated asbestos was chosen as representative of the most frangible type of panel and one that is used quite extensively for industrial buildings. When applied to the roof a slope of 3 inches in 12 inches is usual, but the panels may be used with suitable weatherproofing and insulation for flat roofs. The panels usually extend over one intermediate support and hook clips, "Z" clips, or "J" bolts are used for connections to girts and purlins. Wall girt supports are spaced about 5 feet 6 inches apart and roof purlins at about 3 feet 9 inches.

Sheet steel roof and wall panels may be cold-formed into many shapes and those shown in Figure 5 are specific examples of the many types available. The panels are tack welded to the girts, purlins, and to one another. Wall sections may be made up of several sheets, with insulation, whereas roof deck is usually a single sheet covered with insulation board, felt, tar, and gravel. The spans are made continuous over several supports so that the total length of one sheet may be 50 feet.

When a reinforced concrete roof is used, a 4-inch slab for an 8-foot span is a common requirement. Walls in industrial buildings are 8 or 10 inches thick with a single plane of reinforcing or 12-inch with double plane reinforcing when greater height warrants. Slight modifications in these standard designs (as shown in Fig. 5) will greatly increase ultimate strength.

Curves showing the deflection resistance of several of the roof panels are shown in Figure 6. The information was obtained partially from test results and partially by approximate analyses. The great variation in energy absorbed up to failure is of interest. Brittle panels of small mass and low strength, as in the case of corrugated asbestos, fail very quickly under the sudden application of a pressure load. Table I is presented to provide a comparison of time required for failure of front panels at various distances from a nominal atomic bomb air burst at 2,000-foot elevation.

In the initial elastic range the strength of a wall or roof panel is largely determined by bending resistance. In the case of a brittle panel such as one made of corrugated asbestos or glass, the behavior is essentially elastic up to the ultimate load as determined by the bending resistance at fracture. In the case of a ductile material such as corrugated steel or steel deck, when the bending resistance decreases due to a combination of causes, such as local crippling or buckling, the section may develop much more resistance provided it is fastened securely at each end and further provided that the ends are either part of a continuous series of spans or are restrained against movement by suitable bracing. As a ductile panel deflects and loses its bending resistance, it starts to develop direct tensile "membrane" stresses that resist the load much as does the cable on a suspension bridge. If the end connections are weak the panel will have only a small increase in resistance above that due to bending. Sheet steel decking has been tested by various investigators both as a simple beam and as a roofing member in place. These latter tests bring in the effect of membrane action directly. Test results available to the writer were not carried to complete failure but the curves were extrapolated to a deflection estimated for collapse. The interior spans of this type of panel depend on the exterior spans for lateral reaction required at supports. If the pressures are acting inward the end panel is assumed to wrap itself around the girts or purlins. This will allow the panel to carry load for somewhat larger deflections. If the pressures are acting outward then the panel is assumed to tear free from the girts or purlins.

The load-deflection curves for the reinforced concrete panels were determined by methods commonly used in calculating the ultimate strength of reinforced concrete as reviewed by Jensen (17) with some modifications in assumptions based on more recent tests. The analysis of standard construction showed that concrete slab resistance could be greatly increased by reasonable increase in the percentage of steel used. Approximate formulas were developed and checked against a number of tests of concrete panels that included load-deflection measurements to final failure (18).

Wall panels are supported by "girts," usually consisting of angles with the outstanding leg down. Sag rods may be used to help support the weight of the wall but the angle must resist the wind loads. Roof panels are supported by "purlins," usually channels or wide-flange beams. Rods are sometimes used between purlins to give lateral resistance if the roofing material is considered too weak in this direction. Girts or purlins, as in the case of panels, may develop great additional "catenary" strength after losing their bending strength. Such added resistance will depend on the lateral rigidity of supports and the efficiency of the bolted or welded connections between purlins and supports or between purlins and adjacent purlins in the case of continuous spans. Figure 7A shows the hypothetical tensile load-deflection curve for a girt, together with its connections, including bolt slip at the end connections. If the girt were under uniform pressure and had no bending resistance it would deflect in a circular form as shown in Figure 7B and the deflection at the center could be determined for any given value of tension load as taken off of Figure 7A. The lateral load resistance of the girt is also a function of the tension and thus point by point a curve of lateral pressure versus center deflection can be plotted (Fig. 7B). If initial yielding at the location of maximum moment causes a "plastic hinge" to develop with possible reduction in resistance due to local crippling, the deflected shape would be that of a shallow V, also shown in Figure 7B. The actual resistance due to direct tension, herein termed "catenary" action, will be intermediate between the two curves shown in Figure 7B. In the case of the angle girt as used in the frames that were studied, it was assumed that after the panels were torn off the girts would twist into a position of minimum bending resistance as indicated in Figure 7C. The "catenary" and "bending" resistances may be superposed to give an approximate deflection-resistance curve as shown in Figure 7D. In the case considered the standard end connection of two bolts was insufficient to develop direct tension force at the yield level, hence, the full potential resistance to lateral force could not be realized.

As illustrated in Figure 8 the over-all resistance of a one story building frame (8A) may be satisfactorily represented by a single load-deflection curve (8B) for the cases considered in this report where the significant loads are primarily in the lateral direction. This is the case when frangible walls are used and the downward components of motion are relatively small. When complete enclosure of structure is attempted by use of reinforced concrete walls and roof the yielding process becomes much more complex as downward failure of the roof structure may occur. When frangible walls are used the lateral loads result primarily from drag on the roof structure. Hence the use of a resistance function wherein effective load applied at the roof level is plotted vs. deflection at this point seems the best means of indexing frame resistance. Figure 8B illustrates in a general way the resistance function of a frame which (as in the case of a structural member) has elastic, yield, and collapse ranges of behavior. Initial yield occurs when the maximum moments result in stress at the yield point level. In the case of wide flange shapes, bent in their strong plane, little additional bending moment can be resisted after the yield point is reached and the inelastic range is entered. Yielding then progresses rapidly and deflections increase at an increasing rate. The maximum load resistance of a structure is a significant value since the applied load must be reduced progressively if static equilibrium is to be maintained for greater deflection than that corresponding to maximum load. Continued load will cause accelerated motion ending in complete collapse. The maximum load will be determined either by over-all buckling, by fracturing of some key part or parts of a structure, or simply by straining the material beyond its ultimate load capacity.

In the case of the frames considered in this investigation the calculation of deflection resistance functions is rather simple and comparative examples are shown in Figure 9 for both continuous frame and frame-truss construction considering both pinned and fixed column base to footing connections. Figure 9 permits the following comparisons to be made with respect to lateral resistance of the simple frame types considered.

(1) For both continuous welded-frame construction and truss-frame construction, fixed base design approximately doubles the lateral resistance in comparison with pinned or hinged bases.

(2) Continuous welded-frame construction has several times as much plastic lateral resistance as the corresponding truss-frame construction. This is primarily due to the fact that in continuous frame design the columns participate by bending as well as by direct stress in resisting the vertical roof loads and are therefore of heavier section.

(3) Frames designed for heavy roof panels, such as reinforced concrete, have about double the plastic lateral resistance in comparison with lightweight roof panel construction. (The greater mass of the heavy roof also provides greater inertial resistance to acceleration under shock load.) In some of the truss designs, especially in case of small roof dead load, the lower chords in the end panels are not highly stressed under vertical design load but under lateral force tending to cause column failure these locations become of primary importance. Tensile failure of the connection between the lower chord and the column on the windward side or the buckling of this member on the leeward side may reduce the resisting moment at the top of the frame to less than the amount necessary to develop the full plastic bending strength of the column at this section. Possible failure of the upper chord to column connection should also be considered since a combination of tension due to vertical load and sidesway on the leeward side would be likely to induce abnormally high tensile components in the connection.

III. LOAD TRANSMISSION WITHIN THE STRUCTURE

Referring again to Figure 4, the approximate dynamic analysis during load transmission from panels to girts to frame will be reviewed. To simplify the work, an average pressure-time relationship for a typical panel at mid-height of the wall was taken as representative of wall behavior. Newmark's numerical method (2) was used for the panels not expected to fail and his semi-graphical trial and error procedure was used for the panels that failed. Results are probably more accurate than the uncertainty in resistance or load functions warrants. Distributed force and mass were replaced by equivalent center load and mass for each panel.

In general, the dynamic analysis of girts and purlins is similar to that used for the roof and wall panels. The deflection-resistance function must be determined up to collapse; the load-time relation is obtained from the panel analysis, and the numerical procedures are applied to obtain the end reactions from the girts and purlins which are the loads on the frame.

Since, as shown in Table I, the asbestos and steel panels fail in a relatively short time, the influence of girt or purlin motion on their behavior is insignificant and the two problems (panel behavior and girt behavior) can be considered separately.

The dynamic behavior of the girts and purlins is typified by the end reaction versus time curves shown in Figure 10 which is for 7,500 feet from ground zero.

Also shown is what might be expected in the case of a concrete wall section that does not fail. Detailed investigation of the behavior of girts was not carried out after it was found that the impulse of panel reaction was extremely small in comparison to the drag forces acting on the members. For the analysis of the frame with corrugated asbestos walls, it was found that only the time for failure of the front wall was required for an approximate solution.

Figure 11 carries over the front panel impulses of asbestos and steel panels shown in Figure 10, but with reduced vertical scale and increased horizontal scale. Thus the impulse of the girt reactions on the frame is shown in Figure 11 by the sharp initial and nearly vertical lines. In the case of the steel panels there is also shown a negative effect due to the pressure inward when the blast wave reaches the rear of the structure. This is immediately followed by another positive impulse as the interior retarded shock front reaches the back of the building and carries the rear panels out with it in the blast direction. The principal force-time functions shown in Figure 11 represent drag on the skeleton steel structure that builds up rapidly as the blast wave progresses across the building. The successive accumulation of drag force effectively alters the shape of the force-time function from that obtaining from impingement of a blast pressure wave on a solid immovable box structure. The magnitude of the drag force on the skeleton frame is, of course, much less than the over-all drag on a solid box structure of the same size. The greater drag on the truss frame is due to the greater area of exposed framing members as compared with continuous frame construction. The greater drag for asbestos roofing in comparison with steel deck is due to the greater number of purlins used for the asbestos roofing. In calculating drag on the purlins a fifty per cent reduction to account for shielding was used in comparison with the drag coefficient of 2.0 used on fully exposed steel members. A correction was made for the increase in mass density of air due to increased pressure and decrease due to increased air temperature. This is partially offset by aerodynamic lag in drag build-up, by the relatively high drag coefficients used, and by the uncertainties as to duration and exact nature of the wind gust that follows the shock front.

As shown by the dashed line, the drag-time function is nearly triangular in shape. In elastic cases the maximum deflection may be determined by multiplying the equivalent static deflection for the peak drag force by an "amplification factor," the solution for this shape forcing function being given in graphical form for all time relations by Mindlin (19).

IV. STRAND REINFORCEMENT FOR EXISTING FRAMES

The economy of attempting to materially strengthen existing frames against atomic explosion may be open to question, but it might be desirable in certain cases. Figure 12 shows a possible use of steel wire strand sidesway stays detachable by means of pins and standard cable sockets. The pin plate at the upper end usually could be welded directly to the outer surface of the column in an existing frame. One strand would be attached on each side of each bent and additional bracing in a direction normal to the plane of the bent would not be necessary if cross bracing in the plane of the walls were feasible. Figure 13 shows the horizontal component of lateral resistance obtainable from a one and one-eighth inch diameter galvanized steel strand cable installed as shown in Figure 12. The additional strength obtainable from such an installation is relatively great in comparison with that existing in the frame and the sum of the two resistances as shown in Figure 13 will greatly increase the chances of survival and reduce permanent lateral deflection in any fringe area. It is seen that the wire strand increases the effective lateral resistance of the truss frames by a factor varying from three to six, depending on roof loading and column base fixity. The lateral resistance of the continuous frame with pinned base is doubled. Strand strengthening would be especially desirable for existing truss frames. Such an installation would not introduce bending moment into the column footings and as shown in Figure 13 would increase the lateral resistance of the frame by several times. Strands would be removable and could be installed during periods of emergency. There are obvious objections to the use of such strands since they would not be feasible if adjacent buildings interfered or if the ground area next to the buildings were needed for storage or other purposes. In some cases, however, it might be possible to interconnect adjacent buildings by strands, and anchors to ground would then be needed only on one side of the buildings so interconnected.

V. RESULTS OF DYNAMIC ANALYSES

The results of the dynamic analyses of the frames under the various conditions that have been outlined are summarized in Table II. All frames were designed for the same roof live load of 40 pounds per square foot of snow and lateral wind of 20 pounds per square foot on outside walls. A vertical line is indicative of survival of main frames with little permanent damage. The "N" indicates complete collapse. A slanting line indicates permanent lateral deflection at the top of the frame with a number to indicate the amount in inches. This table illustrates very well the advantages of using fixed base, continuous frame construction, as well as the strengthening possibilities in strand sway bracing. The relative lateral weakness of the truss frames is evident from a consideration of the four columns to the right of Table II. In the case of the commonly used hinged base construction, two of the three conditions considered indicate survival at 7,500 feet from ground zero but all exposures at 5,500 feet or less indicate complete collapse. With fixed base columns requiring special attention to footing design, the truss frames survive in all cases at 7,500 feet but fail at 5,500 feet or less when lightweight roofing is used. The use of a concrete roof provides enough added resistance to indicate survival at 5,500 and 4,500 feet with permanent deflections of 8 and 26 inches, respectively. The use of steel strand sway bracing prevents complete collapse in nine of the eleven cases where failure without such bracing is predicted. Sway bracing also greatly reduces permanent deflection in non-failure cases.

Referring to the first four columns of Table II, the relative superiority of continuous frames as compared to truss frames is quite obvious — only two continuous frames collapsed under conditions that produced 11 failures in truss frames. The continuous frames with concrete roof construction not only survived but remained elastic at all distances considered.

A study was also made of reinforced concrete wall and roof construction, applied to the continuous welded steel frame but redesigned for the heavier dead load. A twelve-inch wall and five-inch roof slab was assumed, with somewhat greater than nominal reinforcement. Calculations indicated that at 7,500 feet from ground zero (assuming no wall openings) the wall and roof might survive, but the main 62-foot span steel roof girders would be at the point of collapse, with a permanent downward deflection of about 6 feet. There appeared no point of attempting analyses for any closer distance to ground zero and emphasis of the investigation was directed primarily toward consideration of frangible wall construction. Obviously, the use of appreciable wall openings would permit entry of equalizing blast pressure and walls but would result in the possibility of complex interior blast reflections. Since the primary purpose of these studies was to make relative comparisons of structural frame behavior, it was decided at the outset to simplify the blast problem by considering the simplest and limiting condition of completely frangible walls.

VI. SUMMARY OF RECOMMENDATIONS AND CONCLUSIONS

Recommendations for the strengthening of existing construction or for modifications in new construction depend entirely on whether or not complete interior protection, insofar as possible, is desired, or whether the alternate of permitting the blast to pass through the building with as little impedance as possible is to be adopted. In the relatively narrow buildings that have been considered in this report, the use of frangible walls seems a generally desirable aim. It is assumed that personnel shelters would be provided and that important machinery would be protected by individual shelters or roof coverings. In a broad industrial building consisting of many bays and incorporating interior partitions or equipment that might impede the progress of a blast wave passing through the building, the lag of the interior blast with respect to that on the exterior might be considerably greater than in the case of mill buildings with only outer walls. In such cases roof failure could not be prevented since the outer pressure would not be balanced quickly and it would seem necessary and desirable to follow the recommendation previously suggested (14, 15) and use not only frangible walls but frangible roof covering as well.

Wall Construction. When the conditions are favorable, wall construction should be frangible over as large areas as possible, and should be non-inflammable and of a material that will minimize the damage resulting from flying fragments. The use of frangible walls under correct conditions accomplishes two major objectives:

(1) The over-all drag is reduced to that of the skeleton frame rather than that of the entire building structure.

(2) The pressures on the two sides of the roof panels are partially equalized, and it then becomes possible to prevent roof failure without major changes in structural design of panels or supporting purlins and girders or trusses.

Roof Construction. Although not essential, a reinforced concrete roof offers the following advantages.

(1) The additional mass increases inertial resistance to acceleration and the additional weight results in a heavier structure better able to resist lateral forces.

(2) With but slight increase in nominal design thickness and reinforcement (assuming flat roof construction and frangible walls) the reinforced concrete roof will remain intact at 4,500 feet and upward from ground zero a nominal atomic bomb air burst of 2,000-foot altitude.

(3) The reinforced concrete roof provides protection from the weather for building equipment during the period of rehabilitation after exposure to bombing.

Choice of Frame. In new construction, if a choice of frame is considered, fully continuous frames with a reinforced concrete roof and fixed column bases offer the greatest resistance of those considered. At the opposite extreme is the conventional truss frame with lightweight roof and pinned base columns. If columns are to be fixed at the base, special attention must be given to footing design, which must be adequate to resist the full plastic moment resistance of the column together with the moment caused by horizontal reaction at the column base. Combined footing and floor slab construction is recommended if feasible and it is to be remembered that considerable footing moment may be developed under blast load even in the case of the pinned base columns because of the above mentioned horizontal shear applied at the top of the footing. Integration of footing with floor slab will minimize the effect of the horizontal reaction load.

The present report does not cover truss frames in which heavy duty crane girders are supported by the columns, either by use of double columns or crane brackets that introduce bending moment into the column. Such buildings will have much greater lateral resistance than the framed truss structures considered in this report.

If a truss frame is chosen for a building without heavy duty cranes, some advantage for a two or three bay building would result from the use of a reinforced concrete roof slab supported on steel purlins. An additional advantage results from the fact that a steel truss (of the type shown in Fig. 3b), if designed for the heavier dead load resulting from a concrete slab, will have chords better able at the ends to develop full bending moment resistance inherent in the steel columns during lateral sidesway. Also, as for fully continuous frame construction, the truss frame may be strengthened by installing fixed column bases with footings designed for moment.

Design Procedures. It would be premature to suggest detailed procedures of design and the scope would be limited by the small number of frame types that are considered in this study. Furthermore, the primary objective in design for wartime uncertainties can never be more than the achievement of increased and balanced resistance to meet arbitrarily chosen nearness of exposure to an arbitrarily chosen magnitude of atomic bomb. If frangible walls are used, the primary consideration will not be pressure distributions but will be drag on skeleton members resulting from the wind gust accompanying the arbitrarily selected conditions of exposure.

For sharp cornered structural shapes a drag coefficient of 2 was used. In view of the importance of drag on the skeleton frames of industrial buildings, more information is needed regarding this action during the sudden gust that follows the shock front and involving the effects of changing pressure and temperature. The allowance to be made for shielding is another uncertainty. In the present studies, when successive purlins offered shielding, the drag was reduced by one half for all except the first purlin.

In the simple cases of plastic behavior considered in this report, all of the plastic "hinges" form about simultaneously. A modified procedure of dynamic analysis may be used in such case, based in part on the existing solution (19) for the amplification factor of a single degree of freedom elastic system acted upon by the triangular pulse that approximates the drag force-time function (Fig. 11). The general yield strength and corresponding deflection of the structure may be calculated, using the usual plastic analysis procedure. Since the behavior will be nearly linear up to general yield, the elastic amplification factor may be multiplied by the static deflection corresponding to peak drag force. If this is about equal to the calculated deflection for general initiation of plastic behavior, a good approximation of permissible peak dynamic load will have been determined. This procedure is not usually applicable as the plastic hinges do not usually occur simultaneously.

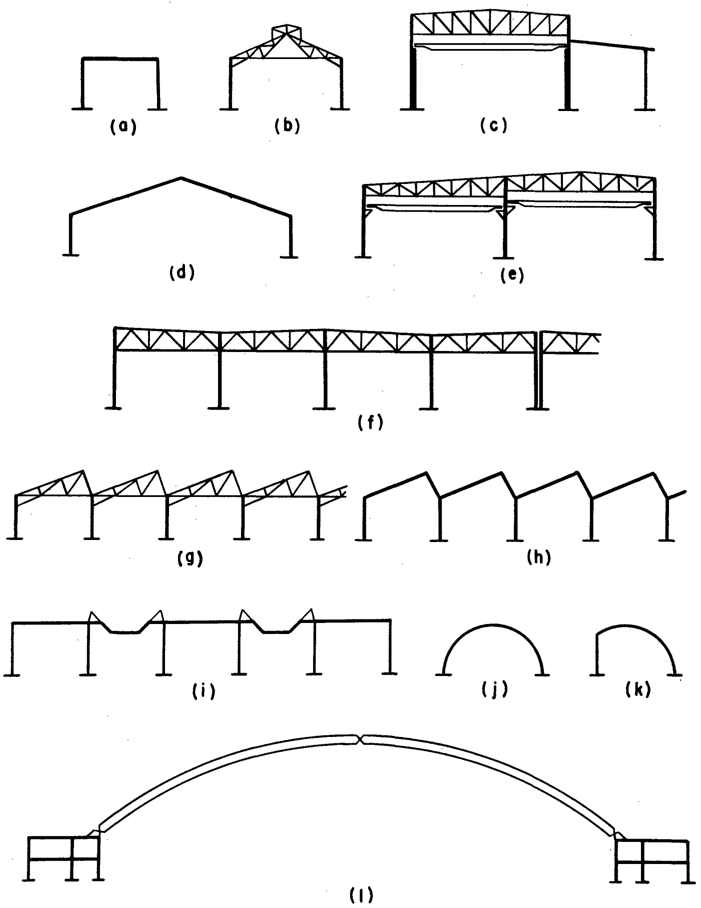
Although the foregoing suggestions appear reasonable on the basis of the present studies, the danger of sweeping generalizations should be kept in mind. Relatively small changes in design of panel and girt connections, for example, may have a large effect in altering the behavior of buildings that appear superficially to be identical in every respect. Each building is a problem in itself and it is only by individual evaluation of the interrelated behavior of panels, girts, purlins, and frames that its resistance can be assessed. Use of shapes favorable to reduction in aerodynamic drag will also increase the chance of building survival if frangible walls are used.

Acknowledgments

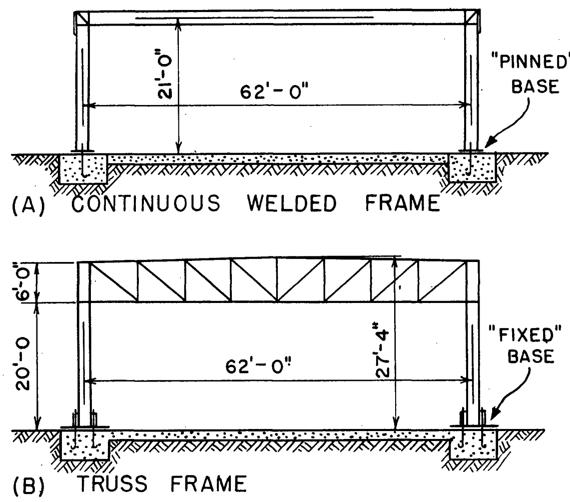
This study has been supported by the Engineering Research Institute and the Civil Engineering Department of the University of Michigan. The cooperation in various ways of the American Steel Band Company and the Mahon Steel Company is gratefully acknowledged. Particular acknowledgment is also due to the John Roebling & Sons Company of Trenton, New Jersey, who authorized a special test of galvanized steel strand and furnished load deformation curves to final failure. Information regarding certain connection tests was furnished by Dr. N. M. Newmark and William Munse of the University of Illinois. The authors also received guidance in the initial direction of the work and in selection of certain typical steel designs through the cooperation of the American Institute of Steel Construction and especially through the interest of Mr. John Griffiths, District Engineer, A. I. S. C., Omaha, Nebraska, and Mr. T. R. Higgins, Director of Engineering.

References

1. Los Alamos Scientific Laboratory. "The Effects of Atomic Weapons," U. S. Government Printing Office, Revised Edition 1950.
2. NEWMARK, N. M. "Methods of Analysis for Structures Subjected to Dynamic Loading," Directorate of Intelligence, U. S. Air Force, Washington, D. C., 1950.
3. JOHNSTON, B. G. "Structural Steel Members and Frames," *Earthquake and Blast Effects on Structures*, University of California Symposium, (1952.)
4. TIMOSHENKO, S. P. "Strength of Materials," Vol. 2, D. Van Nostrand Company, Inc., First edition 1930, Second edition 1941.
5. BAKER, J. F. "A Review of Recent Investigations into the Behavior of Steel Frames in the Plastic Range," *Journal of Institute of Civil Engineers* 3:185–240, 1949.
6. SYMONDS, P. S. AND NEAL, B. G. "Recent Progress in the Plastic Methods of Structural Analysis," *Journal Franklin Institute* 252:383–407, 469–492, 1951.
7. YANG, C. H., BEEDLE, L. S. AND JOHNSTON, B. G. "Plastic Design and the Deformation of Structures — Progress Report 3 — Welded Continuous Frames and Their Components," *Welding Journal* 30:348s–356s, 1951.
8. KETTER, R. L., BEEDLE, L. S. AND JOHNSTON, B. G. "Column Strength under Combined Bending and Thrust — Progress Report 6 — Welded Continuous Frames and Their Components." To be published in *Welding Journal Research Supplement.*
9. BEEDLE, L. S., TOPRACTSOGLOU, A. A. AND JOHNSTON, B. G. "Connections for Welded Continuous Portal Frames — Progress Report 4, Part III — Welded Continuous Frames and Their Components." To be published in *Welding Journal Research Supplement.*
10. RUZEK, J. M., KNUDSEN, KNUD-E., JOHNSTON, E. R. AND BEEDLE, L. S. "Welded Portal Frames Tested to Collapse — Progress Report 7 — Welded Continuous Frames and Their Components." To be published in *Welding Journal Research Supplement.*
11. BAKER, J. F., WILLIAMS, E. L. AND LAX, D. "The Design of Framed Buildings Against High-Explosive Bombs," *Civil Engineer in War* 3:80–112. Published by Institute of Civil Engineers, London, 1948.
12. National Industrial Conference Board, "Damage Control in Wartime," *Studies in Business Policy*, No. 53, The Conference Board, 1951, 247 Park Avenue, New York.
13. National Industrial Conference Board. "Protecting Personnel in Wartime," *Studies in Business Policy*, No. 55, The Conference Board, 1952, 247 Park Avenue, New York.
14. BOWMAN, H. L. "Bombs vs. Buildings," *Engineering News-Record* 144:4-24–28, 1950.
15. Federal Civil Defense Administration. "Interim Guide for the Design of Buildings Exposed to Atomic Blast Loads," March 1952.
16. FEIGEN, M. "Air Blast Pressures on a Cantilever Wall," University of California Report 51.11, December 1951.
17. JENSEN, V. P. "Ultimate Strength of Reinforced Concrete Beams as Related to the Plasticity Ratio of Concrete," Engineering Experiment Station Bulletin 345, University of Illinois, 1943.
18. OFJORD, A., HANSEN, R. ET AL. "Behavior of Structural Elements Under Impulsive Loads, M. I. T. Publication, April 1950.
19. MINDLIN, R. D. "Dynamics of Package Cushioning," Bell Telephone System, Technical Publications, Monograph B-1369, 1945.



INDUSTRIAL BUILDINGS
FIG. 1



FRAMES CONSIDERED IN DYNAMIC ANALYSES, FIG. 3

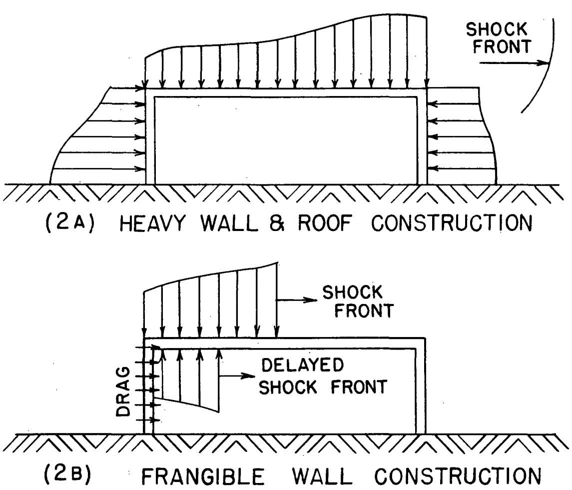


FIGURE 2

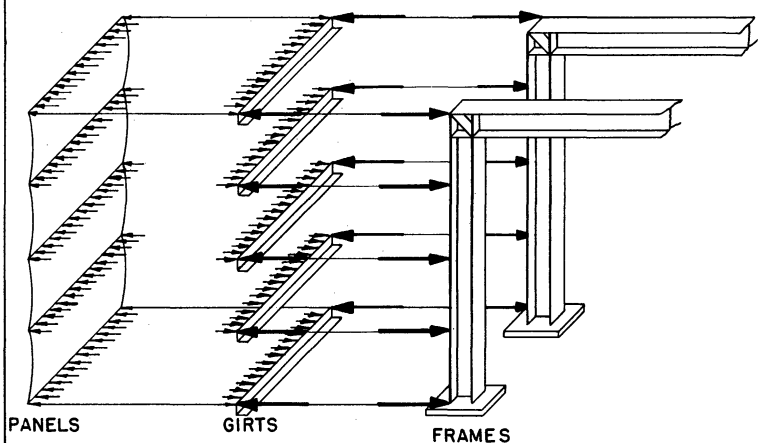


FIG. 4 TRANSMISSION OF LOADS AND REACTIONS

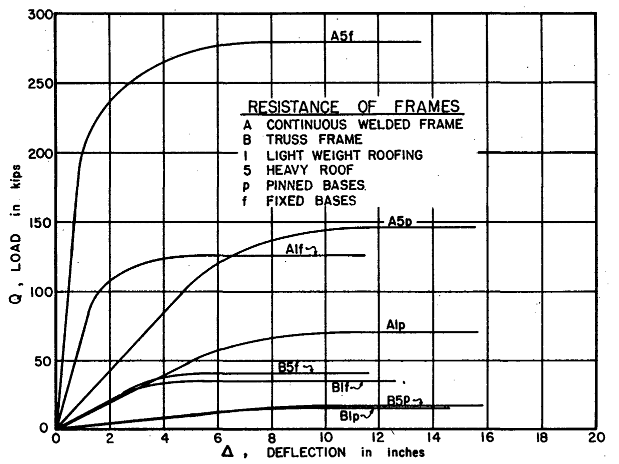


FIGURE 9

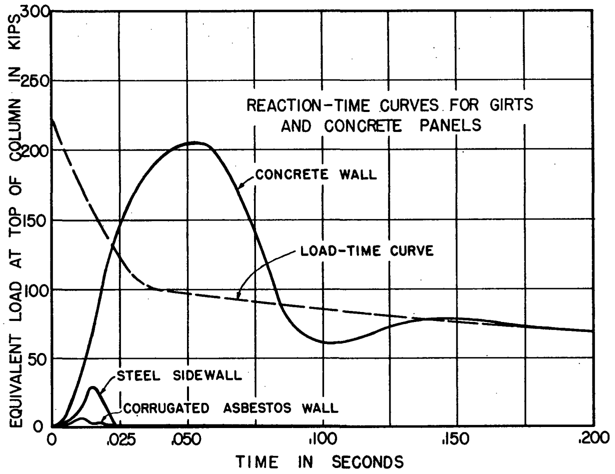


FIGURE 10

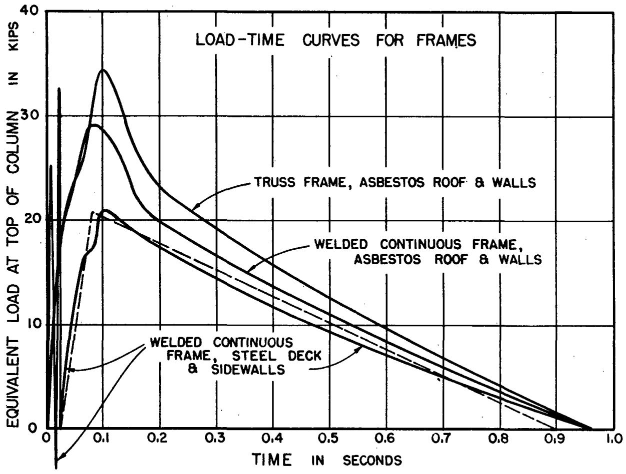


FIGURE 11

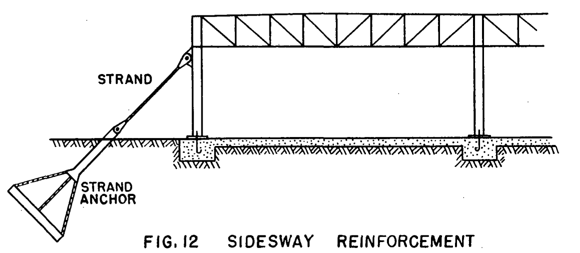
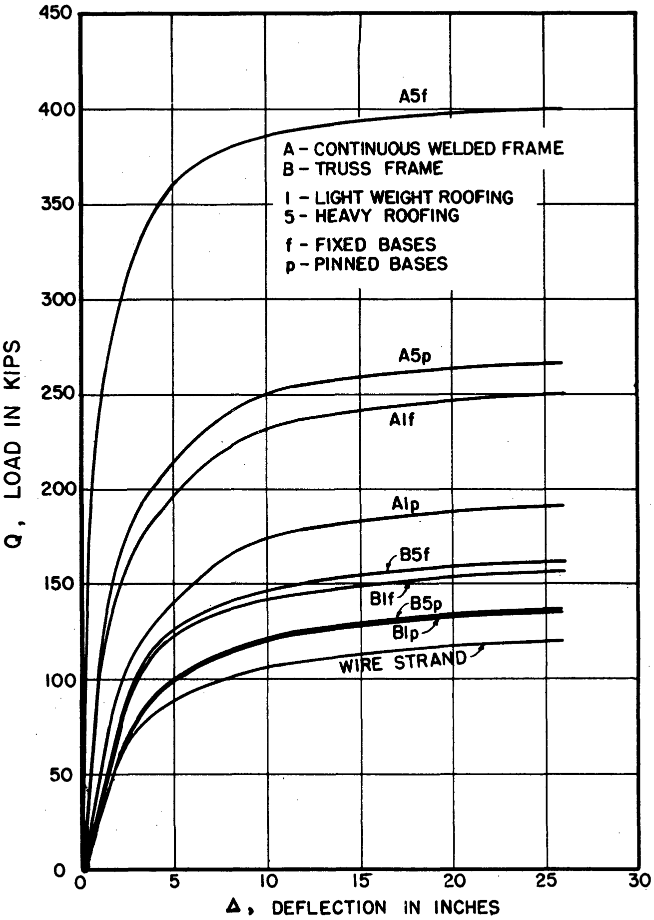


FIG. 12 SIDESWAY REINFORCEMENT

TABLE I
TIMES REQUIRED FOR FAILURE OF PANELS
FRONT WALLS— IN SECONDS

PANEL	7500′	5500′	4500′
CORRUGATED ASBESTOS	0.0048	0.0032	0.0019
STEEL DECK	0.0136	0.0085	0.0064
SPECIAL R.C. DESIGN	NO FAILURE	0.09	0.04



RESISTANCE OF FRAMES WITH CABLES
FIGURE 13

TABLE II
COMPARISON OF FRAME AND TRUSS BEHAVIOR

TYPE OF WALL & ROOF	DISTANCE FROM GROUND ZERO FT.	CONTINUOUS FRAME				TRUSS FRAME			
		HINGED BASE		FIXED BASE		HINGED BASE		FIXED BASE	
		AS DESIGNED	PLUS CABLE	AS DESIGNED	PLUS CABLE	AS DESIGNED	PLUS CABLE	AS DESIGNED	PLUS CABLE
CORRUGATED ASBESTOS ROOF & WALLS	7500						2	6	
	5500		1	1			8		6
	4500		8	18					
STEEL DECK ROOF & WALLS	7500					7			
	5500	4					1		1
	4500	28	1	1			7		4
CONCRETE ROOF ASBESTOS WALLS	7500					2			
	5500						1	8	
	4500						3	26	2

NOTATION		REMAINS ELASTIC	N	"N" INCHES PLASTIC DEFLECTION		FRAME COLLAPES