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A CRITICAL SURVEY

of

BRITTLE FAILURE IN CARBON PLATE STEEL STRUCTURES OTHER THAN SHIPS

by

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Massachusetts Institute of Technology

Prepared for

NATIONAL RESEARCH COUNCIL'S

COMMITTEE ON SHIP STRUCTURAL DESIGN

Advisory to

SHIP STRUCTURE COMMITTEE

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Dear Sir:

The enclosed report entitled "A Critical Survey of Brittle Fracture in Carbon Plate Steel Structures other than Ships" by M. E. Shank, Massachusetts Institute of Technology was prepared for the Committee on Ship Structural Design to facilitate in assessing the present state of knowledge of the structural aspects of brittle fracture. This is one of a group of reports which have materially assisted in determining areas in which research directed toward the elimination of brittle fracture in welded steel merchant vessels may be most successfully undertaken.

Professor Shank's survey is of more than usual interest because it shows that the type of fracture which has been experienced in welded steel merchant ships during the past ten years had, in fact been observed in earlier structures including riveted tanks, bridges, etc. This indicates that the phenomenon of brittle fracture is not of recent origin, even though it is only within the past decade that this type of failure has been satisfactorily classified.

Other reports in this series will be released shortly.

The report is being distributed to those individuals and agencies associated with and interested in the work of the Ship Structure Committee.

Very truly yours,

K. K. Cowart

K. K. COWART
Rear Admiral, U. S. Coast Guard
Chairman, Ship Structure Committee

A Critical Survey of

Brittle Failure in Carbon Plate Steel Structures Other Than Ships

by M. E. Shank
Massachusetts Institute of Technology

Prepared for

National Research Council's
Committee on Ship Structural Design

Advisory to
SHIP STRUCTURE COMMITTEE

under

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ABSTRACT

The failure of ships at sea and at dockside during World War II brought the problem of brittle fracture into sharp focus. Data from ship failures have been well correlated, and as a result, much has been learned from research stimulated thereby. No similar correlation on nonship failure data exists, and this survey was therefore undertaken in order to supplement the study of ship failures. A total of 64 structural failures, plus failures in gas transmission lines, was studied. These failures occurred in both riveted and welded structures such as tanks bridges, pressure vessels, a smoke stack, a penstock, power shovels, as well as gas transmission lines. It is shown that the history of brittle failure extends back at least to 1879. It is concluded that: (1) Brittle failure in nonship structures is the same phenomenon as occurs in ships; (2) brittle failure occurs in many types of nonship structures; (3) brittle fractures can cross riveted joints; (4) there is no evidence to show that the percentage incidence of brittle failure has either decreased or increased with the advent of welding; (5) in conjunction with other factors, thermal stress may be important; (6) residual stresses are not the prime cause of brittle failure, but such stresses may, in conjunction with other factors, initiate such failure; (7) the effect of metallurgical variables is important; (8) cold forming promotes susceptibility to brittle failure, but its role cannot be assessed due to lack of data; (9) in such cases where data are available, Charpy impact values of plate were generally low at the failure temperature; (10) in most cases of nonship brittle failure, the fracture originated at defects arising from fabrication. A few originated at design defects; (11) it seems evident in all cases that fracture originated at a geometric discontinuity; (12) no evidence exists for these failed structures to show the effects of various welding processes on susceptibility to brittle failure; (13) except in the case of exceptionally poor welds, there is no tendency for fracture to follow welded seams; (14) the great majority of nonship brittle failures apparently occur under conditions of entirely static loading; (15) age of structure seems to have no bearing on brittle failure; (16) most engineering codes permit the use of steel which is known to be particularly susceptible to brittle failure. At the same time, under all codes but one, the stress levels are held to quite conservative values; (17) finally, it is demonstrated that brittle failure results from a combination of many factors. There is no readily available material which would entirely prevent its occurrence, and there is no known test which will surely predict from the behavior of small specimens the performance of a given steel in circumstances where structural brittle failure might occur. In short, careful design, selection of materials, and good workmanship are of the greatest importance in the prevention of brittle failure in nonship structures. This is also true of ships.

22 Brittle fracturing, Pressure vessels, Bridges, Welded joints.

Avail. Ship Structure Committee, NAS, etc.

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A Critical Survey of Brittle Failure in Carbon Plate Steel Structures Other Than Ships

by M. E. Shank

INTRODUCTION

ANY critical survey of brittle failures of carbon steel plate, nonship structures must necessarily be oriented by reference to the problem of brittle failures of ships. The ship failure problem was brought into sharp focus during World War II with the breaking up at sea and at dockside of welded merchant vessels, especially Liberty ships and T-2 tankers. Data relating to ship failures have been well correlated and much research on the brittle failure problem has been undertaken. As a result, much light has been shed on problems relating to the brittle failure of steel. No similar central repository of information relating to failures exists in the case of nonship structures. This survey was therefore undertaken to gather and correlate such data, in order to supplement the study of the ship failure problem.

In particular, such a nonship survey reveals how widespread is the brittle failure problem in nonship industries, how long the problem has existed, and to what extent the problem is being met and solved. It is hoped, moreover, that the publication of such a survey will help to categorize and set forth the circumstances in which brittle failures are likely to occur.

It might be well, therefore, to briefly summarize the manifestations of brittle fracture in carbon steel plate. Three conditions can combine to bring about such failures. They are first, low temperature, such as exists in the ambient atmosphere. Second is the presence of a notch (introducing triaxial stress). Any defect, such as a welding crack, or void, or a crack left by a punching or shearing operation, can serve as a notch which will initiate brittle failure. Thus brittle failure is sometimes called "notch brittleness." The third factor is high strain rate or impact loading. This third factor, however, is not necessary for the initiation of brittle failure. As will be later shown, many brittle failures have been initiated under what appear to be completely static conditions.

When brittle failure occurs it may be recognized by several earmarks. Among these are the speed at which fracture occurs (approaching several thousand feet per second), almost complete lack of ductility, negligible energy absorption, and a brittle or faceted appearance of the fractured surface. Moreover, the fractured surface often has a characteristic "chevron" or "herringbone" appearance, the apices of the herringbones pointing to the origin of the fracture. Figures 34 and 35 present an excellent, if extreme example of the physical appearance of such a fracture. Finally when steel plate, taken from a structure which failed in a completely brittle manner, is tested in an ordinary tension test, it manifests a high degree of ductility and strength. As will be seen,[1] it was this last characteristic that was so baffling to the engineers who first encountered the phenomenon.

In 1856, the Bessemer process of steelmaking was announced to the world, and shortly thereafter steel became available in comparatively large quantities. A few years later (1861) the open-hearth process became available. Prior to this time steel was made by the cementation or carburizing of wrought iron (blister bar). It was scarce and expensive, therefore limited to such uses as cutlery and springs. Wrought iron, which because of its slag inclusions is an extremely tough material, was used for structural purposes. By 1860 however, Bessemer steel was available in such quantity that it was used for boiler plate, and in 1863–64 two steel vessels of 377 tons and 1283 tons were built of steel plate. In Great Britain, as late as 1877, Board of Trade Regulations prohibited the use of steel in construction, and removal of these regulations in that year provided a great stimulus to the steel industry in that country. Thus during the period of 1860 to 1890, both in Europe and the United States, wrought iron was gradually being supplanted as a structural metal by steel. A general reluctance on the part of engineers to discard a reliable material like wrought iron caused the change to come about slowly. In the long run, the cheapness, greater availability, and superior strength of

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steel won out. As more steel came into use, troubles with brittle failure began to appear.

In the *Journal of the British Iron and Steel Institute* for 1879 appears a paper, presented at a meeting that year, by Nathaniel Barnaby on "The Use of Steel in Naval Construction." Mr. Barnaby deplores "Recent cases have occurred of fracture in Bessemer bars . . . from some trifling blow or strain . . . they nearly all took place during the late severe weather at Chatham." In the ensuing discussion of this paper Mr. Barnaby was roundly denounced by the assemblage. However, in the same discussion, one Mr. Kirk complains of the cracking of steel in a mysterious manner. In particular, he cites a steel plate that "when cold, on being thrown down, split right up. Pieces cut from each side of the split stood all the Admiralty tests. Now given a material capable of standing without breaking an extension of 20 percent he wanted to know . . . how a plate . . . could split with a very slight extension . . . not to the extent of 1 percent." Mr. Kirk thereupon asked the steelmakers for a remedy to this problem, and if a remedy was not available, at least a rational explanation. His question was totally ignored by the members present. Today the problem is yet with us, and modern engineers and metallurgists are still striving to satisfy Mr. Kirk's request.

Before examining in detail the history of nonship brittle failures, it is well to glance for a moment at the statistics[79] of brittle failures of welded ships over 350 ft long. Very few failures have occurred in smaller vessels. In the period of 1942–52 about 250 welded ships suffered one or more brittle fractures of such severity that the vessels were lost or in a dangerous condition. Nineteen of these 250 ships broke completely in two, or were abandoned after their backs were broken. Eleven of these 19 were tankers, 7 were Liberty ships. In the same 10-yr period, 1200 welded ships suffered brittle cracks generally less than 10 ft in length. These cracks did not endanger the ships, but were potentially dangerous.

Riveted ships, however, are not immune to brittle fracture.[79] Since 1900 over a dozen riveted merchant vessels have broken in two in heavy weather or are listed as missing. It is significant that most of these were of the tanker type, the same category which gives the most trouble in welded structures. Such famous liners as the *Leviathan* and the *Majestic* experienced cracks in their upper strength decks. These cracks usually started in square openings and sometimes extended to the shell. Some breaks even extended down the shell. In at least one case a loud report accompanied the formation of a crack, indicating brittle fracture. The *Europa* had similar cracks. Moreover, frequent mention in the technical literature of cracks in numerous riveted vessels indicates the prevalence of minor failures of this type.

Returning to the subject at hand, the brittle failure of carbon steel plate, nonship structures, it is well to say a word concerning the scope of this survey. The term "carbon steel plate" implies a consideration of plate structures fabricated of plain carbon steel plate. In actuality, one structure made of rolled shapes was considered because of the light it shed upon the failure of a similar structure made of plate. For the same reason, two structures of low-alloy steel were considered. Failures that occurred entirely in welds have not been considered unless brittle fracture of the parent plate ensued as a result.

This report was commenced by a survey of trade publications and technical literature, to secure accounts of failures on record. The number of failures thus revealed was surprisingly small, 39 altogether. Simultaneously, numerous letters were sent to various industries, technical organizations and government agencies asking for data. Data on another 19 failures (plus probable gas line failures) were received from these sources. Thus 58 failures (in addition to failures on gas lines) are here presented. This figure does not count repeated partial failures of single structures. They show that the problem of brittle fracture is present in practically all segments of industry that deal with plate structures. It is a foregone conclusion that many more failures have occurred in the past than are here reported. These, for various reasons, will never come to light. Many failures, when they occurred, were probably attributed to other causes, such as fatigue or, as will later be shown, "bad steel." Moreover, most industries in the past were not anxious to reveal accidents of this type, lest adverse publicity be incurred. As a result, unless personal injury or property damage resulted, failure histories were not revealed and the nature of the fracture was not often understood.

At the present time the situation is somewhat different. Engineers have recognized the progress made in the problem of brittle fracture by the cooperative effort of those industries and agencies working on ship failures. As a result of investigation and research sponsored almost entirely by government agencies, industry in general has a great fund of information on which to draw in preventing brittle failure in service. In consequence, the response to inquiries for this survey was for the most part wholeheartedly cooperative. It is to be regretted, however, that two of the largest industries in the United States have chosen not to contribute information. It might be added that brittle fracture-wise, these particular industries are in more dire straits than any other, and are eager to secure from past ship research all information that might possibly be useful to them.

HISTORIES OF BRITTLE FAILURES—THE ERA OF PREWELDED CONSTRUCTION

Significant brittle failures of steel plate structures in the prewelded period provide a useful background to present day failures. These old failures furnish a perspective and demonstrate conclusively that brittle fracture difficulties did not begin with the advent of welded construction.

1. Water Standpipe, Gravesend, Long Island, N. Y., Oct. 7, 1886[1]

The failure of this riveted water standpipe is apparently the earliest case of brittle fracture of a structure on record. The standpipe was of a very ambitious design for its day, being 250 ft high. It had a diameter of 16 ft to a height of 59 ft, decreasing conically in a length of 25 ft to an 8-ft diam which was retained to the top. The whole was steadied by guy wires. Two plate sizes, 5 by 7 ft and 5 by 9 ft were employed, with thicknesses varying from 1 in. at the bottom to $^1/_4$ in. at the top. All joints were triple riveted. Failure occurred in the hydrostatic acceptance test. Water had been pumped to a height of 227 ft when there was a sharp rendering sound. A vertical crack appeared in the bottom, running up about 20 ft. The whole tower then collapsed. The account[1] states: "Some plates are bent almost double, and others are actually rolled up, showing a very tough metal. . . . The utter destruction of the lower parts of the tower and the appearance of the fallen tower, which is broken in two just above the cone and presents an almost clean square cut just below this cone, can be likened to nothing better in effect than the sudden smashing of the lower part of a high glass cylinder and the vertical drop and then fall of the upper part. The guys on this tower very possibly had some effect in maintaining the structure in a vertical position for a moment of time. . . . In summing up on the general evidence, we should say that the plates were amply thick enough to stand the stress put upon them, even were they a good wrought iron; the workmanship seems to have been generally good, though some of the riveting was not quite up to the standard; the general design was an awkward one and we should not approve of it. But we should say that the main cause of failure lay in the presence of defective steel plates in the lower part of the tower. These plates certainly varied very much in quality, and the wreck shows plates which could not possibly have stood any considerable test for tensile stress. Only a brittle material could have brought the utter destruction there exhibited and it would seem as if this brittle material had unfortunately been concentrated in the portion of the tower exposed to the greatest strain."

The present-day engineer immediately notes that some of the plates were very ductile, others appeared not to be ductile, and that the reporter on the scene believed that many brittle (and thus defective) plates had been concentrated at the bottom of the structure. This fallacy will be seen to be repeated in subsequent early reports of brittle fracture.

2. Gasholder, Brooklyn, N. Y., Dec. 23, 1898[2]

The retaining or sealing tank of this structure failed on its hydrostatic acceptance test. The tank was 178 ft in diameter and 42 ft high, of which height all but 17 ft was underground. The riveted plates varied from $1^1/_4$ in. thick at the bottom to $^7/_{16}$ in. at the top. The design and structure were quite normal for the day. In the resulting failure, fracture went through the body of the plates. There was no tendency to follow the rivet line. To quote an eye witness: ". . .the fracture in some cases taking a curved form similar to that seen in the fracture of a pane of glass. . . . An examination of these fractures shows metal of a rather coarse crystalline structure at the center of the plate, shading off into a very fine grain at the surface, with here and there splinter edges much like a broken case-hardened material."[2] The witness urged a searching investigation into specifications for the plates, their chemical composition, and behavior under test (i.e., tensile test).

3. Water Standpipe, Sanford, Me., 3 A.M., Nov. 17, 1904[3]

This was a riveted steel tank, 40 ft in diameter, 80 ft high. The plate thickness varied from $^5/_8$ in. on the lower course to $^3/_8$ in. at the top. The tank had been standing 7 yr when it broke, and was nearly full at the time. The plates tore through the rivet holes, and it was noted that many small cracks radiated from these holes. The report states: ". . .enough clean fractures were found to indicate that the steel was hard and brittle, showing a crystalline structure. Apparently no rivets were sheared; many plates were torn through the rivet holes. . . . A number of rivet holes were found where there were one or more cracks radiating from the hole. . . . It seems probable that the rupture started in a crack radiating from a rivet hole; and that these radiating cracks may have been caused in the brittle steel . . . due to cutting out the rivet [holes]. It is not evident, however, why failure did not take place immediately upon the initial application of full pressure."[3] One paragraph of the account describes in excellent detail what is now called the "shear lip."

4. Molasses Tank, Boston, Mass., 12 Noon, Jan. 15, 1919[4–9]

The Boston Molasses Tank excited great interest at the time of its collapse, since much damage was done to both persons and property. The tank was erected in 1915–16 on the Boston waterfront, and was used to store molasses. It was 90 ft in diameter, 50 ft high, with lap-jointed plates $^1/_2$ and $^5/_8$ in. in thickness, held by three rows of rivets. At the time of failure it held 2,300,000 gal of molasses, a height of 48 ft 10 in. The failure was a real catastrophe. Twelve persons were drowned in molasses or died of injuries, 40 others were injured and many horses were drowned. Houses were damaged, and a portion of the Boston Elevated Railway structure was knocked over. An extensive lawsuit followed, in which the greatest experts of the day were called to testify. Their testimony sheds a great deal of light, both on the facts of the case as they saw them, and on the general state of knowledge of brittle fracture at that time.

Calculations[5, 6] showed that, at the base of the tank when full, stress in the thicker plates was 26,400 psi, and in the thinner plates 36,000 psi. Thus allowing for a rivet joint efficiency of 66%, stresses in the joint were 40,000 to 50,000 psi.[5, 6] The rivet stresses exceeded by about two times the allowable limits of the building laws.

The witnesses for the defense contended[7] that the tank had been destroyed by a bomb planted by labor agitators. Elaborate tests, with bombs submerged in molasses, were run to demonstrate this. Prof. G. E. Russell of MIT, and others testified that the tank was structurally sound, that it did not rupture at its weakest point, that tests showed the material to have a tensile strength of 55,000–56,000 psi, and that all plate breaks in the tank failure were sharp and not ductile. They conceded that the factor of safety (1.6) was low. Prof. G. F. Swain, of Harvard, testified that the wreckage could not have been propelled to its final location without an explosion. Fatigue was eliminated as a cause. No less a person than Albert Sauveur testified that the Neumann bands (crystallographic twinning) found in the microstructure were usually associated with an explosion of disruptive force.

Witnesses for the plaintiff contended[8] that the tank was weak, particularly in the region of a cleanout manhole. Several breaks had occurred around the manhole. G. G. Lutts of the Boston Navy Yard metallurgy laboratory produced notch-plate fractures, obtained in laboratory tests, showing short, sharp, herringboned fractures, similar to those found on the tank. Mr. Lutts and Prof. R. S. Williams, of MIT testified that Neumann bands would appear in the tank fractures due solely to the action of molasses.* Others testified that the tank design was unsound, that punching of rivet holes had started short cracks and that the tank was stressed beyond the elastic limit in many places.

Finally in 1925, after years of testimony, the court-appointed auditor, Col. H. W. Ogden, handed down a decision[9] that the tank failed by overstress, not by explosion. The auditor's summary is worth reproducing here, since it fairly well summarizes the knowledge (or lack of it), then current among practicing engineers, concerning notch brittle behavior. ". . .The defendant's experts called attention to the presence of Neumann bands in steel of the character herein considered which had been fractured was a proof that the steel in question had been very suddenly fractured and that such bands would not appear if such fracture had been caused alone by static pressure produced by the load of molasses. . . . In the present state of science, however, I find that the conflicting authority in regard to where they occur and why they occur is too fundamental to give their presence any weight in marshalling the proofs in this case. . . . Weeks and months were devoted to evidence of stress and strain, of the strength of materials, of the force of high explosives, of the bursting power of gas and of similar technical problems. . . . I have listened to a demonstration that piece A could have been carried into the playground only by the force of a high explosive. I have thereafter heard an equally forcible demonstration that the same results could be and in this case were produced by the pressure caused by the weight of the molasses alone. I have heard that the presence of Neumann bands in the steel herein considered along the line of fracture proved an explosion. I have heard that Neumann bands proved nothing. I have listened to men upon the faith of whose judgment any capitalist might well rely in the expenditure of millions in structural steel, swear that the secondary stresses in a structure of this kind were negligible and I have heard from equally authoritative sources that these same secondary stresses were undoubtedly the cause of the accident. Amid this swirl of polemical scientific waters it is not strange that the auditor has at times felt that the only rock to which he could safely cling was the obvious fact that at least one-half of the scientists must be wrong. By degrees, however, what seems to be the points in the case have emerged. . . ."

5. Crude Oil Storage Tank, Ponca City, Okla., 6 A.M., Dec. 19, 1925[10]

This tank was 117 ft in diameter, 41 ft 10 in. high, filled with crude oil. The shell was riveted, consisting of seven courses of plates varying in thickness from 1 in. at the bottom to $^1/_4$ in. at the top. The roof was held on framing. The bottom course was welded to an angle iron base ring.

The temperature had been 60° F the day before the failure, and had suddenly dropped to $-4°$ F. At 6 A.M. one or two light, muffled sounds were heard, and fire broke out. Later examination showed that the second course had been torn from the first. The sheets from the second course to the roof were torn along an irregular line.

The investigators eliminated explosion or lightning as a cause, and decided that no defective welding or riveting was involved. The oil company personnel rightly surmised that the sharp temperature drop was responsible. There was, however, no thought given to brittle fracture as such.

6. Eight Crude Oil Storage Tanks, South and Middlewest, U. S., Early 1930's[11]

The data for these failures were recently gathered

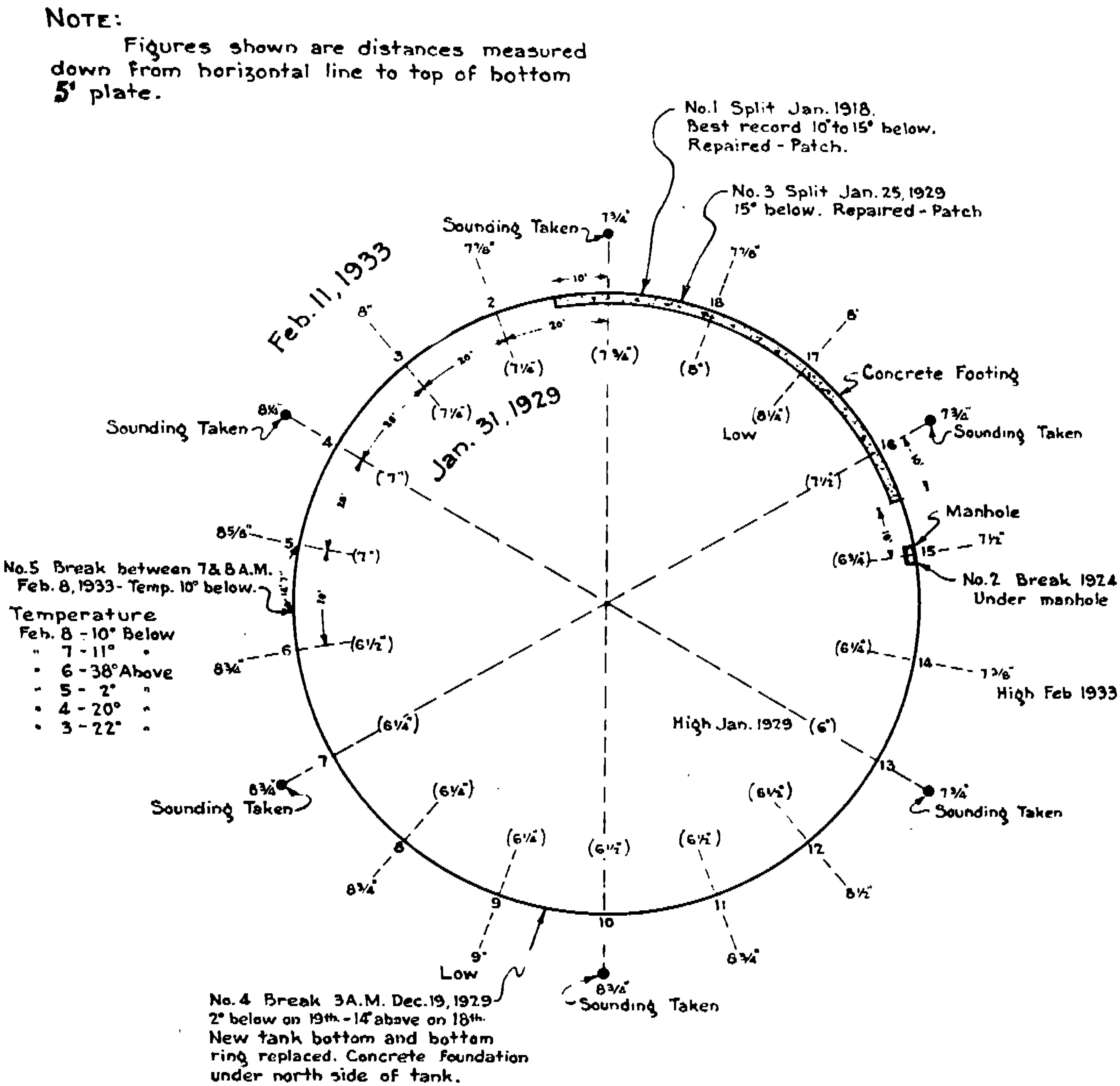
* This author recently talked with Mr. Lutts and Prof. Williams concerning the Boston molasses tank failure. Mr. Lutts recollected that Watertown Arsenal was, during this period, engaged in an impact testing program, and that it was his opinion that engineers of this period were acquainted with notch brittleness to some degree at least. The chevron or herringbone markings found on the tank plates were, in his opinion, an entirely new thing. He recalled that in the fractured plates above the tank manhole, the herringbones pointed down, and in the fractured plates below the manhole, the herringbones pointed up. The significance of these markings was not clear until Mr. Lutts duplicated them in the laboratory. He did this on plate from the tank, by drilling a hole in the center, and cutting a horizontal slot from either side of this hole into the plate. When pulled in the testing machine chevron markings, pointing toward the hole, appeared in the fracture. He also broke some of the tank plates in an impact testing machine, producing Neumann bands in the microstructure.

Prof. Williams recollected that notch brittleness, as we know it today, was not generally understood by engineers and metallurgists in that period.

Mr. Lutts' independent discovery of the meaning of chevron markings is of the earliest on record, and apparently the first understanding of the phenomenon in actual service. In 1914, however, Ch. de Fremenville,[38] in laboratory testing of numerous materials noted chevron markings, and the fact that the apices pointed to the fracture origin.

from old industrial records. As a consequence, it is not complete in all details, but is nevertheless very valuable. A total of eight tanks, with failures of varying severity, were involved.

Tank No. 1 was riveted, of 55,000 bbl capacity. The dimensions are not available. It is believed that it was a secondhand tank when it was erected in 1917. It had a history of five failures (see Fig. 1) as follows:



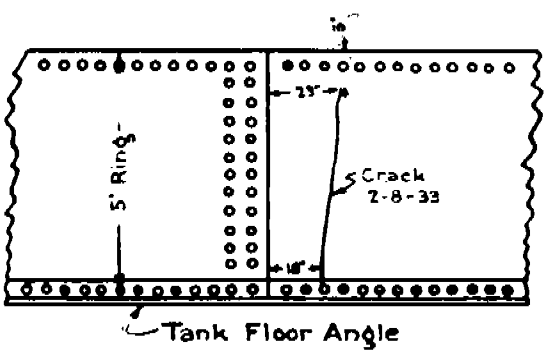


Fig. 1 Failures in riveted crude oil storage tank No. 1

1. January 1918: A split occurred in the lower ring of the tank with the result that leaking oil overflowed the fire walls. The temperature at the time of the break was below zero. The tank was repaired with a patch and replaced in service.

2. November 1924: This failure was a crack in the lower plate just under the manhole entrance. Very little oil was lost. There is no record of the temperature when the break occurred. The cracked plate was patched with a 10- by 20-in. plate at the time, and in 1926 the cracked plate was replaced with a new one. Also in 1926, a new roof and a new bottom were installed, the new bottom being laid on top of the old one.

3. Jan. 25, 1929: The failure evidently was a cracked plate in the lower tank course. The records indicate the temperature was 15° below zero at the time. Repair was made by patching.

4. Dec. 19, 1929: This was a vertical split in the lower ring. The temperature at the time was $-2°$ F. In repairing the tank the entire lower course was replaced with new steel, and a concrete base possibly 18 in. wide and 3 ft deep was run under portions of the tank perimeter.

5. Feb. 8, 1933: This was a vertical split in the lower course. At the time of failure there was a 14-ft oil level in the tank and the temperature was $-10°$ F. The crack was so big that barrel staves were driven into it to reduce the flow of crude oil. Subsequent inspection of the interior revealed that the new bottom sheets had been carried over the bottom leg of the angle and welded to the fillet of the angle, after which an apron covering the angle was welded to the bottom and side plates. When the apron was removed it was found that the bottom angle, 3 by 3 by $^5/_8$ in. in size, had been patched by welding in 12 different places and 22 serious cracks in the vertical leg still existed.

Tank No. 2, riveted, was also of 55,000 bbl capacity, and had been erected in 1917. It failed at 9:50 P.M., Feb. 7, 1933, at a temperature of $-4°$ F. A vertical split occurred extending from the caulking edge at the top of the bottom sheet to the bottom of this sheet. At both the top and the bottom the break ran between rivet holes. The break did not extend into the second sheet. Some 3 years prior to the failure a new bottom angle was installed just below the rivets joining the angle iron to the tank shell. At this time also the sections of bottom angle iron were all butt welded, and a second set of angle shoes was installed over the old shoes on top of the new bottom. (See Fig. 2). There was no concrete ring foundation. When the tank failed, the vertical legs in both of these shoes split directly in line with the split in the bottom sheet. The butt weld in the bottom angle iron was broken at this point, allowing the two sections of angle iron to spread apart. The tank bottom, where welded to the angle iron, was pulled loose for a distance of about 6 in. on both sides of the split. It was thought probable, though by no means certain, that the butt weld in the angle iron was the first to fail and thereby delivered to the shell the impact which split it.

Tanks Nos. 3 and 4. These were erected in 1923, and were both 171 ft in diameter and 42 ft high. The bottoms were $^1/_4$-in. plate, the lower course plate was $^5/_8$ in. thick. There were 7 courses of plates, with 20 sheets per course. The vertical joints were quadruple riveted butt joints, with $^1/_2$-in. thick butt straps inside to include all four rows of rivets. The outside butt straps included only two rows of rivets, one row on either side of the joint. The rivet holes were $^{15}/_{16}$ in. diam for $^7/_8$-in. rivets, and were believed to have been punched full size. Bottom angles were 4 x 4 x $^3/_4$ in. Lighter steel and smaller rivets were used in successively higher courses of plates. Horizontal joints were made with $^7/_8$-in. rivets between the first and second courses, with smaller rivets in higher joints. Tank No. 3 failed Dec. 7, 1932, when the temperature dropped to about $-18°$ F. It was filled to the top with crude oil. Presumably the failure originated in the bottom ring, and extended vertically through two courses of solid plate to the horizontal joint between the second and third courses. At this point the vertical crack presumably stopped momentarily and the cracked sheets began to lean outward, putting a horizontal bulge in the tank about midway between top and bottom. This action was probably accentuated by the collapse of the roof due to the vacuum produced by oil escaping

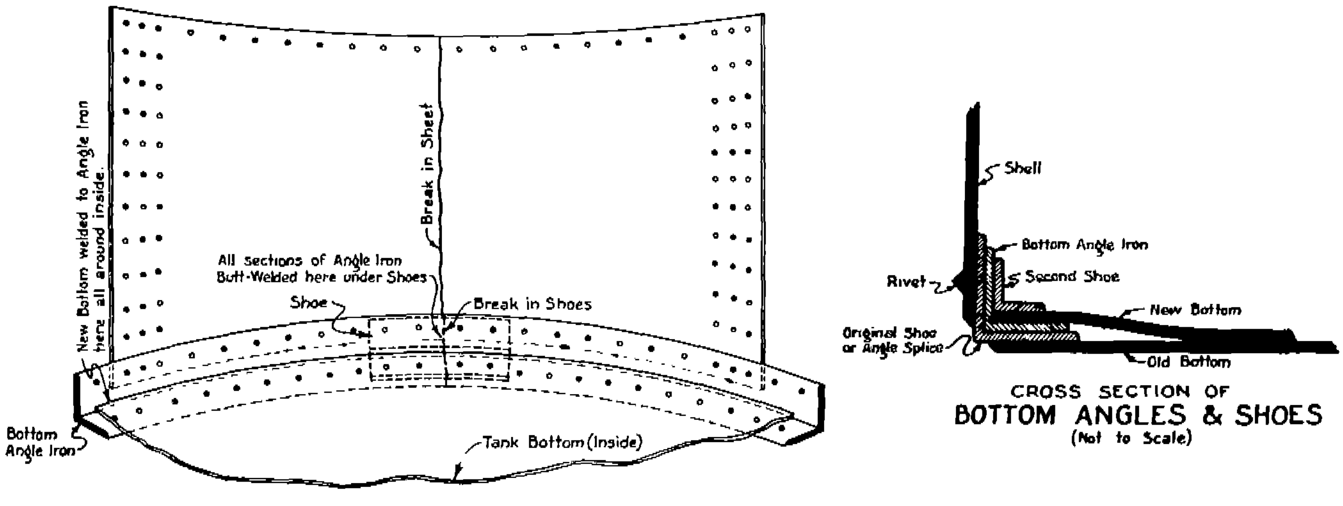


Fig. 2. Section through double bottom, and detail of failure in 55,000-bbl riveted tank No. 2

through the original crack. Evidently the shell then failed in a horizontal direction along the line of the bulge and at the same time cracked vertically until there was a continuous tear from top to bottom. Then the ends adjacent to the tear began to swing outward, the tendency being to straighten out the shell. Pieces were torn off from each end and carried away by the flow of oil. The reaction of the oil approximately opposite the point of original failure pushed over the adjacent shell and laid it on the ground with the inside uppermost. The roof remained attached to this portion of the shell and moved with it. The result was complete failure of the tank.

Tank No. 4 failed during the night of Feb. 8, 1933. It was filled nearly to the top, and the temperature had fallen to below $-30°$ F. It failed in a manner similar to Tank No. 3, except that it broke into a larger number of pieces.

Chemical analyses were made of a piece of each of tanks 3 and 4, giving the following results in percent:

	Tank No. 3	*Tank No. 4*
C	0.29	0.17
Mn	0.42	0.51
Si	0.002	0.006
P	0.013	0.016
S	0.034	0.030
Cr	0.015	None
Ni	None	None

It will be noted that the steel from Tank No. 3 had a comparatively higher carbon content.

The steel had tensile properties usual for such material. In a Charpy test of steel from Tank No. 3, values were 3 to 11 ft-lb at 25° F, 5 ft-lb at 0° F and 1 to 2 ft-lb at -25 and $-50°$ F. Similar values were obtained with steel from Tank No. 4 except that at 25° F the energy was from 21 to 22 ft-lb.

Tank No. 5 was riveted, 120 ft in diameter, 40 ft high. It had been erected in 1922 or 1923, on a concrete ring with cone heads of the rivets attaching the bottom to the bottom angle resting on this ring. The tank had been patched twice where cracks had occurred. At 2:00 P.M. Jan. 9, 1937, leakage was reported through a split in the sheet in the first course. The tank was filled to a height of 20 ft. The weather had been mild to the end of December. On January 5th more severe weather arrived, with temperatures ranging from $-22°$ F minimum on that night to $-11°$ F on January 8th. It is of interest to note that the split either was not detected, or did not occur, until after the coldest weather had passed. Examination of the split, which occurred in the 1st course, showed it to extend vertically across the entire sheet. It occurred about 7 in. from a welded patch. The bottom angle was not cracked, and in this respect it differed from all the other failures.

Tank No. 6 was also 120 ft in diameter by 40 ft high, riveted. It failed partially during the winter of 1933–34. A crack extended from the bottom edge of the bottom sheet, through a rivet hole, and well into the next sheet. The crack was about 7 in. from a weld patch. There was a head of 36 ft of crude oil at the time.

Tank No. 7 was riveted, 117 ft in diameter by 42 ft high. In 1933 inspection showed a crack in the bottom angle iron. The concrete ring was intact. Inspection in 1934 showed cracks in 3 lower course sheets, and cracks in the angle adjacent to two of these sheets. In January 1935, a failure occurred in one of these previously patched sheets, about 6 ft from the patch. The oil level was 32 ft.

Tank No. 8 was riveted, 120 ft in diameter by 40 ft high. Some time during the night of Feb. 25–26, 1934, complete failure occurred. Failure followed a sudden temperature drop from $+5$ to $-20°$ F in 24 hr. The oil level was 37 ft.

The investigators concluded for these 8 tanks, in part, as follows:

1. All tanks were of riveted construction. Presumably in all cases the rivet holes had been punched rather than drilled or subpunched and reamed.

2. Cone head rivets were used throughout. The heads of the rivets joining the tank bottoms to the bottom angle irons rested on the tank foundations. In a majority of cases concrete ring foundations had been used which would afford considerably more resistance to radial movements of the bottom rivet heads than would earthen foundations, particularly since it was found that the weight of a tank and contents was sufficient to force the rivet heads partially into the concrete.

3. All failures here considered occurred when atmospheric temperatures were of the order of zero or below.

4. All failures occurred in tanks in crude oil storage service. In cold weather, crude oil is known to partially solidify against the inside walls and bottom, thus insulating the tank walls from the warmer oil at the center and enabling the tank shell to approach, probably fairly closely, the atmospheric temperature. The tank bottom, however, being protected on top and in contact with relatively warm ground underneath, is usually much warmer. In suddenly cold weather, therefore, a temperature difference between the center and periphery of a tank bottom could readily be of the order of 50° F.

5. All fractures examined and reported were of a crystalline appearance with little or no evidence of necking or elongation. Such fractures are characteristic of brittle failures due to impact or sudden stress increases.

This author would not agree with the implications of conclusion number 5. There is evidence of many brittle failures having occurred in ship and nonship structures apparently under static conditions.

From the above eight failures one conclusion was very apparent to the chief engineer responsible for the above tanks, namely, "...that, because of the resistance which the tank bottom or foundation ring may offer to contraction of the shell when the temperature drops appreciably, the joint between the bottom and shell of a tank is one of the most critical (if not the most critical) of all the joints in the tank. For this reason, it would seem that the utmost care should be used in the fabrication and inspection of this joint to make sure that it is as sound and free of defects and other stress raisers as possible."

7. *Oil Storage Tank, Middle West, U. S., 6 P.M., Dec. 14, 1943*[136]

This oil tank was of riveted construction, 114 ft in diameter, and 30 ft high. The roof was of wood, and the bottom of steel. The bottom course of plates was $^1/_2$ in. in thickness, with a single row of rivets in the horizontal seam. The vertical seams in the bottom course were quadruple riveted lap joints. Higher courses of plates were successively thinner, the top (fifth) course being $^1/_4$ in. thick.

The bottom of the tank had been leaking. To repair it, a large triangular hole was cut in the bottom course of plates with a torch so that a wheelbarrow could be wheeled in. After the bottom was repaired, the piece of steel which had been cut out was electrically welded back into place. It was welded from the outside only. The work was done in May 1943.

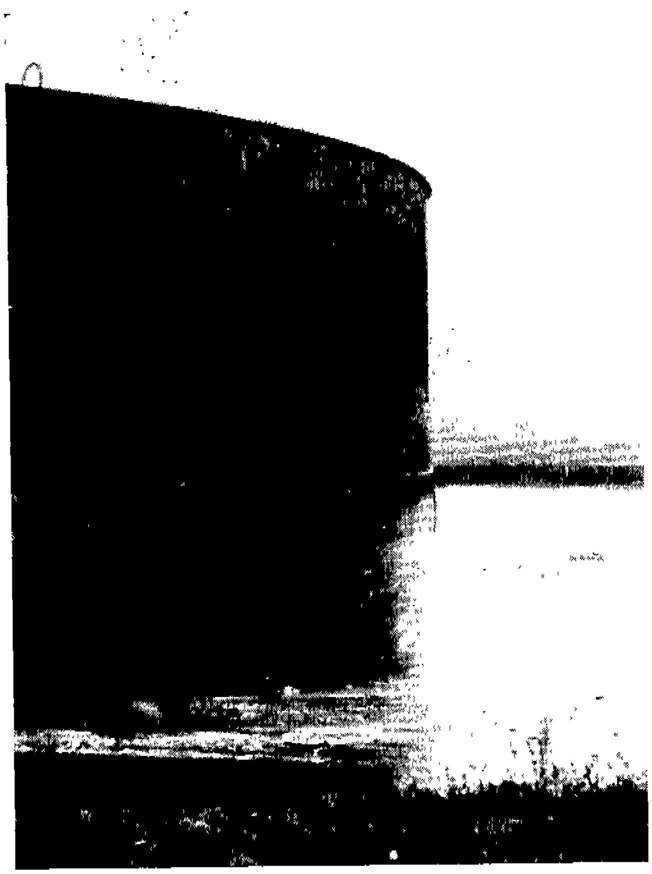


Fig. 3 Oil storage tank with failure emanating from triangular patch plate

The tank is surrounded by escaping oil. The bulge of the broken plate shows in the first course, at the extreme right.

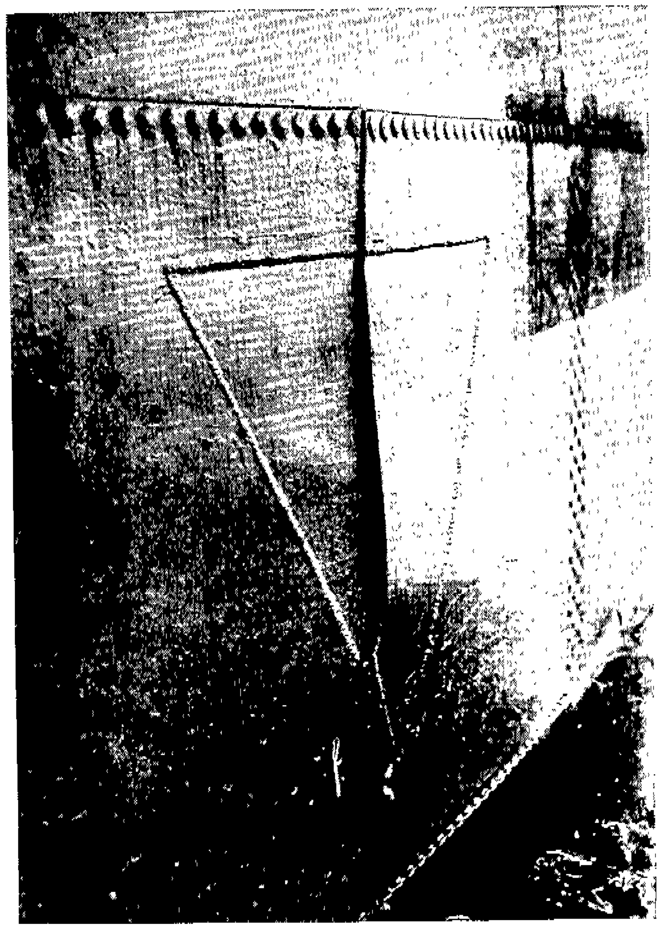
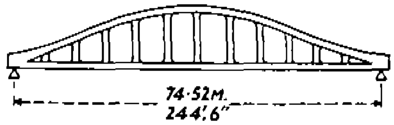
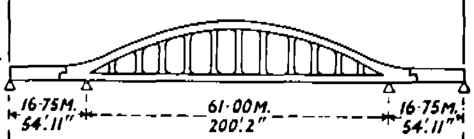
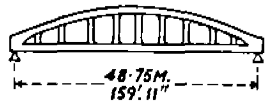


Fig. 4 Triangular patch in oil storage tank

The triangle is about 55 in. on a side. The crack apparently originated in the weld, propagating up and down through the solid metal of the patch and parent plate. Escaping oil obscures the bottom portion of the crack extending downward from the triangle.

The tank was practically full of oil when it burst. The atmospheric temperature was about 12° F, and was rapidly becoming colder. Figure 3 shows a general view of the tank, surrounded by escaping oil. The bulge of the broken plate clearly shows in the first course. Figure 4 shows the patch, and the nature of the break. A triangular section, about 55 in. on a side, had been cut out. The top of the triangle was about 9 in. from the top of the first course, and the bottom apex of the triangle was about 8 in. from the tank base. The rupture shown runs through the entire bottom course plate. Escaping oil obscures the bottom portion of the crack extending downward from the triangle.

Subsequent examination revealed that parts of the weld were poor in quality. Further, the welding of a patch into a solid plate is known to result in a high degree of constraint, with attendant high residual stress. The notch effect caused by poor welding, combined with the low ambient temperature, was sufficient to initiate a brittle failure in the weld (Fig. 4) which then propagated up and down through the solid metal of the patch and parent metal.

№	PLACE	TYPE OF BRIDGE	CLASS	CENTRE SPAN METRES / FEET	WIDTH METRES / FEET	BOTTOM CHORD	STEEL TONNAGE t	DATE OF ERECTION	DATE OF ACCIDENT	REMARK
1	HASSELT	 74·52M. 244'.6"	LIGHT RAILWAY AND ROAD	74·52 / 244'.6"	14·34 / 47'.0"	WELDED	646	1935/36	MARCH 14, 1938	2 COMPLETE EXPANSION JOINTS IN BRIDGE DECK
2	HERENTHALS-OOLEN	 16·75M. 54'.11" — 61·00M. 200'.2" — 16·75M. 54'.11"	"	61·00 / 200'.2"	9·50 / 31'.2"	WELDED	395	1936/37	JAN. 19, 1940	"
3	KAULILLE	 48·75M. 159'.11"	ROAD	48·75 / 159'.11"	9·00 / 29'.6"	GREY BEAM ROLLED	180	1934/35	JAN. 25, 1940	"

(*Railway Gazette* 72, 24 June 14, 1940)

Fig. 5 Details of three failed Vierendeel truss bridges

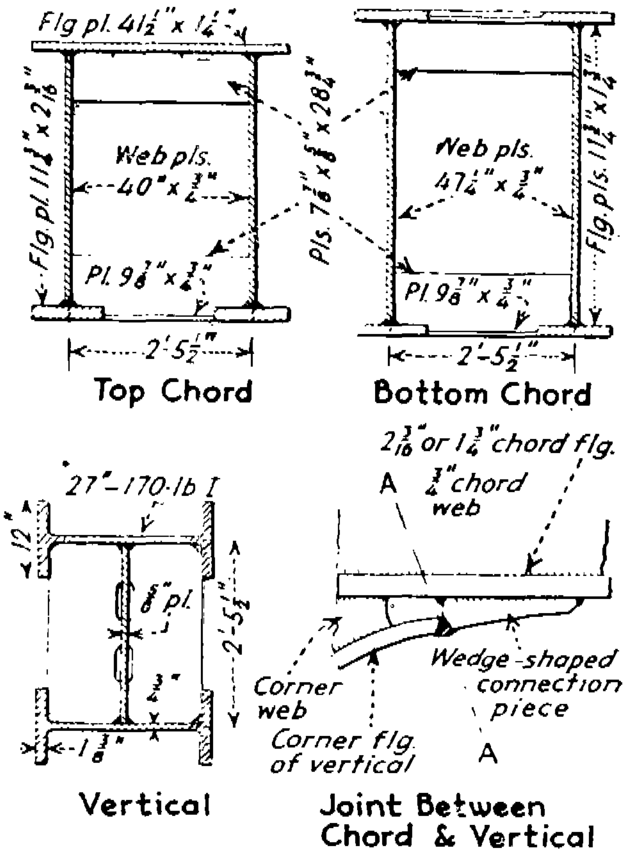
HISTORIES OF BRITTLE FAILURES—THE ERA OF WELDED CONSTRUCTION

8. Vierendeel Truss Bridge—Albert Canal, Hasselt, Belgium, 8 A.M., Mar. 14, 1938[12–44]

This bridge was of a type known as a Vierendeel truss, after its designer. It consisted of straight lower chords, with curved upper chords. The upper and lower chords were joined by verticals. (See Fig. 5.) There were no diagonals. The structure was a very rigid one. Approximately 50 such bridges were built across the Albert Canal, with variations in length and detail to suit the application. Some were built of welded or rolled I-beams and plate, others entirely of plate. The Hasselt Bridge[27, 28, 42] had a span of 245 ft and was made almost entirely of welded plate. (See Fig. 6). The lower chord was of a double I-beam (or box) cross section, with a depth (web) of $47\frac{1}{4}$ in., and a web thickness of $\frac{3}{4}$ in. The flanges were $1\frac{1}{4}$ in. thick. The top chord was also a double I-beam with a depth of 40 in. Again the web was of $\frac{3}{4}$-in. plate, but the lower flanges were $2\frac{3}{16}$ in. thick. The verticals were again welded I-sections of lighter construction. The only parts of the structural portion not made of plate were the gussets, joining chords and verticals. They were castings. The steel was a Belgian St-42, with a tensile strength of 53,000-63,000 psi. The bridge had been in service about one year.

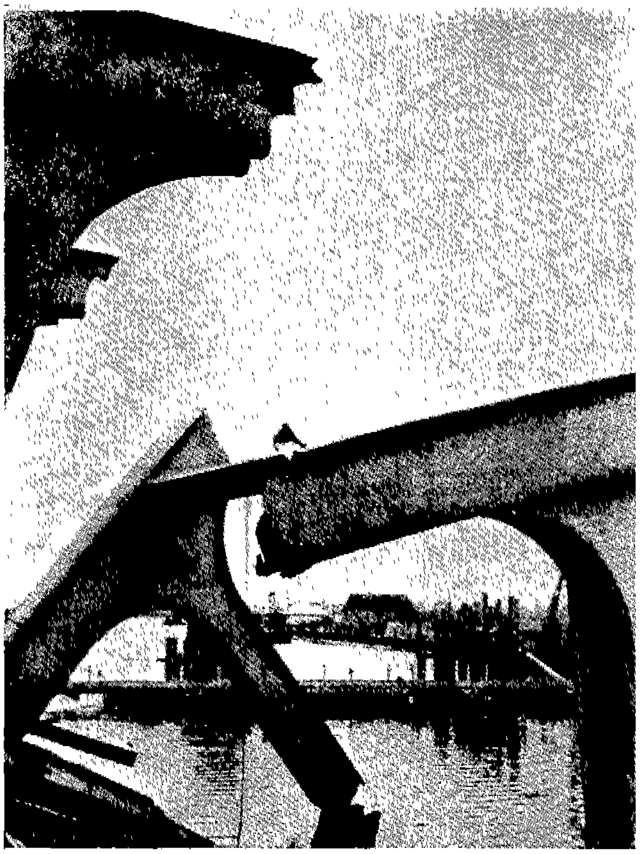
The weather was quite cold when failure occurred. Eye witnesses heard a sound like a shot and saw a crack open in the lower chord between the 3rd and 4th verticals. This left the top chord acting as an arch. Six minutes later the bridge broke into three pieces and fell into the canal. All the fractures were brittle, some through welds, others in the solid plate away from the welds.[24, 27, 28, 35] (See Figs. 7, 8 and 9.) The bridge was lightly loaded at the time.

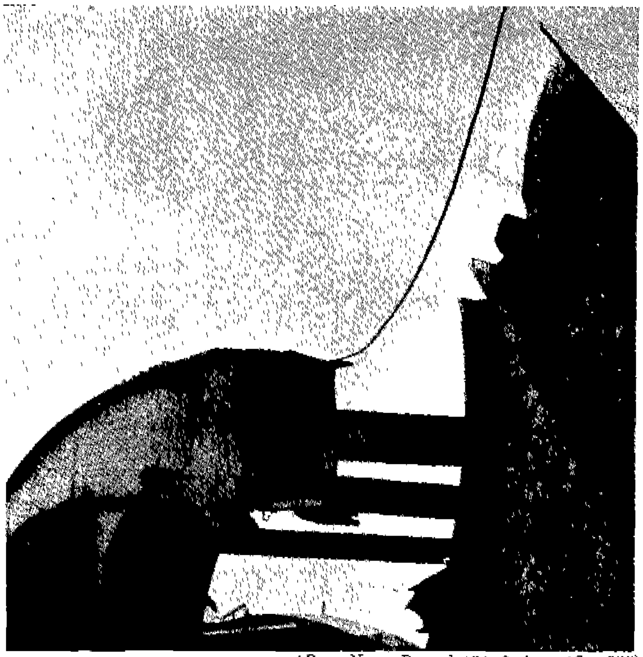
The failure of the bridge set off a great flurry in engineering circles, particularly in Europe. Numerous descriptions[12–23] (mostly fragmentary) of the failure can be found in technical and trade journals. Much speculation as to the cause of the failure took place. The British welding industry, then undergoing a period of growth while struggling for complete acceptance of welding as a substitute for riveting, seemed to be particularly alarmed.[14, 15] One team of British engineers



(*Eng. News-Record* 121, 7 Aug. 18, 1938)

Fig. 6 Structural details of Hasselt bridge members. Numerous fractures occurred along line AA[27]





(*Eng. News-Record 121, 7 Aug. 18, 1938*)

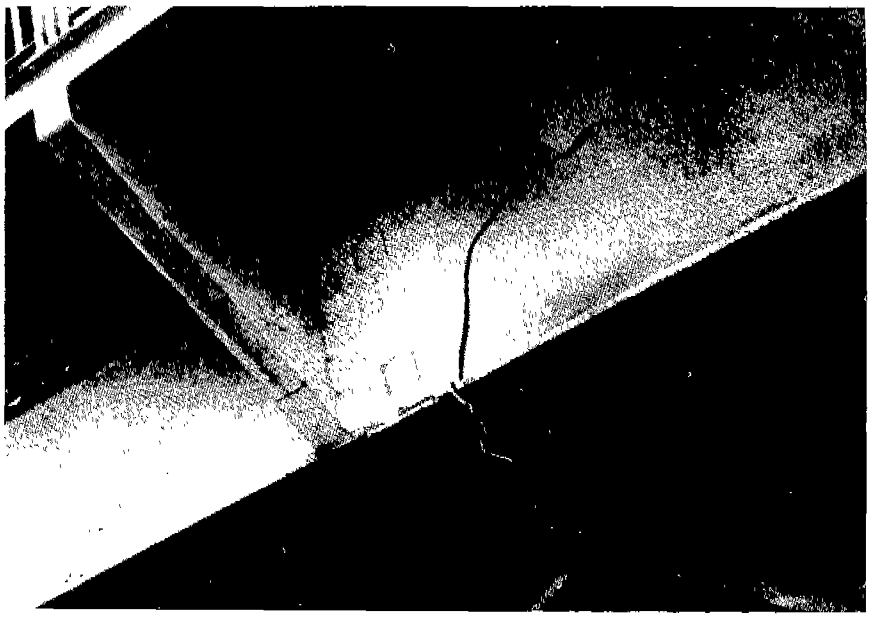
Figs 7 (left) and 8 Top chord breaks in Hasselt Bridge. Extreme brittle behavior of the steel is evident. Breaks occurred at or near junctions of verticals with chord flanges

visiting the site in April 1938, ". . . . .was satisfied that the failure was not due to the weakness or imperfection of the welded joints,"[15] a premature judgment, as it turned out.

Many investigations were started to determine the cause of the failure. An official commission of inquiry was set up, but its report cannot be found in the literature. It must be assumed that the en uance of World War II interrupted its deliberations. Before discussing these investigations, however, it is well to consider the failure of two other Vierendeel trusses.

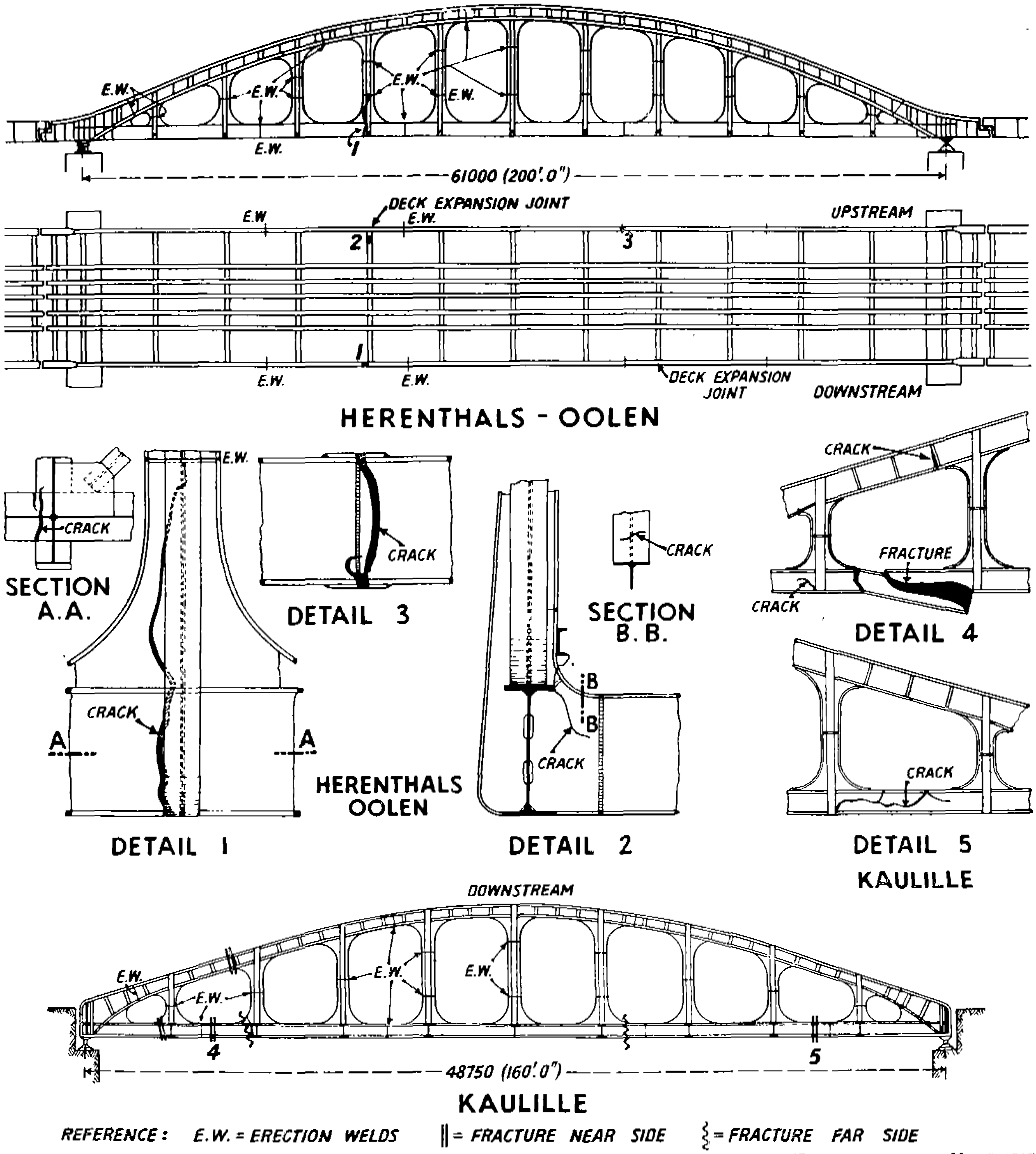
9. Vierendeel Truss Bridge—Albert Canal, Herenthals-Oolen, Belgium, 2:30 A.M., Jan. 19, 1940*[40, 42–44]

The Herenthals-Oolen span was 200 ft in length, with chords of single, welded plate I-beams.[42] (See Fig. 5.) In all other principal respects it was similar to the Hasselt Bridge. Details of plate thickness etc., are not readily available, nor are they really germane. It had been erected in 1936–37. The sentry on duty at the time of failure heard three long reports in succession. The bridge did not collapse. Five hours later, at 7:30 A.M. a 23-ton locomotive passed over the bridge without incident. Afterward cracks were found in the lower chord, one open to 1 in., and 7 ft long.[40] Temperature was 7° F.[43] It will be noted that failure occurred when the bridge was unloaded. All cracks started at weld junctions (Fig. 10).



(*Eng. News-Record, 121, 7 Aug. 18, 1938*)

Fig. 9. Bottom chord fracture in Hasselt Bridge



(*Railway Gazette* 72, 10 Mar. 8, 1940)

Fig. 10 Details of failures in two welded Vierendeel truss bridges

10. *Vierendeel Truss Bridge, Albert Canal, Kaulille, Belgium, 7:30 A.M., Jan. 25, 1940*[10, 42–44]

This bridge had a 160-ft span and was erected in 1934–35. It was constructed of rolled I-beam chords, with welded details. (See Fig. 5.) It is thus not a plate structure, but is included in this survey because of the light it sheds on the failure of other Vierendeel trusses.[40, 42] At the time of failure, the temperature was 7° F. A total of six cracks were found in the lower chord. (See Fig. 10.) The bridge did not collapse.

Numerous investigators agree that the original fracture of the Hasselt Bridge started at a weld between a gusset and the lower chord.[29, 31–33] *L'Ossature Metallique*[29] of February 1939 blamed the failure on residual stress. It stated: (1) That the welds were of good quality "but reveal certain imperfections." (2) The steel is above criticism, its excellent quality being revealed by micrographic examination, chemical analysis, strength tests, impact tests, repeated impact, bending, elasticity and fatigue tests. (3) "It is therefore, incorrect to state as has been done in some quarters, that the brittleness of the steel was either a principal or contributory cause to the accident." (4) No significance is to be attached to the brittle appearance of the breaks since "breaks due to shock always have this appearance." (5) The accident is entirely due to the quality of welds and welding sequence. The reader should note that *L'Ossature Metallique* is a publication edited by the Belgium-Luxembourg Steel Information Center, and thus represents the steel manufacturers' point of view.

The Vierendeel trusses were several times criticized for the welding sequences employed.[17, 24, 27] In the erection of one Vierendeel truss the end lifted 3 cm while still supported on false work. The welders corrected the alignment as they worked.[17, 24] It was also reported that "numerous, sudden fractures, accompanied by detonations have occurred in Vierendeel welded bridges; these sudden cracks manifest themselves for the most part at the works, although some occur during erection, indifferently at the weld, the scarf of the weld, or in solid plate, away from the weld."[24]

Finally a detailed investigation[38] of some length was undertaken in Great Britain, on some steel and welds taken from the Hasselt Bridge. It was found that the steel had a normal chemical analysis, except that the sulfur and phosphorus were high. The steels were of bessemer or fully rimming quality. The mechanical tests of the steel were found to be satisfactory, except that the Izod impact values were low, especially on thick plate. Weld metal showed a high phosphorus content, there were cracks in the roots of important butt welds, and sealing runs were absent on the backs of such welds. The report concluded that the steel, while in some respects unsatisfactory, could not be entirely blamed, that the gravest factor was the welding defects uncovered, and that residual stresses present would have been of no importance had the welds been sound in the first place. The report stated that the exact practical significance of low Izod value is difficult to state, and "In particular there is no definite evidence that such low Izods can be the direct cause of the type of crack known to have developed in this or in similar welded bridges."

Another investigator[39] gave detailed radiographic evidence showing the poor quality of the welds in the Hasselt Bridge. A residual stress of apparently 14000 psi was found in one welded joint. Another source[42] quotes the residual stress as 6.35 to 12.6 tons per square inch (12,000–25,000 psi approximately).

Finally, Busch and Reuleke[43] report a comprehensive investigation undertaken in Germany of the failures of all three of the above-mentioned Vierendeel trusses. They found for the Hasselt Bridge that: (1) most failures occurred at junctions between verticals and the lower chord in butt welds connecting the flanges of both members; (2) on similar joints which were intact, measurement showed the residual stress to approach the yield point;* (3) the design of the bridge caused a high stress concentration at the welds, which was worsened by improper welding sequence; (4) the fractures revealed many fine cracks; (5) welds were defective and contained incipient cracks; (6) the mechanical characteristics of the base metal were satisfactory and complied with specifications.

In their investigation of the Herenthals-Oolen steel, Busch and Reuleke reported: (1) Chemical analysis, percent, as follows:

C	0.09–0.17	P	0.038–0.079
Si	trace	S	0.027–0.038
Mn	0.43–0.94	N	0.011–0.030

The spread indicates the variation from plate to plate. (2) Tensile tests gave:

	Yield point, psi	*Tensile strength, psi*	*Elongation, %*	*Reduction of area, %*
Thickest (1.8-in.) plate	30,000	57,000	36.5	60
Thinnest (0.6-in.) plate	35,000	61,000	35.4	57.5

(3) Impact tests with keyhole notch specimens (from various plates) in the rolling direction gave:

Upper transition temperatures from −40° F and 138 ft-lb to +68° F and 80 ft-lb

Lower transition temperatures from −60° F and 10 ft-lb to −40° F and 10 ft-lb

Specimens from the thicker plate had higher transition temperatures. There was little correlation between the carbon content of the various pieces and the transition range. Practically all specimens were brittle (at least in part) at 7° F, the temperature at which the bridge failed. (4) The steel was not susceptible to cracking during welding and showed no marked increase in hardness due to welding. (5) Small angles of bend were obtained in longitudinal weld bead specimens from the bridge, and fracture was always of the cleavage type. Stress relief gave a greater bend angle, but did not alter the cleavage fractures. (6) The micro- and macrostructures of the steel were satisfactory.

In regard to all three Vierendeel failures, Busch and Reuleke concluded (in part): (1) It should be seriously questioned if nonkilled bessemer steel should be used for welding in the thicknesses of the order of $1^1/_2$ in., in spite of good static tension properties, since the notch impact properties were unsatisfactory. (2) The faulty design of the vertical member-lower girder joint, along with defective welding sequence, was not to blame for the Hasselt failure, since the other two bridges failed elsewhere. (3) The weldability of the steel was not a decisive factor. (4) The accident was caused by (*a*) multiaxial restraint and residual stress, (*b*) low ambient temperature, (*c*) the low notch-impact characteristics of the steel.

As late as 1948, however, the ghost of the Hasselt

* There is some difficulty of translation here.

Bridge had not been laid. Yet another author[44] expressed his opinion as to the cause of the failure.

This author agrees with the implications, if not the formal statement, of the Busch-Reuleke conclusions. The failure was undoubtedly initiated, in the case of some cracks, by weld defects acting as notches. Perhaps the truss structure, which is usually rigid, was contributory. Cracks through the rolled I-beam lower chord of the Kaulille bridge, not near any welds,[40] may substantiate this view. The low ambient temperatures in the presence of the dead-load stresses and residual stresses, combined with the poor notch-brittle characteristics of the steel, did the rest.

11. Fourteen Cases of Brittle Failure in Bridges, Belgium, Presumably 1941–50[45]

Fourteen cases of brittle failure in bridges are here reported in an investigation performed under the auspices of La Commission des Ruptures Fragiles de L'Institute Belge de la Soudure. No locations are given, and no descriptions of the over-all bridge structures or their ages. Some of the bridge sections described were rolled, some were built up from welded plate. In some cases the steel analysis is given. All the structures were of rimming steel, with a probable carbon content of about 0.20%.

The first case presented is of especial interest, since it involves cracks radiating from punched rivet holes in a partly welded structure. Complete failure occurred at 0° C by a crack which seems to have started in one of these holes and progressed across the entire section. Failure was instantaneous, without deformation.

In the other cases, initiation of failure is attributed to residual stress, triaxial stress, bad welds (notch effects) and in one case, poor steel. Low temperature is mentioned as a cause in six cases, and eliminated as a cause in three. The other case descriptions omit mention of temperature.

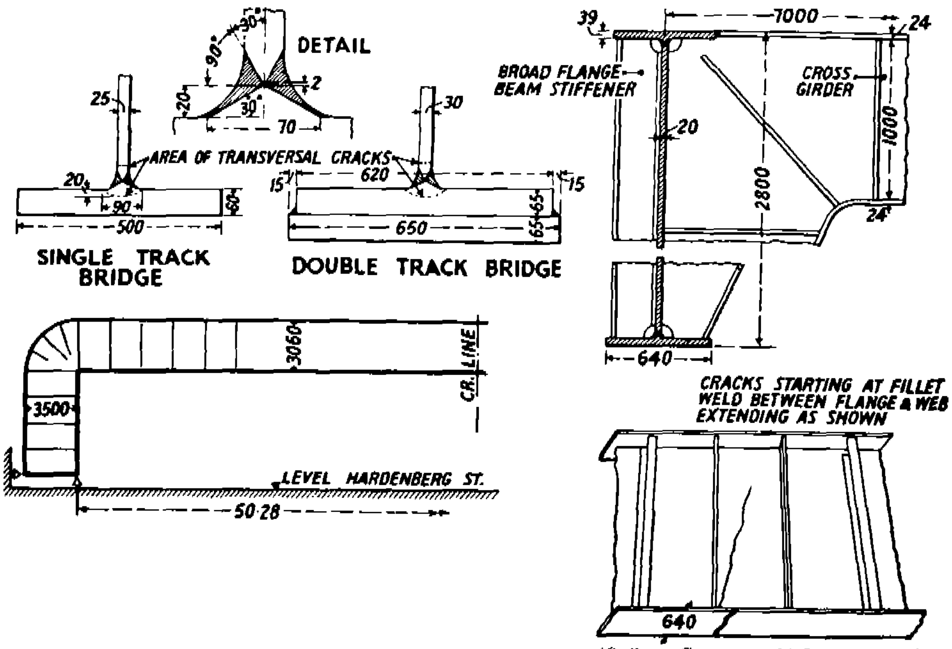
Some of the design details pictured seem to be of the type used in the Vierendeel trusses, with which so much trouble was encountered at the Hasselt Bridge and other locations. With no information given on history of the structure, no definite conclusions can be drawn. The extent of the failure (bridge collapse, etc.) is not stated. Nevertheless the report is a valuable one.

12. Three Welded Plate Girder Bridges, Berlin and Rudersdorf, Germany, 1936–38[42, 46]

Two railway bridges—one single track, one double track—were erected of St-52 steel at the Zoological Gardens Station in Berlin. Girder sections were made of welded plate, with a web 3 m in depth. On the single-track bridge, the flange was of 60-mm plate, 500 mm wide. On the double-track bridge the flange was 620 by 65 mm in cross section. (See Fig. 11.) In 1938, after the single bridge had been in use for half a year, and the double bridge had just been completed, transverse cracks were noted in the fillet welds between web and flanges, extending well into the parent metal. Crack stopper holes were drilled at the crack ends, and temporary supports provided. These spans then carried several hundred thousand trains before being torn down in 1938 and replaced with riveted structures, also of St-52.

At Rudersdorf, near Berlin, an Autobahn bridge, also of St-52 steel, was being completed at the time of the above trouble. It was of plate girder construction, with 17 spans totaling 3280 ft. Because of the Zoo difficulties the welds were carefully X-rayed, and necessary sections repaired. On the night of Jan. 2, 1938,

Fig. 11 Details of welded bridge failures. Left, at Zoological Gardens Station, and right, at Rudersdorf[42]



(*Railway Gazette 72, 24 June 14, 1940*) the temperature suddenly dropped 10° C. Here also cracks started in the fillet weld between the lower flange and web, traveling up into the web, nearly across the girder, which was 2.8 m deep. The flange was 39 by 640 mm in cross section. (See Fig. 11.)

In both cases, investigators concluded that hardening, combined with residual stress, initiated fracture. In the case of the Rudersdorf bridge, vertical stiffeners had been welded to the web plate before the flange plates were connected to it. This apparently resulted in restricting the shrinkage of the fillet welds between web and flanges, causing high residual stress. Extensive tests were made[42] of thin welds on thick flanges of high thermal capacity. Hardening resulted in this thin deposited metal, resulting in transverse cracks.

13. Duplessis Bridge, Three Rivers, Quebec, Canada, 3:00 A.M., Jan. 31, 1951[47, 48]

This bridge consisted of two sections, totaling 1380 ft, containing six 180-ft spans, and two 150-ft spans. It was of continuous welded plate-girder deck construction, resting on concrete piers. The two girders were 32 ft apart, being 12 ft deep at the piers, and 8 ft deep at the centers. The bridge was completed in 1947. In February 1950, in cold weather, 27 months after completion, a fracture was discovered in a down-stream girder of the East Crossing. While this was being repaired a similar fracture was found in an identical location in the West Crossing. Both cracks originated in top flange plates, close to butt-welded joints, and traveled toward the center of the girder. The East break buckled the web and lower flange. The West break stopped because of the tension action of the slab reinforcing. All similar butt-welded joints were checked on this bridge and on the 1548 ft St. Rose and 1520 ft St. Eustache bridges of similar construction. No other defects were found.

Rust colorings in the cracks indicated that they had spread in two or three stages, radiating from the fillet welds joining the web to the flange. Paint was found in the cracks, indicating they had been there (at least in part) before the girders left the shop for the bridge site.[48] To repair these fractures, sections of damaged web and flanges were removed and replaced with web and welded flange sections that were riveted in place to the old material. Following this all tension joints were reinforced with riveted plates.[47]

Finally on Jan. 31, 1951, nearly a year later, the west half of the West Crossing collapsed into the river. Traffic on the bridge was negligible. The temperature was −30° F.[47] Two weeks before the final collapse a Provincial bridge inspector had run a continuous 10-day inspection and reported everything satisfactory.

At the time of the first trouble, February 1950, an exhaustive investigation[48] had been carried out. This revealed (in part) that the flange plate had been ordered to meet C.S.A. S-40 (ASTM A-7) specifications. Although not stipulated in the specification, thick structural plate is usually rolled from semikilled or killed steel. In this case the mill supplied rimmed steel which was passed by inspectors and built into the bridge.

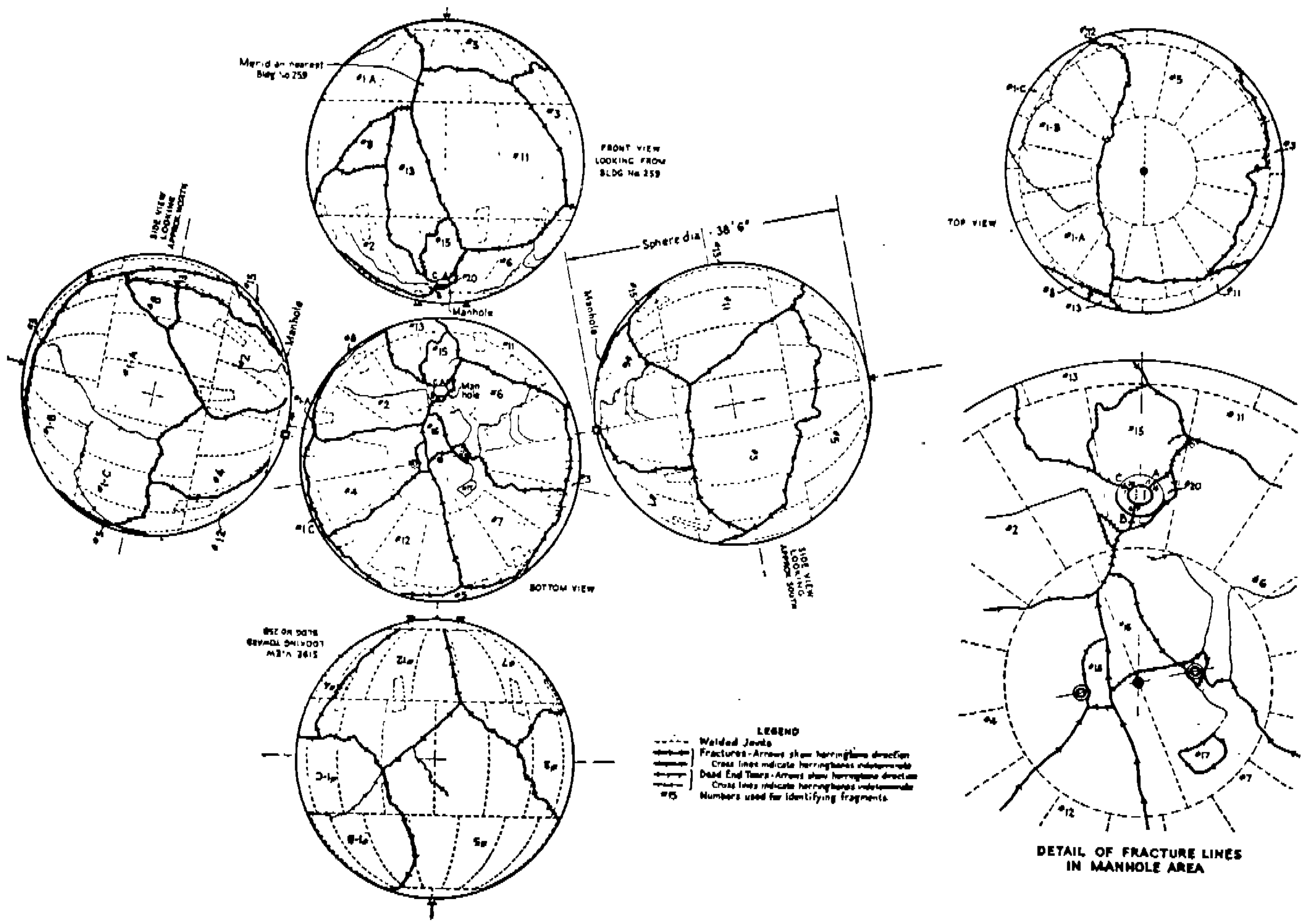
The broken flange plates were found to be of poor quality rimmed steel unsuitable for welding. They contained high local concentrations of carbon and sulfur, with many slag inclusions in the form of strings, particularly in the core section.[48] The postfracture analysis[48] of the $2^1/_2$-in. flange plate showed a variation in carbon content of 0.23–0.40%, and a sulfur content of 0.04–0.116%. Manganese varied from 0.30 to 0.33%. The yield point of the material varied from 27,800 to 57,800 psi, with an average tensile strength of 58,000 psi. Charpy notch-bar tests gave values of 3, 4, 4 and 6 ft-lb at 100° F. The welds were generally satisfactory in quality, showing some slight slag inclusions. D. B. Armstrong[48] concluded speculatively that the original cracks may have been initiated when the longitudinal fillet welds were laid across the butt welds, the combination of restraint and shrinkage stress being too great for the notch sensitive material.

It is the belief of this author that with so brittle a material, any slight defect, combined with dead load stresses, might lead to catastrophic failure at low temperatures. Seventeen other welded continuous girder deck bridges, totaling two miles in length, stand in the Province of Quebec.[48] So far as is known, no trouble had been experienced with them. The government of the Province of Quebec is still conducting an investigation into the fall of the Duplessis Bridge. Results have not been made public.

14. Spherical Hydrogen Storage Pressure Vessel, General Electric Corp., Schenectady, N. Y., 2:47 P.M., Feb. 16, 1943[49–51]

This was a spherical hydrogen tank, 38.5 ft in diameter, 0.66 in. thick, semikilled plate, of welded construction. It had been in service three months. The design was in accordance with Paragraph U-69, ASME Code for Unfired Pressure Vessels. The design called for a working pressure of 50 psi, a working stress of 11,000 psi and a weld efficiency of 80%. In 1942 it had been tested at 62.5 psi, showing no leaks. The manhole of the tank had been made in two subassemblies (bolt flange of neck in one, collar and sphere plate in the other) and welded in place on the ground. All manhole plates were made of $^3/_4$-in. sheared cold-rolled plate. The plates were cold shop formed, and in accordance with Paragraph U-69, no stress relieving was performed.

On the day of the fracture, the ambient temperature had been subzero, had risen 27° F in 7 hr, and was 10° F when failure occurred. The internal pressure was about 50 psi. The sphere burst catastrophically into 20 fragments, with a total of 650 ft of herringboned, brittle tears. The tears were plotted on a model, with directions of herringbones marked by arrows. All herringbones led back to the manhole, which was the origin of fracture. (See Fig. 12.) The intensity of the failure was greatest in the manhole region.



(*Welding Journal 24, 3 Mar. 1945*)

***Fig. 12** Plot of failed hydrogen sphere at Schenectady, N. Y., showing path of brittle tears determined from the herringbone markings*

The general quality of the welding was excellent. Only a few feet of fracture followed welded seams or the heat-affected zones. Later examination of the relief valves showed them to be operating satisfactorily. Fractures did not involve, except in a minor way, support leg attachments where stresses were high. On good evidence, the possibility of internal explosion was eliminated. The field assembly of the manhole neck required heavy welds of many passes. Old cracks were later found in this metal, as well as many small cracks in the inner, sheared edge of the neck. The investigators[49, 50] believed the causes to be: (1) High stresses at the manhole neck, due to the presence of the hole in the sphere; (2) residual stresses approaching the yield point in the manhole neck, due to shrinkage of the heavy weld. There were several old radial cracks in this region; (3) the use of semikilled steel, which was brittle under the present circumstances; (4) probable thermal shock due to the rapid rise of temperature and the sun's rays increasing the hydrogen pressure, or to thermal stress due to uneven heating of the sun's rays. The large amount of energy available from the compressed gas was sufficient to scatter the pieces without an explosion.

The investigators recommended that gas vessels should be tested at twice the working pressure with water, rather than $1\frac{1}{4}$ times the working pressure with gas, and that subassemblies (such as manholes, nozzles, etc.) should be built in the shop, stress relieved and magnafluxed for cracks. The design of these subassemblies should be such that heavy, built-up weld deposits which cause high residual stress are not used.

15. Spherical Ammonia Pressure Vessel, Pennsylvania, March 1943[51]

This sphere was built to contain anhydrous ammonia (density 42 lb/ft^3) at 75 psi. It was 40 ft in diameter, $\frac{7}{8}$-in. plate. Some plates were rimmed steel, others semikilled steel. It was supported on seven columns with reinforcing pads, $\frac{3}{4}$ by 19 by 84 in. where the columns joined the sphere.

Failure occurred while the sphere was being subjected to a hammer test, called for by the 1940 ASME Code for Unfired Pressure Vessels. This test required the seams to be struck with an 8-lb sledge hammer while the vessel was filled with water at 115 psi. A horizontal, brittle tear resulted when the hammer struck a vertical seam. Practically none of the tear followed any welded seams, and the tear extended to the right and left of the hammer blow. Following this, 20% of the seams were examined by magnaflux, with no serious defects detected. The welds were of good commercial quality. Failure had been initiated by a notch effect produced by a slight overlap of weld metal, combined with slight weld porosity. A previously built vessel, twin of the failed sphere was surveyed by strain gage technique while full of water at 40 and 75 psi. The report states that the design was found adequate.

16. Spherical Pressure Vessel, Morgantown, W. Va., January 1944[51, 52]

Following the failure of the foregoing sphere, several similar tanks, which had been in service some time, were checked (by magnaflux) to see if manhole defects were present. Among others were six spheres in an Ordnance plant at Morgantown, W. Va. They were built for a liquid (unspecified) density 42 lb/ft^3, presumably ammonia, at 50 psi pressure, and working stresses of 11,000 psi with 90% joint efficiency. After repair of the manholes, tanks were tested by filling with water at 100 psi. The second sphere while being tested, failed completely at 98 psi. The bottom dropped out, and the top fell in on it. (See Fig. 13.) There were

Fig. 13 Failed sphere at Morgantown, W. Va., showing long brittle tear

350 ft of herringbone tear, and only 4 ft went along a seam. The direction of the herringbone indicated that the tear probably started at a point just below where a column was attached to the shell. Subsequent strain gage readings on a duplicate sphere indicated high local stresses at column attachments. There were 800 ft of welded seams of good quality, though at some points there was lack of complete fusion.[51]

The temperature during the previous night had been about 19° F, and was at 30° F at failure. The water temperature in the sphere was about 38° F. At 32° F the keyhole Charpy impact value for the steel was well below 15 ft-lb. The chemical analysis of the steel showed a carbon content of 0.20–0.21% C, 0.47–0.48% Mn, with the remaining elements as is usual for ASTM A-7 steel of firebox quality.[52]

Engineering personnel of the operating organization recommended in part as follows:

1. Sheared projecting ends of nozzle and manhole necks should be machined or ground to a depth of $^1/_8$ in. to remove cracks.
2. In future, shop-assembled, stress-relieved sphere sections complete with nozzles and manholes should be used for all openings.
3. Existing spheres should not be operated over 50% of maximum hydrostatic test pressure.
4. Top of columns and adjoining sphere sections should be stress relieved.
5. To reduce bending, thicker plate should be used at column connections, rather than a double plate.

A strain gage investigation into the stresses of spherical tanks was performed by G. A. Brewer[53] on a butadiene gas tank 50 ft in diameter at 60 psi pressure. (This was not one of the tanks referred to above.) The plate was 0.822 in. thick, of ASTM A-70, semi-killed steel. The Unfired Pressure Vessel Code allowed a working stress of 11,000 psi. He found that on the juncture of a horizontal and vertical weld bead, on a plate to which a column was fastened, the stress was 33,800 psi, or 2.88 times that predicted by simple theory. Adjacent to the column it was 19,100 psi, or 1.62 times. These stress values have been criticized,[54] however, on the grounds that the figure of 33,800 psi measured at full pressure, on the outside only, may really have represented merely a stress difference, rather than true value. For instance, the stress may have been $-15{,}000$ psi with no pressure, and $+18{,}000$ psi at full pressure. Until the residual stress in the unloaded condition is known, the question will remain unanswered. For further comments on residual stresses, see Discussion, page 36.

17. Cylindrical Gas Pressure Vessel and Spherical Gas Pressure Vessel, East Ohio Gas Co., Cleveland, Ohio, 2:40 P.M., Oct. 20, 1944[51, 55–58]

These tanks,[55] with two other spherical tanks, were built to hold liquefied natural gas at 5 psi and $-260°$ F. A pilot plant was first put in operation in 1940, from which it was concluded that ordinary steel was not safe below $-50°$ F. Charpy tests were performed on various metals, from which it was decided that various metals in order of excellence for a safe Charpy impact test value were copper, bronze, Monel metal, red brass, stainless steel (type not specified), and steel plate with less than 0.09% carbon plus $3^1/_2$% nickel. Erection of the full-scale plant was begun September 1940 and completed January 1941. The storage facilities consisted of three double-shelled spherical tanks. The outer shell of each was of welded, open-hearth, mild steel. Inside was a 3-ft layer of granulated cork, then the 57-ft diam storage sphere of the nickel alloy steel. This had a specified percentage analysis as follows:

C	0.08–0.12
Mn	0.30–0.60
S	0.045 max
P	0.045 max
Si	0.10–0.20
Ni	3.25–3.75

This steel was deoxidized, rolled, normalized at

1550° F to a McQuaid grain size of 6–7, and a hardness of BHN 149–152. The designer picked this alloy because he believed it to be satisfactory and less costly than other materials considered.[55] Seams were welded with 25Cr–20Ni rod. Weld specimens from all welding positions and plate thicknesses were, according to one report,[55] impact tested at −260° F and gave Charpy values of greater than 15 ft-lb in both welds and heat-affected zones.[55] The cork space between the shells was vapor tight and kept dry by low-pressure gas discharge. Each sphere was supported on 12 columns, the liquid content having a density of 26 lb/ft^3. Design stress was 13,750 psi and tanks were equipped with safety valves.[51]

In 1943, after the spheres had been in use approximately two years, it was decided to add an additional cylindrical-toroidal storage tank. The designer felt that this tank would have a safer shape, inasmuch as plate flexure would be decreased. Again the inner shell was of the same $3^1/_2$% nickel alloy and the same welding procedures were used. The inner shell was 70 ft in diameter, 42 ft high. The top and bottom were dished heads within a dished annulus. The bottom inner shell was supported by wood posts. The outer shell was 76 ft in diameter and 51 ft high, the inner space being filled with rock wool. Design stress was 12,496 psi.

The cylindrical shell was given a hydrostatic test by filling it half full of water and pumping the remaining air space to 5 psi. When the tank was first filled with liquid gas in June 1943 a plate in the bottom cracked. In July 1943 the cracked section of this plate was drilled and chipped out, and a patch was welded in. The patch plate was dry-ice cooled during the process so that no residual stress would result. The residual stress was checked by strain gages. The tank was again tested, then uniformly cooled as it was filled with liquid gas. It was put in service with no further incident.[55]

On Oct. 20, 1944, as the plant was being shut down, witnesses saw vapor or liquid issuing from the cylindrical tank, one-third or one-half way up from the bottom. The atmospheric temperature was 51° F at the time. There was a rumble and flames. Explosion followed. Twenty minutes later an adjacent spherical tank failed in the heat, due to weakening of supporting columns. Liquid gas flowed into the sewers, spreading the holocaust. One hundred and twenty-eight persons were killed, damage was $6,800,000.[51, 55]

Several simultaneous investigations ensued. One account states that fragments of the inner shell of the cylindrical tank showed that rupture had started at the center of the roof, had run radially outward, down the shell, and in through the bottom.[51] The fractures were of the brittle type. There were some failures at the weld, but these may have been caused by the heat of the fire. There seemed to be no evidence of an initial explosion, but rather just disintegration of the tank. Analysis of the steel showed that it conformed to specifications.[55] External explosions were eliminated as causes, but some type of seismic shock load, either from an adjacent railroad or nearby drop hammer, remain possibilities. It was pointed out by one group of investigators[55] that most industrial concerns use [austenitic] stainless steel, or nonferrous metal for low-temperature applications.

Another investigator[56] performed a detailed metallurgical examination of the failed No. 4 vertical cylindrical tank. The chevron markings on a great many tank fragments were checked and plotted on a model with little success. All the evidence indicated that there were a great many origins of fracture. There was nothing to indicate that the patch plate in the bottom of the tank had been a failure origin. The material was generally of good quality, and generally free of serious defects. The plate was found to be hot rolled, and in the as-rolled condition. Some weld defects were discovered by X-ray, but they were not serious. Charpy tests (with keyhole notch specimens) were performed at −248° F. Specimens from plates in Tank No. 4 (which failed) gave values of 3–5 ft-lb at that temperature. Specimens from Sphere No. 1, which had stood undamaged through the fire and was cold several days later, gave values of 1–6 ft-lb, as did a spare plate from this sphere. It will be noted that some of the findings here reported are at direct variance with the plate properties reported above.[55]

This same investigator[56] concluded that the lack of adherence to the 15 ft-lb minimum Charpy value at service temperature was of primary importance as a cause of the disaster. He pointed out that even at −194° F, well above the service temperature, his findings showed that a relatively small fraction of the specimens tested by him exhibited sufficient ductility in the Charpy test. He also criticized the design of the cylindrical tank, in that the vertical member of the belt ring was stressed in tension in the direction perpendicular to the surface. Spalling was observed here when the bottom plates tore loose.

One conclusion of another group of investigators[57] points out that the designer calculated only the membrane stess for this cylindrical tank. Calculations of secondary stresses at discontinuities showed that in one location the bending stress approached 50,000 psi. They also were of the opinion that because a spherical tank has fewer discontinuities, it would be better where low-temperature brittleness is a factor. It is the opinion of this author that that simplicity of structure in itself will not necessarily bar brittle failure. This is demonstrated in gas line failures (see below).

The designer of these tanks, in an article published before the disaster[58], stated that this type of gas container cost about $1839 per million cubic feet of storage (regasified) versus $47,600 to $99,000 per million cubic feet in normal gas holders. In the course of the investigation[55] following the disaster he stated that the nickel alloy was to all intents and purposes brittle at −260° F, despite a satisfactory Charpy value. He indicated that when a sheet of this steel was at a low temperature, a sledge hammer could be driven through it, but that in his opinion this should not obviate its use for construction purposes. He cited examples of a large number of brittle materials used in construction.[55]

Hindsight is doubtless better than foresight. Today, one asks, in view of the large comparative savings in construction, why the more expensive stainless steel was not used. Certainly the greatest danger of notch-brittle steels becomes apparent, in that the lesson to be learned is this—while many brittle materials are used in construction, they are designed for as such; design for ferritic steel assumes the material to be ductile, and sometimes it is not.

18. Five Oil Storage Tanks, Russia, Dec. 12–14, 1947[59]

These were cylindrical tanks of 160,000 cu ft capacity, somewhere in Russia. No details of dimensions, construction or contents are given. The material was an ST-3 steel, with a specified percentage analysis as follows: 0.13–0.20 C, 0.35–0.60 Mn, trace of Si, 0.05 P max, 0.05 S max. The welding electrodes had a thin chalk coating, apparently to stabilize the arc.

In the course of 48 hr all five tanks developed innumerable cracks where the bottom joined the first course of plates. All cracks were on the northeastern side, facing prevailing winds. During the time that the damage occurred the temperature ranged from −31° F at noon, to −47° F at night. There was no snow cover. No tanks burst, but all became leaky. In a tank numbered as 18, a crack started at the bottom where the base angle iron joined the first course in a triple layer of welding. The crack went through a vertical welded joint. In Tank No. 19 a crack started in a weld crater, went through the cover plate angle iron, and the bottom course of welds. In Tank No. 11 a crack went all along the welded joint around the cover plate, up into the first course of plates, and down into the bottom. The other two tanks behaved similarly.

The tanks had been built from 1941 to 1943. There had been no previous trouble. Much of the welding had been done in the winter in temperatures of −32 to +27° F. The cracks had all started at notches (craters, lack of fusion, weld build-up, covered weld cracks). Residual stresses were also blamed, as well as thermal stress. In previous winters the cold had set in gradually and evenly. This year the weather had been mild with no snow. It had turned suddenly cold on December 10th. The tanks rested on unfrozen ground, and the contents were warm. Sudden contraction due to the cold wind caused fracture on the windward side.

19. Crude Oil Storage Tank, Middlewest, U. S., 7:31, A.M. Feb. 2, 1947[60]

This tank was built in 1944. Because of the materials shortage, plates were obtained from dismantled, riveted tanks. The cleanout door and its reinforcing plates were new steel. The rivet holes were trimmed off the plates, and the edges prepared for welding. The original diameter of 120 ft was preserved, but the tank height was increased from 40 to 48 ft, $4^3/_4$ in.

At the time of failure the tank was being filled with crude oil, which had reached a level of almost 45 ft. The oil temperature was 43° F, and the air temperature was approximately 0° F. On the previous day the air temperature had been about 42° F. Failure originated at an upper corner of the reinforcing plate of a shell cleanout door in the bottom course. The crack propagated upward through this plate at 45 deg to the vertical as far as the horizontal weld between the reinforcing plate and the shell. (See Fig. 14.) It then spread up and down through the entire height of the shell. The shell tore loose from the bottom plate, flattened out and floated away. The directions of crack propagation were determined from the herring-bone markings.

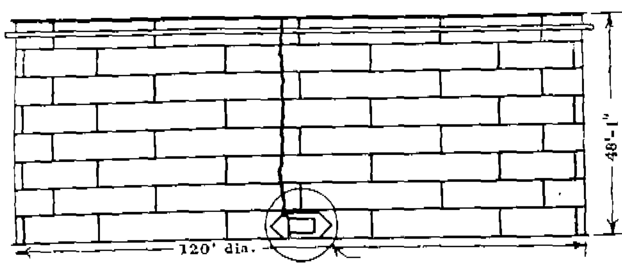


Fig. 14 Failed crude oil storage tank showing path of the crack through the tank shell

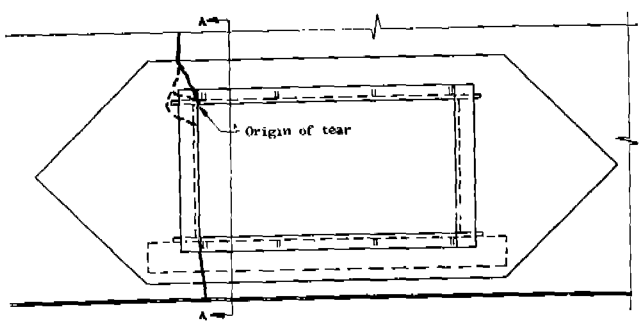


Fig. 15 Details of square cornered cleanout door in failed crude oil storage tank

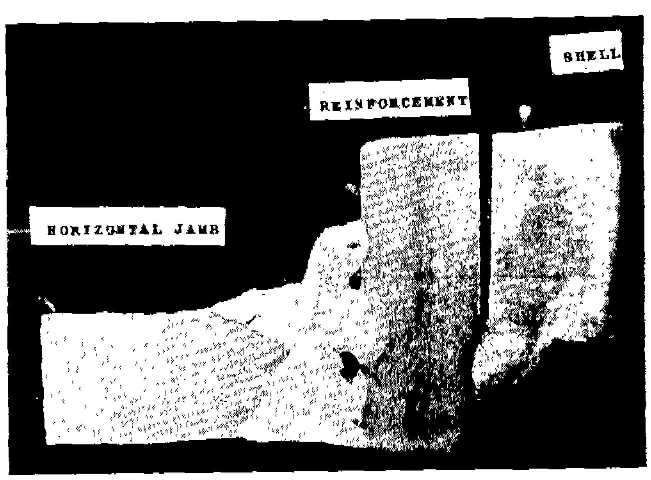


Fig. 16 Section through horizontal cleanout door jamb in failed crude oil storage tank. Note poor quality of welding as evidenced by cavities

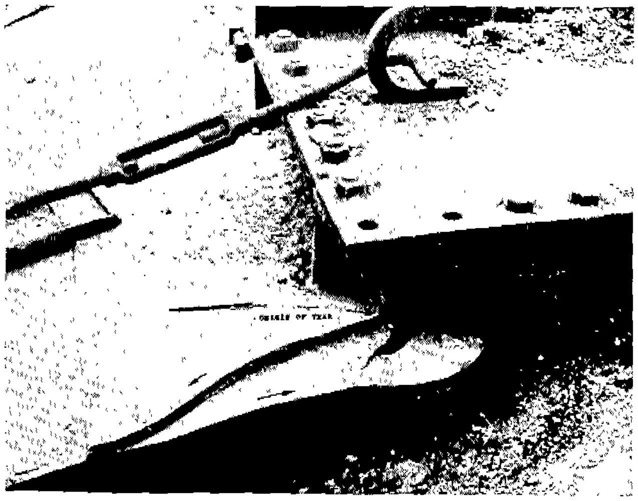


Fig. 17 Failed crude oil storage tank, showing brittle tear, with origin of fracture at corner of cleanout door reinforcing plate

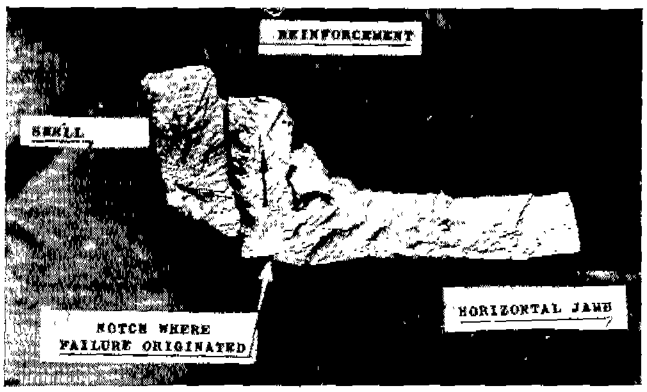


Fig. 18 Origin of fracture in failed crude oil storage tank. Arrow indicates direction of crack propagation

The investigation showed the computed stress to have been 19,000 psi, 1 ft above the lower edge of the bottom shell course, which was 0.66 in. thick. The cleanout door was rectangular with square corners, with a coincidence of a number of built-up plates and welds. It reminds one of Liberty ship hatch corners. (See Fig. 15.) The quality of welding around these corners was poor. (See Fig. 16.) Fractures exhibited typical herringbone markings. (See Fig. 17.) The origin of failure, Fig. 18, shows that the fracture was brittle from its inception, as is typical in most engineering structures.

Percentage analysis of the shell and reinforcing plates showed:

	Shell plate	*Reinforcing plate*
C	0.11	0.28
Mn	0.44	0.49
P	0.010	0.013
S	0.033	0.032
Si	0.002	0.002
Cr	0.01	0.02
Ni	0.03	0.04
Mo	0.01	0.01
ASTM Specs	A-70 or A-10	A-7

Tensile tests showed the material to have the usual strength and ductility of such steels.

Charpy tests with keyhole specimens showed:

Temperature, °F	*Shell plate*	*Reinforcing plate*
Room	30–40	19–20
25°	5–8	4.5–8
0°	3–3.5	3.5
−25°	2.5 ft-lb	2.5 ft-lb

The investigation concludes that, "Both of these materials, while of average quality and similar to those used for tank work, probably should be characterized as notch sensitive. . . . The carbon content of the reinforcing plate is within the range of the so-called welding grade steel."

20. Oil Storage Tank, Normandy, France, Winter 1950–51[61]

This was a tank of 10,000 cu m capacity. The details of construction, tank size, plate thickness, oil height, weather conditions and exact date of fracture are not given. Two cracks appeared, running up from a vertical weld in the first course of plates, joining, and stopping after just crossing a horizontal welded seam. Apparently no further rupture took place. All cracks were roughly perpendicular to the adjoining seams that they crossed. Direction of crack propagation was determined from herringbone markings. Photos showed many of the welds to be of bad quality. One vertical seam was later broken open through the weld. Bad surface irregularities, undercutting and cracks were evident.

Analysis of two plates showed one to have 0.19% C, the other 0.12% C. The analyses were typical of low-carbon plate steel, with sulfur and phosphorus quite low. The crack, while starting in the seam between these two plates, traveled only through the higher carbon plate.

21. One Crude Oil Storage Tank, and One Gas Oil Storage Tank, Fawley, England, Feb. 12 and Mar. 7, 1952[62]

These tanks failed while being given hydrostatic acceptance tests. Tank sizes and data at failure are as follows:

	Crude	*Gas oil*
Size	140 ft diam by 54 ft	150 ft diam by 48 ft
Construction Spec.	API 12 C*	API 12 C*
Water height at failure	48 ft	38-39 ft
Steel	B. S. 13	B. S. 13
Filling rate	6 ipm	9 ipm
Water temperature	40° F	40° F
Air temperature	30° F	47.4° F
Failure date	Feb. 12th	Mar. 7th

* Amended to call for 100% weld penetration on horizontal seams.

Inspectors checked the tanks on erection. The welders were qualified. Weld probe samples were found to be satisfactory. The crude oil tank was the eleventh identical tank built by the same contractor, and the gas oil tank was the fifth by another contractor. There was no trouble with the previous tanks. In the crude tank the plates varied from $1^1/_{32}$ in. thick on the first course to $^1/_4$ in. in the ninth course, and in the gas oil tank from $1^1/_{32}$ in. on the first to $^1/_4$ in. in the eighth.

The crude tank had had a previous partial failure on January 30th. During a hydrostatic test on that date, a crack had started at a weld probe replacement in the first horizontal joint. The crack was 24 in. long and extended across the joint into the first and second course plates. Water height at the time was 35 ft. The tank was drained and repaired. At the time of complete failure at 11 P.M., February 12th, no damage occurred in this earlier repaired area, which was located about half-way around the circumference from the final failure. The tank split into two sections as it washed out. The gas oil tank split at 10 A.M., March 7th, in a manner similar to the other, but the shell stayed in one piece. Prior to the water test in the gas oil tank, some cracks and unfused welds had been removed and repaired. These repairs did not fail when the final fracture occurred.

Herringbone markings in the crude tank showed that the crack had started at a weld probe replacement in the first horizontal joint. The crack progressed vertically in both directions, traveling in a brittle manner up to the fifth course ($^{19}/_{32}$ in. thick) where it changed to a shear type of failure. The weld probe had been cut from the outside and apparently had just barely penetrated the inside surface. The replacement weld metal did not penetrate to the inside of the groove. At the back of the probe location a single cover bead had been laid over the opening. There had been no back chipping to remove slag and provide a clean surface for the back weld. Thus a void had been left about 2 in. long, extending 20 to 25% into the plate thickness.

In the gas oil tank herringbone markings showed that fracture had started at a partially repaired crack in the top 10 in. of a vertical weld in the first course. This crack had extended about 2 in. into a second course plate. The crack was old, and its surfaces were coated with a black oxide film from subsequent welding operations. The final failure progressed vertically in both directions, changing from brittle to ductile failure in the fourth course of plate ($^{21}/_{32}$ in. thick). For the greatest portion of its length, the crack traveled through plate rather than welds. In the fifth course, however (where failure was in shear), it traveled through a vertical joint. This joint showed a serious lack of penetration throughout its length.

The old crack, from which the failure started, had been partially, but improperly, repaired. It had not been entirely chipped out before rewelding, and only a cover bead had been laid over the part that extended into the second-course plate.

The investigators concluded that the failures were initiated as a result of poor workmanship, and that in many respects the mode of failure was similar to that in welded bridges and ships. Accordingly, the properties of the steel were investigated.

Analyses (in part) showed the following, given in percent:

	Crude tank 1st course	*Crude tank* 2nd course	*Gas oil tank* 1st course	*Gas oil tank* 2nd course
C	0.165	0.21	0.245	0.22
S	0.036	0.041	0.027	0.03
P	0.024	0.025	0.031	0.031
Mn	0.54	0.56	0.62	0.54
Ni	0.86	0.062	0.11	0.08
Cr	0.02	0.02	0.05	0.05

Tensile tests gave the usual results for such material, within the values for B. S. 13 steel, which had been specified.

Charpy V-notch tests on steel from the crude oil tank gave the following range of values:

	Ft-lb at temp, ° F 4	32	50	68
1st course	3–4	3–7	5–12	18–30
2nd course	3–5	5–9	5–15	12–22
3rd course	7–9	14–20	15–21	34–45

Charpy V-notch tests on steel from the gas oil tank gave:

	Ft-lb at temp, ° F 4	32	68	104
1st course	3–7	6–10	18–27	30–45
2nd course	4	6–8	9–20	30

U-notch tests gave somewhat higher values.

The investigators then further concluded that attention must be paid to the notch brittle characteristics of the material, as manifested by Charpy impact tests.

One very pertinent point discussed by the investigators was the sectioning method of weld inspection specified by the API Code Section 120. A few defective welds were found by this method and necessary repairs were effected. However, a gross defect in the gas oil tank welding went undiscovered. Moreover, on the crude tank, the unsatisfactory replacement of a probe provided the defect which initiated complete failure. Some probes were, in addition, not cut deep enough, thus raising the possibility that lack of penetration in the root was not disclosed. Radiography, by X-rays and gamma rays was therefore proposed as a better means of inspection. It is stated that radiographic means have since been used at Fawley to reveal lack of penetration, underbead cracking and inclusions in other tanks.

Finally the report points out that the Fawley steel was in the transition range at the operating temperature of approximately 40° F. The conclusion is reached that steels less subject to brittle failure, such as

ABS (American Bureau of Shipping) Class C steel, or steel prepared under Lloyd's Register of Shipping Specification P-403, should be used for the time being. This survey discusses these steels at a later point.

22. Three Empty Oil Storage Tanks, Europe, 1952[63]

Three floating roof tanks, 144 ft in diameter by 45 ft high had been built of eight courses of plate. Plate thickness ranged from $^7/_8$ in. in the lower course to $^1/_4$ in. at the top. The steel had a tensile strength of 26 to 33 tons per square inch, with an elongation of 20% in 8 in.

In the course of erection, the contractor had chipped flush the weld overfill at the seams inside the tank. In addition, from the marks on the plates along the welded seams, it was evident that an excessive amount of hammering had been done to correct distortion. (See Fig 19.) Several weeks after completion, when the tanks were still empty, the ambient temperature fell to $-4°$ C. A large number of cracks developed in all three tanks. The cracks had originated at the chipped or hammered surface of the welds, extending transversely across the welds, entering the plates for a distance of about 3 in. (See Fig. 20). Except in one instance, the cracks occurred in plates over $^1/_2$ in. thick.

In V-notch Charpy tests, the temperature range for 15 ft-lb was $+10$ to $-10°$ C, for both the parent material and the weld metal. In the opinion of the investigator, the causes of the failures were:

1. The formation of transverse surface fissures caused by the chipping tool. A section through one of these fissures (Figs. 21 and 22) showed it to be quite a sharp notch.

2. The existence of tensile residual welding stresses, acting in a direction normal to the surface fissures, along the line of the weld.

3. The increased notch sensitivity of the steel due to the fall in temperature, and to the work hardening of the weld surface layers by the action of the chipping tool and by excessive hammering.

Since the fissures left by the chipping tool were of such small size, it was believed necessary to show that such a discontinuity would produce this effect. Specimens from the tank weld were prepared and bent at various temperatures with the chipped weld surface in tension. At 0° C a brittle fracture occurred without deformation. (See Fig. 23.) In other specimens with the chipped surface ground off, a bend of 45 deg at this temperature gave no indication of a brittle failure.

23. Water Storage Tank, Tucumcari, N. Mex., Dec. 13, 1951[64]

This tank had been designed for oil stoage. In 1938 it was torn down and in 1940 re-erectedr in Tucumcari for water storage. It was of lighter construction than permitted by standards of the American Water Works Assn. The tank was 115 ft in diameter, 30 ft high. Plates at the base were $^1/_2$ in. thick and butt welded, $^3/_8$ in. thick and lap welded at the center, and $^1/_4$ in. thick and lap welded at the top. There was a light, column-supported roof, and a plate floor resting on pea gravel.

At fracture 2,300,000 gal of water were released. A butt weld seam at the $^1/_2$-in. thick base course had let go. This tear propagated through the solid plate to the top. Later examination revealed that the butt weld in the $^1/_2$-in. plate had been faulty. The plates had been flame cut apart on the original disassembly and had been given no edge preparation before rewelding. As a result the weld on the $^1/_2$-in. plates was only partially filled with filler metal. Blackened edges of the original flame cut were plainly evident in this weld. Penetration of less than 0.1 in. had been obtained.

Some of the welds that did not break were offset or were filled with slag covered by weld metal. The steel

Fig. 19 European tank failures. Strip of weld joining $^7/_8$- and $^3/_4$-in. courses of plate, showing severe hammering on and around weld. Approximately $^1/_7$ actual size

itself was of good quality, judging by tensile tests. Notch-bend tests were not performed. It will be noted that here again, bad welds initiated failure which propagated into sound plates.

24. *Power Shovel Dipper Sticks (Location Not Given) Circa 1952*[65]

A dipper stick of a power shovel is the long member

Fig. 20 European tank failures. General appearance of crack across the weld between $^{5}/_{8}$- and $^{1}/_{2}$-in. courses. Approximately $^{1}/_{2}$ actual size

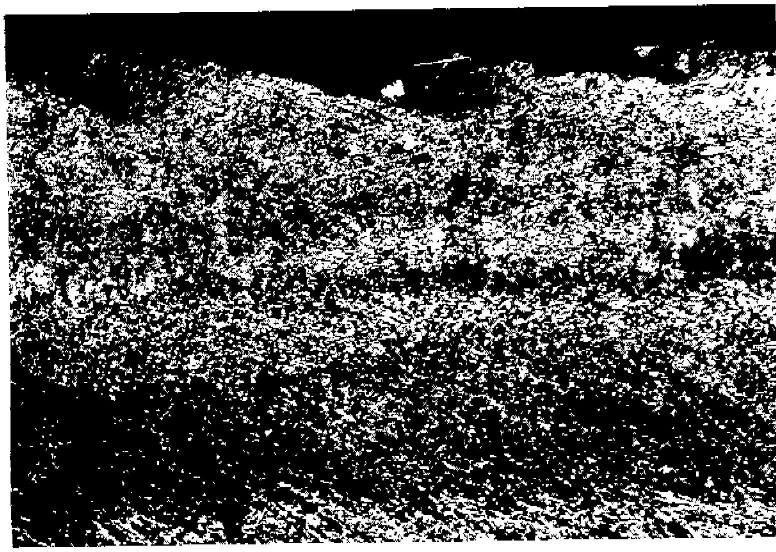


Fig. 21 European tank failures. Longitudinal section of weld, showing fissures caused by chipping tool. × 20

Fig. 22 European tank failures. Fissure caused by chipping tool. × 75

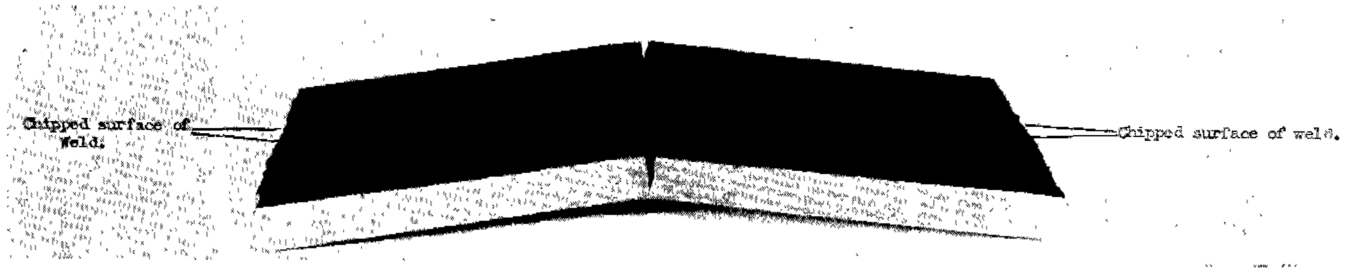


Fig. 23 European tank failures. Test plate bent at 0° C with chipped surface in tension. Brittle failure resulted. Approximately 1/4 actual size

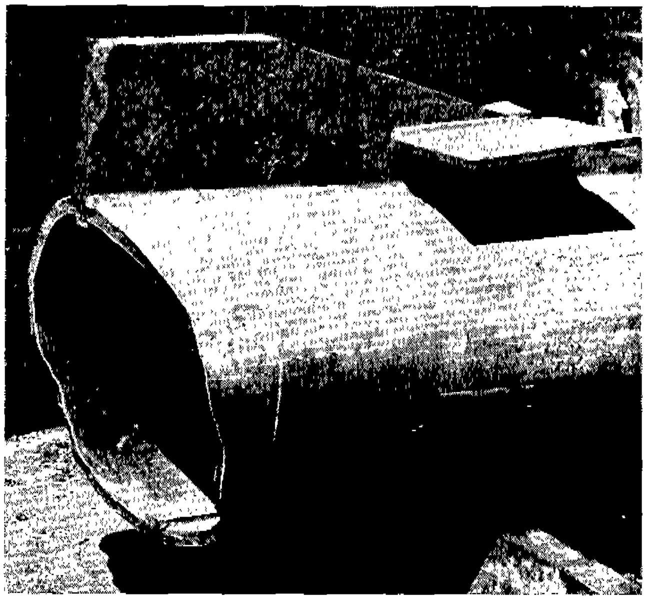


Fig. 24 Power shovel dipper stick, showing brittle failure which occurred at the bumper, which acted as a discontinuity

the end of which carries the shovel bucket. These sticks may vary in length, depending on the size of the equipment. In the case in question, the stick was a tube 37 ft long, circular in cross section, with an outside diameter of 20 in., and a 7/8-in. wall. The tube was made in half sections (semicylinders) 6 ft long, cold formed to a 10-in. outside radius. Longitudinal welds, joining the two halves of each section, were made using a 3/8-x 1 1/2-in. flat bar as a backing plate. Both the longitudinal welds and the circumferential welds to join the tubular sections were made by the submerged arc process.

The material was a "Man-Ten" plate, which would be classified as a low-alloy structural steel. The carbon content is usually about 0.12% but may be varied somewhat from this figure. Remaining nominal composition in percent is: Mn 1.25–1.70, Si 0.30 max, Cu 0.20 min, P 0.04 max, S 0.035–0.055 max. The copper in this material is added for corrosion resistance. Because of the high manganese, carbon must be kept low to prevent air hardening on welding. In general, alloy structural steels of this type have yield points in the range of 45,000–65,000 psi, and tensile strengths up to 90,000 psi. Their impact toughness is higher than ordinary carbon steels.[66]

Failure in this particular stick occurred at a temperature of −15 to −20° F. The fracture occurred at the

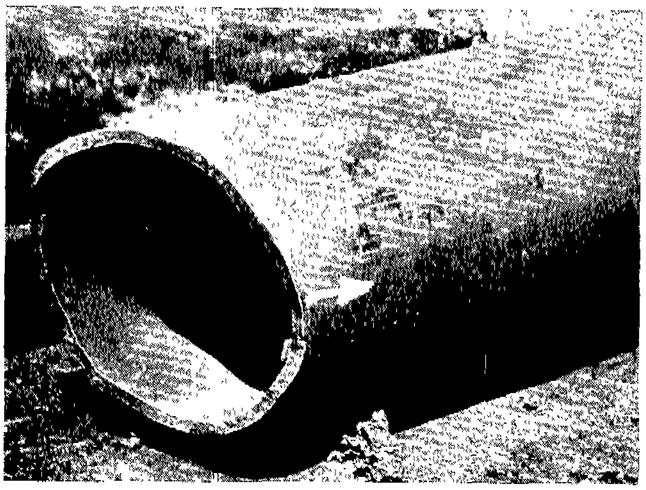


Fig. 25 Power shovel dipper stick. Mating half of fracture shown in Fig. 24

bumper, a piece which keeps the stick from moving too far. The bumper is a plate, sticking out in a radial plane from the tube, and as such is a discontinuity. Figures 24 and 25 show both halves of this failure. Similar failures had also occurred on other sticks at temperatures around 0 to 32° F, in all cases the failure passing through some obvious stress concentration or abrupt change in section. The circumferential and longitudinal welds have never been the source of any trouble.

The design of the bumper was subsequently modified by a sort of extended fillet which decreased the abruptness of the section change. This has to date prevented further failures.

25. Power Shovel Boom and Dipper Stick, Middlewest, U. S., January 1949[67]

The boom of a power shovel is the long member attached to the frame carrying the dipper stick with its shovel. In the case in question, the boom was 33 ft long, rectangular in section, with dimensions of approximately 16 x 20 in. The section was made of 1/2-in. plate specified according to ASTM specification A-7, formed in two halves and Unionmelt welded lengthwise down the narrower sides of the rectangular section with E6012 weld metal. A backing bar is used behind this longitudinal weld. The assembly is not stress relieved.

Figure 26 shows a boom which has had the end broken off. Figures 27 and 28 show the mating fractured sur-

Fig. 26 Failed power shovel boom. The end of this boom has broken off

Fig. 27 Failed power shovel boom showing half of the fracture surface. Diaphragm plate and one of the backing bars appear

Fig. 28 Failed power shovel boom, showing other half of fracture. Both Unionmelt backing bars are shown

Fig. 29 Cracked dipper stick of $2^1/_2$-in. ASTM A-7 plate. Crack occurred at an abrupt change in section

faces. The weld backing bars are clearly shown. This failure took place, it is believed, at −20° F. The crack propagated in and along a transverse butt weld. Failure was apparently initiated by the proximity of the diaphragm shown in Fig. 27 as well as a discontinuity of the Unionmelt backing bar, and poor root fusion at one point. Poor impact properties of the plate and weld metal aggravated the situation.

Failure of a dipper stick is also reported. This dipper stick consisted of two rectangular sections, each $2^1/_2$ in. thick of solid plate, ASTM A-7. One section passes on either side of the boom. Figure 29 shows a fracture in such a member. This failure was due entirely to an abrupt change of section, along with cold weather, and impact loading. There were no welds in the failure area.

26. *Penstock Anderson Ranch Dam, Boise, Idaho, Jan. 4, 1950*[68]

The penstock in question consisted essentially of a 15-ft diam pipe inside a 20-ft diam, concrete lined tunnel. The material was ASTM A-285, of firebox quality (formerly A89–43, Grade B), with 0.22% C max, 0.80% Mn max. This specification called for a yield point of 27,000 psi, tensile strength of 50,000 psi and usual elongation. An allowable design stress of 13,500 psi was used, and the penstock was figured for a static water head of 326 ft plus a water hammer head of 94 ft. The pipe sections were fabricated in the field. So far as it was applicable, the 1943 API-ASME Code for Unfired Pressure Vessels was followed. This code calls for thermal stress relieving for welds on plate over $1^1/_4$ in. thick. In field erection, however, mechanical peening may be substituted.

Hydrostatic pressure tests were to be performed at 225 psi. On the third portion tested, when a pressure of 200 psi was reached, a crack appeared which ran across three pipe sections having plate thicknesses of $1^5/_{16}$ and $1^9/_{16}$ in. The crack was 50 ft long, having lateral end branches. Two stiffener ring supports, one at each end of the crack, were also fractured. These stiffener rings caused the crack to turn at these points. (See Figs. 30 and 31.) The fracture was through the plate, parallel to but not closer than 5 in. from a longitudinal weld. The water temperature was 41° F.

Investigation showed no defects in the plates, and all specifications for the material had been met. The fracture had apparently started at a repair weld in a tunnel-welded girth joint. Another small crack radiated from here, and herringbone markings all pointed to this location. Heavy, irregular beads, applied during repair, may have provided the notch effect. (See Fig. 32.)

After removal and replacement of the fractured plates subsequent tests of the system at 275 psi pressure gave no further trouble.

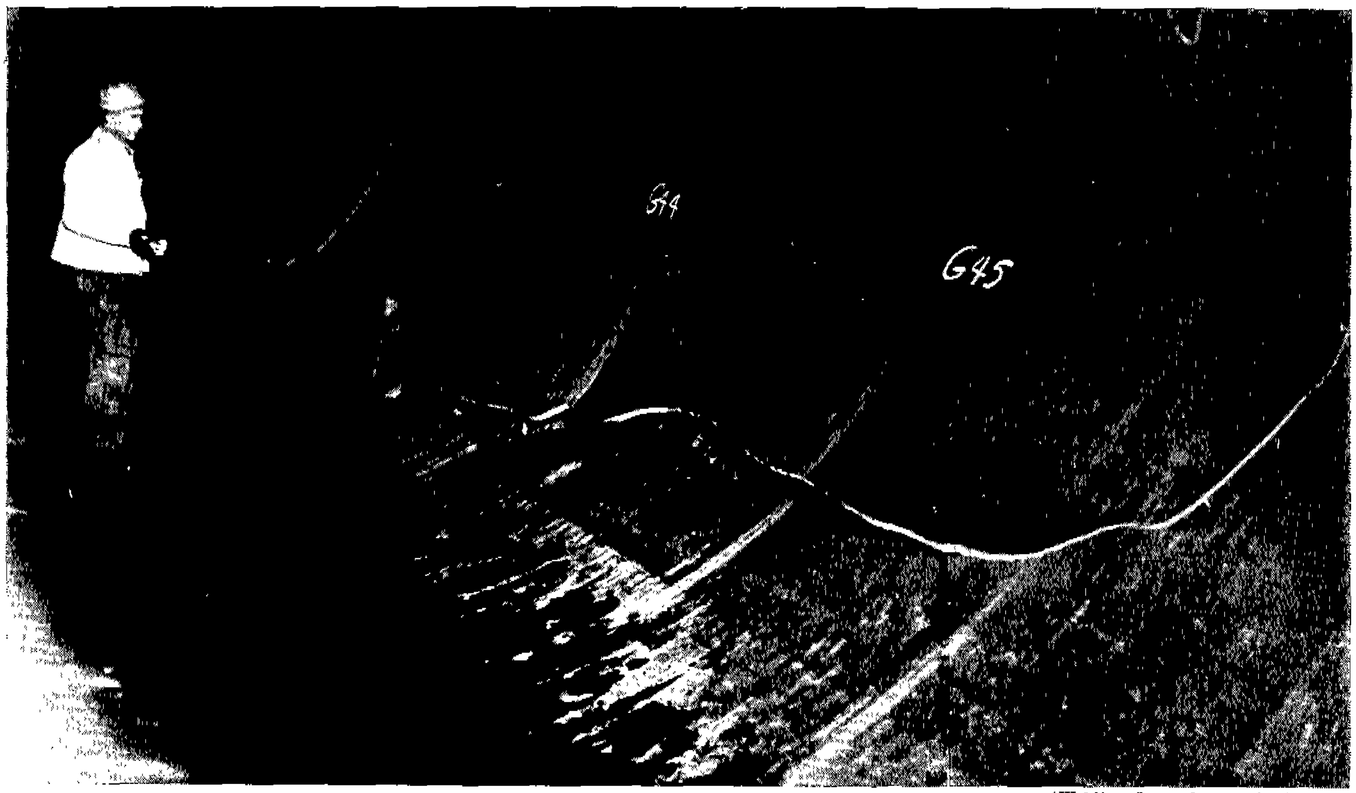
27. *Miscellaneous Failures*[69, 70]

Bursting of very old gas cylinders (some of welded construction) have been reported.[70] These containers were very old, most dating back to the time (circa 1929 and earlier) when all cylinders were periodically annealed. The steel in many cylinders showed carbide spheroidization due to this practice.

Another failure, while not of itself of great engineering significance, is very interesting.[69] A large drum or cylindrical pressure vessel, 66 in. inside diameter, of $^7/_{16}$-in. plate, was hit by a car coupler in a wreck at Windham, Ohio, Dec. 27, 1943. As a result, a large patch was knocked out of the side, the pieces from the patch shattering, much like glass. The hole in the tank is shown in Fig. 33. Shattered fragments are shown in Fig. 34. The piece shown in Fig. 35 exhibits some of the finest herringbone markings this author has ever encountered. At 40° F, the material had Charpy values of 16 ft-lb in the rolling direction, and 10 ft-lb in the transverse direction.

28. *Welded Steel Stack at a Generating Station, Chicago, Ill., November 1951*[71]

A crack about 15 ft long was discovered in a welded



(Welding Journal 32, 4 April 1953)

Fig. 30 Penstock failure. Proof hydrostatic pressure test produced a longitudinal brittle failure

steel stack that had been in use for about ten years. It extended through two sections or courses and through a 4- x 4- x $^1/_2$-in. T bar between the two courses. About $3^1/_2$ ft of crack was in the vertical weld of one course, then the crack branched out, V shape, into two cracks in parent metal across the balance of the course. The crack in the T bar and the other course was vertically below the crack in the weld mentioned above and was entirely in parent metal.

The height of this stack was 177 ft, 8 in. above its base, with a diameter of 9 ft, $11^1/_4$ in. inside of plates. The crack was in $^3/_8$-in. plates, from 95 to 110 ft above the base. The paving brick lining was 4 in. thick with 1 in. of cement between the brick and stack steel. For about two-thirds of its length the crack in the steel was $^1/_2$ in. wide, the balance was $^1/_4$ in., but diminished to a hairline at the ends. There was also a crack about $^1/_{16}$ in. wide in the bricks and mortar.

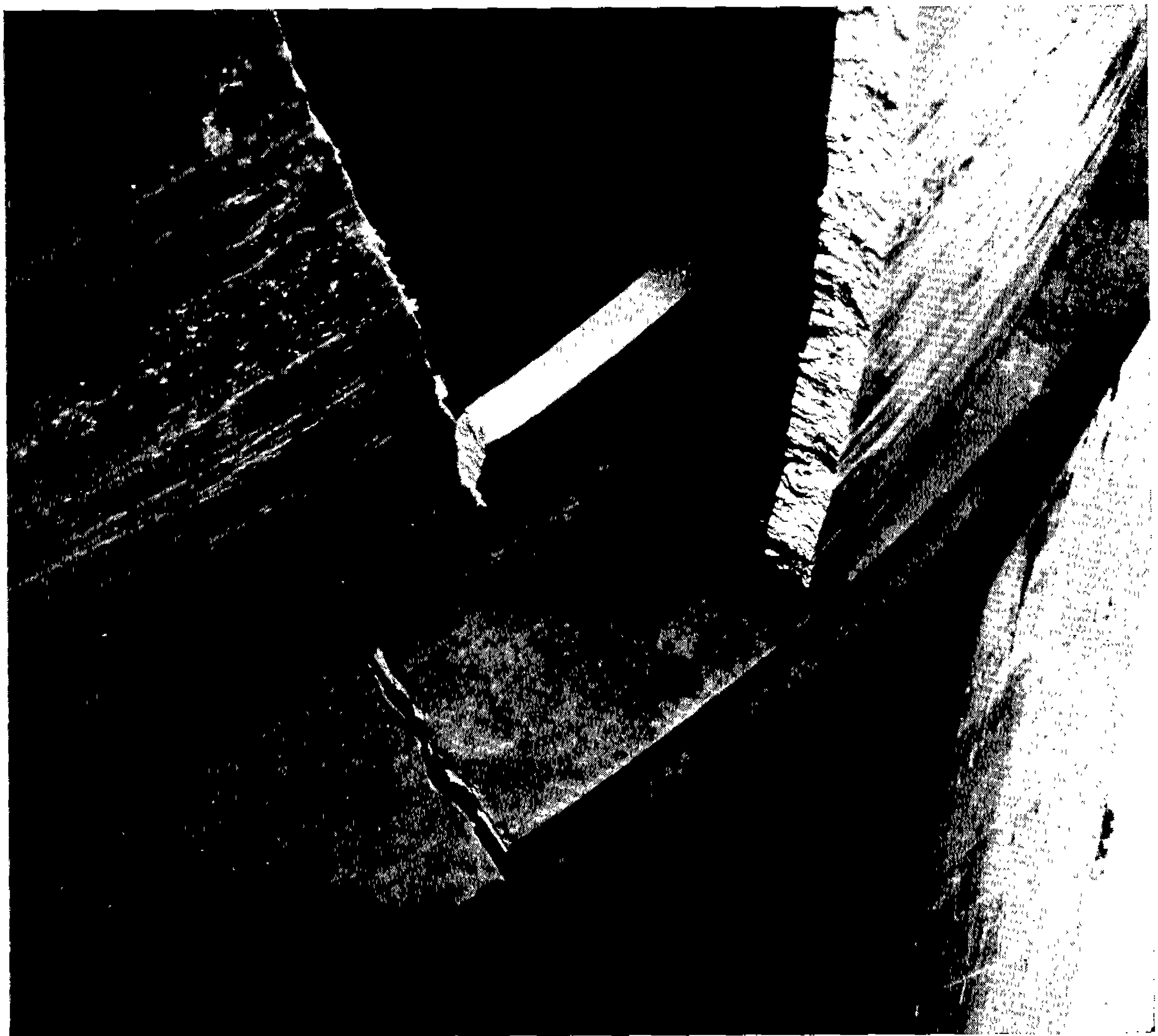
In the two weeks prior to the discovery of the crack there had been a drop in temperature to 12° F. In the same two weeks, the hourly average wind velocity had been from 2 to 24 mph, but maximum velocity was somewhat higher. The stack had been observed to vibrate when moderate winds had occurred from certain directions. Investigation showed the following:

1. The steel in the stack became brittle at low temperatures which existed shortly before the crack was discovered.

2. There was an increase in hoop stress in the steel shell due to temperature changes and expansion of the lining.

3. Wind and design conditions were suitable to cause oscillation of the stack and thus produce additional stresses.

The Charpy transition temperature was 20° F for the course where the crack was entirely in parent



(*Welding Journal 32, 4 April 1953*)

Fig. 31 Penstock failure. The fracture in the shell extended to the stiffener rings in both ends. The resistance of these rings to the propagation of the crack changed its course

(*Welding Journal 32, 4 April 1953*)

Fig. 32 Penstock failure

Investigation showed that the fracture originated at this girth joint, which had been welded in place in the tunnel. A secondary crack, branching off the main fracture, indicates a point of stress concentration which may have been due to an undisclosed defect in the weld repair.

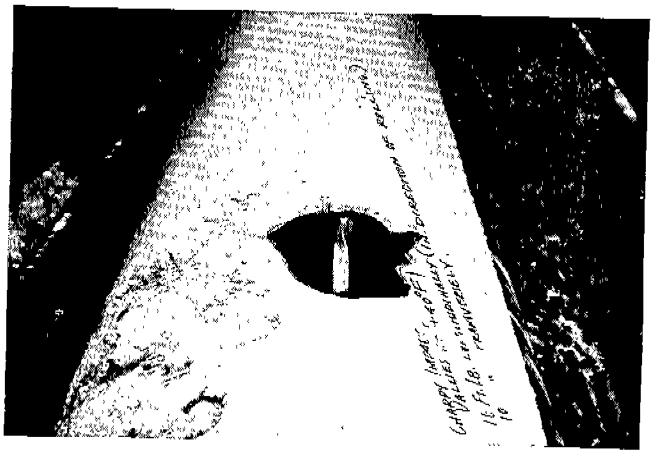


Fig. 33 Fractured drum involved in railroad wreck, showing hole punched in side. Drum is lying in gondola car and is covered with snow

Fig. 34 Fragments from fractured drum, showing how the steel shattered

metal and 35° F in the course where part of the crack was in the welded joint.

The steel had been shipped in 1940 and was in accordance with ASTM A-7 specifications.

Analysis showed the steel to have 0.28% C, 0.50% Mn, with usual amounts for remaining elements. The investigators believed the steel quality to be a major contributing factor in the failure, as other stacks of similar design but different steels were subjected to similar oscillatory and hoop stresses without failure.

29. Brittle Failures of Gas Transmission Lines, U. S. Circa 1948–51[72–76]

One of the most interesting situations that has come to light is that of failures on high-pressure gas transmission lines. Pipe for gas transmission lines is now usually produced under American Petroleum Institute Standard 5LX,[125] first issued in 1948. The allowable percentage check chemical analysis is as follows: C 0.34 max, Mn 1.30 max, S 0.065 max, P 0.055–0.110 (depending on method of steel manufacture. Maximum value is for killed, deoxidized bessemer steel). Ladle analyses, taken from the heat of steel during its manufacture, require slightly lower chemistry. The standard provides for three strength levels, with yield strengths and tensile strengths as follows:

Grade	*Yield strength, min, psi*	*Tensile strength, min, psi*
X-42	42,000	60,000
X-46	46,000	63,000
X-52	52,000	66,000

Fig. 35 Fragment from fractured drum. Note the excellent example of herringbone markings

Certain values of tensile elongation are also specified. How these physical properties are to be obtained is not specified by Standard 5LX. Mill test pressures, varying with pipe diameter and wall thickness, are set forth in detail.

An excellent description of one method of manufacture of this pipe (presumably the X-52 grade) was recently published.[75] In brief, the pipe is cold formed from sheet by several press operations. Following this, the entire 40-ft length of pipe is flash resistance welded, without addition of filler metal. Metal extruded from the flash weld is then trimmed. By means of internal hydrostatic pressure, the pipe is then cold expanded. This straightens and rounds it, at the same time raising the yield strength from about 44,000 psi to a minimum of 52,000 psi. Too much raising of the yield by cold working will lower ductility to the point where field bends cannot be made.

Raising of the yield point by cold expansion also has an important economic consequence. For instance, a 26-in. diam expanded pipe, having a 52,000 psi yield, operating at 700 psi gas pressure, has a wall thickness of 0.250 in. An as-rolled pipe, with a 44,000 psi yield, operating under identical conditions, must be 0.288 in. thick. This 0.038 in. thickness difference amounts to a weight difference of approximately 27 tons per mile. If a cost of $120 per ton is assumed, this means a saving of $1,500,000 in 500 miles of pipe line.[75]

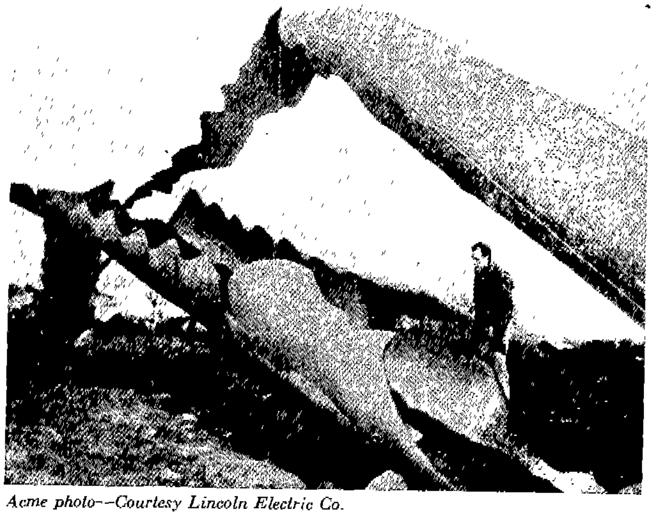
Following cold expansion, the pipes are hydrostatically tested to a stress of 80 or 90% of yield. During the hydrostatic test, the welds are struck with $6^{1}/_{2}$-lb hammer, placed at 2-ft intervals.[75]

Installation and allowable pressure in transmission lines are covered by an American Standards Assn. Code.[76] Under paragraph 807 (C, 1) of this code, in sparsely populated areas it is permissible to carry a pressure which stresses the pipe to 72% of yield strength.* In more densely populated areas, paragraph 807 (C, 2) allows a pressure which stresses the pipe to about 50% of yield.

There is not very much published information concerning gas transmission line failures. One short article[73] describes failures as varying from 180 to 3200 ft in length. The failures here described occurred on test, after installation. The cause (presumably the initiating cause) is stated to be well known—namely gouging or scratching of the plate in transit or installation. The failures always follow a sine wave pattern, and look as though there had been an internal explosion.[73] (See Fig. 36.)

A report contained in the Congressional Record[72] lists hundreds of pipe-line accidents arising from all causes. The information is necessarily rather sketchy, and little can be deduced. Of much greater interest is a report of the Federal Power Commission,[74] upon which,

* An optional provision for use prior to official adoption of the ASA Code, where operation pressure $P = 1.44\ Kt/D$. For electric resistance welded pipe, K is taken as equal to yield strength. t = thickness, D = pipe diameter. Comparison of this equation with the standard thin-walled cylinder equation gives the 72% figure noted above. After adoption of the code other alternative methods are listed for determination of working pressures. One of these provisions allows a working pressure of 80% of stipulated mill test pressure. As an example, in the case of a pipe made under API Standard 5LX, grade X-52, with diameter of 30 in., and wall thickness of 0.344, stipulated mill test pressure is 1020 psi. Eighty percent of this is 815 psi working pressure. The above formula would allow for this pipe a working pressure of 860 psi. The 80% provision, therefore, allows the pipe to be stressed at 68% of yield. The ASA code should be consulted for details. Current pipe line practice, however, seems to reflect use of the 72% of yield figure. This ASA code is now being revised.



Acme photo—Courtesy Lincoln Electric Co.

***Fig. 36** Failure of a 30-in. gas transmission line, showing sinusoidal nature of the fracture. The longitudinal welded seam is seen to be intact. Presumably this failure occurred on test*

apparently, the information in the Congressional Record is based. Data for the FPC report was gathered from 28 major pipe-line companies. Many categories of failures are defined, but of particular interest is what the Commission has termed a "split." In part, the report states, "a number of failures were reported under 'split' pipe. There are failures of the pipe itself, and not in the longitudinal weld. . . . Whatever the cause, where the pipe itself ruptured, such failures have been listed under 'split.'" In addition, the report points out that some failures in bends may have been "splits," but are listed under "bend" along with failures due to other causes, such as corrosion, etc. A table on the report then lists a total of 38 splits which occurred in operation, comprising 2.2% of all failures tabulated, and 30 splits which occurred on tests, comprising 1.8% of all failures tabulated. Thus 68 splits occurred. Details concerning these splits are unobtainable, and indeed much of the data were probably lost in subsequent repair and replacement of the pipe. It seems probable that some of these splits represented brittle breaks, but that others did not.

Because of the paucity of available information few definite statements can be made concerning brittle pipe-line failures. Apparently no technical details on any specific accidents have been released. Some interesting speculations may be made, however. One speculation concerns field welding to join sections of pipe. With the upper limits of chemistry allowed under API standard 5LX, it is possible that trouble may be encountered in field welding of girth joints, in that hardening and cracking might occur in the heat-affected zone. Secondly, the probability of failures initiated by gouging, as described above,[73] is a likely one. With all the handling that is required in the field installation of transmission lines, many possibilities arise for the introduction of defects that will serve as notches for initiation of brittle failure. The effect of the cold work and high chemistry in raising the ductile-to-brittle transition temperature will be considered under Discussion (page 38).

A last speculation concerns the rate of crack propagation in steel versus the rate of pressure release in natural gas (methane) following a pipe break. The gas pressure will be released by an elastic wave traveling at the speed of sound, approximately 1300 ft/sec. This figure is not affected by pressure, and assumes an ideal gas. Secondary compressibility effects (departure from ideal behavior) will not change it greatly.

In brittle fracture the steel is elastic to failure. A fairly recent[77] mathematical analysis considers a moving crack in an elastic solid. This analysis was performed for glass, but since the physical assumptions appear to be valid for the brittle failure of steel, it sheds light on the situation. In brief, it examines the behavior of a straight crack traveling at a velocity V, in a direction normal to the maximum tensile stress. If C_2 is the velocity of propagation of an elastic shear (transverse) wave in the material, there occurs at about 0.6 C_2 a critical velocity at which the crack tends to curve. At a velocity lower than 0.6 C_2 the crack travels in a straight line. As the speed increases, the crack may (but not necessarily) form branches. The original analysis concerns itself with a medium which is isotropic. In steel pipe the anisotropy may be of just the type required for the prevention of branched cracks. At velocities higher than 0.6 C_2, each branch tends to curve. For steel, the value of C_2, velocity of propagation of a shear wave, is approximately 10,000 ft/sec.*

For purposes of this analysis it may be considered that a pipe line is stressed in one direction only (tangential), and is of infinite length. A brittle crack, approaching a velocity of about 6000 ft/sec will thus tend to curve. This in itself may alter the stress field, perhaps slowing the crack, which will then tend to again run normal to the maximum tensile stress direction. The process may then repeat itself, resulting in a sinusoidal fracture. If the action sets up a symmetrical shear wave, this would keep the crack from spiraling the pipe. This repeating action would also limit the average crack speed to about 6000 fps If a branch crack is formed, then it may also behave in the same manner, resulting in perhaps more than one sinusoidal split.

Experimental values of 2750 to 3680 fps in one case,[78] and up to 6600 fps in another case,[90, 91] have been measured in brittle fracture of steel in the laboratory. Thus it appears that the gas discharge pressure wave will never catch up with the brittle crack. The tip of the crack is always traveling in a stressed area. This would account for the long breaks described above. As before stated, no published technical details of accidents are available to corroborate these speculations.

Field testing of pipe with water might tend to prevent long breaks, inasmuch as the velocity of an elastic wave in water is about 4800 fps. In the Anderson

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* $C_2 = \sqrt{G/\rho}$ where G = modulus of rigidity, ρ = density in mass units.

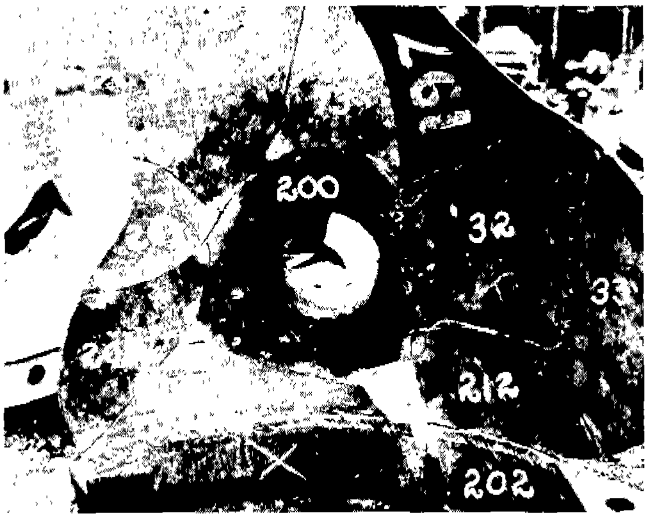

Fig. 37 Failed methane column

This shows the area about the upper 8-in. nozzle, where the defective weld was found between the nozzle wall and shell plate. This is located at the left edge of piece 32 joining piece 200.

Fig. 38 Failed methane column

Edges of pieces 32 and 200 were folded back to show the defective weld. The $^7/_{16}$-in. shell plate was not fused to the nozzle wall for a distance of 3 to 4 in. This weld was covered by a reinforcing ring fillet welded to nozzle and shell.

Ranch Dam penstock failure,[68] however, a 50-ft long crack appeared in a hydrostatic test, and appears to have been stopped only because of the deflecting action of stiffener rings. Gas lines have no stiffener rings, and it is a matter of speculation as to how far the penstock crack would have traveled had the design been different.

30. Methane Column, Eastern U. S., 10:55 A.M., Jan. 29, 1945[137]

This methane column was 43 ft high, 3 ft 7 in. ID, fabricated of firebox quality carbon steel plate. The shell was $^7/_{16}$ in. thick, with five courses joined by oxy-acetylene welding. The bottom consisted of a dished head, also $^7/_{16}$ in. thick, while the top head was flat, made of $3^1/_4$-in. plate. Following fabrication, the entire structure was annealed at 1100° F. The column was installed in 1930. The design stress was 6100 psi, with a gage pressure of 125 psi. The normal operating temperature of the column was −110° C at the top, and −70° C at the bottom. In May 1930 the vessel had been hammer tested while it contained a pressure of 159 psi. It was then tested at 250 psi with water and 188 psi with air. In 1939 the 250 psi hydrostatic test was repeated. Presumably, all these tests were at atmospheric temperature.

When this structure failed, 15 years after its installation, the steel shell broke into 125 fragments. All fractures had a brittle appearance, with no indication of reduction of area or elongation along the fractured edges. Following failure, a defective area was found in the weld of an 8-in. nozzle located near the top of the column. (See Figs. 37 and 38.) In addition, a second faulty weld was also noted in the liquid line near the bottom of the column. (See Fig. 39 and Fig. 40.) These defective welds were located in or near the areas of greatest stress.

In subsequent examination, the chemical composition of the steel was found to be within the composition limits for such steels, namely, C 0.15% max, Mn 0.35–0.60%, P 0.035% max, S 0.04% max. One line attached to the column, however, was found to be of Bessemer quality. In addition, tensile and notch impact tests were performed at room temperature and at temperatures down to −120° C. Tensile properties were quite normal, with the strengths increasing as the temperature was lowered, as would be expected. Izod impact values varied from 39 to 59 ft-lb at room temperature down to 1–3 ft-lb at operating temperatures. It was decided that the strength and ductility of the steels were normal in every respect, and that no apparent embrittlement had occurred since the column was placed in service.

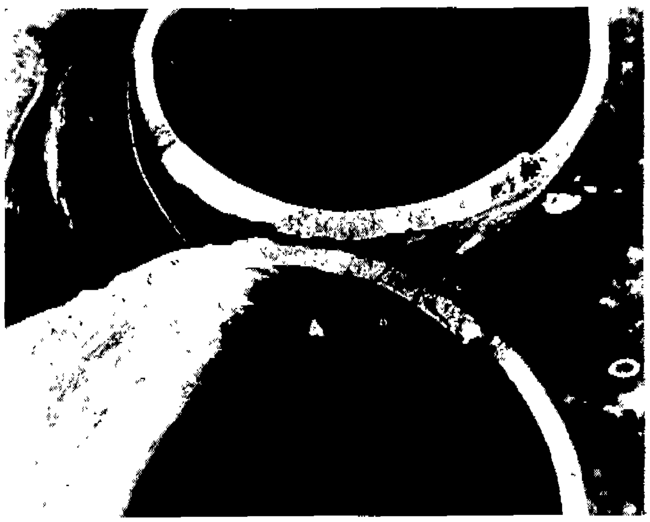

Fig. 39 Failed methane column

Faulty weld between 8-in. tube and pipe. Note lack of penetration and fusion at inside of joint. This was near the liquid line near the bottom of the column.

Fig. 40 Failed methane column

This shows break around another 8-in. nozzle which was a vapor connection near the bottom of the column. This opening was unreinforced, but none of the welds were broken as occurred in the top 8-in. nozzle.

Various investigators concluded (in part) that: (1) there was no evidence to indicate an explosion, since 90% of the fragments were found in a heap at the base of the column, and no appreciable damage was done to the structural steel supporting the shell. (2) Unfavorable factors were the notches present in the defective welds, and stress raising effects produced by side connecting openings. While the examination did not show definitely that either one of the defective welds triggered the failure the opinion was that they played a significant part. (3) The extreme notch brittleness of the steel at the operating temperature was most important.

In addition, several investigators believed that fatigue failure of the defective welds may have transmitted sufficient impact to the column to have caused the catastrophic fracture of the structure. While no data were presented as to the alternating stress conditions present, it is the opinion of this author that there is much evidence to show that brittle failure may readily take place in the presence of a notch, under static loading conditions.

DISCUSSION

1. Background of Early Research

From the foregoing histories it is fully demonstrated that brittle failure of steel structures is not of recent origin nor did it begin with the advent of welding. It is well to point out that research in brittle failure and notch brittleness is not new either. In 1884, Tetmajer[80] carried out repeated-blow impact bend tests on notched T-beams. In the United States, S. B. Russel published in the *Proceedings* of the American Society of Civil Engineers for 1897[81] an account of a new impact testing machine. Two years later, in the *Engineering News*[82] appeared an account of further work by Russel. This account concluded that the shock resistance of mild steel could not be predicted from tensile strength and elongation, and that in time impact tests of the sort demonstrated by Russel might become valuable in judging the quality of structural steel. Charpy[83] developed his pendulum testing machine in 1901 on an extension of Russel's ideas.

In all of this early work, however, all of the testing methods used to reveal brittleness employed impact loading. This supported the opinion, widespread even to fairly recent years, that brittle fracture in steels resulted from impact loading. It was, however, known that if a specimen contained a sharp and deep notch, brittle failure could then be induced by slow bending or slow tension. A. Mesnager,[84, 85] making use of this observation, in 1906 developed the theory of triaxial tension in notch brittleness. A lengthy discussion of the history of notch bar testing is not within the scope of this survey, however, and for further details the reader is referred to the monumental review by Fettweis,[86] which includes a bibliography of 700 references. For a concise development of the theories of brittle failure see a monograph by E. Orowan.[87]

2. Riveted Structure Failures

The earliest failures of riveted structures described in the present survey, Cases 1–3 inclusive, occurred in 1886, 1898 and 1904, respectively. Research work on notch-bar testing was developing over just that period, and though the members of the British Iron and Steel Institute, as before noted, had complained of brittle failures, in 1879 the practicing engineer seemed to be totally unfamiliar with the phenomenon. All three of these failures occurred in the colder part of the year. In all three cases the fractures were described as brittle or glasslike. Hard and brittle steel was suspected as the cause. In two cases, investigation of chemical composition and tensile strength was urged. In Case 3 the failure was correctly related to the cracks radiating from the punched-out rivet holes. This tank had stood seven years before failing, whereas the other two had failed during acceptance tests. The remaining failures of riveted structures occurred in one molasses tank and nine crude oil storage tanks, anywhere from 1 to 16 yr after erection.

Of particular interest is the fact that, in at least three cases of failures in riveted structures, the crack appears to have crossed one or more rivet joints in its passage. This is particularly noted in the accounts of Cases 1 and 6. At this late date the exact details of the crack paths are totally unavailable. A comparison with recent ship failures is of interest, however. Modified practice in the construction of welded ships requires the inclusion of several riveted, longitudinal crack arresters. These are similar to the butt straps used in nonship riveted construction. All plate welds terminate at a slot behind these arresters. Of the approximately 250 vessels which suffered serious failures, 77 were equipped with arresters. Of these 77 ships, 25 casualties did not in-

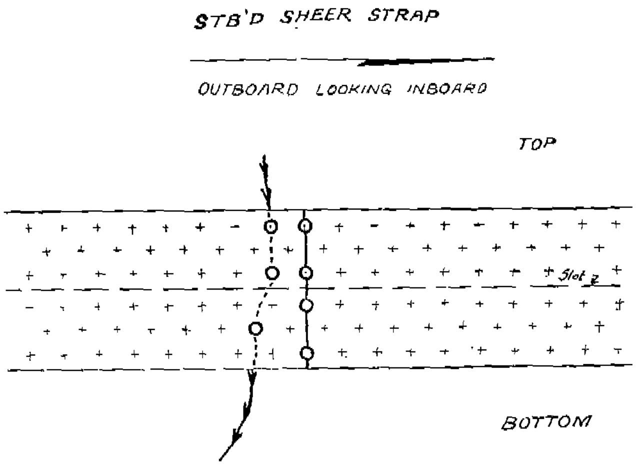

Fig. 41 Diagram of hull crack that crossed ship arrester in a nearly straight line. The arrester fracture is alongside the crack which crossed

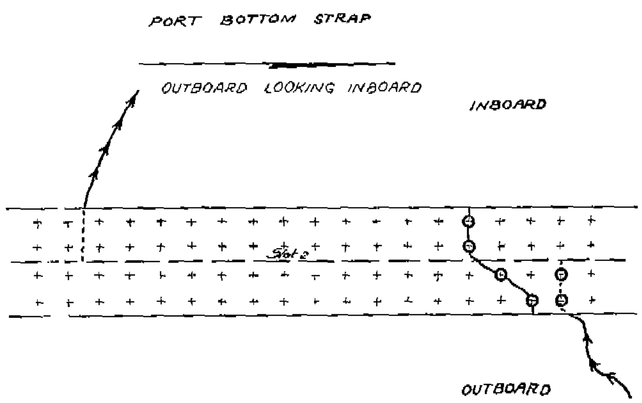

Fig. 42 Diagram of deck crack that crossed ship crack arrester. Entering and outgoing cracks were separated by 15 rivet holes. The arrester strap failed in shear in between

volve the arresters in any way, and 46 casualties involved cracks which were stopped by crack arresters. The remaining 6 vessels had cracks which restarted on the other side of crack arresters, the crossing of the arrester not necessarily being in a straight line. In some cases, there were as many as 25 rivet holes between the end of a crack on one side of the arrester and the start of a new crack on the other side of the arrester.[89, 138] It is possible therefore for a brittle failure to propagate across a rivet joint. Obviously, though, crack arresters in ships have been efficacious in preventing a large number of cracks from propagating to dangerous size.

The crossing of an arrester by a crack seems to take place by either of two mechanisms: (*a*) The incoming crack stops at the edge of the slot or at a rivet hole; the outgoing crack propagates from a notch on the other side of the slot, while in the meantime the strap fractures in cleavage or in shear. (*b*) The incoming crack is stopped, but a crack propagating on the other side in the opposite direction reaches the slot; finally the strap fails.[117] This second mechanism is not really a phenomenon of "crossing," but rather perhaps a termination of failure. There is every indication in the foregoing cases of ship failures that there was a time delay in the crossing of the arresters. Eye witnesses report that the delay varied from one second, in one case, to several hours in another.[117]

Figures 41 and 42 show plotted diagrams of the path of cracks across ship arresters. One crossing shown is in nearly a straight line, with the arrester fracture alongside. The other crossing shows a considerable distance between the entering and outgoing cracks. Figures 43 and 44 are photographs of cracks that crossed arresters.

3. *Comparison of Failure Incidence for Welding Versus Riveting*

Turning from riveted structures to welded structures, failures in the latter seem to have occurred more frequently. This may be a totally deceiving conclusion, however, since no basis for comparison exists. Structural methods have changed greatly in the intervening years, different types of materials are used, and no doubt many more welded structures are now in being than ever were built using rivets. In the past, furthermore, as has been pointed out, brittle failures have probably often gone unrecognized. In short, the sample examined in this survey is too small to permit of any statistical conclusions. For speculative comparative purposes, however, a recent excellent article[92] furnishes data on riveted versus welded ships. About 6000 ships built between 1938 and 1948 are used as a basis. Since 1938 there have been about four times as many welded ships built as ships with riveted hulls or decks. Data presented show that for the same material, and essentially the same quality of workmanship, both the frequency and severity of fractures in ships increased as the amount of welding increased. This fact must however, be considered against a background of wartime urgency in ship production.

4. *Effect of Thermal Stresses*

In 11 cases of welded failures here reported there had been a sharp atmospheric temperature change just prior to fracture. (This was true in five riveted structures also.) Two of these changes were rises to 30 and 10° F, still within the brittle transition range of many structural steels. The remaining changes were temperature drops. As would be expected, no data seem to be available on thermal stresses in tanks and pressure vessels. These stresses seem to be important in some degree, however. In ships[79] several shell failures occurred in tankers when oil in the tanks was being heated. Also, a small coastal vessel suffered a fracture in cold weather (0° F) when launched into warmer water (32° F). Refrigerator ships have had trouble in locations where all-welded decks were exposed in refrigerated (15° F) areas. Studies of thermal stresses in ships have been undertaken[93, 94] but of course results cannot be applied directly to the problems of pressure vessels, storage tanks and other nonship structures. In oil storage tanks containing warm liquids a sudden cooling of the exterior will obviously cause tensile stresses in the shell, and where the tank rests on warm ground, even further restraint will be induced. Thermal stresses in pressure vessels containing gases, or thermal stresses in bridges are probably more complicated and related to the rigidity of the structure. The failure of three empty tanks (Case 22) following a temperature drop is harder to explain, but may be related partly to the resistance to thermal contraction furnished by friction between the bottom and the warm ground. It is this author's opinion that thermal stresses in themselves, without additional factors (notches, defects, etc.) are probably not too important. The point bears further investigation.

5. *Effect of Residual Stresses*

On-the-spot investigators blamed residual stress in eight cases of nonship failure. Following the failure of the tanker *Schenectady* at dockside in January 1943, much controversy was stirred up over the role of residual stress in brittle failure.[92] Since then, however, many

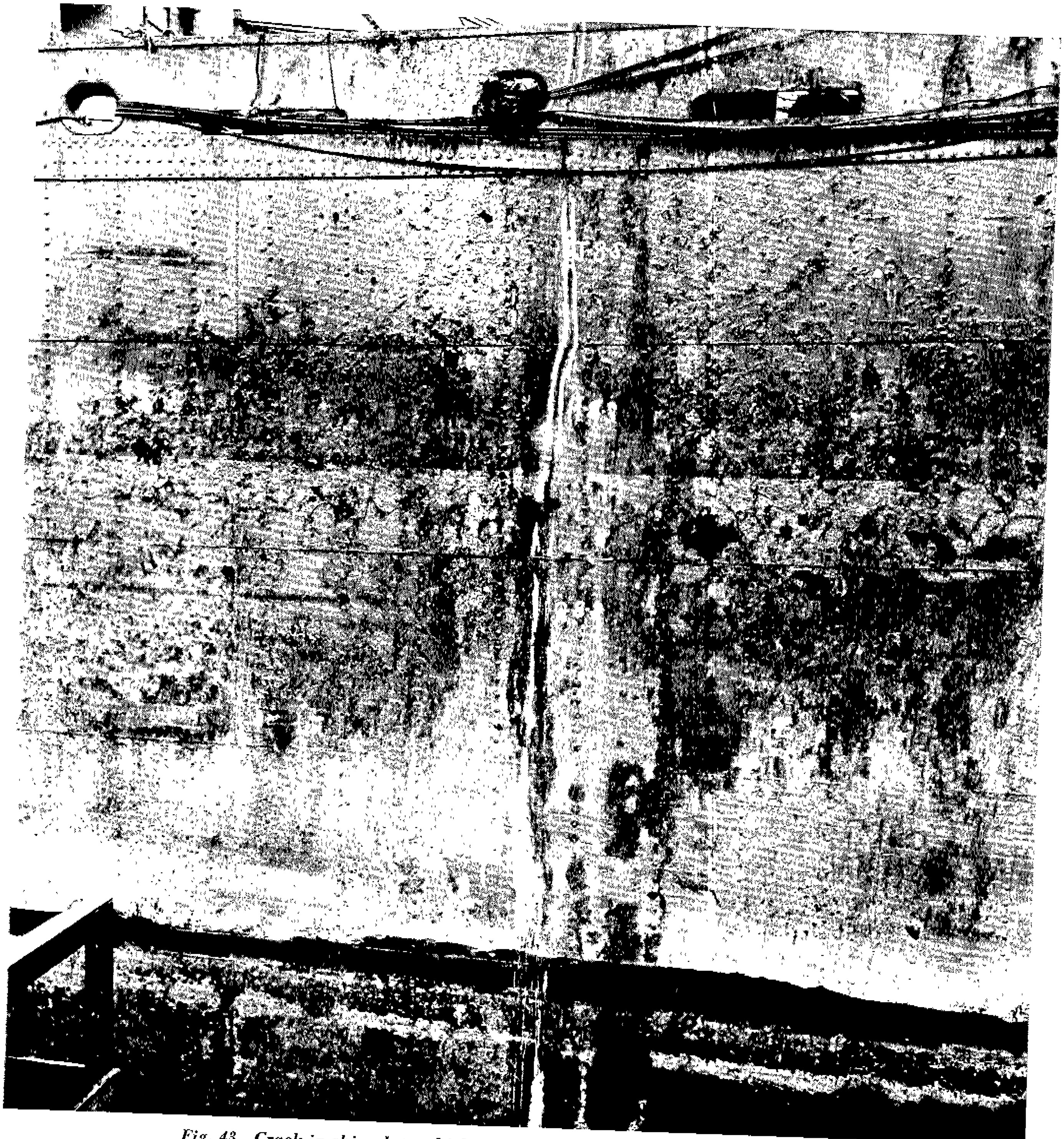

Fig. 43 Crack in ship plate which crossed an arrester in nearly a straight line

investigations have been performed to evaluate residual welding stresses in the butt welding of ship plate and locked-in stress* in ship assembly. Moreover, enough work has been done to show that the residual stresses in welds performed on fairly sizable (4 by 6 ft) plates will give good indications of stresses found in much larger structures.[96] In addition, the results of ship investigations indicated that the basic welding stress patterns were practically the same regardless of the type of ship or where it was built.[79] Thus results of these investigations can probably be directly applied to nonship plate structures, at least in qualitative fashion.

When a weld bead is laid down, the deposited metal solidifies and shrinks. It would thus be expected to be in a state of tension. This is in fact the case. Residual longitudinal stresses approaching the yield point in tension have been measured along the length of the centerline of butt welds in ship plates.[95, 96, 100] Values of transverse stress across the weld are low, about 2000 to 10,000 psi in tension.[95, 96] These results are found in tests performed on both actual ships and smaller plate samples. Stress values measured in automatic Unionmelt seams were found to be more uniform than those measured in hand-welded seams, otherwise the

* In ship reports it has been customary to define residual stresses as those resulting from the welding of unrestrained members. Locked-in stresses have been defined as including residual stresses, and stresses resulting from other assembly and fabrication procedures.

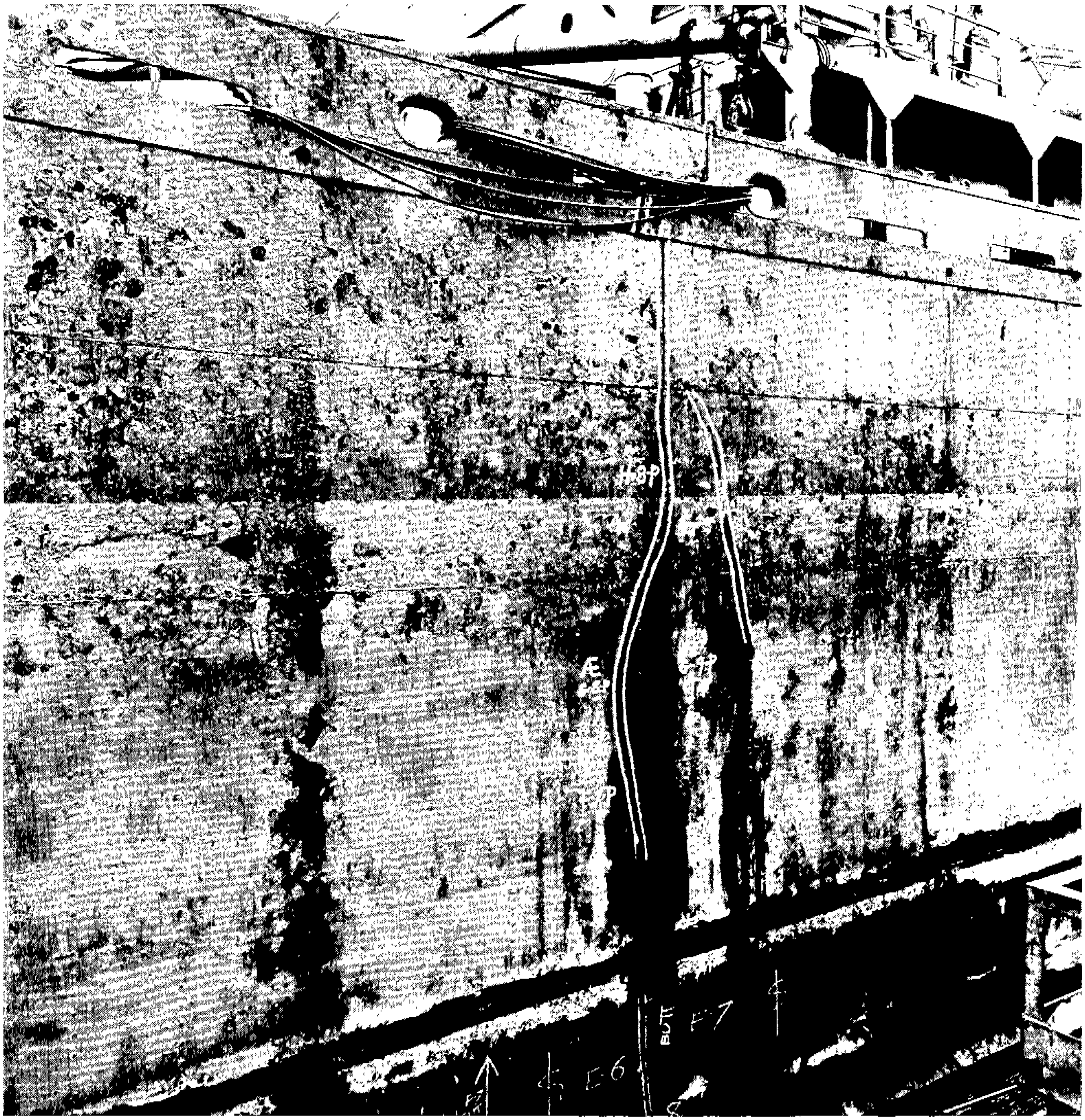

Fig. 44 Crack in ship plate which crossed an arrester, with spacing between the entering and outgoing crack

the stress levels were similar.[95] Naturally, a compressive stress must exist in the parent plate. Such stresses were found to vary up to 10,000 psi in compression about 4 in. from the weld and parallel to it.[96]

In unrestrained butt welds up to about 20 in. long in ship plate the maximum residual longitudinal stress is a function of the weld length. For welds longer than 20 in., this stress is zero at each end, rises to a maximum about 10 in. from each end, and remains constant throughout the remaining length of the weld. Regardless of length the transverse residual stresses are similar, being about 3000 psi compression near the ends, and about 10,000 psi tension in the central portion of the weld.[97] Residual stresses in welds can be decreased somewhat by block or stepwise welding sequence.[95,98] The use of austenitic electrodes will also reduce weld residual stresses slightly, but not enough to offset the cost.[99]

An additional factor enters, that of residual stresses remaining from the rolling or forming of plate. In one case rolling stresses in ship plate were found to approach 4000 psi in tension at the center of the plate, and 6000 psi compression at the faces.[95]

The next question to be considered is the relief of residual stresses in service. Does the stressing of a pressure vessel in service, or the loading and unloading of a bridge in traffic, for instance, cause yielding and relief of residual tensile stresses in a weld? The problem remains largely unanswered for nonship structures. In only one nonship structure (the Hasselt bridge) have residual stresses been measured. Values from 14000 psi up to the yield point were found, depending on the investigator.[39, 42, 43] In ships it has been found that the magnitude of locked-in stresses is not materially reduced by the working of the ship at sea.[93, 100] Thus, since all welded ships contained locked-in stresses, and these stresses are not reduced in service, and since only a fraction of ships suffer casualties, locked-in stresses are not, by themselves, the prime cause of ship failures.[101, 102] Likewise, most nonship structures continue to stand undamaged. Thus the statement of the investigator[38] (in reference to the Vierendeel failures) that residual stresses have no importance if the welds are sound has a good deal of truth in it.

In structures where defects exist (cases 7–15, 18, 20 and 21 for example) residual stresses must be reckoned with as being able to initiate failure, either by themselves or, as is more likely, in combination with other factors. The failure of three empty oil tanks (Case 22) as exemplified in Fig. 15 could have been initiated only by the presence of high residual stress in the weld. Undoubtedly, however, the full role of residual stress in helping to cause failure is not fully understood, even though there is evidence to show that stress relief will improve performance of materials in some measure.[103]

6. *Effect of Metallurgical Variables and Chemical Composition of Plate*

Fettweis,[86] in his 1929 survey, reports the results of early research workers concerning the effect of composition and cold work on notch toughness. The earliest of this work goes back to 1905. More recent work has confirmed, enlarged and established in much more exact fashion the facts known then.

For steels which are otherwise generally similar, a fully killed steel will have a lower ductile-to-brittle transition temperature range than a semikilled steel. Similarly, a semikilled steel has a lower transition range than a rimming steel.[104, 118] As can be seen from the failure histories, rimming steels had been used in at least 18 cases. In Case 14 the use of a semikilled steel did not prevent initiation and propagation of brittle failure. In Case 15, a crack running in rimming steel crossed a weld and continued through semikilled plate. Both of these cases were spherical pressure vessels. It cannot be implied, of course, that the use of killed steel will prevent failure.

Increasing ferritic grain size will raise the transition temperature range.[105, 106] In a very low carbon steel (0.02%) an increase of one ASTM number in ferritic grain size was found to raise the transition temperature range by 30° F.[106] Normalizing lowers the transition range[105, 119] by reducing the grain size, but slow cooling after normalizing will raise the transition temperature,[105] as will an increase in plate thickness.[105, 119] A lower finishing temperature in hot rolling also lowers the transition temperature,[114] no doubt because of a finer grain size.

A very careful assessment of the role of individual chemical elements was performed at the Naval Research Laboratories.[108] Special heats of killed steel were made. A base composition of 0.30% C, 1.00% Mn was used, and all specimens had a coarse pearlitic microstructure. Transition temperatures were measured by the intersection of the average energy line with the energy-temperature curve. For variation of individual elements, the following shifts in the transition temperature were found:

Al. . . . . .Lowers, then no change. Probably acts by decreasing the grain size, deoxidizing or tying up *N*.
B. . . . . .Increases, rapidly and regularly.
C. . . . . .Increases 5° F per 0.01% below 0.30%. Increases 6° F per 0.01% above 0.30%.
Cr. . . . . .Little effect.
Cu. . . . . .Raises slightly and decreases maximum energy.
Mn. . . . . .Decreases, up to 1.5%, at approximate rate of 1° F per 0.01%. Amounts smaller than 0.30% were not studied.
Mo. . . . . .Increases, almost as rapidly as C, and decreases max energy
Ni. . . . . .Slightly beneficial up to 1.80%.
P. . . . . .Increases, at rate of 13° F per 0.01%.
S. . . . . .Induces laminations, and in such cases increases the energy to failure.
Si. . . . . .Increases, at rate of 1.25° F per 0.01%. (This was later found to be valid for greater than 0.25% Si only. Below this figure Si decreases the transition temperature. See Reference 112.)
Ti, V. . . . . . . .First increases, then lowers. This may be because of the effect on carbides.

There was some additivity noted in the above figures. Another investigation, using semikilled steels, qualitatively confirms many of the above findings.[114] This latter work was performed on American Bureau of Shipping Class A and B steels. Nitrogen has also been found[111] to raise the transition temperature, but there is indication[131] that its effect may depend strongly on other variables.

The foregoing work[108] also considered the effect of the manganese and carbon together, and found that a high Mn/C ratio in itself will not lower the transition temperature. As an example, for two steels with the same Mn/C ratio of 1.5, a 0.67% C steel had a transition temperature of 232° F, while a 0.27% C steel had a transition temperature of 65° F. Other research[111, 135] work on the effect of the Mn/C ratio also indicates that a high ratio will of itself not necessarily promote a low transition temperature. There is, however, evidence to show that high Mn may be as efficient as grain refining in lowering the transition range, despite the fact that the grain-refined, low Mn steel may have a finer ferritic grain size than the steel with the higher Mn content.[110] One investigation, however, has found that a high Mn/C ratio is of importance in lowering the transition temperature.[109, 110]

In an investigation of fractured ship plates[112] at the National Bureau of Standards there was no readily evident relationship between failure incidence and the Mn/C ratio. Of all these plates, however, only one had a manganese content higher than 0.60%. This same investigation showed that for source plates (i.e., plates in which a fracture originated), the range and average value of carbon content was higher in each plate thickness group than for nonsource plates.

This same report[112] tentatively proposed a formula for the calculation of the 15 ft-lb transition temperature. Of 113 ship plates 96% had transition temperatures less than indicated by:

$$\text{Max 15 ft-lb transition temperature, } ^\circ\text{F} = 100 + 300 \times \%\text{C} + 1000 \times \%\text{P} - 100 \times \%\text{Mn} - 300 \times \%\text{Si} - 5 \times \text{fracture grain size number}$$

This formula, however, is not applicable for compositions including more than about 0.35% carbon, 0.10% phosphorus, 0.25% silicon,0. 25% copper or 0.2% molybdenum, chromium and arsenic combined, which may raise the transition temperature above the limit indicated.

The method of determining fracture grain size is similar in technique to the Shepherd method (*Metals Handbook*, 1948 ed., p. 405) for determining austenitic grain size. The fractured surfaces of Charpy bars, broken at a temperature low enough to give an almost completely brittle fracture, were compared with the fracture surface of standards for which the ferritic grain size was known. Assuming that the austenitic grain size is equal to the ferrite (plus pearlite) grain size in steels of this composition, it is then possible, using Shepherd's correlation between austenitic grain size and fracture grain size number, to assign a fracture grain size to each of the standards, and hence, by comparison with the standards, to each Charpy specimen of fractured plate.[113]

In view of the above, it would seem that the practice of using rimming steels in the past may have contributed to brittle failure in some cases. It also indicates that the practice of using higher carbon steels, either inadvertently, or deliberately to obtain high strength, may also have been contributory. This last may be an important factor in gas line failures. Similarly, high phosphorus is equally damaging. As noted in Case 29, it is permissible, in the manufacture of gas lines, to use steel with 0.34% C and 0.110% P. In general, the use of a high manganese content is to be recommended, but it should also be borne in mind that manganese is one of the most effective single elements in promoting hardenability (i.e., ease of forming martensite on cooling from above the critical range). Consequently, hardening and cracking can result following welding if the manganese content, in conjunction with the carbon content, is too high. Such cracking can serve to initiate brittle failure.

In the failure histories here reported, for structures in which the chemical compositions were known, the carbon contents varied from 0.09% C up to as high as 0.40% C, often with considerable variation within a single structure. If anything, this latter fact indicates lack of attention in the past as to steel compositions, at least insofar as the effect of composition on possible brittle failure is concerned. Manganese contents varied, in these failures, from very low values all the way up to 1.70%. It is interesting to note in Case 24 that the failure occurred with one of the lowest carbon contents and highest manganese contents considered. This points up that while composition is important, it is not the sole controlling factor. It also indicates that brittle failure can occur in service with a low-alloy steel at ambient temperatures.

7. *Effect of Cold Forming*

Cold forming of steel plate is a necessary part of the fabrication of almost all engineering structures. Two interesting pieces of work[107, 115] have been performed on steels commonly used for plate structures. These steels were ASTM-201 (killed) and ASTM A-70 (now A-285, rimmed). It was found that a tensile strain of 1% in the rolling direction raised the upper end of the keyhole Charpy transition temperature by about 20° F for the killed A-201, and by about 60° F for the rimmed A-70.[107] The large difference is probably due to the great susceptibility to strain aging in the rimmed steel. Normalizing at 1600° F consistently restored ductility and lowered the transition temperature. Heating to 1150° F was not consistently equally effective. Heating at 500 or 800° F only worsened the situation, presumably because of the strain aging.[107] Straining to 20% in the rolling direction raised the upper end of the transition range by about 80° F for both steels.[115]

Thus the initial cold working is most damaging in this rimming steel, so far as raising the upper transition is concerned, and more cold work has only a little more effect. In the killed steel, however, the upper transition is raised steadily and continuously by increasing cold work.[107]

No data seem to be available on the effect of cold forming on the transition temperature for steel taken from failed nonship structures. Even so the implications of the foregoing research work are clear. The work was performed on only two steels, and generalization of the results might be considered an overoptimistic extrapolation of the data. Nevertheless, extensive cold work, it seems, will tend to contribute to susceptibility to brittle failure. Outstanding examples of such cold forming in practice are the fabrication of pipe for gas lines, or the severe cold forming of pressure vessel heads.

8. *Effect of Welding Processes*

As shown by several laboratory investigations,[132-135] welding in itself contributes many metallurgical variables to the state of the metal in the weld and in the heat-affected zone. Moreover, behavior of the as-rolled plate gives no evidence of characteristics in the welded material.[135]

However, practically no data are available from failed nonship structures as to the details of welding procedures, such as types of rod, speed of welding, weather conditions, etc. As a consequence it is impossible to assess the role of metallurgical variables, resulting from welding, in the initiation of brittle failure.

In the case of five Russian oil tanks (Case 18) the tanks were erected and welded in extremely cold weather. This is known to produce weld deposits having reduced ductility and toughness. The practice is not permitted by present American Welding Society Codes. In Case 12, the Zoo and Rudersdorf bridges, light welds on heavy plating, with consequent quench cooling, were no doubt a factor in failure.

9. *Notch Bar Impact Values in Failed Plates*

In the ten cases where data are available for plates from failed nonship structures, the Charpy or Izod energy values are seen to be quite low at the temperature of failure. Examination of Table 1 shows the following:

(*a*) In 4 cases, the impact energy value at failure temperature was below 5 ft-lb.

(*b*) In 2 cases, the impact energy value at failure temperature was below 10 ft-lb.

(*c*) In 2 cases the impact energy value at failure temperature was below 15 ft-lb.

(*d*) In the remaining 2 cases the data are not in such form as to show the energy at the failure temperature.

No attempt at statistical interpretation can be made of so small a data sample. It seems to be in line, however, with data obtained in the investigation of fractured

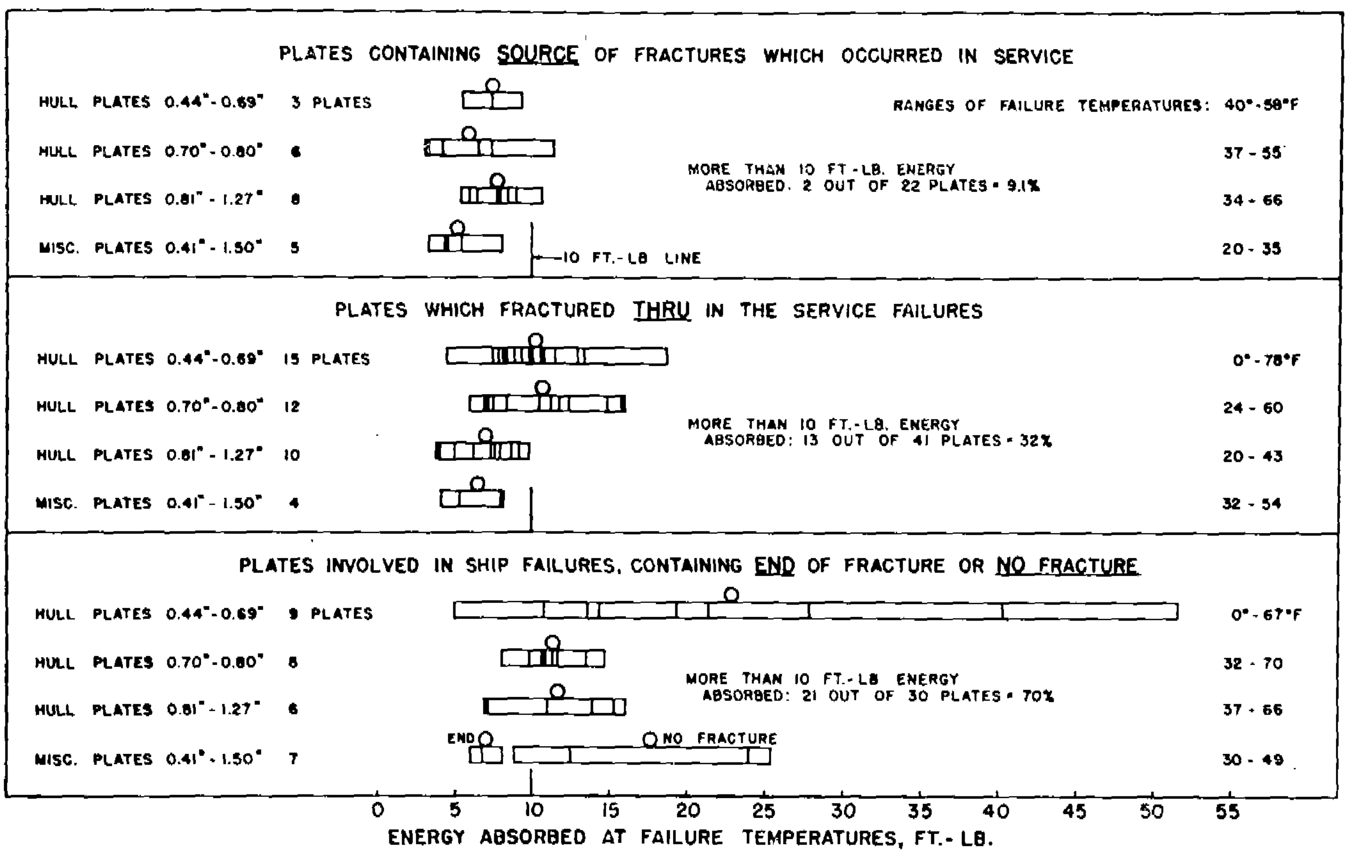

Fig. 45 Relation of energy absorbed by Charpy V-notch specimens at the temperatures of the ship failures to the nature of the fractures in ship plates. (From M. L. Williams, et al., Ship Structure Committee, NBS-3, "Investigation of Fractured Steel Plates Removed from Ships.")

Table 1—Condensation of Nonship Brittle Failure Data of Engineering Structures

(1) *Case no. and year*	(2) *Structure*	(3) *Details of structure*	(4) *Age ai failure*	(5) *Weather conditions or time of year*	(6) *Charpy, ft-lb*	(7) *Type of steel or partial chemical analysis, %*	(8) *Details of failure*	(9) *Failure*
1 1886	Standpipe	Rivet 225 ft high x 16 ft diam	Acceptance	October	.....	.....	Crack ran up 20 ft. Must have crossed rivet joints.	Complete
2 1898	Gasholder	Rivet 42 ft high x 178 ft diam	Acceptance	December	.....	.....	Fracture through body of plates.	Complete
3 1904	Standpipe	Rivet 80 ft high x 40 ft diam	7 yr	November	.....	.....	Tore through rivet holes. Many small cracks radiated from holes.	Complete
4 1919	Molasses tank	Rivet 50 ft high x 90 ft diam	3 yr	January	.....	.....	Fractured through manhole. Cracks radiated from rivet holes. Low safety factor.	Complete
5 1925	Crude oil tank	Rivet, weld 42 ft high x 117 ft diam	....	Temperature drop 64° F in 24 hr	.....	.....	Second course torn from first. Sheets torn up to roof.	Complete
6 1918–33	Crude oil tank, 1	Rivet, weld 55,000 bbl capacity	1 to 16 yr	Very cold	.....	.....	5 minor failures. Cracked plates and angles.	Partial
6 1933	Crude oil tank, 2	Rivet, weld 55,000 bbl. capacity	16 yr	−4° F	.....	.....	Failure started in weld angle going through plate.	Partial
6 1932	Crude oil tank, 3	Rivet 42 ft high x 171 ft diam	9 yr	Sudden temperature drop to −18° F	5 at 0° F, 1–2 at −25°, −50° F	0.29 C 0.42 Mn	Crack started at bottom, through 2 courses crossing riveted joint	Complete
6 1933	Crude oil tank, 4	Rivet 42 ft high x 171 ft diam	10 yr	Drop to −30° F	5 at 0° F, 1–2 at −25°, −50° F	0.17 C 0.51 Mn	Manner similar to Tank 3.	Complete
6 1937	Crude oil tank, 5	Rivet 40 ft high x 120 ft diam	4 or 5 yr	Sudden temperature drop to −22° F	.....	.....	Split in first course.	Partial
6 1933–34	Crude oil tank, 6	Rivet 40 ft high x 120 ft diam	....	Winter	.....	.....	Crack extending through lower course, through rivet joint and into next course.	Partial
6 1934–35	Crude oil tank, 7	Rivet 42 ft high x 117 ft diam	....	Winter	.....	.....	Several cracks in sheets on various occasions.	Partial
6 1934	Crude oil tank, 8	Rivet 40 ft high x 120 ft diam	....	Sudden temperature drop to −20° F	.....	.....	Complete failure. No details.	Complete
7 1943	Oil tank	Rivet 30 ft high x 114 ft diam	....	Winter	.....	.....	Crack in welded patch.	Partial
8 1938	Hasselt Vierendeel truss bridge	Welded plate 245-ft span	About 3 yr	Quite cold	.....	S & P high	Poor welding, high residual stress. Cracks starting from welds. Forcing of alignment in erection. Steel not susceptible to hardening in welding. Rimming steel.	Complete
9 1940	Herenthals Vierendeel truss bridge	Welded plate 200-ft span	3 yr	7° F	Upper transition varied from −40 to 68° F	0.09–0.17 C 0.43–0.94 Mn	Same as for Case 8.	Partial
10 1940	Kaulille Vierendeel truss bridge	Rolled sections 160-ft span	5 yr	7° F	.....	.....	Same as for Cases 8 and 9.	Partial
11 1941–50	14 bridges	.....	....	Low temperature a cause in 6 cases	.....	Rimming steel	All rimming steel. Full details of surrounding circumstances not given.	Not known
12 1936	2 Zoo bridges	Welded girder	6 mo	......	.....	St-52	Cracks in web and flange radiating from fillet welds, due to hardening and residual stress.	Partial
12 1938	Rudersdorf bridge	Welded girder, 17 spans totaling 3280 ft	....	Sudden temperature drop of 10° C	....	St-52	Same as Zoo bridge, stiffeners resulted in even more residual stress.	Partial
13 1951	Duplessis bridge	Welded, continuous plate girder deck, six 180-ft spans and two 150-ft spans	3 and 4 yr	−30° F	Flange, 3–6 at +100° F	0.23–0.40 C 0.04–0.116 S 0.30–0.33 Mn	Material ordered to ASTM A-7, but flange steel found later not to meet specs. Cracks present in girder before bridge sections left shop.	Partial and complete

Table 1—(*Continued*)

(1) Case no. and year	(2) Structure	(3) Details of structure	(4) Age at failure	(5) Weather conditions or time of year	(6) Charpy, ft-lb	(7) Type of steel or partial chemical analysis, %	(8) Details of failure	(9) Failure
14 1943	Spherical pressure vessel	38.5 ft diam, welded	3 mo	Temperature rise of 27° F in 7 hr. Temperature 7° F at failure	.....	Semikilled	Shearing cracks in manhole neck, combined with high residual stress in heavy welds, and thermal stress caused failure. Very little tear in welded seams.	Complete
15 1943	Spherical pressure vessel	Welded, 40 ft diam	....	March	.....	Rimming and semikilled steel	Plate split in hammer test. Tore across weld, starting at slight offset in joint.	Partial
16 1944	Spherical pressure vessel	40 ft diam	....	Temperature rose from 19° F right before to 30° F	Below 15 at 32° F	0.20 C 0.47 Mn ASTM A-7	Failed on hydrostatic test at twice working pressure. Only 4 ft of total 350 ft of tear along seam	Complete
17 1944	Cylindrical pressure vessel	42 ft high x 70 ft diam, insulated, to operate at −260° F	1 yr	......	3–5 at −248° F	$3^1/_2$% N 0.08–0.12 C	Split and shattered. Killed 128 persons. Apparently the Charpy specifications had not been met.	Complete
17 1944	Spherical pressure vessel	57 ft diam, insulated to operate at −260° F	3 yr	......	.....	$3^1/_2$% N 0.08–0.12 C	Failed when legs collapsed due to heat from above failure.	Complete
18 1947	5 oil tanks	Welded, 160,000 cu ft capacity	4–6 yr	Temperature suddenly dropped to range of −31 to −47° F	.....	0.13–0.20 C 0.35–0.60 Mn	Poor welds. Innumerable cracks in all 5 tanks starting in weld defects and heavy welds. Thermal stress important. Welding done in extremely cold weather.	Partial[5]
19 1947	Crude oil tank	Welded 48 ft high x 120 ft diam	3 yr	Temperature dropped from 42° to 0° F. Oil at 43° F, being pumped in.	3–4 at 0° F. 19–40 at room perature	0.11–0.28 C ASTM A-70, A-10, A-7 0.44–0.49 Mn	Failure originated at cleanout door corner. Poor welds.	Complete
20 1950–51	Oil tank	Welded 10,000 cu m capacity	....	Winter	.....	0.12–0.19 C	Very poor welds initiated cracks	Partial
21 1952	Crude oil tank	Welded 54 ft high x 140 ft diam	Acceptance	30° F, water at 40° F	3–9 at 32° F in lower 2 courses	0.16–0.21 C 0.55 Mn	Failure initiated at poor weld probe replacement in 1st horizontal joint, had prior partial failure.	Complete and partial
21 1952	Gas oil tank	Welded 150 ft high x 48 ft diam	Acceptance	47° F, water at 40° F	6–10 at 32° F in lower 2 courses	0.22–0.25 C 0.54–.62 Mn	Failure started from partially repaired crack in weld.	Complete
22 1952	3 oil tanks	Welded 45 ft high x 144 ft diam	New	Temperature fell to −40° C	15 at +10 to −10° C	.....	Cracks initiated from fissures left from hammering and chipping, aided by residual stress. Tanks empty at failure.	Partial[3]
23 1951	Water tank	Welded 30 ft high x 144 ft diam	11 yr	......	.....	.....	Oil tank, re-erected to hold water. Lighter than permitted by AWWA Code. Very poor welding.	Complete
24 1952	Dipper stick	Welded tube, 20 in. diam	....	−15 to −20° F. Others at 0 to 32° F	0.12	0.12 C 1.25–1.70 Mn	Failed at stress concentration. Not related to welds.	Complete
25 1949	Boom	Welded, rectangular 16 by 20 in.	....	−20° F	.....	ASTM A-7	Poor weld initiated failure.	Complete
25 1949	Dipper stick	$2^1/_2$-in. solid plate	....	Cold	.....	ASTM A-7	Initiated by stress concentration Not related to welds.	Partial
26 1950	Penstock	20 ft diam, welded	Acceptance	41° F	.....	ASTM A-285 0.22 C max 0.80 Mn max	Fracture initiated in repair weld. Crack traveled 50 ft. Heavy irregular beads of weld metal deposited	Partial
27	Miscellaneous Items: Old gas cylinders and a tank	.....	....	......	.....	.....	....	Complete

Table 1—(*Continued*)

(1) Case no. and year	(2) Structure	(3) Details of structure	(4) Age at failure	(5) Weather conditions or time of year	(6) Charpy, ft-lb	(7) Type of steel or partial chemical analysis, %	(8) Details of failure	(9) Failure
28 1951	Stack	Welded 178 ft high x 10 ft diam	10 yr	Cold	Transition temp. 20 to 35° F	0.28 C 0.50 Mn	Thermal stress and vibration helped initiate a long Y-shaped crack.	Partial
29 1948–1951	Gas lines	Welded up to 30 in. diam	....	......	.....	ASTM A-7 1.30 Mn max 0.34 C max 0.110 P max	Yield raised by cold work. Operate at 72% of yield. Cracks initiated at gouges or other defects. Sinusoidal split, up to 3200 ft in length. Perhaps 68 splits. Crack travels faster than gas pressure wave lowers the stress. Little published data.	Partial and complete. No numbers available
30 1945	Methane column	Welded, 43 ft high x 4 ft diam	15 yr	......	1–3 ft-lb at operating temperature	Firebox	Low carbon plate being used at −110° C. Defective welds.	Complete

plates from welded ships,[112] noted above. This ship plate investigation divided plates into three categories: (*a*) Source—a plate in which fracture originated, (*b*) through—a plate through which a fracture traveled, (*c*) end—a plate in which a fracture terminated. The highest value of impact energy for a ship source plate was 11.4 ft-lb at the temperature of failure. Of 22 source plates only 2, or 9.1%, had energies over 10 ft-lb. For end plates, or plates with no fracture, 21 out of 30 plates, or 70% had over 10 ft-lb. These data are shown in Fig. 45. Further, of 31 plates which were fracture sources only 10%, or 3 plates, had 15 ft-lb transition temperatures below 70° F. Of 82 plates which did not contain fracture sources, 67% had 15 ft-lb transition temperatures below 70° F. The report is a most valuable one. Its perusal is most highly recommended to those interested.

In considering results of notch bar tests, it is interesting to note in passing, that poor quality, dirty steels often have higher impact values than supposedly good steels. This was noted by Mesnager[85] who, in 1906 observed that imbedded inclusions cause individual metal laminations to separate from each other, preventing a brittle crack from traversing the specimen. He also noted the fact that gas holes and other faults caused a similar effect. Fettweis[86] stated in 1929 that faulty material can have a higher impact resistance than sound material. The action of high sulfur in inducing laminations leads to the same effect.[108, 114] Wrought iron owes its toughness to its highly laminated structure.

Another interesting fact is that fractures which are mainly cleavage can be obtained in the laboratory with high values of energy absorption.[105] This is probably related to the fact that cleavage fractures can be propagated with a velocity as low as 150 fps.[91]

10. The Role of Cracks, Stress Concentrations and Other Defects in Initiating Failure

In nonship structures for which data are available the preponderance of failures have been initiated in both riveted and nonriveted cases at cracks left by punching or shearing, at plate offsets, weld voids, poor weld probe replacements, poorly repaired welds, and other defects resulting from improper fabrication procedures. Two cases (Nos. 24 and 25) were initiated by the effect of stress concentrations designed into the structure. Modification of the design in these structures seems to have eliminated subsequent failures. Other cases (Nos. 4 and 19) seem to have been initiated at a combination of fabrication and design defects.

In the case of welded ships built during World War II however, fractures often originated at points where poor welded design practice had been utilized (i.e., sharp hatch corners in the Liberty Ships).[102, 121, 138] Following design modifications, on the other hand, the origins of most recent failures in these ships have been traced to defective workmanship.[138] It was also concluded for ships that every fracture investigated could be traced to a starting point at a definite geometrical discontinuity due to design or workmanship.[102] While the data are not complete for nonship structures, it would appear that the latter conclusion is equally valid here.

The importance of workmanship cannot be overemphasized. One ship is known to have failed as the result of fracture initiating at such a small thing as a crater left by an arc strike.[112] Equally small defects are seen to have initiated nonship failures.

11. *Crack Paths*

Unless a weld is exceptionally bad, as in Case 23, there is no tendency for brittle cracks to follow welded seams. An outstanding example of this is Case 14.

12. *Static Versus Impact Stresses in Initiation of Brittle Failure*

The historical development of the notched-bar impact test (see paragraph 1 above) has led to the association of brittle failure with impact. In the nonship failures here reported, only 5 (Cases 24 and 25 which are power shovels, Case 27 which was a rail wreck, and Case 15 initiated by the hammer test) can be definitely connected with the phenomenon of impact. In the case of ships, 23 or about 10% of the 250 very serious failures (see Introduction, page 4) occurred at dockside, or in a calm sea.[79]

Brittle failure can apparently occur in the presence of static loading if the proper conditions of temperature, triaxiality (notches or defects) and stress are present.

13. *Age at Failure, and Degree of Failure*

From the figures presented in Table 2 it does not seem that age of the structure has any bearing on the occurrence of brittle failure. In ships this same conclusion was found to be statistically valid.[102]

For 50 structures (excluding gas lines) for which complete data are available, 22 suffered complete collapse due to brittle fracture, and 28 suffered partial failure. (See Table 2.)

14. *A Glance at Codes and Specifications*

It is not within the scope of this survey, nor is it the intention of this author to pass judgment on codes and specifications. Codes and specifications are usually the product of long and careful deliberation, conservatively based on experience in service. A critical survey of brittle failure, however, would be in some degree lacking in orientation if it did not take cognizance of some of the codes under which engineering structures are often fabricated, or some of the specifications under which materials are usually purchased.

The ASME and API-ASME unfired pressure vessel codes[122, 123] allow use of quite a variety of steels made in accordance with ASTM specifications.[128] Among other types, A-201, A-212, A-283, A-285 are allowed. Type A-7 is allowed by both these codes, but certain restrictions are put on its use. All of these steels can be used at temperatures down to $-20°$ F, with no impact toughness tests required. An American Water Works Assn. Code[126] allows, among other steels, types A-7 and A-285. API Standard 12C for welded oil storage tanks[124] calls for types A-7 and A-283, grades C or D. Naturally, oil and water tanks operate at ambient atmospheric temperatures.

Under the ASTM standards, for example, types A-7 and A-285 can be furnished rimmed, semikilled or fully killed. Specifications for type A-7 (Steel for Bridges and Buildings) set limits on phosphorous and sulfur only, and the former can be as high as 0.138% for acid Bessemer steel. In type A-212, for instance, the carbon content can be as high as 0.35% in steel plate 2 to 6 in. thick. The high values of carbon and phosphorus al-

Table 2—Summary of Nonship Brittle Failures of Engineering Structures (Riveted or Welded) Reported Herein

(Includes repeated partial failures of a single structure except where noted. Riveted oil tanks with welded base angles are included as riveted structures)

1. Total failures:
   Other 66
   Gas lines Unknown
2. Number of failures based on lifetime of structure (not including gas lines):

	Rivet	*Weld*	*Total*
(*a*) On acceptance test	2	3	5
(*b*) 0–1 yr	1	10	11
(*c*) 2–5 yr	2	13	15
(*d*) After 5 yr	7	3	10
(*e*) Not known	1	24	25
	13	53	66

3. Number of failures which occurred after sharp atmospheric temperature changes:

Rivet	6
Weld	11
	17

4. Number of welded failures where residual stress was deemed by investigators to be of importance: 8
5. Number of failures (excluding gas lines), that were:

	Rivet	*Weld*	*Total*
Complete	8	15	23
Partial	10	19	29
Unknown	0	14	14
	18	48	66

6. Breakdown of riveted structures which failed totally or partially (not counting repeated partial failures):

Standpipes	2
Gas holder	1
Molasses tank	1
Crude oil storage tanks	9
	13

7. Breakdown of welded structures which failed totally or partially (not counting repeated partial failures):

Bridges	21
Spherical pressure vessels	4
Cylindrical pressure vessel	1
Oil storage tanks	12
Water tank (converted from oil)	1
Power shovel dipper sticks and booms	3
Penstock	1
Stack	1
Gas lines	Unknown
Miscellaneous	2
Methane column	1
	45 plus

lowed in gas transmission lines under API Standard 5LX[125] have already been discussed. At additional expense, certain of the foregoing ASTM steels (A-201, A-212) can be purchased under ASTM Specification A-300, which calls for a minimum Charpy V-notch impact value of 15 ft-lb at some specified temperature. This latter requirement is mandatory only for service conditions below $-20°$ F.

Both the API 12C and AWWA codes allow partial penetration of horizontal welded joints in cylindrical tanks. This is permitted in square-groove and double-beveled joints providing that the unwelded portion is located substantially at the center of the thinner plate, and that the unwelded portion, plus any undercutting, does not exceed one-third the thickness of the thinner plate. In a cylindrical tank, the horizontal joint is a region of secondary stress, and such practice, so far as this author knows, has never led to any mishap. In fact, one manufacturer of tanks and pressure vessels has stated to this author that his company will make full penetration joints in such cases only if requested. He states that a full penetration joint causes the seam to draw inward, making the tank unsightly.

Because of the high incidence of ship failures, the American Bureau of Shipping (ABS) in 1947 established new specifications for structural steel for hulls.[127] Under these specifications, all hull plate steel of $^1/_2$ to 1 in. in thickness (Class B) must have a carbon content of 0.23% maximum and a manganese content of 0.60–0.90%. Steel over 1 in. thick (Class C) must have a maximum carbon content of 0.25%, with 0.60–0.90% manganese and 0.15–0.30% silicon. Further, Class C steels must be made to fine grain practice. This, in effect, excludes rimming steels in larger ships, and requires a fully killed steel in heavy plate. Plate less than $^1/_2$ in. thick (Class A) is limited only in phosphorus and sulfur contents. This last recognizes the fact that there have been no recorded failures in small ships which are built of lighter plate.[92] Several industrial organizations which submitted failure reports to this survey have stated their intention in future to use ABS Class B and Class C steels for such varying structures as oil tanks, power shovels and smoke stacks.

It can be shown that for a given initiating defect brittle fracture requires a certain critical value of the applied tensile stress.[120] Therefore, it is well to glance at design stresses allowed by some codes. The pressure vessel codes[122, 123] usually allow a design stress of 25% of the ultimate strength, except for certain steels for which about 21% of the ultimate is used. The American Water Works Assn. Code[126] allows a maximum design stress of 15,000 psi, regardless of ultimate strength. The end result of either of these two methods is a working stress of about 50% of the yield point. Such values are, perhaps, conservative, but they do not place a premium on cold working to achieve strength, a practice which can promote susceptibility to brittle failure. This is not the case in the ASA code for gas transmission piping[76] which allows in certain circumstances a working stress of 72% of yield, after the yield strength has been obtained by cold work. On the other hand, as has been pointed out[130] research may demonstrate the suitability of low alloy steels which can be safely stressed to a figure of 75% of yield in nonship structures.

The foregoing merely serves to point up some of the difficulties of design and of steel selection and use in nonship engineering structures. A very able exposition of this problem (in regard to pressure vessels) is presented in an interpretive report[129] by H. C. Boardman. To further show the difficulties of the problem, it is well to point out that while brittle failures have occurred in structures built under API and pressure vessel codes, there is no known recorded failure of a structure built under the AWWA code.

CONCLUSIONS

It is not the function of this paper to propose a remedy for brittle failure, nor to evaluate techniques of fabrication and manufacture of nonship structures. The task at hand was to survey nonship brittle failures of carbon plate steel structures and determine the factors of importance relating to such failures, in order to supplement the study of the failure of ships. The following conclusions seem justified:

1. Based on the examination of nonship failures, it is concluded that brittle failure in nonship carbon plate steel structures is the same phenomenon as occurs in ships. This may seem to be an obvious statement, but one that should be made. Moreover, brittle failures affect a wide variety of different types of nonship plate structures.
2. Brittle failure of steel did not originate in the era of welded construction. Its history goes back to 1879 or earlier. Riveted structures suffered brittle failure, with fractures originating at cracks radiating from rivet holes, or other defects.
3. As in ships, brittle fractures in nonship structures can, on occasion, cross riveted joints.
4. There is no evidence available to demonstrate that the percentage incidence of brittle failure in nonship structures has either increased or decreased with the advent of welding. Certainly it can be said, however, that brittle failure in welded structures, once initiated, can travel across welds with ease, thus perhaps causing a greater extent of damage in a welded structure.
5. In certain circumstances, thermal stresses (in conjunction with other factors) may be of importance in initiating brittle failure in nonship structures.
6. Residual or locked-in stresses, by themselves, are not the prime cause of nonship brittle failures. It is probable that, as in ships, residual and locked-in stresses are not relieved in service. In conjunction with other factors, however, residual stresses may serve to initiate failure.

7. The effect of metallurgical variables in brittle failure is important. Increasing the ferritic grain size, carbon and phosphorus contents (also certain other elements) and plate thickness will increase susceptibility to brittle failure. Increase of manganese and application of normalizing will lower the susceptibility. Killed steel, in general, is less susceptible to brittle failure than rimmed steel. Other, more subtle metallurgical variables are also important. However, in nonship structures that have suffered brittle failures, there was a wide variety of chemical analyses, types of steel and thicknesses of plate. One structure that failed at ambient temperature was made of low-alloy steel. The data do not permit statistical conclusions.

8. Cold forming promotes susceptibility to brittle failure, but there are no data to show the role that it has played in actual failed structures.

9. No data exist to show in general the effect of various welding processes on the initiation of, or susceptibility to, brittle failure in nonship structures. In five cases, the structures were welded at very low temperatures, and in two other cases, light welds on heavy plating were a factor.

10. In failed nonship structures, where data are available, the Charpy impact values were generally low at the failure temperatures. The sample is too small for any statistical interpretation.

11. For cases of nonship brittle failure where data are available, the great majority of fractures originated at defects arising from fabrication. In only two cases did fractures originate solely in design stress concentrations. Despite the lack of complete data it seems probable that in all nonship cases (as in ships), brittle failure originated at some definite geometrical discontinuity involving design or workmanship.

12. Except in the case of exceptionally poor welds, there is no tendency for brittle cracks in nonship structures to follow welded seams.

13. The great majority of brittle failures of nonship structures apparently take place under conditions of static loading. In only five nonship cases of brittle failure here reported were there clear indications of impact loads.

14. As in ships, age of the structure seems to have no bearing on the brittle failure of nonship structures. There is, however, no broad statistical basis for this conclusion.

15. At the present time, most engineering codes permit the use of steels which are known to have particular susceptibility to brittle failure. At the same time, all of these codes but one specify very conservative stress levels, which would tend to decrease the possibility of brittle failure. It should be stated, however, that it is not the intention of this survey to pass judgment on any codes or specifications.

Finally, it has been shown that brittle failure results from a combination of many factors. There is, at the present time, no material readily and economically available which would, if built into bridges, pressure vessels, and other nonship structures, totally prevent brittle fractures. Moreover, there is no known test which will surely predict from the behavior of small specimens the performance of a given structure in circumstances where brittle failure might occur. In short, careful design, selection of materials and good workmanship are of the greatest importance in the prevention of brittle failure in nonship carbon plate steel structures. This is also the case in ships.

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Bibliography

1. "The Water Tower Failure at Gravesend, Long Island," *Eng. News*, **16**, 264 (Oct. 23, 1886).

2. "The Failure of a Large Gas Holder in New York City," *Ibid.*, **40**, (25), 406–407 (Dec. 22, 1898).

3. Sherman, C. W., "A Standpipe Failure at Sanford, Maine," *Ibid.*, **52** (22), 507–508 (Dec. 1, 1904).

4. "Disastrous Explosion of a Tank of Molasses," *Sci. American*, **120** (Feb. 1, 1919).

5. "Bursting of Molasses Tank in Boston Charged to Bad Design," *Eng. News-Rec.*, **82** (7), 353 (Feb. 13, 1919).

6. Brown, B. S., "Details of the Failure of a 90 Foot Molasses Tank," *Ibid.*, **82** (20), 974–976 (May 15, 1919).

7. "Boston Molasses Tank Trial: The Case for the Defense," *Ibid.*, **85** (15), 691–692 (Oct. 7, 1920).

8. "Experts Deny Bomb Caused Collapse of Boston Molasses Tank," *Ibid.*, **87** (9), 372–373 (Sept. 1, 1921).

9. "Bursting of Boston Molasses Tank Found Due to Overstress," *Ibid.*, **94** (5), 188–189 (Jan. 29, 1925).

10. "Oil Tank Fails after Sudden Drop in Temperature," *Ibid.*, **94**, (16), 638–639 (Apr. 16, 1925).

11. Privately communicated industrial report, undated, received May 7, 1953.

12 "Effondrement du Pont de Hasselt," *Ann. inst. tech. bâtiment et trav. publ.*, **3** (2), 39–46 (Mar.-Apr. 1938).

13. "Effondrement du pont de Hasselt, sur le canal Albert en Belgique," *Bull. tech. Suisse Romande*, 153–154 (May 21, 1938).

14. "The Belgian Bridge Which Collapsed," *Electric Welding*, **40**, 98 (April 1938).

15. "Hasselt Bridge—Failure Not Due to Imperfect Welds," *Welder*, **53**, 124 (April 1938). (Translated from *L'Ossature Metallique*, No. 4 (1938)).

16. Gerbeaux, H., "A propos du pont soude de Hasselt," *Rev. soudure autogene*, **30** (289), 371 (Apr. 1938).

17. Francois, E., "La Construction soudee—Impressions consecutive a l'accident du pont de Hasselt," *L'Ossature Metallique*, **7** (5), 201–207 (May 1938).

18. "Collapse of a Welded Bridge," *Ry. Gazette*, **68** (20), 984–985 (May 20, 1938).

19. "La rupture du pont de Hasselt," *L'Usine*, No. 21, 37–39 (May 26, 1938).

20. Schleicher, F., "Einsturz einer Brücke uber den Albert-Kanal," *Bauingenieur*, **19** (21/22), 334–335 (May 27, 1938).

21. "La rupture du Pont de Hasselt (Belgique,) *Jnl. soudure—Z. Schweisstechnik*, 104–107 (May 1938).

22. "La Rupture du Pont—Route Soude de Hasselt," *Génie civil*, **112**, (24), 497–500 (June 11, 1938).

23. Bondy, O., "Collapse of an All-Welded Bridge at Hasselt, Belgium," *Engineering*, **145** (3779), 670–671, 682 (June 17, 1938).

24. "Failure of the Hasselt Welded Bridge," *Engineer*, **165** (4301), 675–676 (June 17, 1938).

25. Rosch, H., "Einstruz einer geschwessten Strassenbrücke in Belgien," *Stahl u. Eisen*, **58** (30), 807–809 (July 28, 1938).

26. "Commentaries suscites par l'accident du pont de Hasselt," *L'Ossature Metallique*, **7** (7–8), 336–339 (July-August 1938).

27. Bondy, O., "Brittle Steel a Feature of Belgian Bridge Failure," *Eng. News-Rec.*, **121** (7), 204–206 (Aug. 18, 1938).

28. "Welded Bridge Failure in Belgium," *Ibid.*, **120** (18), 654–655 (May 5, 1938).

29. "Etat actuel des recherches relatives aux causes de l'accident du pont de Hasselt," *L'Ossature Metallique*, **8** (2), 101–102 (Feb. 1939).

30. Meslier, R., "A Propos du pont de Hasselt," *Rev. soudure autogène*, **31** (299), 616 (February 1939).

31. "The Hasselt Bridge Collapse," *Engineering*, **147** (3816), 258 (Mar. 3, 1939).

32. Rucqoi, L., "Le progres techniques en construction soudee," *L'Ossature Metallique*, **8** (3), 151–155 (Mar. 1939).

33. "Considerations relative aux causes de la rupture du pont de Hasselt en Belgique," *Génie civil*, **114** (9), 201 (Mar. 4, 1939).

34. "Collapse of the Hasselt Bridge—Present Position of the Research and Its Causes," *Civil Eng.*, **34** (393), 93–92 (May 1939).

35. Bierett, G., "Uber die Schweissung in Brückenbau," *Ill. Schweisstechnische Rundschau*, **49** (9/10), 386–390 (May 1939).

36. Portevin, A. M., "Collapse of the Hasselt Bridge," *Metal Progr.*, **35** (5), 491–492 (May 1939).

37. de Marneffe, A., "Fragilité de forme—Reflexions à propos de l'ecroulement du pont de Hasselt," *L'Ossature Metallique*, **7** (6), 284–287 (June 1938).

38. Reeve, L., "Examination of Welded Steel Specimens from the Hasselt Bridge," *Quart. Trans. Inst. Welding*, **3**, 3–13 (January 1940).

39. Hecq, F., "La position de la construction soudee apres l'accident du pont de Hasselt," *L'Ossature Metallique*, **9** (2), 81–88 (February 1940).

40. "Welded Bridge Failures in Belgium," *Ry. Gaz.*, **72** (10), 335–336 (Mar. 8, 1940).

41. "The Welds in a Collapsed Bridge," *Metals & Alloys*, **11**, MA 324–326 (June 1940).

42. "Causes of Welded Bridge Failures," *Ry. Gazette*, **72** (24), 830–832 (June 14, 1940).

43. Busch, H., and Reuleke, W., "Investigation of Failure in a Welded Bridge," THE WELDING JOURNAL, **25** (8), Research Suppl., 463-s to 465-s (August 1946). (Translated from *Stahl u. Eisen*, **62** (4), (Jan. 22, 1942), by D. Rosenthal.)

44. Campus, F., "Questions fondamentales en matiere de constructions soudees," *Schweizer Arch.*, **14** (5), 29–37 (May 1948).

45. Louis, H., "Cas de rupture fragiles constatees dans la construction de ponts,' Rev. soudure, **6** (2), 96–110 (1950).

46. Schaper, G., "Das Schweiffen im Brückenbau und im Ingenieruhochbau," *Die Reichsbahn*, 733–767 (July 19, 1944).

47. Merritt, F. S., "Bridge Collapse in Quebec Charged to Brittle Steel,' *Eng. News-Rec.*, **146** (6), 23–24 (Feb. 8, 1951).

48. Armstrong, D. B., "Canadian Welded Girder Bridge Practices," *Highway Research Abstr.*, **21**, Part 24 (June 6, 1951).

49. Brown, A. L., and Smith J. B., "Failure of Spherical Hydrogen Storage Tank," THE WELDING JOURNAL, **24** (3), 235–240 (March 1945).

50. Brown, A. L., and Smith, J. B., "Failure of Spherical Hydrogen Storage Tank," *Mech. Eng.*, **66**, 392–397 (June 6, 1944).

51. Plummer, F. L., "Field-Erected Pressure Vessels," THE WELDING JOURNAL, **25** (11), 1081–1089 (Nov. 1946).

52. Privately communicated industrial report dated Feb. 25, 1946.

53. Brewer, G., "Electric Strain Gage Analysis of a Fifty-Foot Horton Sphere," *Proc. Soc. Exptl. Stress Anal.*, **6** (2), 88–94 (1948).

54. Boardman, H. C., Private communication dated Apr. 14, 1953.

55. Elliot, M. A., Seibel, C. W., Brown, F. W., Art, R. T., and Berger, L. B., "Report on the Investigation of the Fire at the Liquification, Storage and Regasification Plant of the East Ohio Gas Co., Cleveland, Ohio," U. S. Dept. Interior, Bur. Mines R. I. 3867 (February 1946).

56. Gensamer, M., "Metallurgy of the Cleveland Liquified Gas Tank Failure," Jan. 15, 1945. (Report of the Technical Consultant to the East Ohio Gas Co.).

57. Barnes, G. E., Braidech, M. H., and Donaldson, K. H., "Report of the Technical Consultants, Board of Inquiry for the Mayor of Cleveland on the East Ohio Gas Company Fire," July 1945.

58. Jackson, J. O., "Liquified Gas Storage Containers," *Gas Age*, 37–42 (Apr. 22, 1943).

59. Shabalin, V. I., "Some Cases of Failure of Welded Tanks at Low Temperature," *Avtogennoe Delo*, **19** (6), 29–30 (1948).

60. Privately communicated industrial report, dated Feb. 2, 1947.

61. Tourret, J., "Fissuration du Reservoir a 8 de Brut a la Raffinerie de Normandie," Campagnie Francaise de Raffinage, Direction Recherches and Procedes Rapport No. 1152, July 23, 1951.

62. Feely, F. J., and Northup, M. S., "Failure of Two Oil Storage Tanks, Fawley, England," Standard Oil Development Co., Esso Engineering Dept., Nov. 3,1952. (Report for API Meeting, Chicago, Nov. 11, 1952.)

63. Barr, W., "Failure of Welded Oil Storage Tanks," Colvilles, Ltd., Motherwell, Scotland, Mar. 23, 1953.

64. "The Tucumcari Tank Failure," N. M. Soc. Prof. Engrs., *Jnl. Am. Water Works Assn.*, 44 (5), 435–441 (May 1952).

65. Privately communicated industrial report dated Apr. 21, 1953.

66. Heyer, R. H., *Engineering Physical Metallurgy*, D. Van Nostrand Co., New York, p. 297.

67. Privately communicated industrial report dated Jan. 3, 1950.

68. Bier, P. J., "Anderson Ranch Dam Penstock Test Fracture Repaired by Welding," THE WELDING JOURNAL, **32** (4), 313–319 (April 1953).

69. Huge, E. C., Babcock and Wilcox Corp., Report No. 2894, Jan. 21, 1943.

70. Sopwith, C. D., "Particulars of Failures in Gas Cylinders," Admiralty Ship Welding Committee Document, No. FE 4–370, December 1950.

71. Oldacre, M. S., Chicago Utilities Research Commission Report dated May 1, 1953.

72. United States of America: The Congressional Record, 82nd Congress, First Session. Appendix **97**, Part 11, p. A33, A437–8: Part 12, p. A2563–4, Part 15, p. A6658–9.

73. Saylor, W. A., *Iron Age*, **167** (13), 97–98 (Mar. 29, 1951).

74. "Natural Gas Pipe Line Failures and Causes as Reported to the Federal Power Commission," A report submitted to the Committee of Congress Investigating H.R. 88.

75. Hoppe, W. C., "Producing Large Diameter Pipe to Exacting Specifications," *Iron Age*, 118-121 (Sept. 25, 1952).

76. ASME, Gas Transmission and Distribution Piping Systems. ASA B31.1.8—1952, Section 8 of American Standard Code for Pressure Piping.

77. Yoffe, E. H., "The Moving Griffith Crack," *Phil. Mag.*, Serial 7, **42**, 739-750 (1951).

78. Hudson, G., and Greenfield, M., "The Speed of Propagation of Brittle Cracks in Steel," *Jnl. Appl. Phys.*, **18** (4), 405–407 (April 1947).

79. Acker, H. G., "Review of Welded Ship Failures," Report to the Committee on Ship Structural Design, National Research Council-National Academy of Sciences, Jan. 12, 1953. (To be published as Ship Structure Committee Report, Serial No. SSC-63.)

80. Tetmajer, L., "Methoden und Resultate zur Prüfung von Stahl und Eisen und Anderer Metalle," *Mitt. Antaltz z. Prüfung v. Baumaterialen am Eidg. Polytechn. Zurich, Heft*, **3**, 99 (1886).

81. Russel, S. B., "Experiences with a New Machine for Testing Materials by Impact," *Proc. Am. Soc. Civil Engrs.*, **25**, 550–577 (1897).

82. "Impact Tensile Test of Steel," *Eng. News*, **42** (20), 323–324 (Nov. 16, 1899).

83. Charpy, G., "Note sur l'essai des metaux a la flexion par choc de barreux entaillées," *Bull. Soc. Ing. Civils*, 213–238 (June 1901).

84. Mesnager, A., "Etud comparative de differentes methodes d'essais aux choc sur barreaux entailles," Reunion de membres français et belges de l'association internationale des methodes d'essais, Meeting of Dec. 26, 1902, pp. 395–405.

85. Mesnager, A., International Association for Testing Materials, Brussels Congress, 1906. Rapport non-offic. No. A 6f, pp. 1–16.

86. Fettweis, F., "Die Kerbschlageprobe Entwicklung und Kritik," *Arch. Eisenhüttenw.*, **2**, 625–674 (1929). (Translated by L. Emerich and E. P. Klier, and Distributed by the Ship Structure Committee, U. S. Coast Guard Hq., Washington 25, D. C., April 1951.)

87. Orowan, E., "Fracture and Strength of Solids," *Repts. Progr. in Physics*, The Physical Society, Gt. Brit., **12**, 185–232 (1949).

88. de Freminville, Ch., "Recherches sur la fragilité," *Rev. mét., Memoires*, **11**, 971–1000 (1914).

89. Ship Structure Committee: Minutes of Preliminary Meeting on Crack Arresters, Feb. 24, 1953.

90. Davis, H. E., Troxell, G. E., Parker, E. R., and O'Brien, M. P., 'Cleavage Fracture of Ship Plate as Influenced by Design and Metallurgical Factors (NS-336) Part II—Flat Plate Tests," OSRD Rept. M-608, Jan. 10, 1946. (Report not available.)

91. Boodberg, A., Davis, H. E., Parker, E. R., and Troxell, G. E., "Causes of Cleavage Fracture in Ship Plates—Tests of Wide Notched Plates," THE WELDING JOURNAL, **27** (4), Research Suppl., 186-s to 199-s April (1948).

92. Brown, D. P., "Observations on Experience with Welded Ships," *Ibid.*, **31** (9), 765–782 (September 1952).

93. Unpublished Studies by the Bethlehem Steel Co., Shipbuilding Division, Central Technical Dept., as quoted in Reference 79.

94. Howe, E. D., Boodberg, A., and O'Brien, M. P., "Temperature Studies of Liberty, Victory, and Refrigerated Cargo Ships," OSRD Rept. M-630, February 1946. (Report not available.)

95. Field, P., "Control of Welding Stresses by Welding Sequence," THE WELDING JOURNAL, **25** (11), 1109–1122 (November 1946).

96. DeGarmo, E. P., Meriam, J. L., and Jonassen, F., "Residual Stresses in Intersecting Butt Welds," *Ibid.*, **25** (8), Research Suppl., 541-s to 562-s (August 1946).

97. DeGarmo, E. P., Meriam, J. L., and Jonassen, F., "Effect of Weld Length upon the Residual Stresses of Unrestrained Butt Welds," *Ibid.*, **25** (8), Research Suppl., 485-s to 491-s (August 1946).

98. Jonassen, F., Meriam, J. L., and DeGarmo, E. P., "Effect of Certain Block and Other Welding Procedures on Residual Welding Stresses," *Ibid.*, **25** (9) Research Suppl., 492-s to 496-s (September 1946).

99. Jonassen, F., Meriam, J. L., and DeGarmo, E. P., "Reduction of Residual Welding Stresses by the Use of Austenitic Electrode," *Ibid.*, **25** (9), Research Suppl., 489-s to 491-s (September 1946).

100. Howe, E. D., Boodberg, A., and O'Brien, M. P., "History of Residual Stresses in Welded Ships, Part I: Ships Hulls of Liberty and Oil Tanker Types," OSRD 4866, Serial M 445, March 1945. (Report not available.)

101. Jonassen, F., "Some Research Activities Related to Welded Structures," *Ohio State Univ. Eng. Exptl., Sta. News*, 11–17, 38–42 (June 1950).

102. United States Government Printing Office, July 15, 1946, "Final Report of a Board of Investigation to Inquire into the Design and Methods of Construction of Welded Steel Merchant Vessels."

103. Greene, T. W., "Evaluation of Effect of Residual Stresses," THE WELDING JOURNAL, **28** (5), Research Suppl., 193-s to 203-s (May 1949).

104. Kahn, N. A., and Imbembo, E. A., "A Method of Evaluating Transition from Shear to Cleavage Failure in Ship Plate, and Its Correlation with Large-Scale Plate Tests," *Ibid.*, **27** (4), Research Suppl., 169-s to 186-s (April 1948).

105. Barr, W., and Tipper, C. F., "Brittle Fracture in Mild Steel Plates," *Jnl. Iron Steel Inst.*, **157**, 223–238 (October 1947).

106. Hodge, J. M., Manning, R. D., and Reichold, H. M., "The Effect of Ferrite Grain Size on Notch Toughness," *Trans. AIME*, **185**, 233–240 (1949).

107. Osborn, C. J., Scotchbrook, A. F., Stout, R. D., and Johnston, B. G., "Effect of Plastic Strain and Heat Treatment," THE WELDING JOURNAL, **28** (8), Research Suppl., 337-s to 353-s (August 1949).

108. Rinebolt, J. A., and Harris, W. J., Jr., "Effect of Alloying Elements on Notch Toughness of Pearlitic Steels," *Trans. ASM*, **43**, 1175–1214 (1951).

109. Barr, W., and Honeyman, A. J. K., "Effect of the Manganese/Carbon Ratio on the Brittle Fracture of Mild Steel," *Jnl. Iron Steel Inst.*, **157**, 239–242 (October 1947).

110. Barr, W., and Honeyman, A. J. K., "Some Factors Affecting the Notched-Bar Impact Properties of Mild Steel," *Ibid.*, **157**, 243–246 (October 1947).

111. Kahn, N. A., Imbembo, E. A., "Notch Sensitivity of Steel Evaluated by Tear Test," THE WELDING JOURNAL, **28** (4), Research Suppl., 153-s to 165-s (April 1949).

112. Williams, M. L., Myerson, M. R., Kluge, G. L., and Dale, L. R., "Investigation of Fractured Steel Plates Removed from Welded Ships," Ship Structure Committee Report, Serial No. NBS-3 (June 1, 1951).

113. Williams, M. L., "Examinations and Tests of Fractured Steel Plates Removed from Welded Ships," Ship Structure Committee Report, Serial No. NBS-4 (Apr. 2, 1953).

114. Banta, H. M., Frazier, R. H., and Lorig, C. H., "Some Metallurgical Aspects of Ship Steel Quality," THE WELDING JOURNAL, **30** (2), Research Suppl., 79-s to 89-s (February 1951).

115. Tör, S. S., Stout, R. D., and Johnston, B. G., "Further Tests on Effects of Plastic Strain and Heat Treatment," *Ibid.*, 30 (11), Research Suppl., 576-s to 583-s (November 1951).

116. Scotchbrook, A. F., Eriv, L., Stout, R. D., and Johnston, B. G., "Effect of Welding on Pressure Vessel Steels," *Trans. ASME*, **72**, 539-548 (July 1950).

117. Letich, M. J., American Bureau of Shipping letter dated (June 4, 1953).
118. Bruckner, W. H., and Newmark, N. M., "Axial Tension Impact Tests of Structural Steels," THE WELDING JOURNAL, **28** (2), Research Suppl., 67-s to 80-s (February 1949).
119. Osborn, C. J., Scotchbrook, A. F., Stout, R. D., and Johnston, B. G., "The Composition and Property Variation of Two Steels," *Ibid.*, **28** (1), Research Suppl., 24-s to 34-s (January 1949).
120. Orowan, E., "Fundamentals of Brittle Behavior in Metals," *Fatigue and Fracture of Metals*, Technology Press-John Wiley & Sons (1952).
121. Ship Structure Committee Report, Serial No. SSC-37, (Oct. 1, 1952): "Considerations of Welded Hatch Corner Design."
122. ASME, Unfired Pressure Vessel Code, Section VIII (1952), Table USC-23.
123. API-ASME, Code for Unfired Pressure Vessels (1951). Table W-306.1.
124. API, Standard 12C, Specification for Welded Oil Storage Tanks, (September 1952).
125. API, Standard 5LX Specification for High-Test Pipe Line (March 1953).
126. American Water Works Assn., Code D-100-52 Standard Specifications for Elevated Steel Water Tanks, Standpipes and Reservoirs (September 1952).
127. Am. Bur. Shipping, Rules for the Building and Classing of Steel Vessels, Structural Steels for Hulls (1952).
128. ASTM, Standards, 1952, Part I, Ferrous Metals.
129. Boardman, H. C., "Interpretive Report, Fabrication Division, Pressure Vessel Research Committee" THE WELDING JOURNAL, **29** (9), Research Suppl., 422-s to 432-s (September 1950), Discussion, *Ibid.*, **30** (6), Research Suppl., 291-s to 301-s (June 1951).
130. Pressure Vessel Research Committee, "The Pressure Vessel Research Committee Realizes Its First Objectives," *Ibid.*, **31** (10), Research Suppl., 492-s (1952).
131. Enzian, G. H., and Salvaggio, G. J., "The Effect of Nitrogen on the Brittle Behavior of Mild Steels," *Ibid.*, **29** (6), Research Suppl., 537-s to 544-s (June 1950).
132. Grossman, N., and Shepler, P. R., "The Effect of Welding Technique on Brittle Transition Temperature," *Ibid.*, **26** (5), Research Suppl. 321-s to 329-s (May 1947).
133. Stout, R. D., McGeady, L. J., Sun, C. P., Libsch, J. F., and Doan, G. E., "Effect of Welding on Ductility and Notch Sensitivity of Some Ship Steels," *Ibid.*, **26** (6), Research Suppl., 335-s to 357-s (June 1947).
134. Stout, R. D., and McGeady, J. L., "Metallurgical Factors in the Embrittlement of Welded Plate," *Ibid.*, **26** (11), Research Suppl., 683-s to 692-s (November 1947).
135. McGeady, L. J., and Stout, R. D., "Notch Sensitivity of Welded Steel Plate," *Ibid.*, **29** (5), Research Suppl., 243-s to 251-s (May 1950).
136. Boughton, N. J., and Lewis, C. W., Jr., Test and Inspection Dept. Missouri-Kansas-Texas Railroad Co., Report of Feb. 18, 1944, and letter dated May 21, 1953.
137. Privately communicated industrial report dated May 27, 1953.
138. Third Technical Progress Report of the Ship Structure Committee, (Aug. 1, 1953).