HIGH STRENGTH STEEL: CRISIS OR NO?

Some failures, yes. Problems so serious as to call for a ban or radical curtailment, no. Here's how to design with high strength steel, and case histories of failures.

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A front-page headline in the *Wall Street Journal* on January 16, 1984, read "High Strength Steel is Implicated as Villain in Scores of Accidents."

Reporter Bill Paul interviewed many experts, described failures in a bolt in a Beech E90 private aircraft, the offshore structure Alexander Kielland, a hydrogen storage tank in California, the wire cable supported roof of a Berlin auditorium, the fears of possible failure in several U.S. nuclear reactors' steel pressure vessels, a bolt in a University of Texas arena roof, bolts in something like a million General Motors 1981 autos, 20 or more train derailments over 10 years linked by some to faulty steel rail, cracking of welded steel in the Grumman bus, and in two steel bridges.

One might conclude from the article that high strength steel is dangerous, that its use should be stopped or radically curtailed. In fact use of high strength steel has grown if anything. And this seems appropriate.

Most of the failures involved not faulty steel but mistakes in design, fabrication, etc.

Those interviewed for this article say the problems—and especially in civil engineering structures—are essentially technology transfer, not ignorance. The need is for education and for quality control.

PRIMER

It's almost as if high strength steel were a different material than tried and true A36 steel. Certainly some of the ways it can fail are different. Of course, there's not just one high strength steel but a variety, and each behaves differently. But some useful generalizations can be drawn:

1. *Metallurgy or chemistry.* Its strength can be extraordinary—100 ksi structural steel is readily obtainable, and in prestressing wire or rod and in bolts you can get strengths near or over 200 ksi. This strength is obtained at the price of reduced ductility and, sometimes, more brittleness.

Very high strength steels have less, sometimes almost no, margin of strength above yield, in the plastic or ductile zone. Fracture and failure, when they occur, tend to be precipitious—little warning. Design stress that controls is often half the tensile strength, and may be lower than the percent of yield used in lower strength steels.

The higher the steel's strength, the more critical the brittleness problem is. Ultra-high strength steels fail in the presence of a *smaller crack* than will less strong steels, and when subjected to *less of a shock* when fast-impact loaded (highway vehicle loads, incidentally, are not shock loads). And they will fail with less extreme *temperature cycling* (which induces stresses) and tend to be more *corrosion-sensitive*. See Fig. 1.

2. *Ambient temperature.* Each type of steel has its own "transition temperature," below which it has little ductility or ability to stretch plastically at stresses beyond yield. If it fractures, it does so brittly. A rubber band offers an imperfect but useful analogy. At room temperature it is highly elastic, but at extremely cold temperatures it becomes brittle and will shatter when struck.

Not only ambient temperature but the steel's—and a weld's—temperature history during its creation is important. Especially with thick plates or complex joints, if you weld without applying preheat or postheat to the base metal, trouble may result. One solution, which was used in critical joints in Chicago's John Hancock Building, for example, is to prefabricate them offsite, place them in an annealing furnace, and reheat them and then slowly cool to ambient. This reduced the high levels of residual stress introduced by welding.

DISCONTINUITIES

3. *Initial discontinuities.* One type of discontinuity, and one of the more common problems, is that introduced by welding. Properly chosen weld metal and welding technique will result in a weld that is more nearly flaw-free and resistant to brittle fracture. Even then it's a good idea to make your welds more fracture-tough than the base metal, because the weld is more likely to have a flaw.

Several years ago a pressurized steel tank of A514 steel ruptured. Explains Dick Roberts of Lehigh University, it failed for three reasons: (1) An initial crack 6 ft (1.8 m) long and ⅛ in. (3 mm) deep at a weld, was introduced during stress relief (the steel was sensitive to stress-relief cracking).

(2) Pressure of the gas in the cylinder was cycled up and down, promoting crack growth. (3) The pressurized (1500 psi) hydrogen gas in the cylinder accelerated crack growth. Had the gas been other than hydrogen, Roberts says, the crack would never have grown large enough in the tank's lifetime to cause rupture.

Too, remember that repairing a flaw in a weld does not always

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eliminate the problem—the repair weld, too, may be flawed, and may introduce residual stresses. When doing repair welds, consider preheating. Often a repair weld is short, and even when perfect it has two locations that are stress-raisers—its two ends.

Do not design with the assumption you can eliminate all flaws or stress raisers. Say Rolfe and Barsom in their text *Fracture and Fatigue Control in Structures*, a key to co-existing with steel fatigue and fracture is to understand that "real structures contain discontinuities."

4. *Dynamic loads.* Not only the stress level relative to yield is important, but also the rate of application of stress. Steel is a hybrid animal, behaving sometimes ductilly and sometimes brittlely. If you load steel at a high enough rate—a truck crashing into a bridge, an explosion, etc.—and with enough energy, some steels will crack brittlely. Mobilizing the toughness in even a ductile steel requires a bit of time. When you pull taffy slowly, it stretches, but if you pull it fast, it fractures. Fig. 2 shows that a steel's static load capacity is greater than its energy-absorbing ability when impact loaded.

Another way of looking at loads is to differentiate a single load from repetitive or cyclic loads. A bridge over its lifetime is subjected to thousands or millions of vehicle loads. Such dynamic loads increase the likelihood of another form of steel fracture called fatigue cracking. Bridge steel structural members are designed for fatigue loads, and steel building frames generally are not because it is not necessary. If fatigue loads are of concern, permissible design load may be a fraction of design loads in the absence of fatigue.

STRUCTURAL DESIGN

So much for fracture mechanics of the steel. How the engineer designs his structure also influences fracture susceptibility.

In his book *Fatigue and Fracture in Steel Bridges*, John Fisher writes that there are two kinds of stresses in steel bridges, those due to *load* and those to *load-caused deformations in the structure.* This latter type, which Fisher estimates accounts for 90% of bridge cracks, is discussed in his CE article last month. The former type, which though far less frequent are more serious, are again of two types: (1) those in structural details with low fatigue resistance, and (2) those due to large initial discontinuities. In both types, welding is usually involved. (Fisher writes about all steels, whether or not high strength.)

Asked to single out a typical failure in each class, Fisher nominated Connecticut's Yellow Mill Pond Bridge as an example of (1) and Louisiana's Gulf Outlet Bridge, of (2). Two photos and Fig. 3 describe them.

If you feel you're going to have a problem with high strength steel, seldom is it necessary to abandon steel; two alternatives are to choose a different steel or take added precautions in its use.

A. *Use lower-strength steel.* That was done in replacing the roof bolts at a Texas arena, and cracking has not recurred.

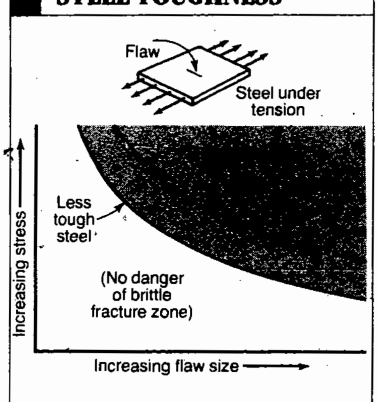
Back in 1979 a bolt in the roof structure of the Irwin Special Events Center at the University of Texas-Austin broke and fell to the ground; it had split in half. Eventually all 12,000 bolts in the roof were replaced.

Cause, says the school's Karl Frank, was stress corrosion cracking. Corrosives, in extreme cases even such seemingly mild ones as water, in super-high-strength bolts may cause accelerated corrosion which increases chances of fracture. (See Fig. 5.)

The original bolts had ultimate strengths of about 210 ksi, even though the specification for these A490 bolts called for ultimates in the range 150-170.

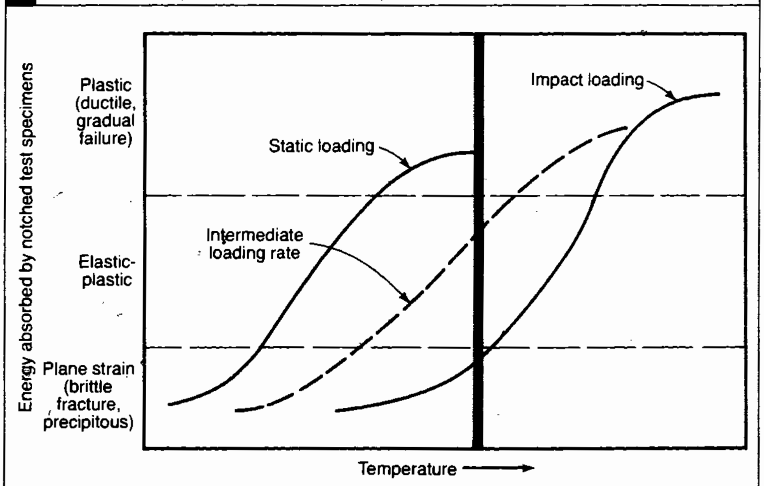
It has been found that when bolt steels (they are highly tensioned, and this contributed to the problem) rise much above 170 ksi they are subject to stress corro-

FIGURE 1
FRACTURE AND STEEL TOUGHNESS



The tougher the steel (upper curve), the more resistant it is to fracture. This figure shows the relationship of steel toughness, stress applied to the steel, and critical flaw size (size at which the flaw will grow into a complete fracture). (From Rolfe & Barsom)

FIGURE 2
FRACTURE, LOADING RATE, TEMPERATURE



This figure shows how the steel's temperature and loading rate influence its susceptibility to fracture. The thick vertical line represents a given steel. Note that (1) it can take a larger static load than impact load, and (2) its fracture resistance rises with temperature. (From Rolfe & Barsom) sion cracking, but when under 170 they are not.

The failed roof of Kansas City's Kemper Arena used the same grade bolts from the same maker, and they too were over spec in strength and hardness. Another problem was improper pretensioning of bolts. If the Kemper bolt-tightening had been done correctly, Karl Frank says, either the bolts would have broken or the erector could not have tightened them up to spec. He recommends not using bolts in diameters over 1 in. because of difficulty in installation and in controlling bolt hardness.

(Concern about fatigue fractures of very strong bolts has led to over-conservatism or corrective overkill in some cases, says Reidar Bjorhovde of the University of Arizona. Some engineers are specifying bolts of lower strength than necessary, because it gives them more fracture-toughness. But Bjorhovde and others say that most structural applications do not involve the cyclic loadings that might lead to fatigue cracks, so fewer, stronger bolts could be used instead and money saved.)

In their bridges, some state highway departments are trying to avoid use in thick tension members of the highest strength hot-rolled steels (A514 quenched, tempered steel with 100 ksi yield). As in the case of the highest strength bolts and critical welds, it can be used successfully, but requires more care, says Karl Frank.

Years ago engineers thought that higher strength steels produced a structure with better fatigue performance. But fatigue tests of welded details have shown the fatigue performance of high strength steels is the same as for lower strength steels, Karl Frank says.

B. *Introduce structural redundancy.* Two inherently redundant structural systems are the wire-cable suspension bridge and the reinforced concrete beam. A suspension bridge's two cables each consist of hundreds or thousands of wires, and if one or a few fail there is no danger. As for RC beams, almost all have more than one rebar on the tension side—the beam is unlikely to fail without warning if one rebar breaks.

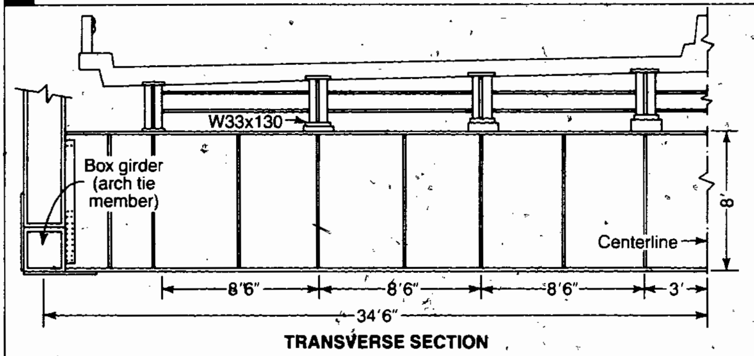
The same was true years ago of steel structures. They were redundant, because the beams and columns consisted of built-up sections, that is, of relatively small plates and angles riveted together. One plate or a few rivets could fail and the structure would retain its integrity.

Much of that redundancy has been lost in the past 50 years, as engineers and steel mills turned to ever-bigger rolled sections or sections built-up by welding. The trend was to fewer, bigger pieces.

But since AASHTO revised its Bridge Spec about 10 years ago, at least in bridges there has been a partial turning back to more redundancy. For example, more and smaller stringers are often used rather than fewer, larger ones.

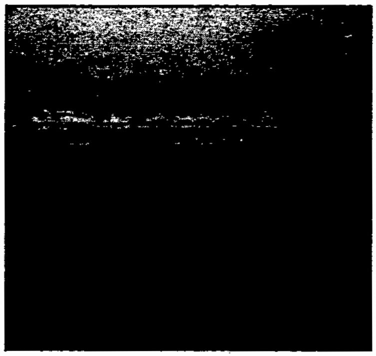
In some cases, adding redundancy is not possible or desirable. In such cases the AASHTO Guide Spec for bridges mandates all of these three steps: a lower allowable stress range, higher fracture toughness of steel and weld, and special procedures in welding and weld inspection.

FIGURE 3
LARGE INITIAL DISCONTINUITY



The Gulf Outlet Bridge is a tied-arch structure of 702 ft (214 m) main span, crossing the shipping channel of the Mississippi River near New Orleans. In the main span's tie-beam box girders, transverse cracks were found. Each box had been welded up from four plates, using corner fillet welds. Transverse cracks were found where manually made weld passes had been added to the original submerged arc welds to increase their size. Each crack studied originated at a porosity or entrapped piece of slag in a weld. All cracks were removed and repaired. Apparently too little preheat was applied prior to the manual weld passes, leading to the cracking. (Special caution is in order when welding steels, such as this, of 100 ksi and higher yield strength. Most fabricators are not accustomed to dealing with them.)

At the welded ends of their girder coverplates, several bridges cracked in recent years. Two photos show an example of this, on the Yellow Mill Pond Bridge carrying I-95 in Connecticut. The girders were of A242 steel. The fractures were fatigue cracks, caused by the repeated loadings of cars and trucks. (This is an example of the most fracture-susceptible type of bridge detail. Today designers either omit partial coverplates [for example by specifying coverplates extending to the girder's end or by using thicker girder flanges] or use them only on bridges getting infrequent loadings.)



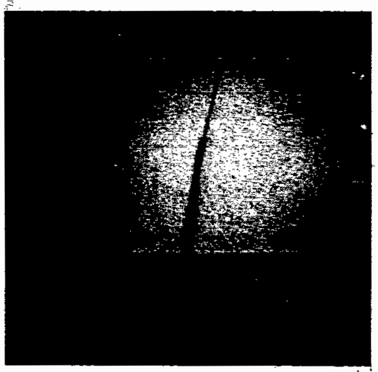
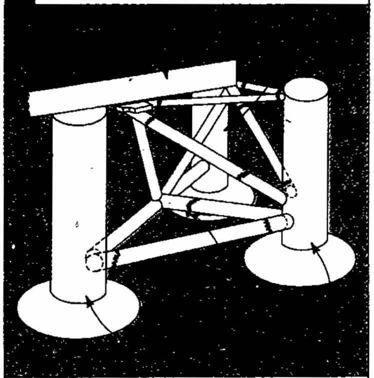


FIGURE 4

OFFSHORE PLATFORM FAILS



In 1980, the semi-submersible offshore platform the Alexander Kielland broke up and sank in the North Sea, and 123 died. Failure sequence began at a seemingly insignificant detail—a hydrophone mount. It was welded to a bracing tube as shown, and contained an initial flaw that went undetected during construction inspection. Combined stillwater and wave loading stresses at time of failure were only 40-50% of yield. In addition, stresses were increased as follows: If the fillet weld that tore had been intact, by a factor of 1.6; and if the weld had been completely fractured, factor of 3.0. Once that brace failed, lack of redundancy in the lower bracing system (it was designed with incomplete triangulation on the lower bracing to permit small boats access to the platform) permitted progressive failure of all the other braces (see jagged marks) supporting one leg, leading to collapse.

C. *Predict fatigue-crack growth rates.* The rate at which a fatigue crack will grow, the crack length at which it will fracture or fail, and the time to failure can be predicted. These are keys to the fracture control plans used for U.S. Air Force jet fighter aircraft.

Such techniques are also used by civil engineers to predict the service lives of structures subjected to repeated loads, such as bridges and offshore structures.

D. *Design for inspectability.* Critical details should be readily accessible. One engineer thinks that if the hanger-pin assembly at the Mianus River Bridge had been readily inspectable, the failure might not have happened. But the only access walkway under the bridge deck ran down its centerline, many feet from the hangers and pins. Another hard-to-examine detail is a closed box girder.

E. *Peer review, field inspection.* Rather than going overboard on fracture control plans and other fancy reforms, Rolfe suggests paying more attention to two quality control alternatives already in use on civil structures in some places. They are peer review and better construction inspection. Fig. 4 is an example illustrating an inspection failure.

In other words, maybe for most civil structures the need is not more attention to fracture mechanics but more quality control.

AGENDA FOR THE FUTURE

Problems with high strength steel structures have mostly been solved. But some remain, and bolted and welded connections account for most of them.

Bjorhovde believes the problems of bolt failure due to incompletely understood fracture mechanics are largely behind us. Problems remain with bolted structures, he says, but now the #1 concern is field inspection of bolting. Often the inspector arrives after field erection is done, and checks bolt tension by using a torque wrench. But static fraction (which is what is measured after erection) does not correlate with bolt tension, Bjorhovde says. So the post-construction inspection is meaningless (fortunately it errs on the conservative side), and should be abandoned in favor of inspection during the original bolt tensioning. The Research Council on Structural Connections is working to write corrective language, which will be proposed for inclusion in the AISC *Manual of Steel Construction.*

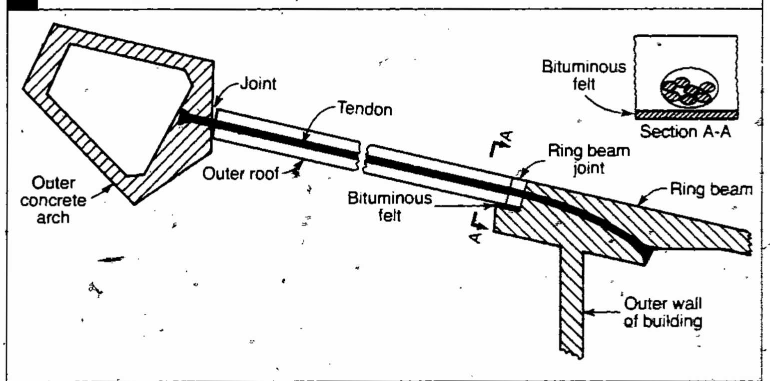
As for welding problems, John Fisher of Lehigh University says that while welding properly done is a reliable technique, question marks in the state of the art do remain, among them:

1. Aluminum in weld metal. "Once aluminum goes above 0.04%," Fisher says, "fracture toughness goes to pot."

2. Torch-cut edges of steel plates (this is a weld-related matter but not welding narrowly defined). Torch cutting of plates hardens the base metal near the plate edge, and being imperfectly smooth, it also introduces stress raisers. Fisher says both factors have contributed to failures, even on steels with ultimate strengths as low as 60 ksi.

FIGURE 5

PRESTRESSED TENDON FRACTURES



In 1980 the roof of the Berlin, West Germany, Congress Hall collapsed. The roof was a hyperbolic paraboloid in shape, and cantilevered out far beyond the outer wall thanks to dozens of post-tensioned tendons. There were several causes of failure: At the failure location (see section A-A), there was almost no concrete cover in the roof panel around the duct. Duct grouting was incomplete. The bituminous felt pad beneath the roof apparently acted as a water reservoir. The ring beam joint's concrete was of poor quality, apparently contributing to movement of the adjacent precast roof panel. This in turn led to changes in the tendons' geometry, which led to stress concentrations. Some of the tendons failed over time due to stress corrosion cracking; when enough tendons had thus failed, the remaining tendons' capacity was exceeded and half the roof collapsed. Today corrosion protection of prestressed elements gets much more attention.