Lessons from Crane Runways

JOHN E. MUELLER

MANY ENGINEERS consider the design of a mill building to be 90 percent judgment and 10 percent perspiration. While the exact percentages are debatable, there is little question that judgment forms an important part of the design of such structures, particularly with regard to basic load assumptions and the evaluation of "indeterminates"—those mathematical problems with too many variables for exact solution.

Some of the questions that must be resolved by the designer are: How many cranes can be expected to operate simultaneously in the general vicinity of a particular column? How often, if ever, will all cranes pick up their maximum loads with the trolleys adjacent to the column? What lateral thrusting forces can occur simultaneously? What allowance should be made for impact? Would the plant be in operation when there is maximum snow load on the roof? How stiff is the variable stiffness truss? How much resistance to rotation will the foundation afford?

The need for sound engineering judgment is equally important in the design of crane runway details. This paper will deal primarily with such details. The suggestions offered are not the result of exotic computerized stress theory, nor are they based on laboratory experiments. They are founded on experience with a great variety of actual installations and on the lessons learned from personal observations over a period of nearly 25 years.

CRANE GIRDER-TO-COLUMN DETAILS

The first important detail is the connection of the crane girder to the column. The predominant loading is vertical, and the crane girder is directly supported by its seated connection on the column. The girder web stiffeners should be chosen for size and location to provide a direct flow of stress to the column web and flange below, with a minimum of applied eccentricity. The welded stiffener provides this feature most simply.

John E. Mueller is Chief Engineer, Structural Division, Whitehead & Kales Company, River Rouge, Mich.

The next principal loading is transverse. Figure 1a shows a detail used frequently on lighter crane runways.

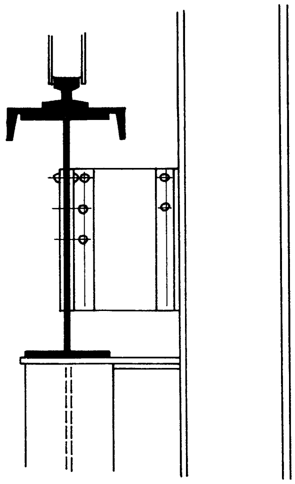
It is a dangerous detail. It is an attempt to connect for lateral forces through the use of costly detail material, and a connection which is hopelessly incomplete.

Figure 1b illustrates the reversible strain to which the girder web is subjected. This exaggerated position of the flange will, of course, never occur. But the sketch is an attempt to picture the action on the web—a ripping action leading to the result shown in Fig. 1c. The cracking shown in this sketch has occurred many times. To a very impressionable new engineer, the first observation of this defect, as a widespread occurrence on a crane runway only three years completed, can be startling. It is a needless failure, because it is very simple to connect the top flange directly to the face of the column. The top flange acts as a horizontal beam delivering its reaction to the column. When the flange connection will amply withstand the lateral thrust, there is no longer a need for the diaphragm. It will no longer receive a loading or stress of any kind; therefore it should be discarded.

Another effect of the diaphragm is demonstrated by Fig. 2. The simple span vertical deflection of the girder tends to rotate the end of the girder on its column seat. The diaphragm attempts to restrain that rotation. But it is not designed for that purpose. The invariable result is high shear on the upper fasteners, and local tension in the web adjacent to the diaphragm—a second contribution to trouble in that area of the web. Diaphragm material sometimes is fractured, and fasteners are loosened or sheared.

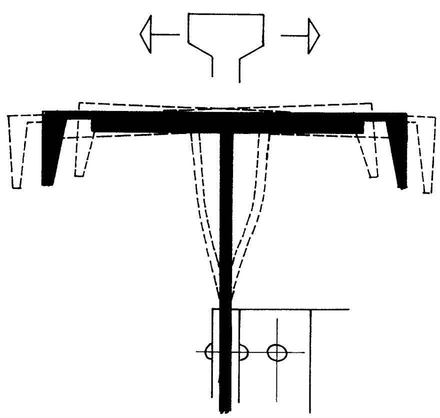
On occasion, these fasteners have been replaced, and have failed again. A second attempt at repair involved reaming the holes to the next larger size, and the installation of larger fasteners. The second discouraging repair generally was followed by the refusal to inspect further.

At this point, it would be well to refute the idea that the diaphragm angle might "breathe" with the end rotation of the girder, if no direct outer splice plate is



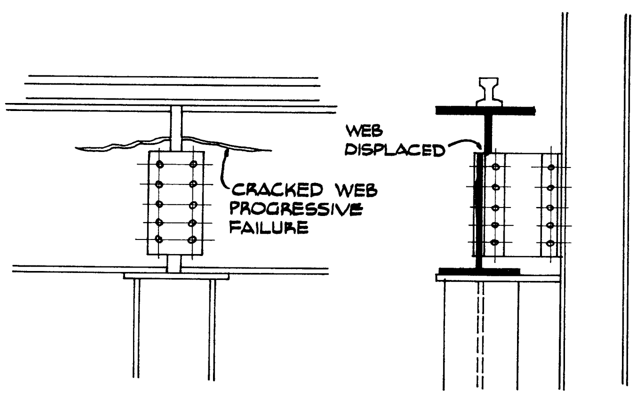
THE DIAPHRAGM

(*a*)



THE ACTION

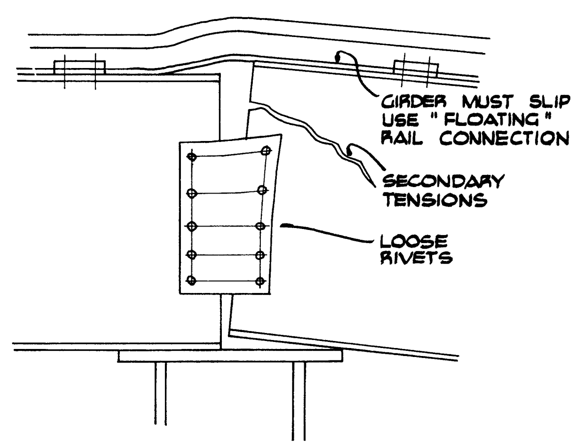
(*b*)



THE RESULT

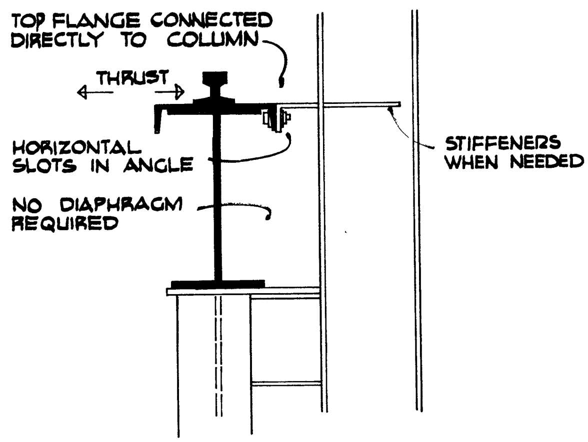
(*c*)

Figure 1



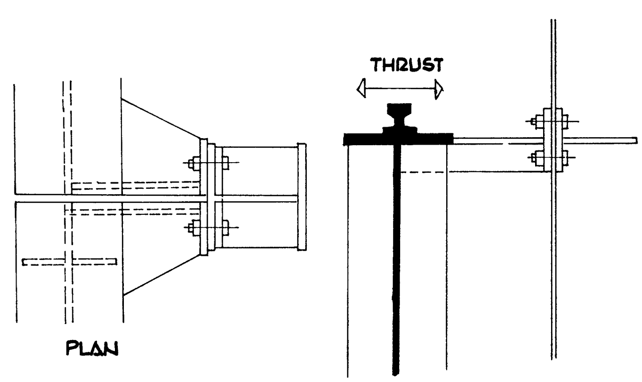
END ROTATION

Figure 2



THE SOLUTION

Figure 3



THE SOLUTION —
HEAVIER RUNWAYS

Figure 4

used. This bending of the angle within the elastic range cannot accommodate the amount of movement expected.

One should not attempt to rationalize the need for an unnecessary expenditure such as the diaphragm. It serves no need or purpose, other than to provide for runway lateral forces. And this it does very poorly.

The characteristic of continuous crane girders would offer a simplification of the rotation problem. However, the continuous or cantilever installation is not generally as economical as the simple span girder.

For new work, or for repairs of old, the diaphragm should be eliminated as an unwanted troublemaker. It should be replaced with a direct top flange connection which will allow the girder unrestricted rotation.

Figure 3 shows a basic connection usable with lighter runways. It prevents movement in a transverse direction, but allows complete movement longitudinally to permit the girder to rotate without restraint.

For heavier runways, a detail such as Fig. 4 is a good solution. It provides for free movement. With due allowance for resulting torsional stresses in the column section and with proper design of bolts and parts, a "pure" design can be achieved which will give the desired results.

There are other possible variations of the details—any of which is acceptable if the flange is connected directly for transverse forces and allowed to move longitudinally.

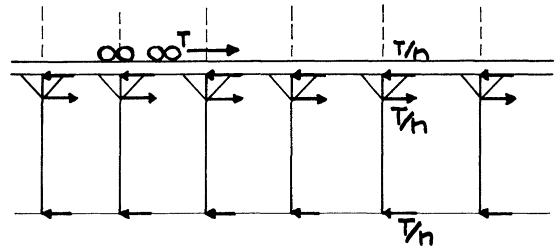
KNEE BRACES

Another important element of a steel mill building is the crane girder knee brace. This small member might seem rather insignificant in a large steel mill building, but assuredly it is an important member, sometimes poorly designed.

Its intended function is shown in Fig. 5. However, it is subject to secondary effects, not usually recognized, which render it non-functioning and even deleterious to companion structures. It is for this reason the author believes the knee brace is better eliminated.

In the absence of base-of-column restraint, the knee brace provides the sole stabilizing moment to the column. This permits the column to resist movement in the direction of crane traction thrusting. Without the knee brace, the column and girder structure would be unstable. However, the resistance to movement is partial, and not enough in terms of a good rigid structure. Engineers therefore always provide a degree of restraint in the base of the column, achieved by the arrangement of anchor bolts and proper connecting material, and by proper rigidity of the foundation.

As a third consideration, the all-important vertical X-brace is finally added, and this is what, after all, does the job of taking traction forces down to the ground. When the vertical X-brace is functioning in a crane runway, it renders the column at the girder seat so immobile as to prohibit the induction of stress due to horizontal movement of the girder in the knee brace. The knee brace therefore is not performing as intended.



FUNCTION OF THE KNEE BRACE

Figure 5

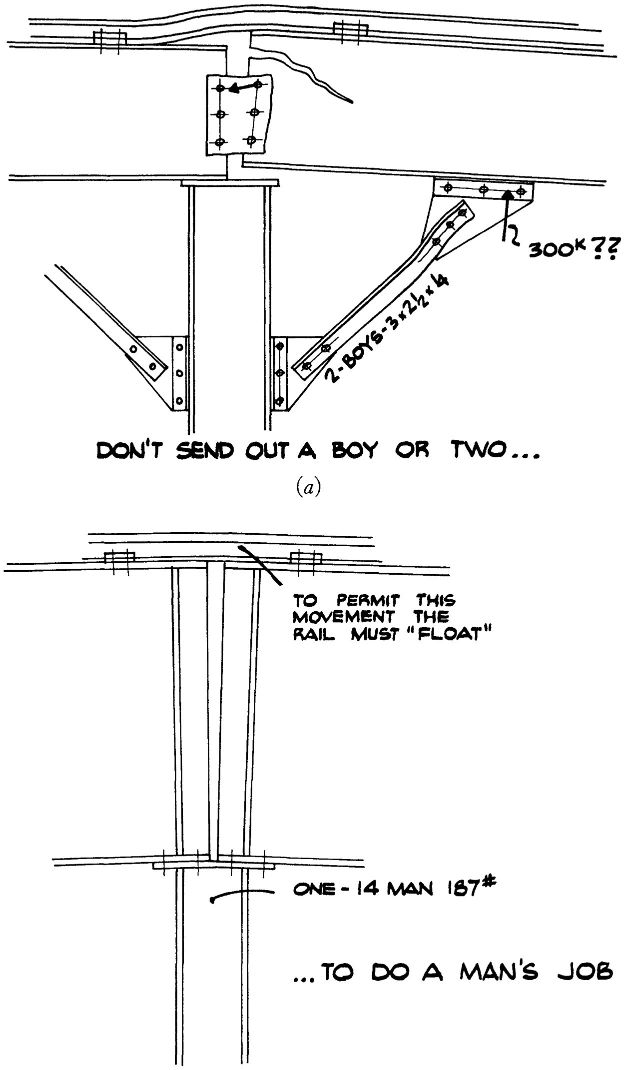
Figure 6 refers to the attempt of the knee brace to support vertical load. The exaggeration in Fig. 6a might seem preposterous, but such cracks in webs have been observed many times. The girder supporting action of the knee brace will not raise the girder from the column seat as shown, but the tendency is there. The resulting tensile stresses in the web above the diaphragm are added to the ripping action caused by lateral thrusting discussed previously.

The author has not actually observed knee brace failure other than occasional loose rivets. But it would be well to attempt to understand the member by considering the action and its effect on the knee brace. The deflection of the loaded girder will shorten the knee brace. There will be partial relief by deformation of the column, shortening of the knee brace opposite, and negative deflection of the girder in the next span. The knee brace will be shortened to the extent of yielding.

A typical crane girder could be expected to deflect almost ¼-in. in the vicinity of the knee brace connection. To shorten a steel member 6 or 8 ft long by ¼-in. can readily stress it beyond yield. Yielding results in permanent deformation of the knee brace, inducing prestressing of the unloaded girder and permanent tensile stress in the web above the diaphragm. Repeated loading on the deformed girder results in eventual fatigue failure.

In one astonishing case of a rather limber, light runway, the column base was observed to jump back and forth on its base plate as the crane passed over adjacent girders. Investigation found the two anchor bolts sheared through at the plane of movement.

As the crane approached this column and loaded the adjacent girder, the simple span deflection was restrained by the diaphragm and knee brace. The resulting column base horizontal reaction exceeded the frictional resistance



(*a*)

(*b*)

Figure 6

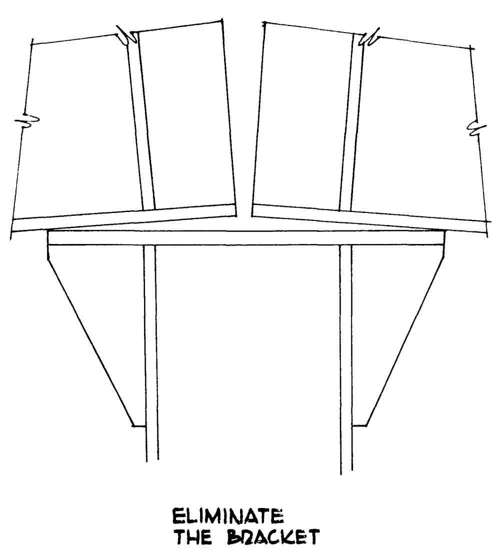


Figure 7

to movement. Repeated impacting on the anchor bolts eventually caused their failure in shear.

Elimination of the diaphragm and the knee brace leads to a simple structure (Fig. 6b) capable of providing for all possible loadings. The simple span girder can carry vertical loads to its seat on the column, and still be permitted to deflect quite naturally without the secondary effect caused by other materials which were added because "we always did it that way".

The simple span girder can carry lateral loads to its top flange horizontal connection direct to the face of the column, without passing through dangerously thin material. The girder acting as a strut can carry the longitudinal traction forces of the crane, and transfer them from the rail through the column girder seat to the adjacent girder and so on to the vertical X-bracing which takes those forces down to the ground.

It should be noted that elimination of the diaphragm and the knee brace results in elimination of the direct connection from one girder to the next, providing a path for longitudinal forces to be carried down to vertical X-bracing. There remain only the bolts through the girder bottom flange and the column cap plate. These bolts must be designed capable of transferring the expected forces.

In Fig. 6b, notice the absence of a frequently specified bracketing at the girder seat somewhat as shown in Fig. 7. As a matter of fact, there can be no justification for this wasteful detail. The presence of such a bracket can only increase the eccentricity on the column below. The resulting increase of column weight plus the cost of the bracket achieves absolutely no saving elsewhere. What, therefore, could be the purpose of the bracket? For direct transfer of crane girder reaction to column, the Specification allows some 33,000 lbs per sq in., while maximum column stress would be 19,000 lbs per sq. in. Selection of stiffeners and cap plate thickness can accommodate this transfer.

TRANSFER OF LOADS TO FOUNDATION

The next problem is that of transferring to the foundation structure the longitudinal crane traction thrust plus accompanying longitudinal wind forces on the building. This is often accomplished by the system shown in Fig. 8—a pattern of vertical bracing located intermittently in an expansion field with three, four, or sometimes five unbraced bays between braced bays. It has always seemed advisable to locate bracing in the end bays. First, there may be a crane stop and the bracing will directly dispose of the shock. Second, the erector has a tool to "plumb" his structure at the onset. The frequency of braced bays was deemed necessary to minimize the travel of traction forces to a point of dissipation. There is a fallacy however, in the use of braced bays located at the ends of an expansion field.

Figure 9 graphically indicates the deformations of bracing caused by the thermal expansion of the runway and upper parts of the structure. Deformation of 1½ in. in a 30 to 35-ft diagonal will stress it well beyond yield. Contraction of the structure will bow the diagonals. The result of this action is observed in loose vibrating diagonals, broken angles, or broken gusset plates, loose rivets, and generally non-functioning panels of bracing which many are prone to attribute to operating abuses.

Figure 10 shows the general pattern of bracing recommended. The center panel is an anchor bay which holds the center columns plumb, and permits free expansion or contraction outwardly. It forces the expansion two ways, and thereby minimizes total movement. Only the column below the crane girder is deformed; that is unavoidable. It is the magnitude of the secondary stress associated with this deformation which should limit the length of the expansion field.

The wide 48-ft bay, now recommended as most economical, ably spreads this center panel of bracing to where the dead load is ample "hold-down" for the uplift reaction of the bracing. While on the subject of vertical bracing, note that the wide flange section is far superior to the customary double angle member. The most frequently heard complaint about steel mill buildings in the past has been "too much sidesway, too much vibration, too much movement of the structure." Bracing can vibrate even though adequate in strength. Members should therefore be selected which have maximum rigidity per pound of steel.

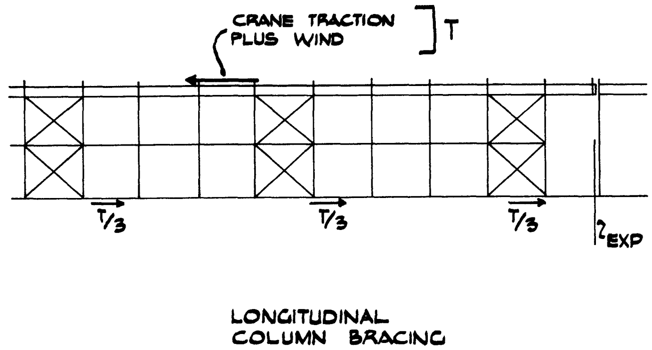


Figure 8

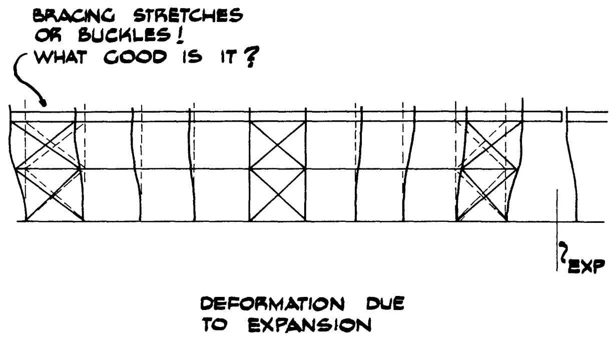


Figure 9

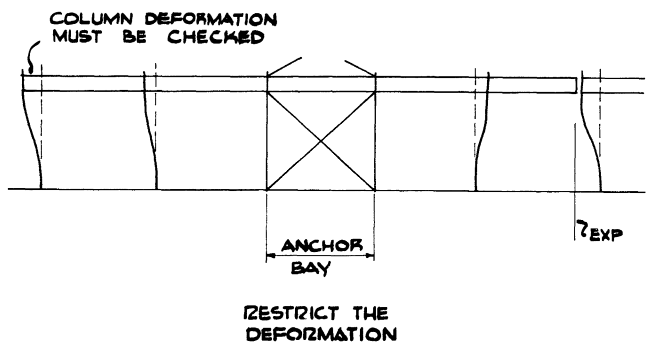


Figure 10

OTHER DETAILS

Certain other details of construction make interesting discussion, although they may not be involved with common failure. One is the arrangement and selection of members in the assembly of the column. The stresses for which the column is to be designed are the subject of much discussion among engineers. Much literature has been written on the subject. Engineers cannot honestly predetermine the precise stresses at each point in the height of the column, and it is not vital that they should.

Each of the methods known to be used will be successful. Security is realized in the knowledge that the portal is a rigid frame and that redistributions of stress will readily occur. Keep the stresses low in the selection of parts of the column, at no great sacrifice of steel. The upper shaft roof truss supporting element of the column should be as rigid as possible. The deepest wide flange section space will allow provides a bonus of structural rigidity achieved at no extra weight or cost when compared with the more slender, more flexible sections generally selected.

There is much more to discuss regarding the problems of steel mill building design, but those selected were considered most important.

It is hoped that the observations here recorded will be of value in the design of better crane runways.