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DESIGN OF CRANE RUNWAY STRUCTURES

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by

B. Tooma

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AUTHOR: B. Tooma

SUPERVISOR: Dr. A. Smith

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ABSTRACT

At present there is no code of practice or design guide for the complete design of crane runways. Many sources of information apply to steel structures in general and do not address some of the more important design and practical aspects of crane runways. It is the purpose of this report to review the various standard procedures together with rules and guidelines which result from practical experience in design, construction and operation. In particular it is hoped to identify those questions around which there appears to be some uncertainty or lack of substantiation. Some of these topics are identified as areas for possible future research.

The report considers the version components of the runway system and the loads which act on them. After discussion of the dynamic nature of loading and the allowances made for vertical and horizontal loads, the supporting system is described with reference to accepted guidelines, design details and sketches of connections. Interaction of the various components considers the design and detailing of the rail the girder, the horizontal girder (or surge plate) and columns and foundations. Use is made of a computer program to compare the behaviour of alternative girder support systems and the advantages and disadvantages of each is summarized.

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF TABLES

CHAPTER 1

INTRODUCTION

The design of mill buildings in general, and crane runway structures in particular, has been neglected in the literature. Considering the large annual capital investment committed to this type of structure, very little attention has been paid to methods of designing, detailing, fabricating and erecting these structures. The relative scarcity of literature is readily apparent to anyone engaged in research in this area of structural design.

The main deficiency has been a failure to incorporate the principles of design and detailing into a common practical procedure which may be used as a guide. To depend on existing design codes such as CISC*, NBC*, AISC*, AISE*, BS449*, etc., and to follow the normal design procedures is necessary but not in itself sufficient to produce a proper practical structure.

Although general principles of design are understood there is a significant measure of uncertainty with respect to the following aspects of design.

CISC - Canadian Institute of Steel Construction
NBC - National Building Code
AISC - American Institute of Steel Construction
AISE - Association of Iron and Steel Engineers
BS449 - British Standards Institution

1) The response of the structure to imposed loads.

2) The types and nature of applied loads.

3) The way in which load-carrying members interact at connections.

There is at present no code of practice, design guide or specification for the complete design of crane gantries. Designers may make reference to a number of sources of information (CISC, AISC, AISE, etc.), which define design criteria. Some of these sources apply to steel structures in general and much of the content is not relevant to the design of gantries. On the other hand there are many aspects of gantry design which are not addressed. In general, the somewhat sparse and inadequate code guidelines must be substantially augmented by practical rules and considerations which have been accumulated over many man-years of design experience. In reviewing these practical guidelines, one is impressed by the somewhat arbitrary nature of many recommendations, occasional contradictory recommendations and the general absence of supporting proof or documentation.

It is the purpose of this report to endeavour to bring together the various standard procedures along with many rules and guidelines from practical experience and to highlight those questions around which there appears to be some uncertainty or lack of substantiation.

To achieve the above objective, the main body of this paper will address the following topics.

a) <u>Loads and their types</u>: This includes the origin of loads, their values and the way in which these loads are applied to the structure.

b) Support of the loads: This involves the structural members and their connections, which together form the supporting system. Types of connection and fabrication details will be discussed and illustrated, with a brief review of the advantages and disadvantages of each.

c) Design of the Girder: This includes the criteria and the allowable stresses used in the design, as well as alternative types of girder with advantages and disadvantages given for each. The comparative results of computer analysis are discussed in Chapter 7 to further clarify the advantages and distadvantages.

d) Columns and Foundations: This involves a brief discussion of the types of columns and their design approach. It also includes a brief discussion of the foundations and their effect on the structure.

Following the review of these various aspects of gantry design, the important questions of uncertainty or contradictions are summarized. Suggestions are made concerning possible ways in which future research or study may be brought to bear on the subject.

CHAPTER 2

LOADS AND FORCES

Most civil engineering structures are designed to resist essentially static loads on which certain dynamic loads are superimposed as a result of random and occassionally extreme events. The operation of a crane gantry in a steel plant presents a startling contrast in that extremely large loads are handled with a speed and near-violence that can arouse feelings ranging from surprise to alarm in the visitor to such a plant. In order to give the reader an idea of the magnitude of loads acting on the runway, a 15 ton capacity crane which is considered moderate in size can be given as an example. This crane spans 150 feet centre to centre of the rails and the total weight of its bridge is 230 tons and that of its trolley is 35 tons. It can travel at speeds as high as 300 feet per minute. Crane runways are thus dyanmically loaded structures, subjected to various types of loads and forces. The main loads acting on the structure are:

a) Vertical loads and impact.

b) Horizontal forces: in the lateral and longitudinal direction.

In this chapter the above loads will be discussed with regard to their origin, values to be used and the way in which they are applied.

2.1 VERTICAL LOADS

Vertical wheel loads are the sum of the weight of the crane bridge, its trolley and its pay load, applied to the structure through the crane wheels. The structure experiences stresses which are applied nearly instantaneously and are of short duration. This impact factor is over and above the static stresses induced by the wheel loads. This overstressing is usually a result of operational conditions, some of which are:

a) Chain slippage.

b) Sudden lifting or jerking of loads.

c) Sudden lowering of the load.

d) Dragging of load.

e) The physical condition of the rail: for example, the flange of the rail may be rough and damaged.

f) Lack of, or damage to, damping material under the rail.

g) Poor rail splices forming rough joints thus causing high shock effect.

h) Excessive stiffness of the crane bridge and supporting structure, resulting in less energy being absorbed by these components and thus increased impact effects.

Due to the short duration of these stresses, and also due to the variable degree of flexibility of different crane bridges, there is a degree of uncertainty with regard to the amount of energy induced into the structure. An impact factor is introduced in the design to provide a relationship between the static imposed load and the dynamic load which must be resisted by the structure. For vertical loads, an impact value of 25% of the maximum wheel load is added to the wheel load. This value does not appear to be the result of scientific studies or measurements. Rather, it seems to have been arrived at by consensus among groups of engineers and owners with considerable practical experience relating to such structures. It has been accepted and used as such, even though there is no assurance that it reflects actual conditions.

This is an area where research and studies would be most welcome in order to arrive at an impact factor which would be more representative of the true conditions. These studies will have to be on site measurements since it is not practical to represent the crane and its operating conditions in the laboratory.

The stresses in the crane runway girder can be measured by using strain gauges attached to it at predetermined locations. The following procedure may be possible for finding the value of impact.

1) Locate the trolley with its full load at the extreme end of the crane bridge in order to produce maximum vertical wheel loads.
2) Locate the crane bridge at predetermined locations and measure the stresses in the runway girder section, which are the result of the static maximum wheel loads.
3) Continue to measure the stresses over a period of time with the crane performing its usual operations.

4) Any increase in the stress level above that produced by the static maximum wheel loads will be an indication that the vertical loads induced in the runway girder are higher than the normal static wheel loads. This increase may be attributed to the dynamic effect of the crane.

The above steps may require a considerable period of time if reasonable values are to be achieved. This is due to the fact that the conditions which will produce the maximum effect do not occur on the regular bases.

An alternative to the above would be to artificially create impact conditions, and measure the level of stress in the runway girder. Values obtained from this may not be as valid as those obtained from the first method mentioned above, since it is produced under controlled conditions. Impact which is the result of accident and carelessness could be more severe. Until such time, as a more rational value can be found through future research, the minimum value of 25% impact factor should be accepted.

The fact that the crane bridge has a degree of flexibility in addition to its suspension system, provides some dampening effect which reduces the level of impact. Also the use of a damping pad (see section 3.1.1 and 3.1.2) installed under the rail will assist in dampening of the shocks produced by impact. Therefore the use of 25% as a minimum value for impact factor may be considered justifiable, subject to allowance by the designer for exceptional circumstances. This factor should be evaluated in the light of the type of operation and conditions that the crane will be used for, and if necessary its value should be increased accordingly. This is because in the opinion of this author, some operations may produce higher impacts, as for example stripping and soaking pit cranes.

2.2 HORIZONTAL FORCES

The crane runway structure has to resist horizontal forces in addition to the vertical wheel loads. The main causes of these forces are:

- Traction forces
- Crane bridge impact on the crane stop
- Skewing of the crane.

The horizontal forces that are caused by friction are more regular in occurrence. They are the result of the crane bridge accelerating and decelerating on the runway and the trolley accelerating and decelerating on the bridge. The maximum value of these forces is the maximum friction force between the wheels and the rail. If this friction force is exceeded the wheels will either spin or skid. The horizontal forces can be classified as follows:

- Longitudinal forces which act along the top of the rail
- Lateral forces which act transversely to the top of the rail.

In a similar way to the impact factor, values assigned to these forces are not the result of scientific measurements. To do so would require on site measurements and data collection, using expensive equipment, over a long period of time which might be years rather than months.

In general, values obtained from investigating one crane may not be easily generalized and applied to other cranes due to the following variable factors:

a) Difference in their types.

b) Difference in their speeds.

c) Difference in their bridge spans.

d) Difference in bridge flexibility.

e) Difference in operating conditions.

f) The manner with which operator drives the crane.

g) Misalignment of rail on the structure.

h) Condition of top of flange of the rail.

i) Misalignment of crane bridge in relation to the runway.

j) Difference in magnitude of forces acting on crane stop.

Therefore, to undertake such an investigation, it may be necessary for more than one crane to be studied. Such an investigation, over and above requiring time, money and expert manpower, might be hindered by the owners reluctance to participate. This reluctance on the part of owners is mainly due to the fact that such investigations will cause production interruptions that can be very expensive.

2.2.1 Longitudinal Forces

These forces are caused by the bridge accelerating and decelerating on the runway (traction), and by the crane running into the stop. They act along the runway in the direction of the bridge travel.

A. Traction Force:

The value used as a general practice, by a majority of designers is 10% of the total wheel loads on one side applied to the top of rail.

Table 2.1 shows minimum values assigned to the longitudinal force by a few authorities. It can be noticed in the table, that all of these authorities, except AISE, give the values without specifying the type of crane.

These values are more the result of agreement among various groups, and may also be based on the assumption that the coefficient of friction between the rail and the wheel is 20% with half (10%) applied to each side. This author has not been able to establish conclusively the logic behind using the 10% factor. One can only speculate.

Since the coefficient of friction is the property of surface in contact, therefore the value of the longitudinal force should be 20% of the total wheel loads on that side, assuming that the coefficient of friction is 20%. On the other hand, the 20% factor may be questionable by being based on conditions which may be too ideal.

To assume that the rail and wheel surfaces are smooth and in good condition may not be true. They could be rough due to wear and tear, thus producing a higher coefficient of friction. For example, Maas [12] reports a measured maximum value for the coefficient of friction between 35% and 40%.

The preferred course of action by this author would be to use a minimum of 20% of the total maximum wheel loads on one side applied to the top of the rail. The designer, by using his own judgement, should

TABLE 2.1

VALUES FOR LATERAL AND LONGITUDINAL FORCES

AUTHORITY	LATERAL (one side)	LONGITUDINAL (one side)
CISC Section 6.4	10% of Sum of Weight of Lifted Loads and Trolley	10% of Maximum Wheel Loads
NBC Section 4.18.4	10% of Sum of Weight of Lifted Loads and Trolley	10% of Maximum Wheel Loads
AISC Section 1.3.4	10% of Sum of Weight of Lifted Loads and Trolley	10% of Maximum Wheel Loads
B.S.499 Part 1: 1970 Part 2: 1069	10% of Sum of Weight of Lifted Loads and Trolley	5% of Maximum Wheel Loads
AISE St'd 13: 1969	1) 10% of combined lifted load and trolley.	20% of Maximum Wheel Loads
	2) 5% of the lifted load and entire crane weight, including trolley, end trucks and wheels.	
	3) Mill Cranes: 20% of Lifted Load	20% Maximum Load on Driven Wheel
	Ladel Cranes: 20% of Lifted Load	20% Maximum Load on Driven Wheel
	Clamshell & Magnet: 50% of Lifted Load	20% Maximum Load on Driven Wheel
	Stripping Cranes: 100% of Lifted Load	20% Maximum Load on Driven Wheel
	Soaking Pit: 100% of Lifted Load	20% Maximum Load on Driven Wheel

evaluate this value for each case and adjust it accordingly if so required.

B. Crane Stop:

Located at the ends of the runway, the crane stop is an important structural component of a runway system. The main funtion of the stop is to be able to sustain medium impact without any appreciable damage to itself or the structure. In addition, the stop must be capable, under full impact force, of preventing the crane from leaving the runway.

The general philosophy appears to be that damage to the stop and the structure below it can be tolerated, if major damage to the crane can be prevented. A crane is an expensive piece of equipment, upon which production depends. Also by preventing the crane from leaving the runway, major accidents may be prevented where lives are involved.

In performing its function, the crane stop becomes subjected to an impact force which is dynamic in nature. This force may be mild if caused by the crane docking against the stop or running into it at low speeds. However, it may be severe if the crane runs into the stop accidentally. The degree of severity will depend on the velocity of the crane at the instant of impact, and its mass. The impact force on the crane stop is resisted longitudinally by the runway structure and so can be detrimental to the structure.

Idealistic, optimistic and sometimes questionable assumptions have been made in arriving at the value of the impact force on the stop. As mentioned below, calculating the force involves two design groups:

a) Engineers involved in the design of the crane and its bumpers.

b) Engineers involved in the design of the crane runway structure including its crane stop.

Those involved in design of Hydraulic Bumpers, design crane bumpers according to specifications which may generally be based on arbitrary assumptions. The following are examples of such specifications:

a) The AISE standard 6 specifies that bridge bumpers must stop the crane at 50% of full load speed with a maximum deceleration of 16 ft/sec^2, using the total weight of crane, exclusive of lifted load. All hydraulic bumpers must be capable of absorbing the total energy at 100% speed with corresponding increase in deceleration rate, to maintain the same travel distance in the bumper.

b) The OSHA* specifies that the bridge bumper must stop the crane at 20% of full load speed with maximum deceleration of 3 ft/sec^2. It must also have sufficient energy absorbing capacity to stop crane at 40% of full load speed. The total crane weight is used exclusive of the lifted load.

c) The CMAA* specifies that the bridge bumper must stop the crane at 20% of full-load speed with a maximum deceleration of 3 ft/sec^2. It must have energy absorbing capacity to stop the crane at 40% of full load speed. The total crane weight is used exclusive of the lifted load.

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*OSHA - Occupational Safety and Hazard Association

*CMAA - Crane Manufacturers Association of America

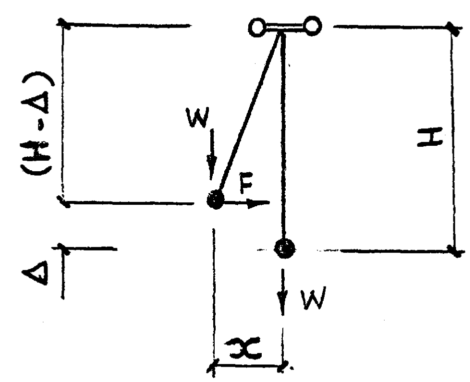
The first two authorities (AISE and OSHA) state that energy absorbing bumpers must be used on the crane bridge. The third authority (CMAA) states that the bumpers must be used, but they do not have to be energy absorbing, thus giving designers a choice between the two.

It should be noted that the above mentioned sources exclude the lifted load in calculating the bumper force. The explanation normally given is that the load is hanging at the end of the cable and the assumption is made that when the crane stops after hitting the crane stop, the load continues in its forward motion, thus not contributing to the impact force in a major way. As a result of a crane hitting the stop, the pay load will swing like a pendulum and it can be shown that the maximum longitudinal force F expressed as a percentage of the pay load W is given by the expression:

$$\frac{F}{W} = \frac{\sqrt{2\phi - 2\phi^2}}{1 - \phi} \qquad \text{where } \phi = \frac{2V^2}{2gH}$$

in which V = crane speed ($\simeq$ 7 ft/sec)

H = distance from cable drum to centre of gravity of the load.



For example if H 30 ft. and V = 400 ft/min. then F = 21.5% of W. Also the period of the swing will be approximately 6 seconds ($T = 2\pi\sqrt{H/g}$), so that the application of the longitudinal force F will occur approximately 1.5 seconds after impact. Now if the deceleration of the bumper is 16 ft/sec^2, the crane is brought to rest from a speed of 400 ft/min. in less than 1/2 second ($t = (V_0)_{/a} - (V_1)/a$). Thus it is reasonable that the impact force and the component of the load need not be considered as occuring simultaneously.

Using AISE recommendation as an example, it should be noted that the crane stopping force is designed for a speed less than that for the bumper design. The reason given for this approach is that the chances of a crane running at full speed into another crane or into the crane stop are relatively remote. Therefore to design all the cranes for 100% rated speed will result in larger and heavier cranes. Also the bumpers would have to be longer because of the longer stroke required to keep the deceleration rate to an acceptable limit (16 ft/sec^2 is considered acceptable), thus requiring longer buildings. Therefore the crane is designed to accept a force of 50% of its rated speed.

The bumpers, on the other hand are designed to absorb the total energy at 100% rated speed. This is to ensure that the bumper will not collapse in case the crane does crash at speeds higher than 50% of the rated speed, and cause extensive damage. It should be noted that at least one steel producer, very recently, has adopted the above specification of AISE standard 6.

Groups involved in designing the runway structure have their own interpretation of the operating conditions and their own appraoch to the design of crane stops. Some use the above mentioned specifications in calculating the bumper force on the crane stop. Others use values based on their own approach which may be different than above. For example, one major steel producer, in its updated deisgn standards for designing the crane stop, specifies a crane stop force of 20% of the sum of maximum wheel loads of the heaviest crane on that runway. Depending on the interpretation of individual designers this wheel load may or may not include impact.

From the above discussion, the assumptions used for calculating the force on the crane stop may be classified as follows:

a) The crane is assumed travelling at a speed less than its rated maximum at the instant of impact.

b) The hydraulic bumpers on the crane are assumed to be in good operating condition.

Validity of Assumptions

A crane bridge and its assembly is a heavy piece of equipment, travelling at speeds as high as 400 ft. per minute. They may be subject to severe operating conditions, depending on type of operation, such as mill cranes, stripping cranes, etc. Their condition depends also on the driver. On the average, in a 24-hour work day, three different drivers operate the crane. Each driver has his own characteristics for handling it.

To assume that the crane will not run into the crane stop at maximum or near maximum speed may be unjustifiably optimistic and thus lead to a false sense of security. Case histories demonstrate that the speed criteria are often exceeded. Crane operators, for example, have been known to race each other at full speed. In another incident the crane ran into the crane stop at such high speed that it sheared the crane stop and went through the gable end of the building. The crane was left hanging on the end of the runway. This may have been caused by absent mindedness, or by the operator falling asleep behind the controls.

Basing design specification on the capacity of hydraulic bumpers is a valid one, providing that they perform properly with no major malfunctions. To accept this may be highly idealistic. The Manufacturers of hydraulic bumpers, as a rule, always stress the advantages and reliability of their products. One particular manufacturer claims that his product is 100% maintenance-free under normal operation. He goes on to claim that no maintenance, whatsoever, will be required for the life of the bumper. However the phrase "under normal operation" is very vague. It would be very hard to define normal conditions in a heavy industrial operation.

If and when maintenance is required, it is very probable that it may be overlooked and neglected. Therefore, due to such abuse or due to some form of mechanical failure, the crane bridge bumpers can not be fully relied upon to absorb the impact force that they are rated for.

In one case two cranes travelling on the same runway collided with such impact that the hydraulic bumpers collapsed causing extensive damage to the crane bridges. This collapse may have been caused by:

- The cranes colliding at higher speeds.
- Under designed or malfunctioning bumpers.

As for the 20% design value, it is not clear where it originated. It may have been based on the assumption that the coefficient of friction between rail and the wheel is 20%. So when the crane is in the process of full braking the maximum longitudinal force induced in the structure is 20% of the sum of all wheel loads on that side of the runway. This is the force for which the bracing is designed. Therefore an aribtrary consensus may have been reached that the stop should be designed for the same force. Another assumption may have been that this force is always equal or larger than the design force of the crane bumper. Whether a coincidence or not, the calculations shown in Section 2.2.1 indicate that the force is approximately of the order of 20% of the pay load, which might be another source of origin for this value.

Recommended Approach

As yet, there is no established and universally accepted approach for calculating the force acting on the stop.

The confusion and lack of communication between various groups is a great contributor toward this non uniformity. For example, the above mentioned steel producer crane design groups have revised their approach to crane bumper design, while the runway design group is still using the 20% value. In other words, the crane bumper is now being designed to absorb total energy at 100% of the rated speed of crane.

The author's preferred course of action would be as follows:

a) Design the crane stop for 100% of the rated speed with crane empty, for a deceleration rate of 16 ft/sec^2.

b) Design the bracing and the foundation for the above crane stop force or 20% of the total maximum wheel loads, which ever is greater.

These criteria allow for the deviations discussed above. They also take into account the fact that if there is an increase in the price of the stop and the structure due to an increase in the design load, it will be relatively low. For example: The total price of a crane may range from 1.5 million to about 4 million dollars. Over and above its price, the lose in production caused by a damage to it may run into thousands of dollars.

The braced bays are the only parts of the structure effected by the crane stop in a major way. Assuming that the braced bays are located at the end of the runway, there will be a total of four bays envolved. Therefore assuming a 100% increase in the design force, the increase in the cost of the structure, including the foundations, may range between 1% to 2% of the total cost of the crane, not including the cost of production loss. In this author's opinion, the extra cost of the structure is relatively small.

Types of Stops

Crane stops detail, shape, size and their methods of mounting vary depending on the designer, owner and the crane. Figures 2.1, 2.2, 2.3 illustrate some of the common crane stops in use, along with their methods of support.

For light to medium capacity cranes the stop shown in Fig. 2.1 is occasionally used. It consists of a solid steel block constructed from plates. A piece of wood is attached to its bumper shaft to absorb some energy. Two rods, straddling the girder provide a tie to the runway. This type of crane stop has been avoided by many, due to low energy absorbing capacity. It may still be found in use for light duty cranes.

The crane stop shown in Fig. 2.2 has proven to be relatively efficient. The shaft is made of a pipe. The collapsable coil which is an extra heavy duty pipe is wrapped around the shaft. If the impact force is high this coil will collapse by straining beyond its elastic limit thus absorbing a certain amount of energy without recovery. A resilliant pad, sandwiched between two steel plates, is added to redistribute the load and also to provide a cushioning effect.

One major steel producer has used the above arrangement in areas of heavy cranes, and it has produced good results. Recently, efforts have been made to replace it with a hydraulic bumper.

Support of Stop

The crane stop force may be transferred directly through the girder into the bracing as shown in Fig. 2.3. This method may be used when bracing is located at the end. Figure 2.2 shows another mounting method whereby the force is transferred through the girder into the column cap plate, through the keeper bars, into the adjacent girder and so into the bracing. This occurs when bracing is located at the mid span of the runway.

2.2.2 Lateral Forces

These are caused by the crane trolley travelling on the bridge. They may also be the result of the crane bridge skewing during its travel. Lateral forces act in a transverse direction to that of the bridge travel, and laterally to the rail.

A. Trolley Force

The trolley is the unit which lifts the load. It travels on a set of rails on the crane bridge, in the transverse direction to that of the bridge travel. The lateral force created by the trolley acceleration and deceleration is similar to the longitudinal force in the sense that they are both tractional. The value assigned to the lateral force is 20% of the sum of the lifted load and the weight of the trolley, with one-half applied to each side of the runway (see Table 2.1).

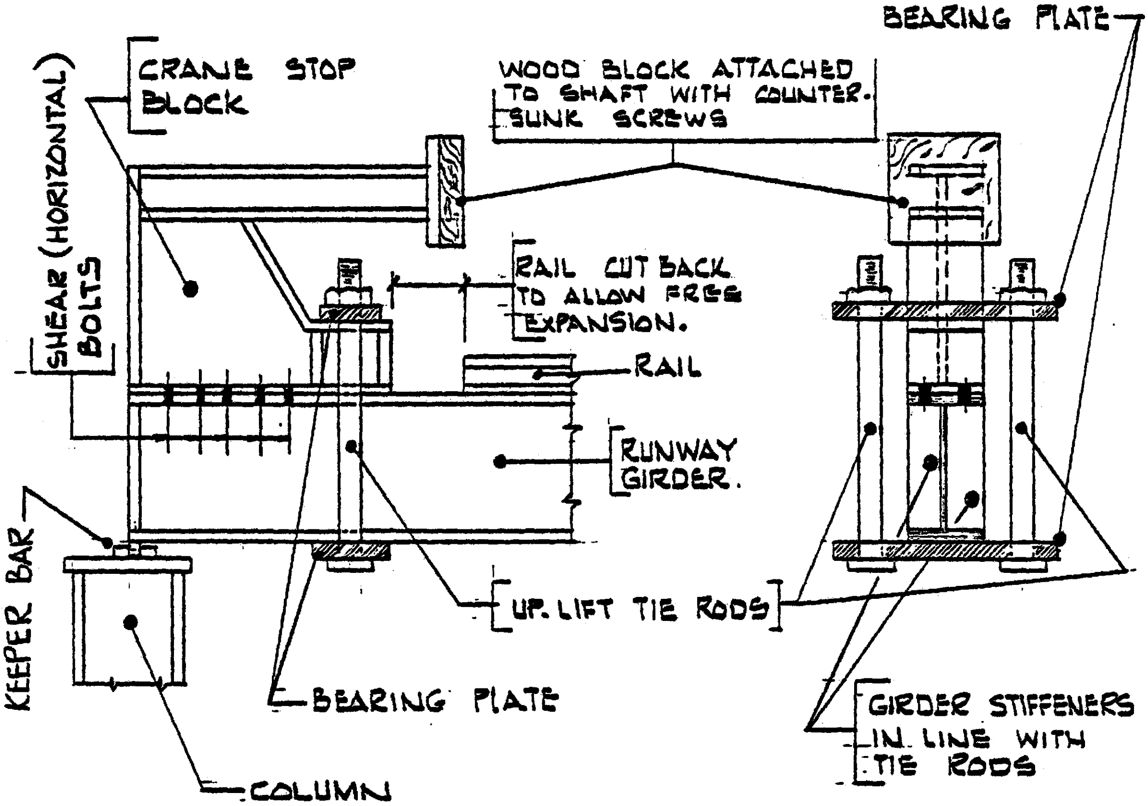


Figure 2.1 Solid Plate Crane Stop and a typical Method of Support

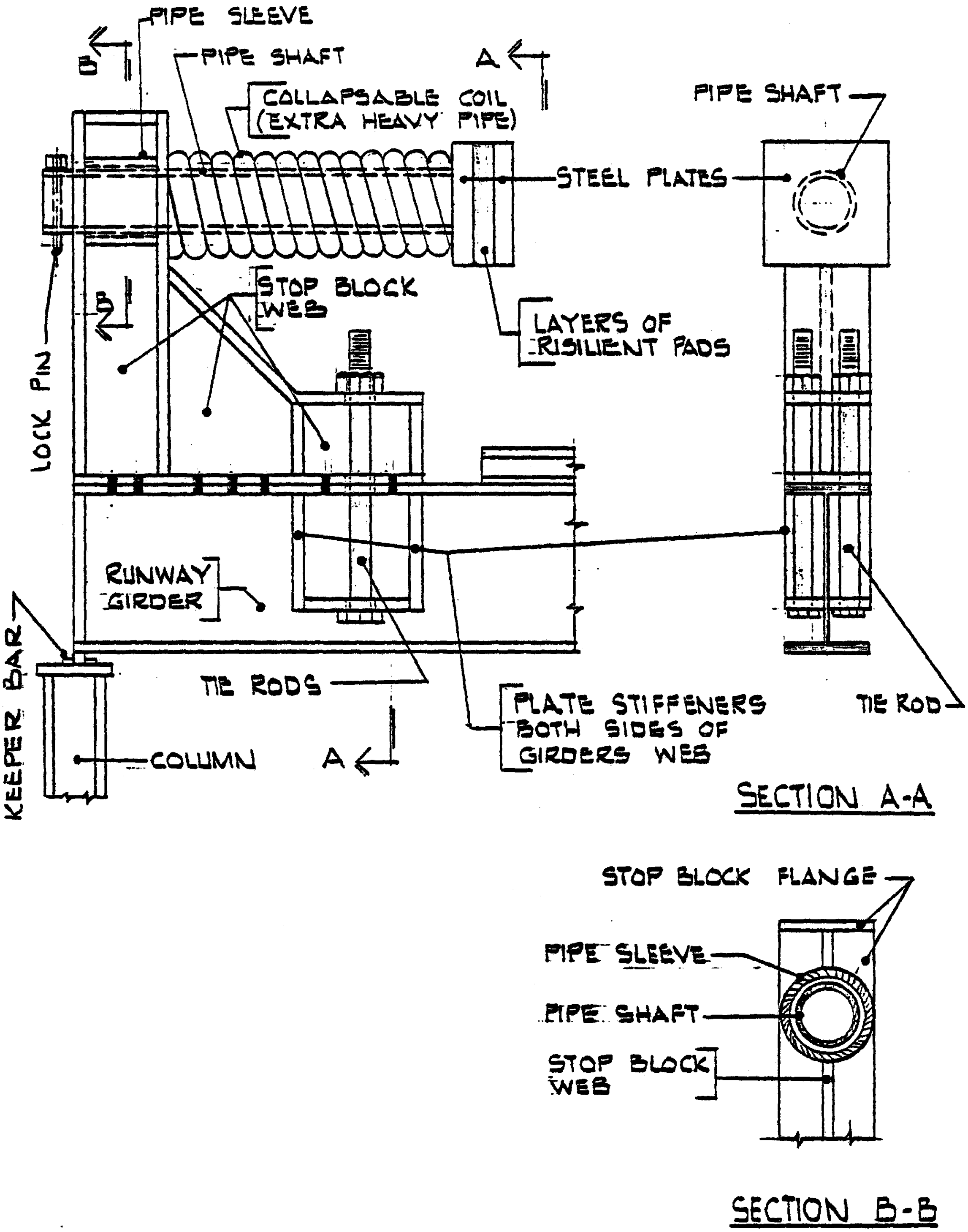


Figure 2.2 Collapsible Crane stop and a typical Method of Support

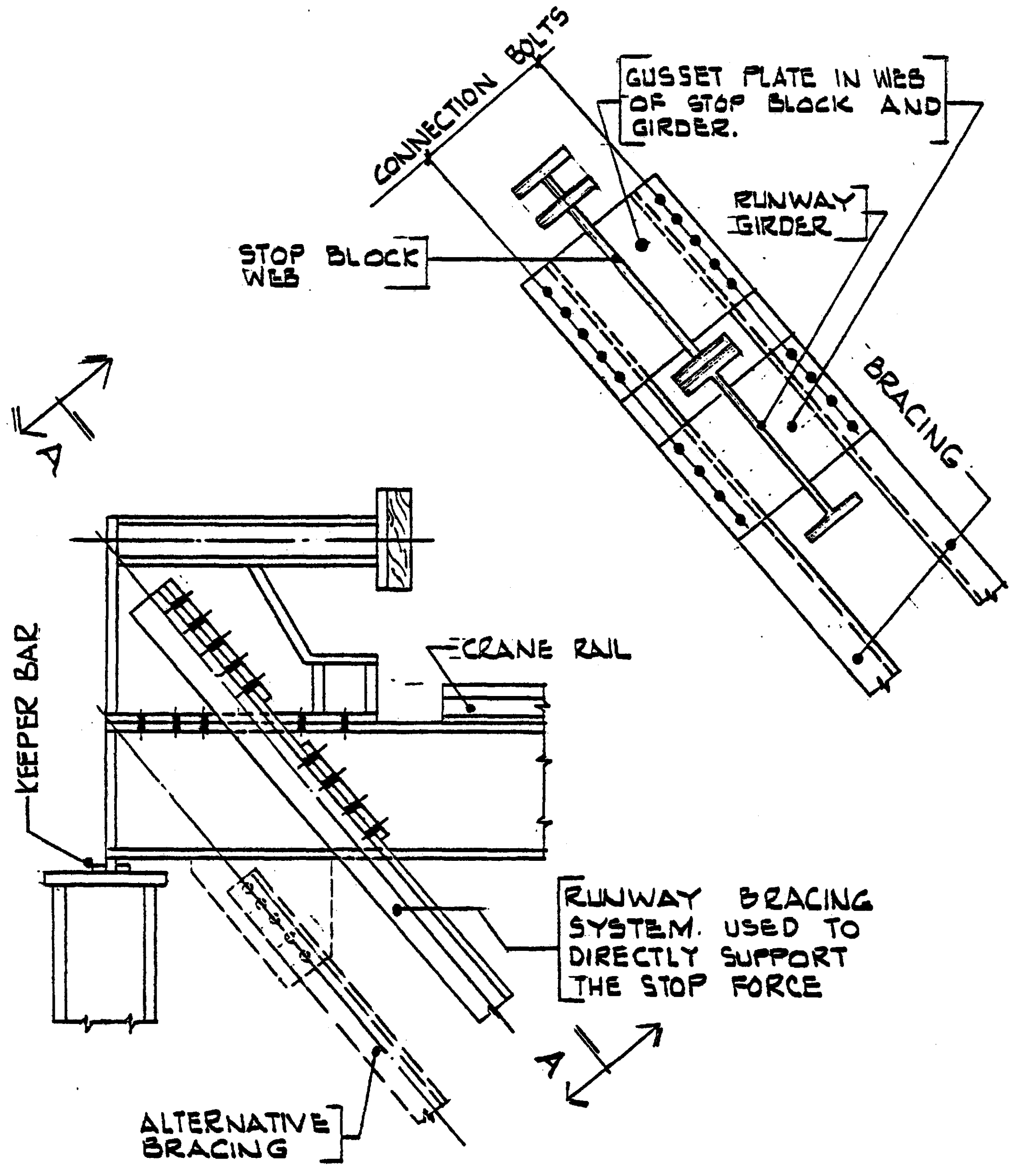


Figure 2.3 Typical Bracing Connection to Crane Stop and the Girder

The above value may be considered valid if the wheels on both sides of the runway bear against the head of the rail in the horizontal direction. But if the wheels do not bear on the head of the rail, then the lateral force will be wholly transmitted by friction. In such a case the above value will be valid only if the trolley was located at mid span of the bridge, thus producing equal vertical wheel loads. Hence the lateral load will be divided equally between the two sides of the runway.

In the author's opinion the above conditions do not exist at all times. The possibility that wheels on one side may bear against the rail head, will cause the whole lateral force to be induced in that side of the runway. If the wheels do not bear against the rail then the lateral force will be transmitted by friction. In such a case the trolley may be operating at the extreme end of the bridge (near one side of the runway), thus producing larger vertical wheel loads on that side. Since the maximum lateral force is the friction force between the rail and the wheel, there will be more force applied to one side than the other.

The author's preferred course of action would be to design for the maximum condition. This would be to assume that the wheels are bearing laterally on the head of the rail on one side of the runway. In such a case the whole lateral force will be transmitted to that side.

If the wheels are such that the above approach is not true, then the lateral force should be calculated by placing the trolley to one extreme, keeping in mind that the maximum lateral force can not be larger than the friction force between the wheel and the rail, and also that a coefficient of friction higher than 20% may exist, depending on the surface conditions of rail and wheels.

B. Crane Bridge Effect on Structure

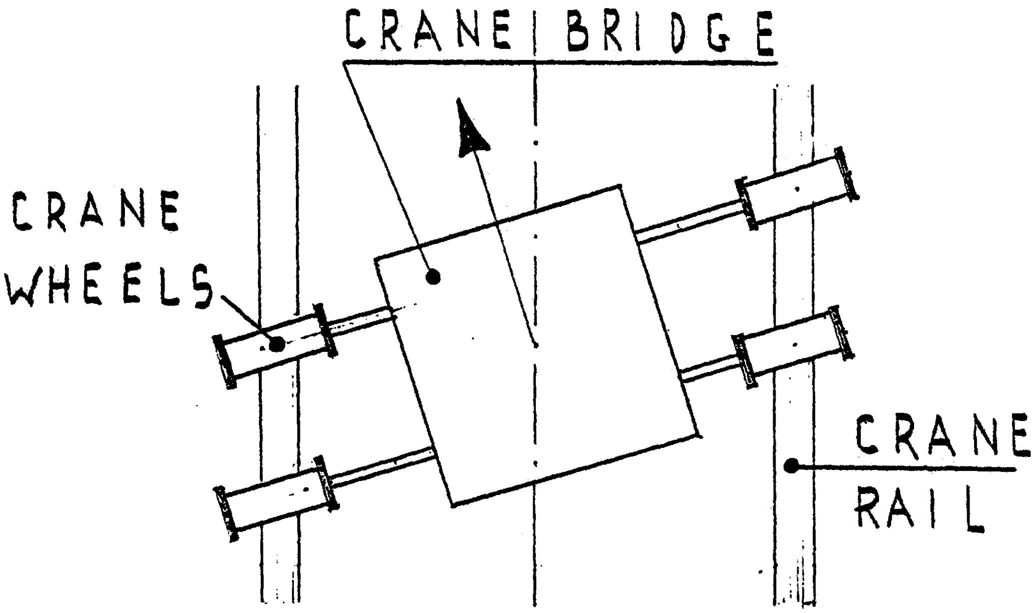
The skewing action of the crane, when travelling on the runway, can be the cause of horizontal forces in the structure (see Fig. 2.4). Skewing action causes the wheel to bear against the rail thus causing lateral thrust. It also produces surface wear in wheels and rail. Since horizontal forces are also a result of friction, their magnitude can be higher due to rough surface. Many studies have been, and are being carried out. Different crane systems are currently being tested for lessening the skewing action. Methods vary, ideas and approaches differ, with no common consensus for an acceptable solution.

A crane system which can eliminate the skewing action, will reduce the wear in wheels and rail, thus keeping the coefficient of friction low, hence lower longitudinal forces. But the effect of lateral force can not be totally removed, because lateral forces have to be applied to the rail in order to stabilize the crane.

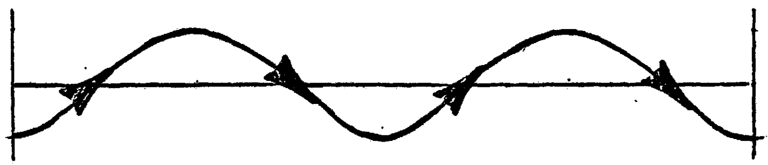
2.3 APPLICATION OF LOADS

When calculating the maximum bending moment, it is normal for the following procedure to be used:

a) A full impact value of 25% is included in the wheel loads (see section 2.1).



"a"=PLAN SHOWING SKEWING OF CRANE



"b"=TRAVEL PATH

Figure 2.4   Skewing Action of Travelling Crane

b) The 25% allowance should apply also for the lateral forces.

c) If two cranes are operating on the same span, impact is added to one crane only (the heavier of the two), and the bending checked with the two cranes bumper to bumper.

d) In the same way the estimation of lateral forces with two cranes on the same bay sould be treated as in (c) above.

The reasoning behind this practice can be summarized as follows: it is possible for the crane to be located such that it will produce maximum moment while at the same time the trolley can be moving laterally toward one side of the runway. While all these conditions are occurring, it is possible that impact may occur simultaneously. Thus the inclusion of the impact factor may be considered valid.

If two cranes are operating on the same runway, it is possible that they will be working bumper to bumper to produce maximum bending. But it is unlikely that both will produce impact at the same time. This is the reason for adding impact to one crane only.

When calculating maximum reaction in the column, the following assumptions are commonly used:

a) Full impact is included for two cranes operating on adjacent spans.

b) In the same way the estimation of lateral forces should be treated as in (a) above.

The approach used for calculating the column reactions is based on the idea that it is possible for two cranes to be operating on adjacent bays in such a manner that they will produce maximum column reactions in the vertical and lateral direction. While this is occurring, it is also possible that both may produce impact force.

When calculating the maximum longitudinal force that acts on the structure, it is proposed that the sum of forces produced by all the cranes be considered acting on the runway. The reasoning for this assumption is that it is possible for all the cranes to brake or accelerate in the same direction simultaneously while carrying their full pay load.

Summary

Table 2.1 shows the values for lateral and longitudinal forces recommended by some of the more common authorities. The first three authorities (CISC, NBC, AISC) recommend a minimum value of 10% for the lateral and longitudinal forces. BS449 recommends the same for lateral force with a lower value of 5% for longitudinal. These authorities seem to take the same application approach. The forces are applied literally to all design cases with no consideration for the type of operation and conditions that the crane may be used for. AISE ST'D. 13, is the only authority that specifies the type of operation and adjust the values accordingly. In giving the values for lateral forces, the largest value produced by one of the three cases is used. As for the value of the longitudinal force a 20% value is recommended. It should be noted here that the 20% value is twice as large and it is, in the author's opinion, more appropriate than the 10% value as it was discussed before.

Conclusion

From the discussion in this chapter, it can be concluded that the value assigned to the impact factor of vertical wheel loads and the values assigned to the lateral and the longitudinal forces may not be representative of the true conditions. The same applies to the value asssigned for the design of crane stop. Therefore research and study in these areas will be most welcomed.

CHAPTER 3

VERTICAL LOAD SUPPORT

The rail and the girder are the main components of the runway through which the vertical loads are supported, and hence are the areas of discussion in this chapter. The methods of detailing and mounting of the rail and their associated effects on the girder will be discussed.

The discussion of the girder will centre on the design criteria such as the stiffness and the allowable design stresses, and on the methods of detailing along with their advantages and disadvantages. This will include the torsion in the top flange, connecting the flange to the web and the connection of the stiffeners to the web and the flanges.

The aim is to bring to light the problems and inconsistencies in current design practice and to high light topics for future studies.

3.1 RAIL

The growing demand by industry for higher production has resulted in the development of heavier cranes with very high speeds. Hence the demand for efficient and durable crane runway girders and their support structures has been constantly increasing.

Rail is the member on which the crane depends and runs. It is also the member through which forces and loads are transmitted into the structure.

The cost of rail compared to the total cost of the crane and its supporting structure is relatively small. If the rail is faulty, damaged or poorly mounted, however it may result in costly shutdowns and interruption in the production by causing failures such as wear in the wheels and bearings, and axles breakage. It will also cause failures in the structural members of the runway as discussed later in this Chapter. Therefore, to avoid unnecessary damage to the rail, the manner in which it is mounted on the girder becomes very important.

3.1.1 Improper Mounting and Associated Problems

It is usual to find rail mounted directly on the girder, in segments and connected by bolted splices on the rail web. The rail is frequently attached rigidly to the top of the runway by either clamping or welding, such that it has a tendency to act as part of the girder top flange thus experiencing the same stresses. Due to this type of attachment, the rail expansion joint will have to match that of the structure since it will expand in the same direction. The expansion joint is built in by splicing it with sliding bolted connection. Experience has shown that this method of detailing and mounting can be a source of many problems.

Rail mounted directly on top of the girder can result in poor bearing between the underside of the rail and the top flange of the girder. Full bearing is essential for proper load distribution. Lacking full contact area, the load will be transmitted into the top flange in a concentrated manner (point load), which may result in one or more of the following undesirable conditions.

(a) High local stresses in the top flange and at the toe of the web.

(b) High fatigue stresses caused by shock forces, transmitted through incomplete surface contact.

(c) High eccentricity in the top flange.

(d) Damage of the rail itself.

The lack of proper contact area appears to be the result of the built in camber in the underside of the rail. Since the rail and the top flange of the girder are stiff members, wheel loads may not be sufficient to flatten the camber and increase the contact area. Compounding the problem of a small contact area, the rail may wander laterally on top of the girder. This will cause both the size and position of the contact area to be randomly distributed. Complex and undetermined stress patterns will thus be created.

Bolted splices and bolted expansion joints are a poor method of mounting. In the author's opinion, this type of splice can result in poor and rough joints due to loose fitting bolts and splice gap with the resultant bad effects of rail roughness. Rough and uneven joints create high localized shock forces. These forces are directly transmitted into the girder top flange causing increased stress concentration which in turn may lead to an increased chance of fatigue failure in the top flange and vicinity. For example, it has frequently been found during maintenance inspection that a local failure in the upper flange area is located directly beneath rough joints, holes or other irregularities in the rail which result in increased impact loading.

To avoid rough joints which can act as stress raisers, the device is sometimes employed of mounting the rail continuously over the girder. By connecting the rail rigidly to the top flange, it will have a tendency to act as part of the girder as mentioned above. The rail will tend to restrict the end rotation of the girder (discussed in section 3.2) thus experiencing bending stresses which may cause it to break, see Fig. 5.2(b). Welds connecting the rail to the top of the girder will experience similar stresses as those of the girder, and the wheel loads will be transmitted directly through them thus causing them to break.

Rail which is continuous and rigidly fixed to the top flange will distort due to expansion. In general, structures may experience a temperature range of the order of $100^{\circ}F$ or more depending on locality and on whether they are enclosed or not. For example if the runway is anchored (braced) at its ends, it will expand toward the middle. This will cause the rail to distort or break because of its tendency to expand in the opposite direction, away from its centre and toward its ends.

3.1.2 Improved Rail Mounting

Rail should be continuously mounted with its splices welded according to approved welding procedures. Welds should be ground smooth in the direction of stress (longitudinal) in order to eliminate rough joints which were discussed in the previous section.

Rail should be allowed to float, instead of fixing it rigidly to the top flange. It should have a degree of freedom in the vertical and logitudinal direction. It should be restricted in the lateral direction to ensure proper location with respect to the centre line of the girder. Many of the difficulties described in the previous section may be all-but eliminated by the proper use of an appropriate pad of packing material between the rail and the girder.

The vertical movement is necessary to avoid direct contact between it and the top flange of the girder for reasons discussed in the previous section. The longitudinal freedom of movement is essential in order to avoid damage caused by expansion which was also discussed in the previous section.

Floating action can be achieved by using a resillient (damping) pad (very hard rubber or equivalent about 11/32" thick). The pad has a width equal to that of the rail flange and is installed between the rail flange and the top flange of the girder. The main functions of this pad can be summarized as follows:

- Redistributes and recentres the load.
- Eliminates point contact between the rail and the top flange of the girder.
- Reduces impact.
- Reduces noise.
- Reduces vibration.
- Eliminates wear in the top flange of the girder.
- Eliminates strains and distortion caused by expansion.

The advantages of the damping pad have been proven through practical applications and scientific studies. Runways have been known to develope cracks due to fatigue in areas where the damping pad was either worn out or missing. Senior [11] reports a decrease of about 45 percent in direct stress by use of the damping pad. Periodic maintenance and replacement of the pad is essential. The pad loses its resiliency after a certain period of time. Neglecting its replacement will cause extensive capital losses in terms of repairs and shutdowns.

The rail is usually held to the girder with clips. These rail clips are made and installed so that they provide the following:

- Lateral adjustment of rail during and after installation.
- Constrain the rail in the vertical and lateral direction within the required tolerances suggested by AISE [5].
- Allow a degree of movement in longitudinal direction.
- Ensure continuous hold down for the rail. Thus they maintain proper contact between the rail, damping pad and top flange of girder as shown in Fig. 3.1.

3.2 MAIN GIRDER

3.2.1 Stiffness of Girder

The crane runway girder differs from many other structures because of its function and the operating conditions. Stiffness in the girders is an important requirement for many reasons.

Stiff girders are required to avoid the "roller-coaster" conditions which can exist if the girder deflects excessively. It has also been found through experience and feed-back from operators that the

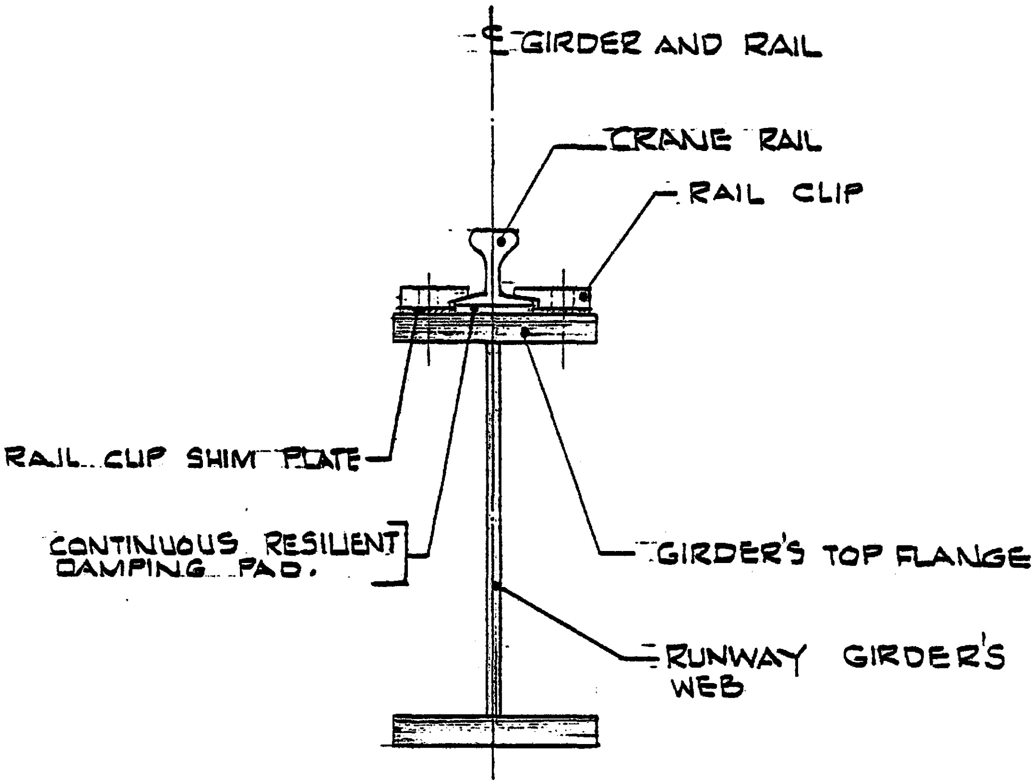


Figure 3.1 Soft Mounting for Rail

crane operators, for psychological reasons, are reluctant to operate the crane at its full speed when the girders are soft or bumpy. These conditions can be agravated by the crane bridge being bumpy also.

Stiff girders are also necessary to avoid excessive end rotation, which can cause high fatigue strains and failures in the connection. End rotation will cause the rail to bend upward thus inducing flexural stresses in it. See Fig. 5.2b.

Since high stiffness is a requirement, the inertia becomes a major design criteria. Therefore the depth to span ratio is an important factor in the design process. It has been found through experience, that a maximum deflection value of span/1000 for the total dead and live load, and a minimum depth equal to span/10 have resulted in functional girders. Such a design will in the long run prove to be economical due to the reduction in maintenance cost arising from proper design and detailing.

Note that the tolerance in terms of the rail floating action discussed in section 3.1.2 are of the magnitude of about .009 inches, whereas the deflection of the girder is 1/1000 of span or 0.6 inches for a 50 foot span. Moreover the deflections in the pad are local whereas girder deflections affects over 50% of the span.

Recommendation made by AISE regarding the maximum deflection is span/1000 for the live load only which is nearly the same as above since the dead load is relatively small compared to the live load. Flexural load due to self weight is typically of the order of 3% - 5% of the total. The value for depth of span/16 recommended by AISE is however smaller compared to the previous criteria.

3.2.2 Allowable Stresses

Crane runway girders are dynamically loaded structures. The loading is cyclic and it varies between the minimum (self weight) when the crane is not on it, and the maximum (dynamic live load and self weight) when the crane is passing over it. The girder will experience positive (downward) flexural stresses if it is simply supported, and the reversal of stresses if it is continuous over the support. (This is discussed in Chapter 7). Therefore the girder should be designed according to fatigue design criteria.

Fatigue design specifications given by the latest issue of the existing codes (namely CISC, AISC and AISE) are different than those for static load design. The main design criteria for fatigue design given by these codes is as follows:

A. The loading conditions are based on the expected number of applications of the maximum load that will occur over the life span of the structure.

The classification of loading conditions is:

1) 20,000 cycles to 100,000 cycles

2) 100,000 cycles to 500,000 cycles

3) 500,000 cycles to 2,000,000 cycles.

4) over 2,000,000 cycles.

Each of the above loading classifications is assigned its own allowable design stresses.

B. For each of the loading conditions mentioned above the allowable stress range is categorized according to the type of girder and its fabrication details, which are mainly:

a) for rolled wide flange section

b) for built up welded plate girder

c) for girders with attachments such as web stiffeners that may act as stress raisers

C. The maximum stress allowed in the design is specified as follows:

1) It must not exceed the basic allowable bending stress of 0.66 F_y which is normally used in the static load design.

2) The maximum stress range (maximum allowable less the minimum), should not exceed certain values, for each particular loading condition and girder category. These values are tabulated in the afore mentioned codes, and are reproduced in Appendix A.

The above criteria given by the existing codes have rationalized the fatigue design of dynamically loaded structures. These specifications are for general application. They apply to structures the loading for which can be classified as a fatigue loading. Therefore to apply these specifications to the design of heavy duty crane runway girders, their relevant points must be identified.

Practical experience and studies* have proven that crane runway girders do experience more than 2,000,000 cycles of loading during there life span. Therefore girder design should be based on loading condition number 4 of the specifications which is over 2,000,000 cycles.

The crane runway girders can be rolled wide flange sections or built-up welded plate girders. Attachments (stress raisers) such as stiffeners are common, especially in deep web girders. Also depending on the type of detailing and member arrangements, other stress raisers may be present. Therefore all the categories listed and tabulated in the codes, and which are illustrated by sketches (see Appendix A) should be investigated in order to arrive at the relevant allowable stress range.

For the loading condition of over 2,000,000 cycles a total maximum allowable stress of 0.66 F_y is specified. The allowable stress range for the same loading condition varies with the type of girder section and its fabrication details. Typical examples are as follows:

a) For a rolled wide flange section an allowable stress range of 24 ksi is specified.

b) For a welded plate girder with no attachments (stress raisers) such as stiffeners an allowable stress range of 15 ksi is specified.

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* AISE studies.

c) For girders with web stiffeners the allowable stress range specified is 12 ksi at the stiffener's location when the shear stress is equal or less than half the allowable shear, and 9 ksi when the shear stress is higher than half the allowable shear stress.

Compared to other structures, the minimum (dead weight) stress of crane runway girders is small relative to its live load stress. Therefore, it can be concluded from the above specified values that the design stress for rolled wide flange section can be very close to the specified allowable of 0.66 F_y. But for the built-up welded plate girders, the design stress will most frequently be less than the maximum allowable of 0.66 F_y.

On the other hand design policy in certain companies allows the use of allowable bending stresses which are different from the values mentioned above, depending on their own interpretation of design cases. Lack of emphasis given to crane runways by the various codes, and lack of understanding of the crane runway structures along with their associated loading conditions and fabrication details, has led to a marked absence of standardization in the approaches used in crane girder design. Despite the existence of general and sometimes detailed guide-lines in Codes of practice, it seems clear (from the authors experience) that different design groups continue to use allowable stresses different from those specified earlier in this section.

Current practice appears to favour the adoption of maximum allowable bending stresses as follows:

a) 0.50 F_y for rolled wide flange girder sections.

b) 0.45 F_y for built-up welded plate girders.

c) For the allowable stress ranges the CSA (Canadian standard association). Standard S16.1 should be followed. The allowable stress ranges should conform to the maximum allowable stress given above, rather than 0.66 F_y value.

The above values can be justified as follows:

- It provides an allowance for the possible over loading of the crane by the operators.
- It is an added factor of safety against possible errors in design or loads. This factor of safety which is between 45% and 50% is reasonable if compared to that of static design which is about 33%.
- Insurance against possible mill rolling irregularities in the material used for construction.
- Insurance against the possibility of poor fabrication.
- Allowable stress values given by the authorities are based on the loading condition of over 2,000,000 applications of the maximum stress cycle. For crane runways in heavy mill operations, this may not be sufficient. Murray [9] and Mass [12] report that in a crane girder, one impact application produced 40 cycles of cyclic strain before damping out. Mass also reports that the 2,000,000 cycles was reached within a period of 18 months.

3.2.3 Torsion in Top Flange

Torsion in the girder can be a major problem to which many failures can be attributed. Torsion can be caused by the lateral forces which act transversely to the top of the rail causing a twisting moment in the top flange. This is especially true if the rail is rigidly fixed to the top of the girder.

Torsion can also be caused by the manner with which the vertical wheel loads are applied to the girder. In theory the vertical loads are applied normal and concentrically to the top of the rail, and the rail is positioned at the centre line of the girder. In practice, this is not always the case. The load might be applied to the top of the rail with a degree of inclination resulting in a horizontal component or it might be eccentrically applied thus causing torsion. These conditions may be attributed to the following:

a) Misaligned rails: Rails which are not aligned with the centre of the girder will transmit the load with some eccentricity.

b) Random contact between the underside of the rail and top of the girder: Full bearing between the underside of the rail and top of the girder may not be possible especially when the rail is mounted directly on the girder. Therefore the load might be applied at some distance from the centre in a concentrated manner rather than being uniformly distributed over the width of the flange (see Section 3.1.1).

c) Physical condition of rail: Distorted or unevenly worn rails will cause the wheel to bear sideways causing the load to be induced at an angle to the vertical. The magnitude of this angle will vary depending on the angle of the surface wear and on the degree of distortion.

d) Wheel taper: This will cause the wheel to bear against the rail at some inclination and thus create a side thrust at the top of the rail. The angle of inclination could be of order of 15 degrees to the vertical.

e) Crane bridge deflection: This causes the ends of the bridge to rotate about the wheels, causing them to bear against the rail at an angle to vertical. This angle might be of order of 7 seconds.

f) Foundation settlement: This causes the wheel to exert an inclined thrust on the rail (see Section 6.2).

The individual torsional effect of the above conditions may be negligible. But if more than one condition exists simultaneously the torsional effect may become significant.

In dealing with problems of torsion, current practice is to design the girder to resist it. It is necessary to design for torsion, but it may not be very practical. Proper and rational values for eccentricity are necessary in order to achieve a rational design. Such values may be hard to establish since they depend upon many undetermined factors such as those mentioned above. Design values used for eccentricity varies from one group to another. There seems to be no agreement among design groups on a valid formula or factor for eccentricity.

For example, Senior [11] reports eccentricity with transverse point contact between rail and the top flange, located at the edge of the rail bottom flange. This suggests that the above value of eccentricity is equal to half the width of the rail bottom flange measured from the centre of the beam. The assumption here is that the centre line of the rail is aligned with that of the girder, which may not be the case. Maas [12], on the other hand suggests values which are different to those of Senior. An eccentricity value of 40% of the rail head measured from centre axis is suggested. He also suggests an eccentricity value of one third (33%) of the width of the rail bottom flange, providing the rail is very straight.

Current practice in designing for torsion is to assume that the entire section of the girder participates in resisting the torsional moment. This may not be true in the case of runway girders which are normally deep and their webs relatively slender. Therefore, the top flange and part of the web will have to be depended upon to resist the moment. This in itself will not be strong enough to resist an appreciable amount of moment.

Therefore, the above practice may not be valid in the case of runway girders especially if the eccentricity values suggested above by Senior and Mass are a possibility. Therefore, to suggest a provision for the eccentricity in the design is not practical. Such an approach is curative rather than preventive, and may create a sense of false security regarding the soundness of the structure. Thus the tendency will have less emphasis on proper details and connection arrangements.

A detailing approach would be more appropriate in dealing with torsion that is caused by eccentricity. It would be more of a preventive measure rather than a curative one. This may be achieved by:

a) Proper rail mounting discussed in Section 3.1.2.

b) Designing the crane bridge for maximum stiffness.

c) Proper and regular maintenance. Periodical replacement of rail and the damping pad is essential.

d) Prompt replacement of damaged wheels.

Proper rail mounting as mentioned above includes the installation of the damping pad between the rail and the girder flange. The pad redistributes and recentres the vertical load evenly across the top flange of the girder. The redistribution of the load will reduce the eccentricity by eliminating the random contact through small bearing areas between the rail and the girder flange.

It may also reduce the torsional effect of the lateral (transverse to the rail) force. As the lateral force is applied at the head of the rail, a moment is created causing one edge of the rail to lift and the other to push downward. Since the pad has the ability to yield under vertical pressure, the effect of the couple created by the moment will be reduced.

The top flange will receive some stabilization against rocking through the surge plate (see Secton 4.1.1, Fig. 4.2) which is connected to it. Through proper detailing part of the plate will resist twisting thus giving some form of assistance to the flange.

3.2.4 Top Flange

The top part of the girder, which consists of the top flange and the top part of the web, is usually subjected to a variety of loads and forces. Vertical wheel loads may create high local bending stresses in the flange. They may also cause high compressive stresses between the top of the web and the underside of the flange. These local stresses can be aggravated by rocking and torsion caused by the eccentricity of loads. As discussed in the previous section, eccentricity is primarily the result of horizontal forces, eccentricity of the wheel on the rail, of the rail on the girder or distorted rail, or flange.

Fatigue failures in the vicinity of the top flange are frequent. They can be attributed to poor methods of detailing and construction, rather than design. For example, in a built-up plate girder the usual approach is to connect the top flange to the web by fillet welding, where full bearing contact between flange and the web is not always possible. This is attributed to mill rolling irregularities in the plates, which cause randomly distributed gaps between surface of the flange and the edge of the web. This condition may cause unevenly distributed bearing stresses, thus creating areas of high stress concentration, the result being cracks in the fillet weld. Also, if the weld is not finished with a smooth transition, it may act as a stress raiser, causing failures in the web at its toe (see Fig. 3.2a).

In one known case a rolled wide flange section girder (Fig. 3.3)*

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* Author's personal expeprience with an ore bridge structure.

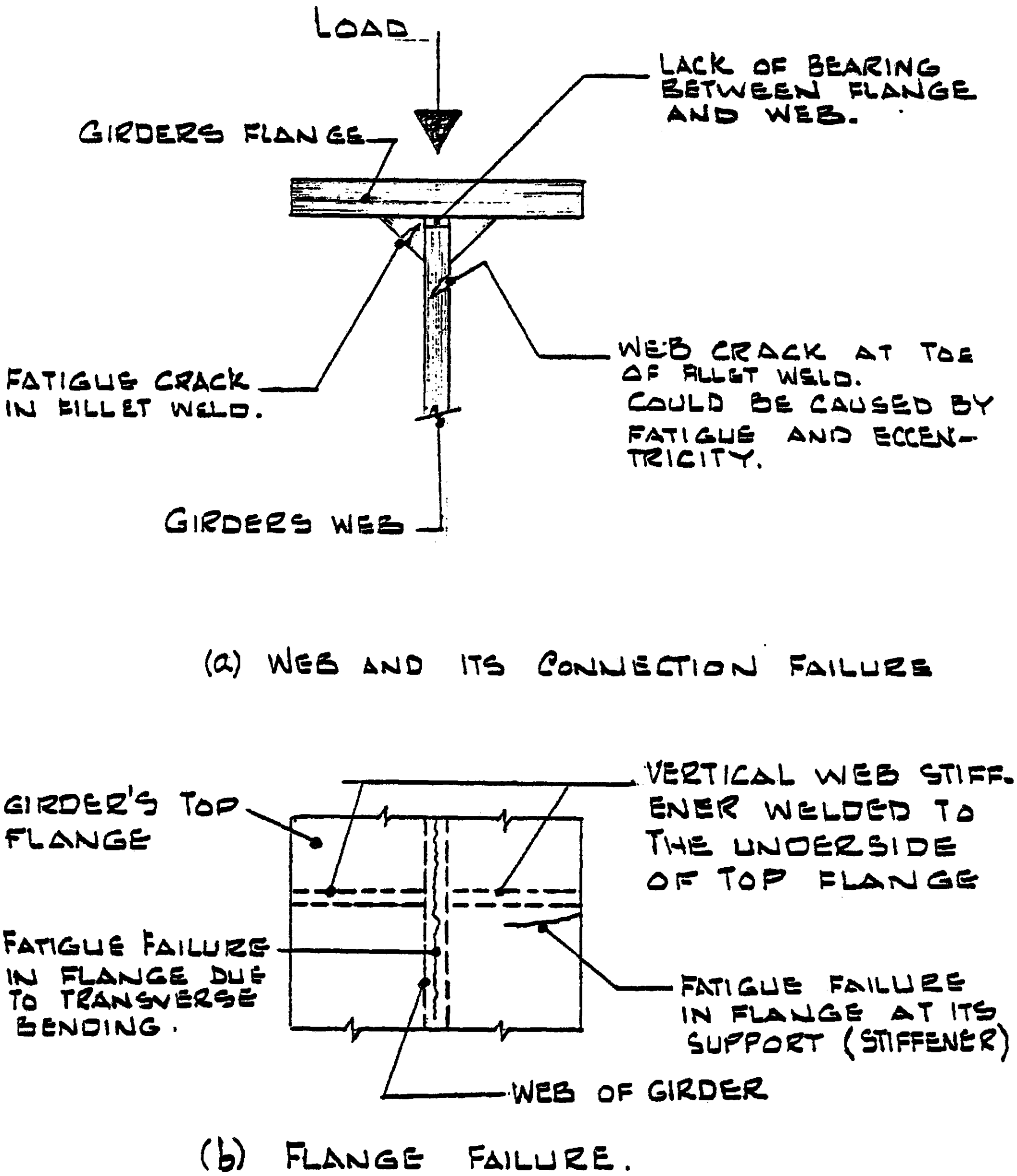


Figure 3.2 Typical Fatigue Failures in Top Part of Girder

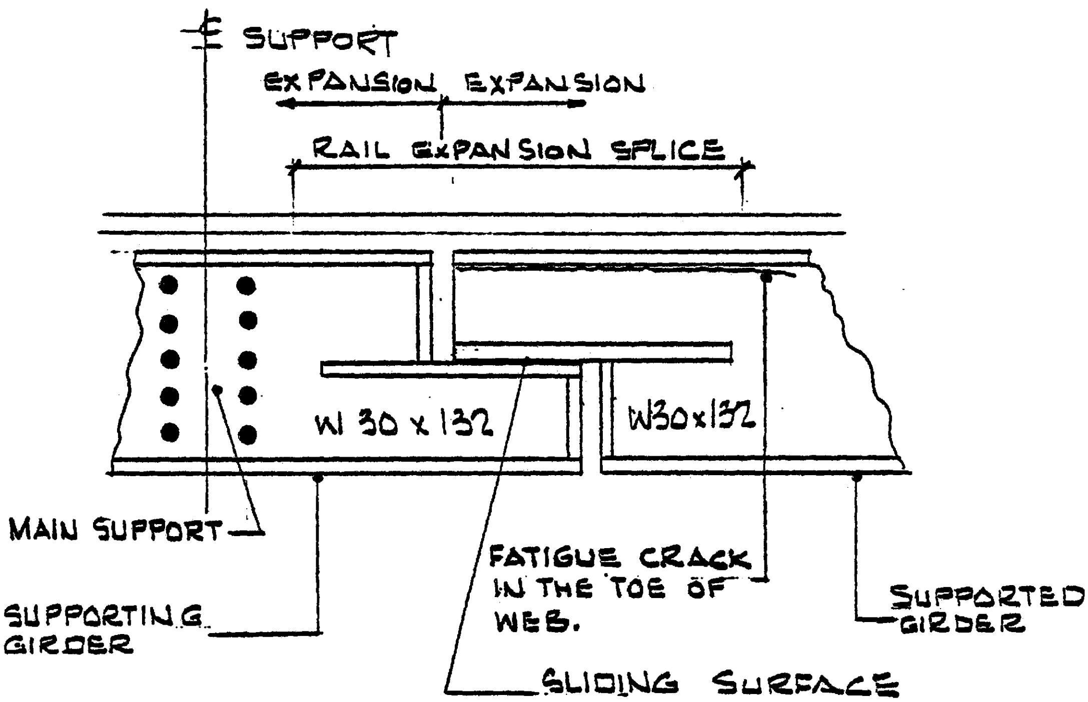


Figure 3.3 Fatigue Failure of the Web in the Top Flange Area, in a Rolled Wide Flange Section

due to excessive shock forces and unstable top flange, developed fatigue cracks, which was attributed to poor detailing and construction. Figure 3.3 shows the expansion joint in an ore bridge. The left beam is cantilevered by about three feet, supporting the right beam. The rail expansion splice consists of two special rail sections with splice bars. In the case cited, the crack appeared in the toe of the web and propagated to more than twelve inches. It was gouged and welded, but it reappeared soon afterwards. Field observations showed that when the trolley passed over the joint, the end of the beam deflected vertically and its top displaced laterally, indicating that the joint was unstable. It was therefore concluded that the failure was a direct cause of rocking due to unstable connection and high shock forces created by the rough rail joint.

To solve the problem, the expansion joint was removed. This was accomplished by installing two girders, both connected to the main support. The rail was mounted continuously over the damping pad. This again is a classic example of the effect of poor construction on fatigue life of a structure.

The current practice appears to favour the following detailing and construction criteria to avoid such failures in the top flange:

a) The flange must be stable. This can be achieved by eliminating the eccentricity (see Section 3.1.2).

b) Full penetration butt welding, preferably double beveled, should be used to connect the top flange to the web. The weld should be ground to a smooth transition (see Fig. 3.4).

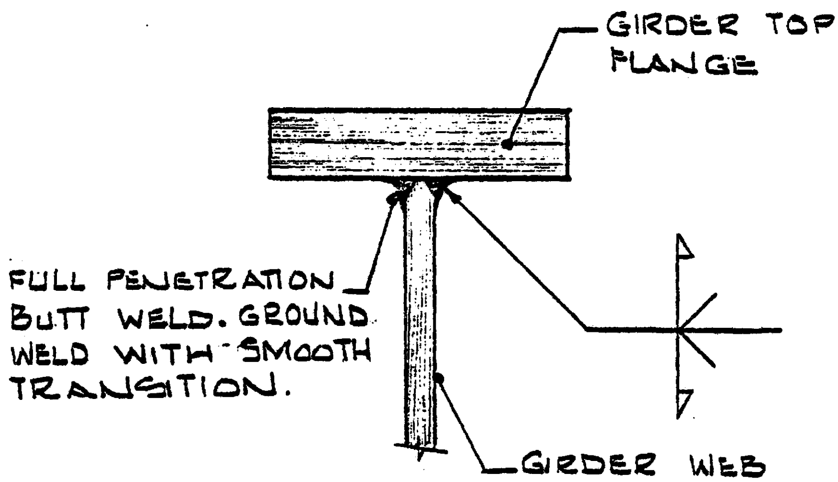


Figure 3.4 Proper Flange to Web Connection

c) Lateral forces should be resisted by stiffening the top part of the girder in that direction (see Section 4.1). The surge plate combined with the top part of the girder and back-up strut form a lateral girder which resists the lateral forces (see Fig. 4.2).

3.2.5 Intermediate Stiffeners

Stiffeners prevent the web from buckling, and restrain the top flange against rotation. They are mostly required in built-up plate girders with deep slender webs. While stiffeners provide these benefits and eliminate the need for heavy web plates, at the same time they can create major problems. They are troublesome from a fatigue point of view, having a high incidence of failure. Even though not catastrophic, these failures can be expensive in terms of shutdowns and production loss.

It has been an accepted fact that the stiffeners are stress raisers that cause failure in fatigue. For this reason, allowances in the design have been recommended. A reduction in principal allowable stress in the region of stiffeners is found to be necessary by the current codes of practice for steel design.

The above design approach is valid but not sufficient. Failures are caused mostly by poor methods of construction and detailing. Opinions and approaches for attaching stiffeners vary while all agree that their connection to the girder require careful detailing and fabrication. For some typical failures, see Figs. 3.2b and 3.5.

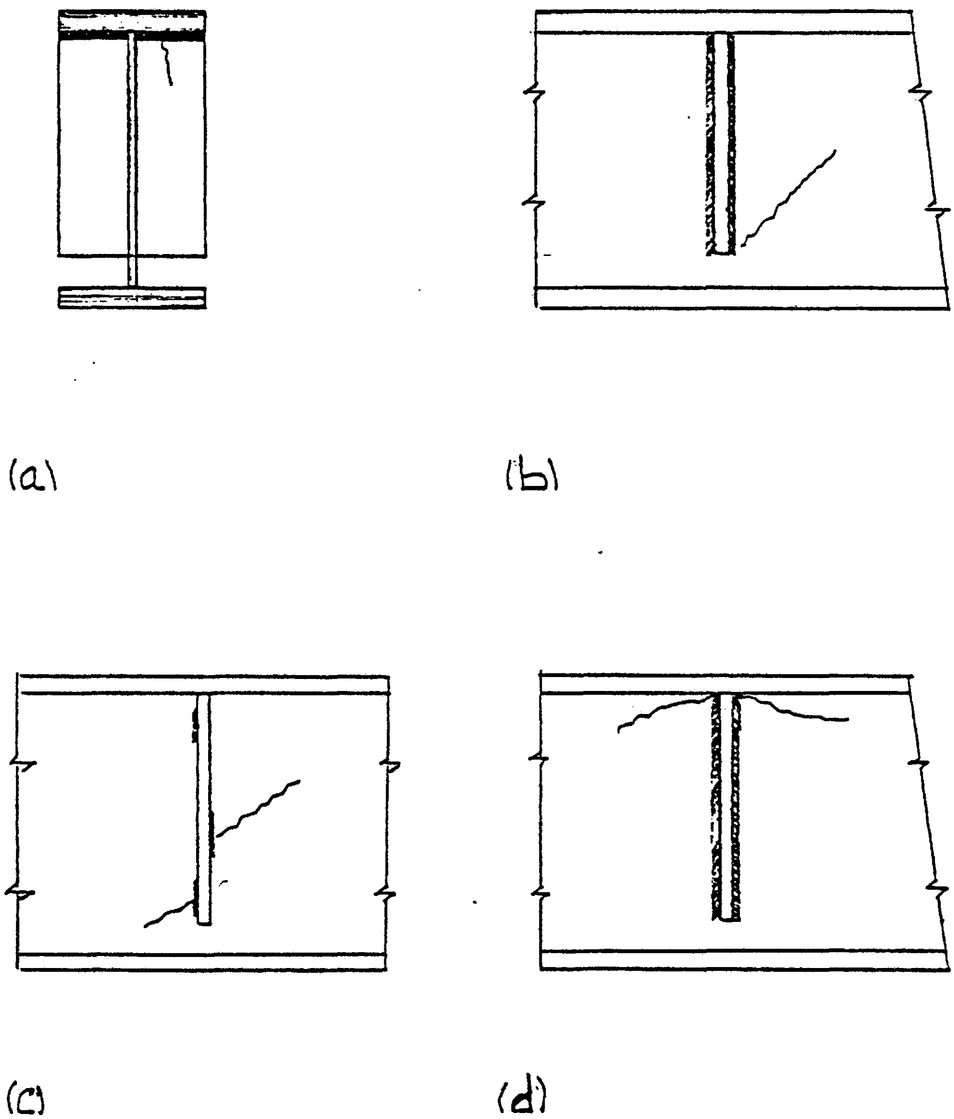


Figure 3.5 Fatigue Failures Caused by Web Stiffeners acting as Stress Raisers

Connecting to Bottom Flange

The crane runway girder is a dynamically loaded structure with its bottom flange and part of the web in the tension field. Welding transversly to the direction of the stress flow in the tension regions is not recommended by the current design practice and the design codes such as AISE and AISC, particularly in dynamically loaded structures. But through personal experience, there are known cases where the stiffeners have been fully welded to the bottom flange or terminated just short of it.

If stiffeners are to be used, current practice is to terminate them at a distance of one sixth of the depth (d/6) from the bottom, since the centroid of the stress distribution area is located at that point.

Connecting to Top Flange

Intermediate stiffeners attached to the underside of the top flange are believed to provide support for the flange against rotation in addition to stiffening the web against buckling. The stiffeners are attached to the flange either by welding or by tight fitting.

Stiffeners welded to the flange may cause fatigue problems since it is generally believed that the underside of the flange, directly under the wheel load experiences local tensile stresses. This may be attributed to the compressive yielding of the web and the stiffener. Stiffeners which are not properly fitted to the underside of the flange can also create fatigue problems in the area of the web to flange connection. The fitted stiffeners, if poorly installed will not provide proper support for the flange against rotation because of the gap. Since the stiffeners create rigidity in the web, the rocking of the flange will cause fatigue failures due to flexural overstressing.

Attaching the stiffener to the underside of the flange, whether welded or fitted, may cause other problems, especially if the rail is mounted directly on the girder. With stiffeners acting as supports, the flange will act as a continuous beam. If the load is located at mid span between two stiffeners, the flange between them will be in positive bending (downward) and the adjacent span will be in negative bending (upward). As the load moves on, the flange will experience stress reversal (see Fig. 3.6). This will increase the likelihood of fatigue (see continuous girder support, Chapter 7). Also at the stiffener to flange joint, the flange is extremely stiff, which may cause high shock effects causing fatigue cracks. For typical failures see Fig. 3.2b.

Connecting to the Web

The welding of stiffeners to the web can also cause fatigue failure. Cracks are more frequent at the termination of the stiffener above the bottom flange. This is due to poor welding which causes high stress concentration in the tensile region. Cracks may also appear in the stitch welding of a stiffener to the web. This can be attributed to rough weld termination which again is a stress raiser. For typical fatigue failures see Figs. 3.5.

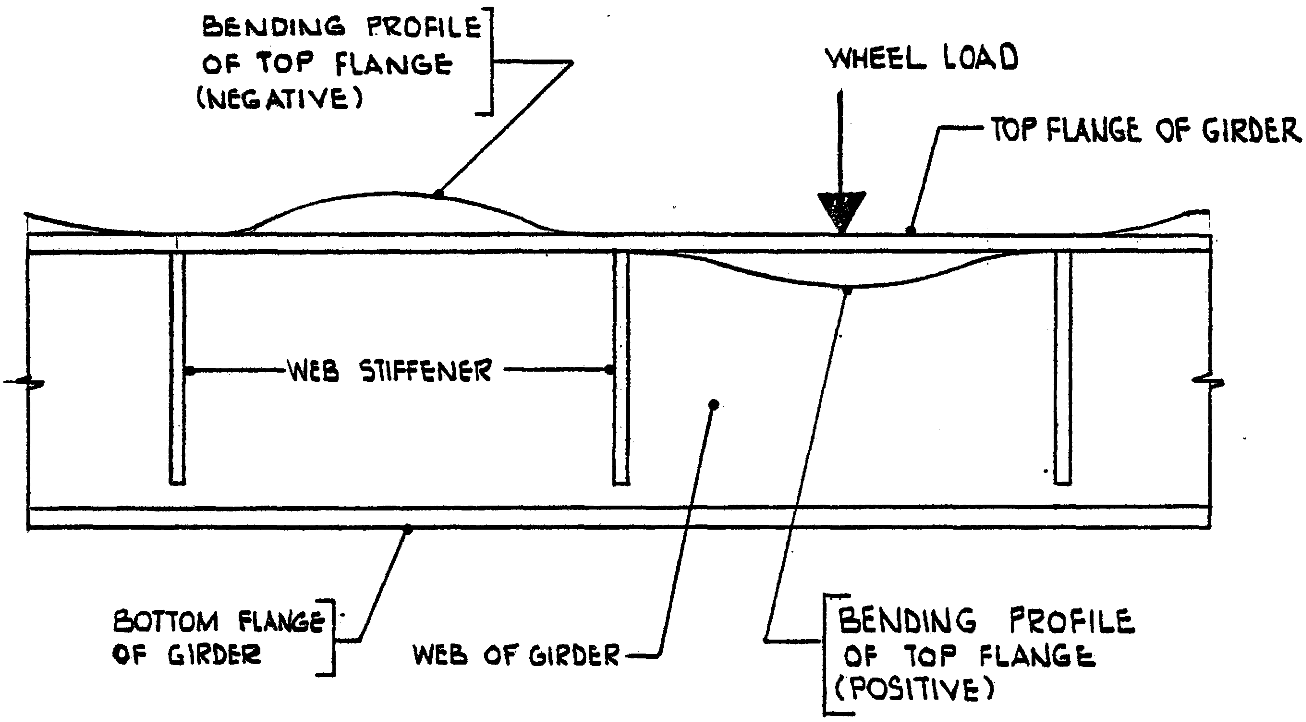


Figure 3.6 Continuous Support of the Top Flange by Stiffeners and the Bending Profile

Conclusion

These observations suggest that the manner in which the structure is detailed, fabricated and constructed will determine its efficiency. The best approach is to minimize the detailing feature that may cause problems, rather than to introduce them and then try to prevent failures.

Stiffeners have always been a source of structural weakness in crane runway girders. Therefore they are to be avoided whenever possible. At times a heavy web plate may prove more economical than stiffeners. As a rule, an increase in the number of pieces to be handled and fabricated is associated with an increase in price per ton.

If stiffeners have to be used, then the following guidelines would be advisable:

a) Stiffeners should be terminated above the bottom flange by a minimum distance of one sixth of the girder overall depth.

b) All welds connecting the stiffener to the web should be ground and finished smooth, especially at the termination point.

c) Stiffeners should be fitted to the underside of the top flange without welding. Complete bearing between them must be ensured.

In order to establish rational and uniform design data and criteria which are representative of the true operating conditions, future field studies and research will be most welcomed in the following areas:

1) Number of cyclic loading that the girder may experience during its life span.

2) Eccentricity in the top flange of the girder and its effect on the section.
3) The effect of the stiffeners on the girder specifically in the top flange region.
4) The effect of physical condition of the rail and different methods of mounting.

CHAPTER 4

HORIZONTAL FORCE SUPPORT

In Chapter 2, the discussion centred on the types and origin of the loads and forces which are normally induced in a crane runway structure. Chapter 3 dealt with the methods of vertical loads support and their design. In this chapter, the discussion is concerned with the different methods of design, detailing and construction which are normally used to resist the horizontal forces. The horizontal forces are:

- Lateral; i.e. acting transversley to the runway (rail),
- Longitudinal; i.e. acting along the runway (rail).

Various methods of detailing and construction illustrated with figures will be discussed and recommendations made.

4.1 LATERAL SUPPORT

In addition to vertical wheel loads, the runway structure has to resist lateral forces which are applied at the top of the rail as a result of acceleration and deceleration of the trolley on the bridge. The lateral force which acts at the top of the rail is transferred into a horizontal girder through the rail and the rail clips. The rail clips hold the rail to the top of the girder in the lateral direction (see Fig. 3.1).

For the crane to operate safely and efficiently, the lateral stability of the girder is essential. For this reason, the crane codes tolerate very little lateral displacement. AISE recommends a maximum lateral displacement of the top of the rail equal to 1/4 inch per fifty feet of span.

Some type of system is needed to resist lateral forces and at the same time maintain the deflection requirement. To rely on the girder alone will not be sufficinet, even though it has been proven that the section as a whole participates in resisting the forces. It may not be practical to apply this to the runway girder because:

a) The flange is shallow and possesses only modest inertia to resist bending in the lateral direction.

b) The web in general, is relatively deep, rendering it relatively flexible in the transverse direction, thus transverse forces cannot be transferred to the lower flange.

c) The lateral forces can be quite severe.

d) Over and above the lateral forces the girder section has to cope with vertical loads. These loads act at exactly the same time and location as the lateral forces.

e) Due to the slenderness, lateral deflection becomes critical.

4.1.1 Methods of Support

Various systems are used to support the top flange against lateral forces. Some are simple in construction, others elaborate. For light cranes with medium spans, Fig. 4.1 shows some typical methods used

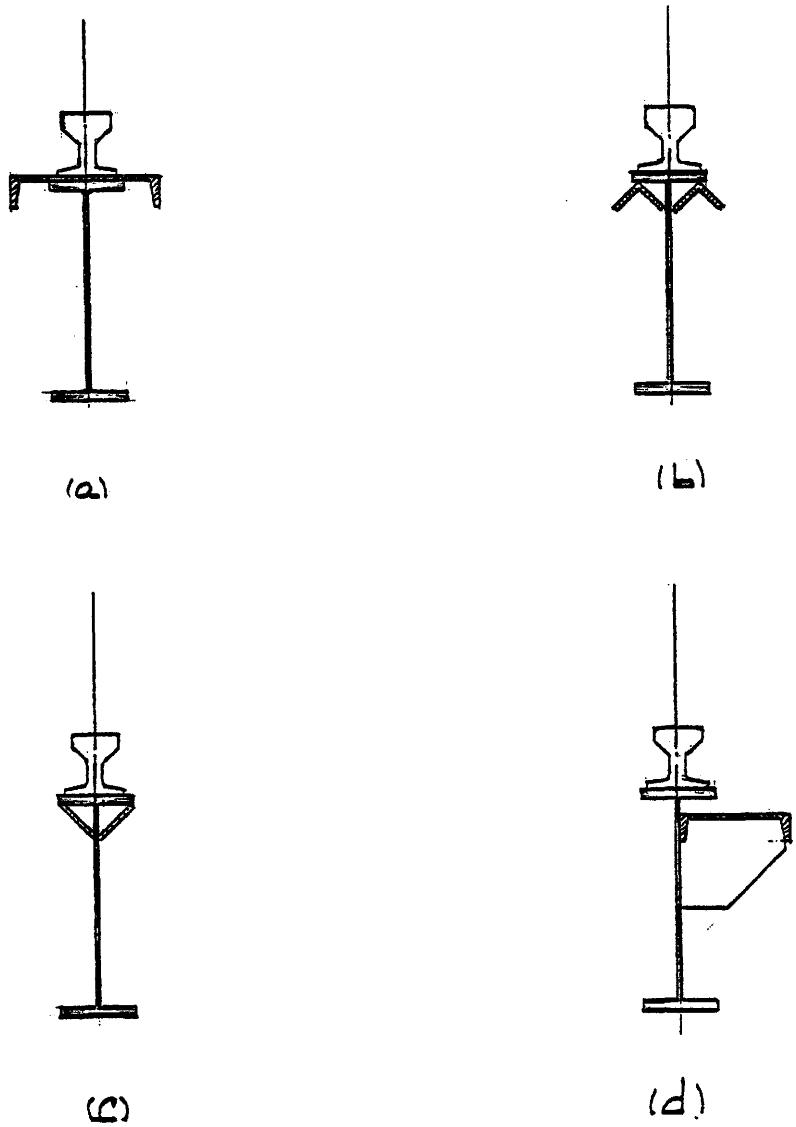


Figure 4.1 Typical Details for the Support of the Top Flange in the Lateral Direction

for reinforcing the top flange against the lateral forces. Figure 4.1(a) shows the most commonly used system. It is economical and efficient and it also provides the top flange with the protection needed against wear. Figure 4.2 shows some of the typical systems used in heavy crane runways. The top part of the girder assembly is designed to act as a girder in a horizontal direction. Methods of detailing may vary, but the principle is still the same.

Systems shown in Fig. 4.2(a)-(c) consist of a plate called a Surge Plate. It is connected to the top flange of the girder at one edge and to a strut at the other. The girder and strut provide vertical support, while the girder top flange and part of its web, surge plate, and the strut form a complete girder system. The surge plate also serves as a service walkway. The strut may take the form of a standard wide flange section for medium spans of up to 30 feet.

When longer spans are involved, a truss is used in place of the wide flange beam. The bottom flange of the girder and the bottom chord of the truss are relatively slender. Due to this slenderness, personal experience has shown, that they could be excited into torsional vibration by falling into synchronism with their natural mode of vibration. Therefore, lateral support is provided by lacing them together. Lacing in combination with the lower part of the girder and vertical truss form a horizontal truss (Fig. 4.2(b)).

The system of Fig. 4.2(d) is similar to the above but employs a truss inclined at an angle relative to the girder. The only difference is that it eliminates the bottom chord lacing. Stability for the bottom

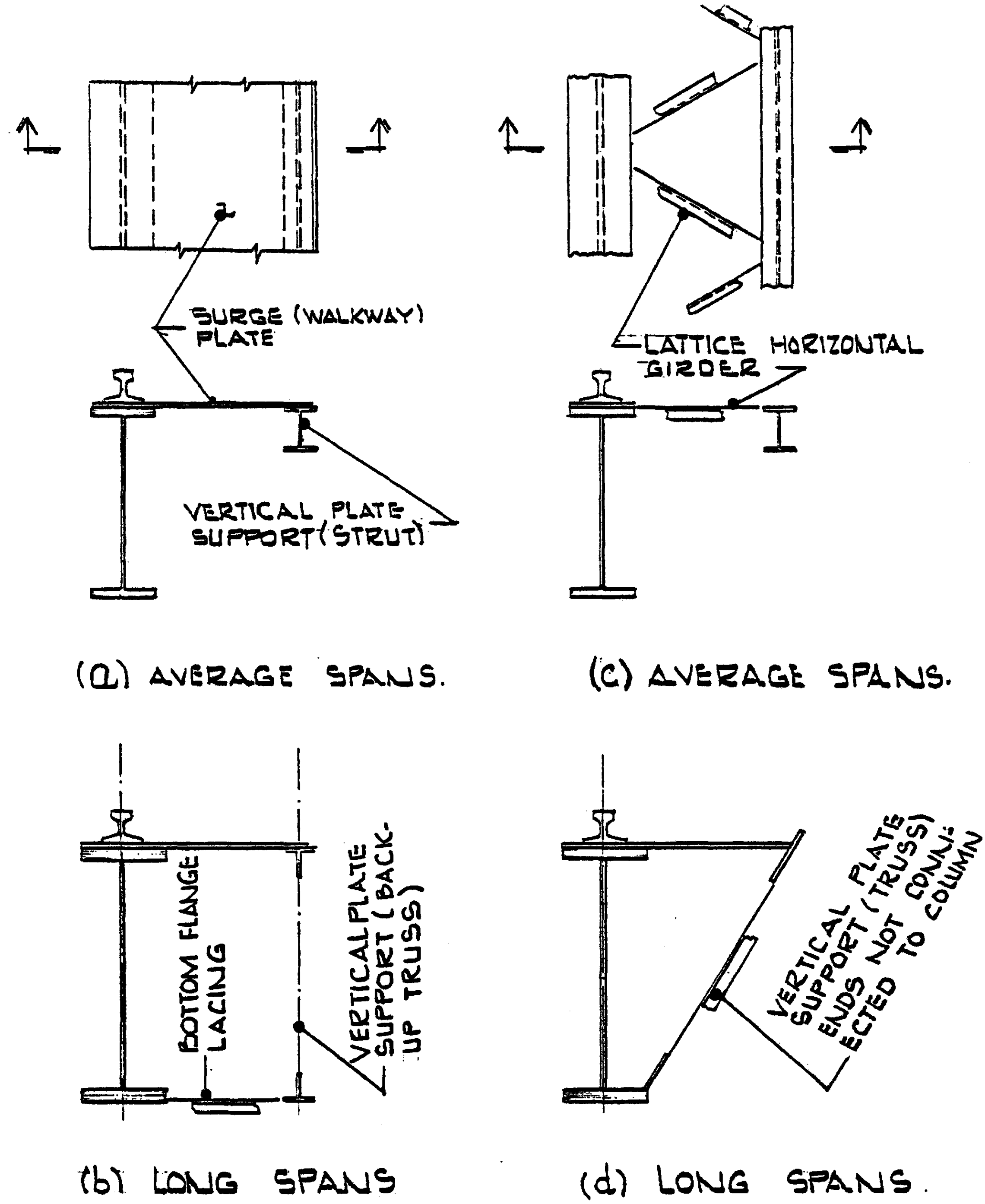


Figure 4.2 Typical Lateral Support for the Top Flange of Medium and Long Span Girders in Heavy Crane Runways

flange of the girder is provided by tying the vertical truss directly into it. This system has become very popular since it is both simple and effective. In Fig. 4.2(c), the lateral girder comprises a lacing system instead of a plate. The horizontal truss is formed with its chords being the top part of the girder and the top chord of the back-up truss. The system has a multiplicity of members and connections, and is therefore sensitive to fatigue. It is uneconomical and not so practical for future repairs or modifications. Also it does not provide a service walkway. Therefore, the system is not so common.

4.1.2 Design Approach

There is a consensus among various groups regarding methods of support. All seem to agree that lateral loads can not be resisted by the main girder alone. Some form of lateral stiffening is required as discussed in section 4.1.1.

To design the system, we have to determine how its different components should interact with each other. Concerning this point there seems to be disagreement. Some suggest that the system should be constructed such that it will act as a box girder, supporting vertical and lateral loads simultaneously. This box girder is composed of:

- Main (veritical) girder.
- Lateral girder (Surge).
- Back-up truss for long span or wide flange section for short span
- Bottom flange lacing for long spans.

To transform the system into a box section, incorporation of cross diphragms is sometimes suggested, see Fig. 4.3.

AISE 13 [5] is one of the authorities recommending this system contending if no diaphragms, or only a few are provided, the vertical wheel loads will cause differential vertical deflection (Fig. 4.3b). This in turn will force the flange upward, creating cross bending at the web to flange joint, thus causing fatigue failures. It is also claimed that the vertical deflection in the main girder will be reduced by the action of the box section (diaphragms). There appears to be some disagreement regarding the usefullness of the diaphragms. While some groups accept and incorporate cross diphragms, others avoid their use. For example, of the two major steel producers in Hamilton, one makes use of them extensively, while the other rejects them categorically.

Mass [12] reports that his test results have shown that the cross diaphragms cause the back-up truss to deform. The results have also shown that they do not reduce the deflection in the main girder, contrary to the suggestion made by AISE.

The deformation of the back-up truss may be attributed to the considerable difference between the inertia of the back-up truss and that of the girder. Vertical wheel loads will cause torsion in the box system, transferring these loads into the horizontal and vertical components. Therefore, the lateral girder, back-up truss and the bottom flange lacing may experience loads which are heavy relative to their strength. Probably this could be a good area where further tests and studies might be required.

The author favours the approach adopted by some groups whereby the use of the cross diphragms is avoided, and instead the structure is designed and detailed in such a way that it will act as a mechanism. A mechanism might be the best approach for supporting heavy dynamic loads. If the components of the structure are relatively free to move, independent of each other, they will not break.

In order to achieve a mechanism action, the structure will have to be designed and detailed such that each of its components will perform its assigned funtion, with a degree of independency of each other. That is to say:

a) The main runway girder supports vertical wheel loads.

b) The lateral girder (Surge plate) resists lateral forces.

c) The back-up truss supports the surge plate.

d) The bottom flange lacing stabilizes the main girder's bottom flange and the back-up truss bottom chord, in the lateral direction.

It is stated by Goreng [15], that there exist an interaction between the above mentioned components of the system shown in Fig. 4.2(b) which may be contradictory to the above proposed approach. He feels that this interaction may be significant enough to cause fatigue failures (Fig. 4.3(b)).

This interaction may not be very significant. For it to occur, the deflection of the main girder will have to be significant and the components which frame into it considerably stiff. On the contrary:

- The deflection allowed in the main girder is relatively small.

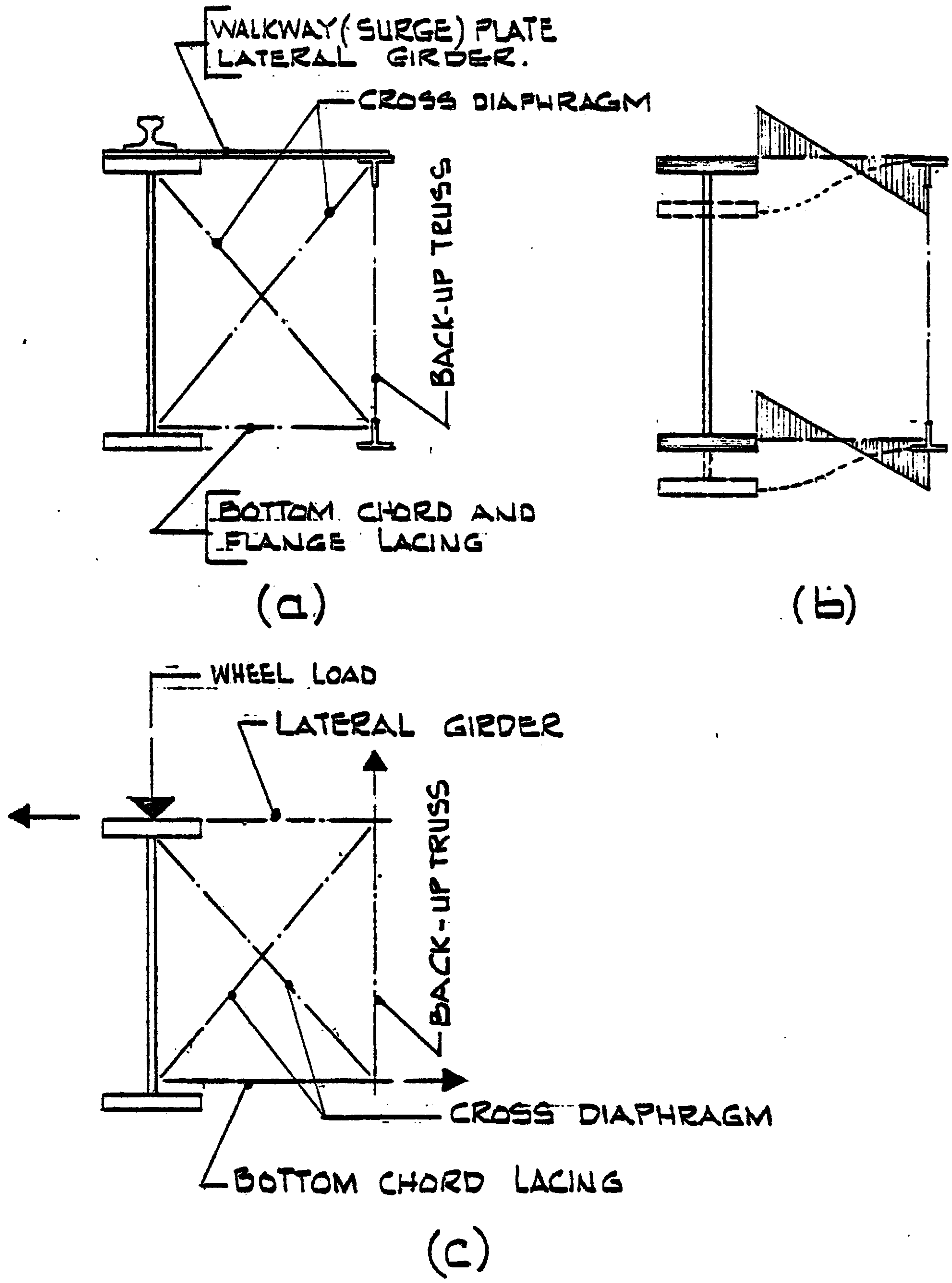
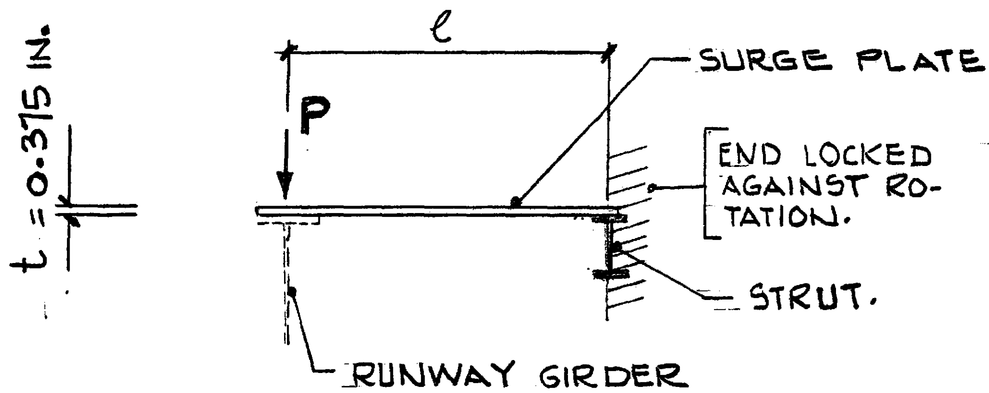


Figure 4.3 Shows the Effect of Cross Diaphragms on a Girder System

- The surge plate is slender in vertical direction compared to the stiffness of flange and web of the girder.
- Bottom flange lacing is also slender compared to the stiffness of bottom flange.

For example, assume that the surge plate shown in Fig. 4.2(a) is rigidly fixed at the strut end with zero rotation, and it is free at the other end (girder). Consider a 12 inch wide strip of the plate and 36 inches in span acting as a cantilevered beam. The thickness of plate normally used is 0.375 inches.



The excercise is to find the load required to deflect the free end of the plate the same amount as that of the maximum beam deflection. Assume a girder span of 25 feet (centre to centre of columns). Considering the maximum allowable deflection to be equal to span/1000, the maximum deflection of girder will be:

$$\Delta = \frac{25 \text{ ft} \times 12 \text{ in}}{1000} = 0.30 \text{ inches}$$

The formula for a cantilevered beam deflection is

$$\Delta = \frac{P\ell^3}{3EI}$$

Δ = deflection in inches

P = point load in Kips

ℓ = span in inches

E = modulus of elasticity (29,000 Ksi)

I = moment of inertia of the plate in $in.^4$

To deflect the plate, an amount of 0.3 inches the required point load will be

$$P = \frac{3EI}{\ell^3}\,\Delta = .029 \text{ Kips/ft.}$$

The above example shows that a very negligible point load is required in order for the plate to experience the girders deflection. This calculation demonstrates that the main girder and the surge plate are essentially uncoupled.

Therefore, these components, due to their slenderness are able to deflect more than the girder. Hence, no significant resistance can be applied by them. As was stated before, if members are allowed to move under the dynamic loading they will not break. With some careful detailing hinged connections may be produced, eliminating any joint rigidity, and making the system act more like a mechanism.

As a demonstration of the soundness of this approach, the system shown in Fig. 4.2(b), has proven to be very economical, practical and efficient. It is being used extensively by one major steel producer in some heavy crane runways with high loading cycles, and its surge plate, back-up truss and bottom flange lacing, to the best of this author's knowledge, have not caused any problems.

4.2 LONGITUDINAL LOAD SUPPORT

Longitudinal forces that must be resisted by the runway structure were discussed in Section 2.2. They are:

- Crane force on the stop.
- Traction forces.
- Thermal expansion and contraction.

These forces exist in the top part of the runway and their safe transfer into the foundation is the aim. It must be kept in mind that maintaining the rigidity of structure ensures optimum conditions for crane operation. To achieve the transfer of the forces into the foundation, three types of systems are available, namely:

a) A simple girder to column connection, where the structure is braced in the longitudinal direction.

b) A rigid frame system, whereby the girder is continuous and rigidly connected to the column.

c) A continuous pinned system, whereby the girder is continuous and pinned over the column. An example would be the knee bracing to the column, or the trussing of a simply supported girder (Fig. 7.6 and 7.4).

Simple girder-to-column connection will be dealt with in this section. Rigid, knee braced, and trussed systems will be discussed in Chapter 7.

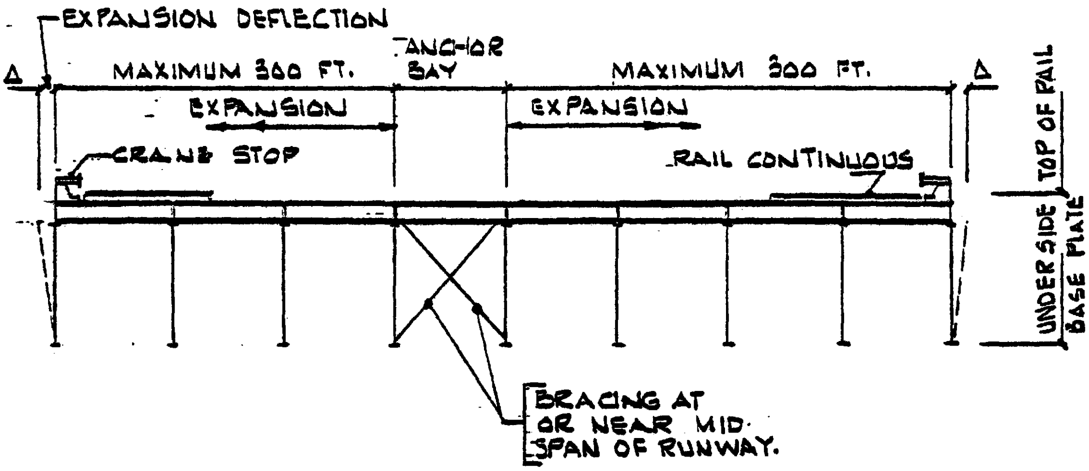
Location of Expansion Joint

The location of the expansion joint which in turn determines the location of the bracing is essential if the longitudinal stability of the structure is to be achieved. Establishing an acceptable maximum length of the structure between the expansion joints is important in order to avoid distortion of the structure which may be caused by excessive thermal expansion and contraction. Authorities appear to differ slightly as to the recommended maximum distance between the expansion joints.

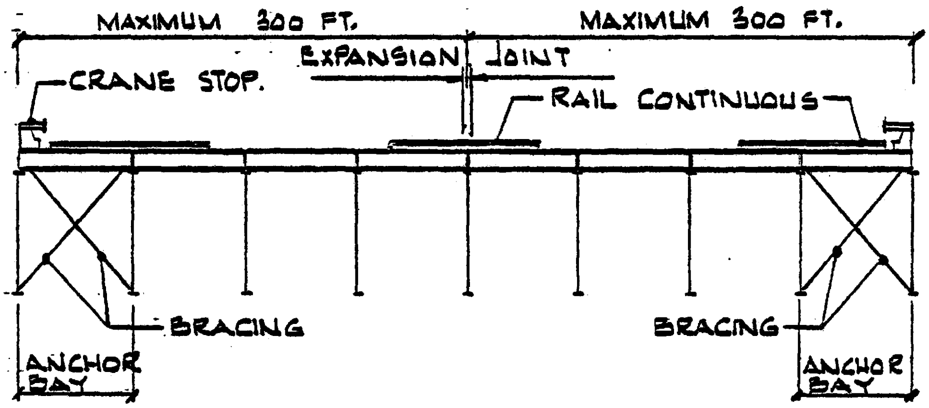
For example, Murray [16] and AISE [5] propose a maximum distance of 400 feet between the expansion joints. Goreng [15], on the other hand suggests a maximum distance of 260 feet. One major steel producer in its latest design practice manual states that thermal stresses must be considered when the length exceeds 300 feet. Current practice, which is to limit the maximum runway length between the expansion joints to about 300 feet has performed efficiently with the above mentioned steel producer.

There are basically two methods in current practice of allowing the structure to expand. One method is to anchor the structure at about mid-span and allow it to expand toward its free ends (outward) (see Fig. 4.4a). The second method is to anchor the structure at the ends and allow it to expand toward its mid span (inward) (see Fig. 4.4b).

The method of anchoring the structure at its mid span appears to be the favoured practice. This method suits a runway which is about a maximum of 600 feet long, assuming that the 300 feet between the



(a) : ANCHOR AT MID-SPAN OF RUNWAY AND EXPANSION TOWARD ENDS.



(b) : ANCHORS AT EXTREME ENDS AND EXPANSION TOWARD THE CENTRE

Figure 4.4 Expansion Systems of Runway with Alternate Bracing Location expansion joint is the design criteria. This type of system eliminates the need for built-in expansion joints which can be costly and inefficient. Practical experience has shown that such joints do not function properly. The built in expansion joints may be necessary if the structure is excessively long requiring more than one anchoring position to maintain a 300 feet maximum span between the expansion joints. Also, in some cases the designers may feel that, to transfer crane stop force through the structure, into a brace located at mid-span of the runway may be detrimental. Therefore they feel that the bracing has to be located at the ends where it can directly resist the crane stop force (see Fig. 2.3). This approach makes it necessary to have an expansion joint at the mid-span. If the built-in expansion joint is used, care must be exercised when detailing it. Different authorities do not appear to agree on the best approach for developing an efficient expansion joint; some methods of detailing are crude, others are elaborate and expensive. As an example, the AISE proposes the support of two girders at the joint, independently by using two columns, one for each. However, to do so may do more harm than good, since this may create excessive relative movement and possibly result in fatigue stresses.

Current practice appears to favour anchoring the runway at mid-span and eliminating the built-in expansion joints where ever possible. As for the crane stop force, argument which requires the bracing at the end may not be valid. With proper detailing of the girder to column connection, the crane stop force can be transferred safely through the structure toward the mid-span and into the bracing, as discussed in Chapter 5.

Conclusion

Since there are conflicting opinions regarding the type of girder system to be used, it might be beneficial if studies were conducted for establishing the effect of diaphragms and also the degree of coupling between the main girder and the surge plate.

CHAPTER 5

GIRDER SUPPORT AT THE COLUMN

In the previous chapters the type of loads along with their origin, and methods of their support and transfer into the vertical and horizontal girder were discussed. In this chapter the discussion will be about the transfer of these loads through the end connection into the column. Methods of detailing and construction of the end connection of the runway girder system will be discussed and illustrated, and their advantages and disadvantages highlighted.

5.1 APPROACH

The end support of the girder at the column is more of a detaling consideration than a design one. This is one area where there is a high degree of agreement among various authorities, at least in principle if not in methods of connecting and detailing. The basic principle is that the end should be a pin connection. It must be free from restraint, thus allowing it the freedom to rotate. Toward this goal, AISE recommends that for a distance of 18 inches from the ends, the flanges should be free from any curvature, due to end fixity.

If we do not allow rotational freedom, the end region will experience stress fluctuation and reversal. Allowing high strains in the ends will cause failure in the connection. As the girder is loaded, its end in the top region will experience tensile stresses, which will disappear when unloaded. If this is allowed to happen, then almost inevitably trouble will be experienced in the form of fatigue failure. Therefore, the method of end supports, and their connecting details must be such that possible strains in the ends of girder are prevented without sacrificing stability, efficiency or the required tolerances.

Many different detailing approaches are used. Some are very elaborate and expensive, while others are relatively simple, yet all strive to achieve the same: a degree of rotational freedom. At the same time, it is still possible to find inefficient and failure prone methods in use. This can possibly be attributed to a lack of communication, or lack of knowledge.

5.2 VERTICAL SUPPORT

Some of the methods which can cause fatigue failures in the ends of the girder are shown in Figs. 5.1 and 5.2(a). In Fig. 5.1(a), the web is directly connected to the flange of the column. This creates excessive rigidity, restricting end rotation and causing failures as shown.

Knee bracing, Fig. 5.1(b), is another method which causes many problems. Bolts connecting the brace to the girder and the column fail in shear, or work themselves loose. Welds connecting the gusset plate tend to crack. Strengthening the connection merely relocates the problem into the member, causing the gusset plate to crack in fatigue or the knee brace to deform. To solve these problems, some designers choose to make the knee brace and its connections relatively strong.

But this does not eliminate the problem either, for the stresses are then transmitted into the girder, causing it to deform or crack in the web. Clearly, it is advisable to avoid this method. One steel producer has spent much time and effort, at a substantial cost, in repairing some of its runway structures that have knee braces. Figure 5.1(c) illustrates the details of one such repair.

Figure 5.2(a) shows another common type of detail which can be troublesome. Its tendency to rotate causes the girder to pivot against the edge of the cap plate, as shown in Fig. 5.2(b), which in turn causes prying action at the bolts. If the bolts, relative to girder, are not strong enough, they will break. If the reverse is true, the girder web might weaken due to fatigue. Installing loose bolts (finger tight and tack welded) ensures end rotation. But by solving one problem, another may be created: The rotation will now be about the edge of the cap plate, (Fig. 5.2(b)), causing the top of the girder to move vertically and horizontally, and causing failures in the rail. Also the toe of the web at the bottom flange may fail due to point load action. This problem can be aggravated when using a deep column, which necessitates a longer cap plate, thus increasing the pivot arm.

The general practice is to avoid the above mentioned methods of support or others similar to them which might hamper the end rotation. Details shown in Figs. 5.2(c) and 5.3 are the most simple, practical and effective methods. Practice has confirmed that they are virtually failure free, and these are described in the following section.

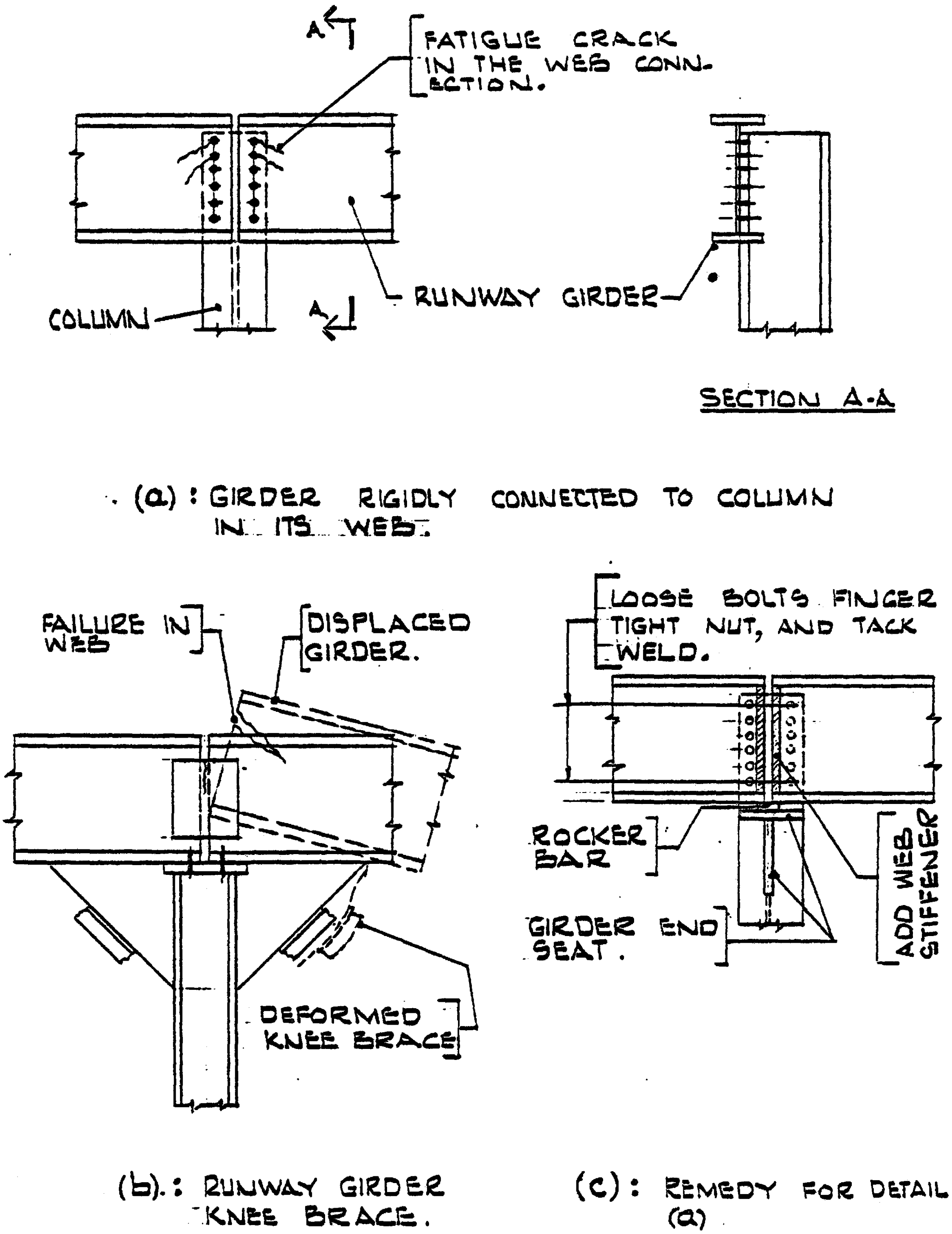


Figure 5.1 Typical Details for the End Support of Runway Girders which are Troublesome

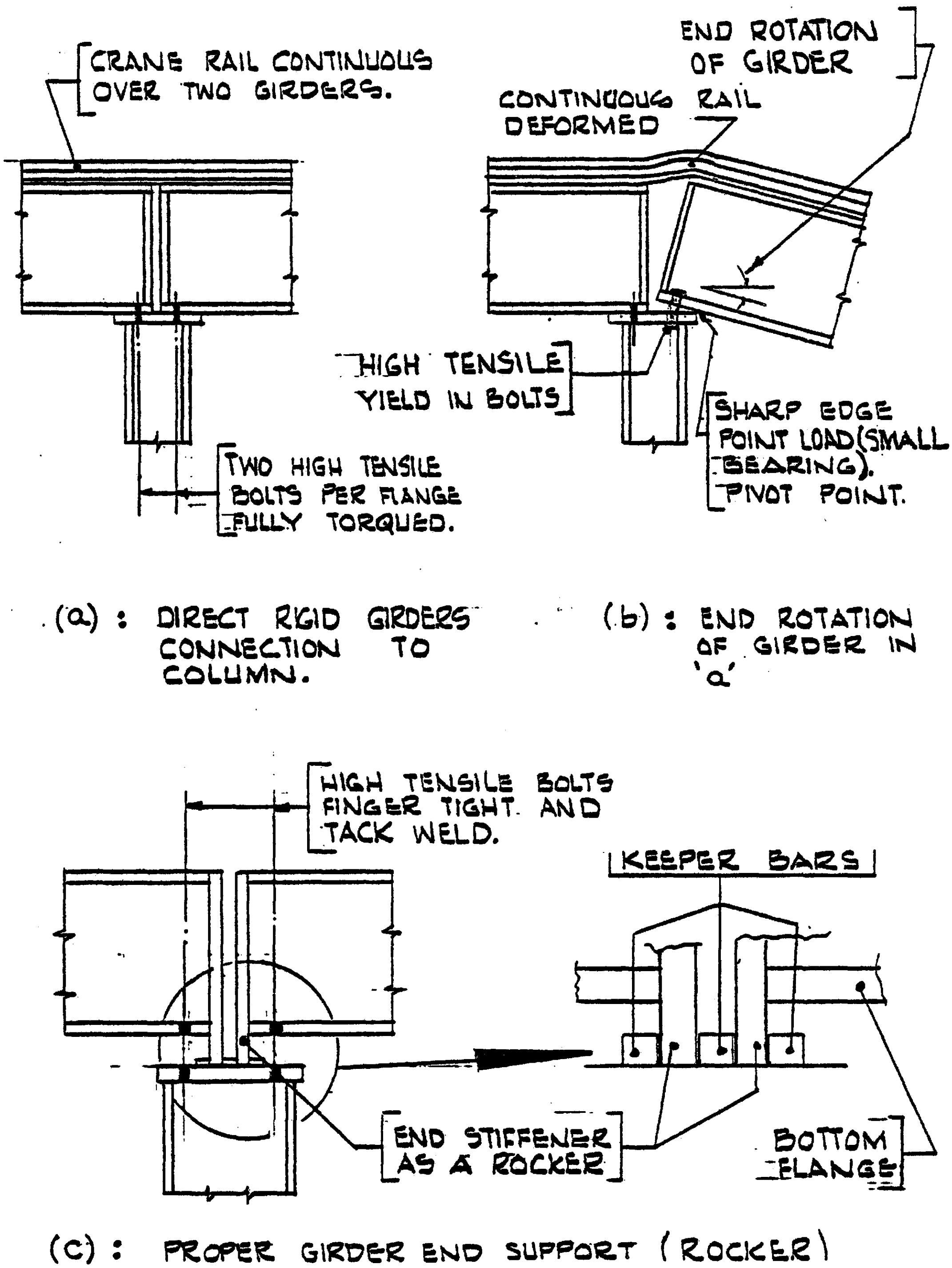
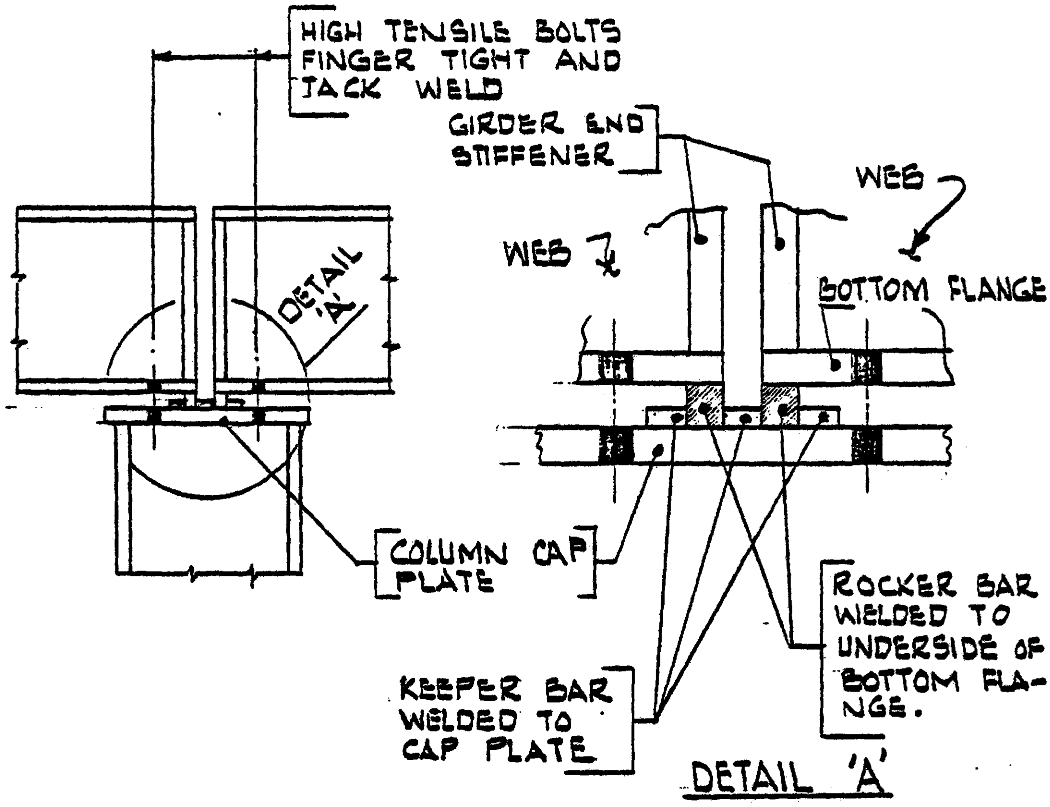
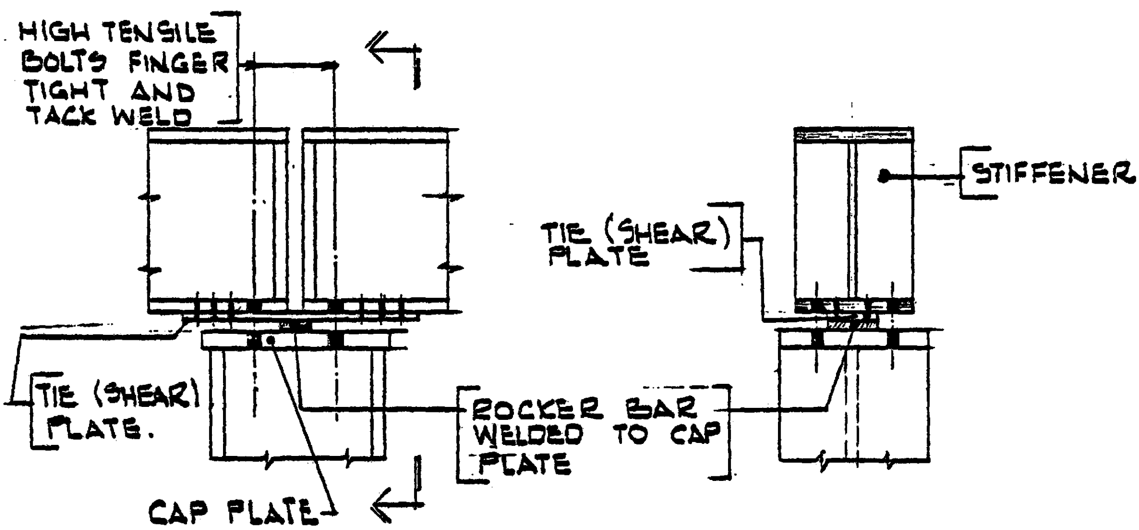


Figure 5.2 Typical Details for the end Support of Runway Girders



(a): PROPER GIRDER END SUPPORT (ROCKER)



(b): PROPER GIRDER END SUPPORT (ROCKER)

Figure 5.3 Typical Proper Details for the End Support of Runway Girders in the Vertical and Longitudinal Direction

5.3 LONGITUDINAL SUPPORT

When proper details for vertical support are adopted, longitudinal support can be incorporated in them (see Fig. 5.3). Longitudinal forces must be transferred from the top of the girder to the top of the column, and hence into the bracing. These forces may be applied at locations some distance away from the bracing. Using keeper bars as shown, load is transmitted from one girder down to the keeper bar, then into the next girder and so forth, finally ending in the bracing. The purpose of the bolts is to hold the girder down on the column. They are normally finger tight and tack welded to ensure against loose nuts.

Figure 5.3(b) shows a method of tying the girders directly to each other. This method has some limitation, but for small to medium forces it functions reasonably well. If the transferred forces are of such magnitude that would require a thicker and stiffer tie plate, the end rotation might be hampered, or the plate may fail in fatigue. In practice it is found that tie plates not exceeding 0.375 inch, perform well.

5.4 LATERAL SUPPORT

It was mentioned in Section 4.1.2 that the main and the horizontal girders should be made to function relatively independently of each other. This means that the lateral support should not be allowed to interfere with and restrict the vertical support. In other words, vertical support is constructed such that it allows the end of the girder to rotate when under the influence of vertical loads. Therefore, the lateral support must be detailed not to inhibit this end rotation.

Methods of support are not developed by design and analysis, but rather through practical applications, involving a multitude of trial and error cases, and by using some engineering common sense. The majority of runway connections and their details are the result of this process.

Figure 5.4(a) shows a support diaphragm, connected rigidly to the girder and the columnn. This type of connection, more often than not, will develop fatigue cracks as shown. This is a result of continuous torsional twisting caused by the tendency of the girder end to rotate.

The supporting method of Fig. 5.4(b) is relatively new. It has been applied in a few heavy crane runways and it might be too early to judge its effectiveness and efficiency. The opinion of the author is that when length and force necessitate a thicker and stiffer plate, intolerable strains due to end rotations may be experienced by these ties.

The method shown in Fig. 5.4(c) has been used in runway structures, varying in capacity from medium to the heaviest, with excellent results. The oversized slotted holes facilitate end movement. Bolts are finger tightened and tack welded, the former to ensure movement, and the latter to ensure against loose bolts.

Some designers who have a tendency to interpret the code literally, point out that code S16.1 does not permit bolts in slotted holes under fatigue loading conditions. However, if such problems are to occur, it will be easier to replace a few bolts periodically rather than to break the whole connection, which can be more troublesome and more costly. On the other hand, to the best of this author's knowledge, no such complications have occurred.

Having supported the end of the girder laterally, the next step is to support the surge plate at the column. Methods are many, some being simple and others expensive and elaborate. Figure 5.5(a) is one method which might provide the required function, but it can be an expensive arrangement, requiring continuous maintenance.

The method shown in Fig. 5.5(b) provides the required function, while being simple and less expensive. A clip angle is welded to the column, acting as a prop and stabilizer for the plate, connected to it by a single bolt. The long leg of the angle is welded to the column, along the lower edge and two inches vertically up the side. This allows the angle enough flexibility to deflect away from the column when the end rotates and ensures that during rotation the plate will push rather than pull on the angle. This detail, in combination with that of Fig. 5.4(c) constitute one complete lateral support system.

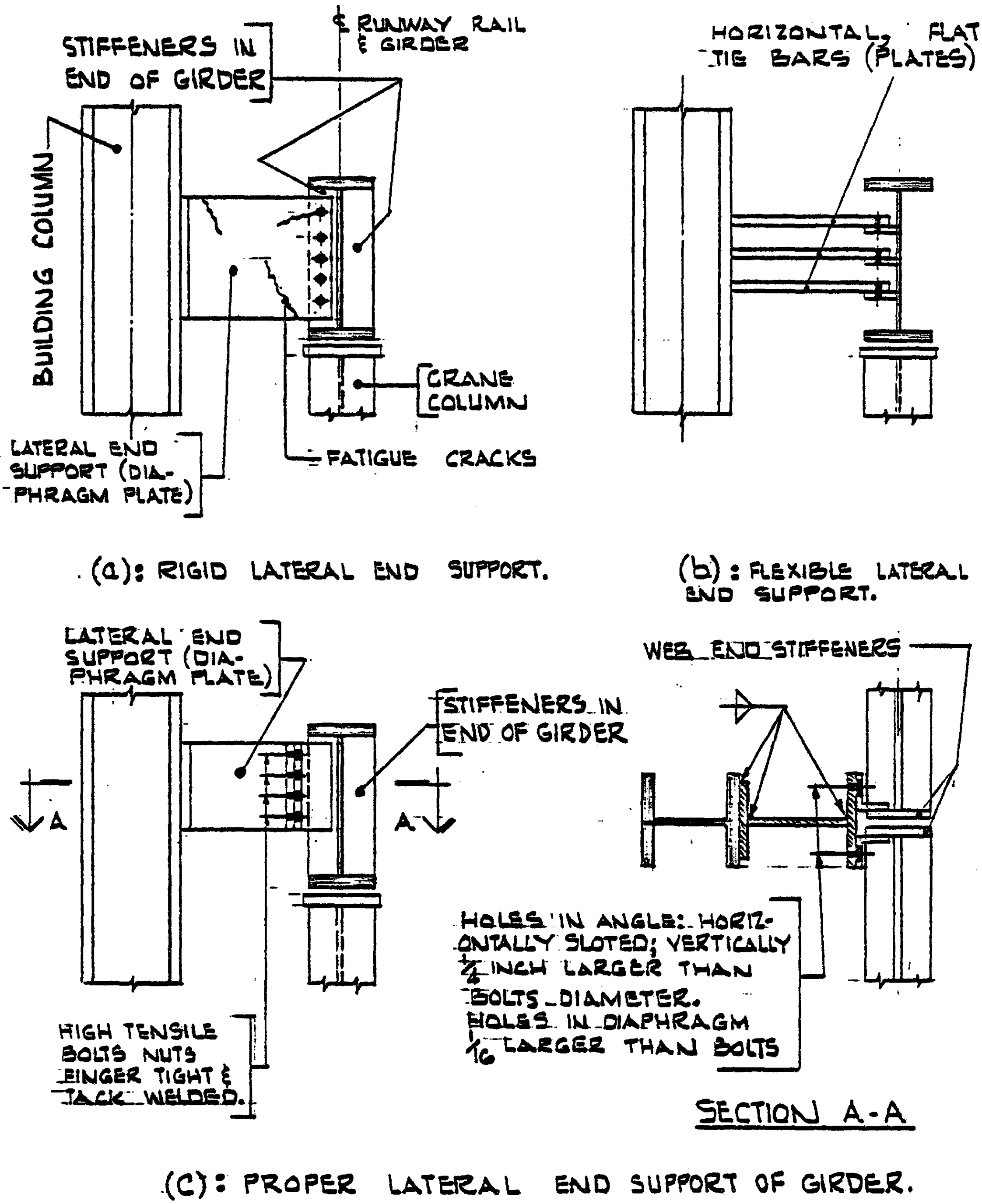


Figure 5.4 Typical Details for the end Support of Runway Girder in the Lateral Direction

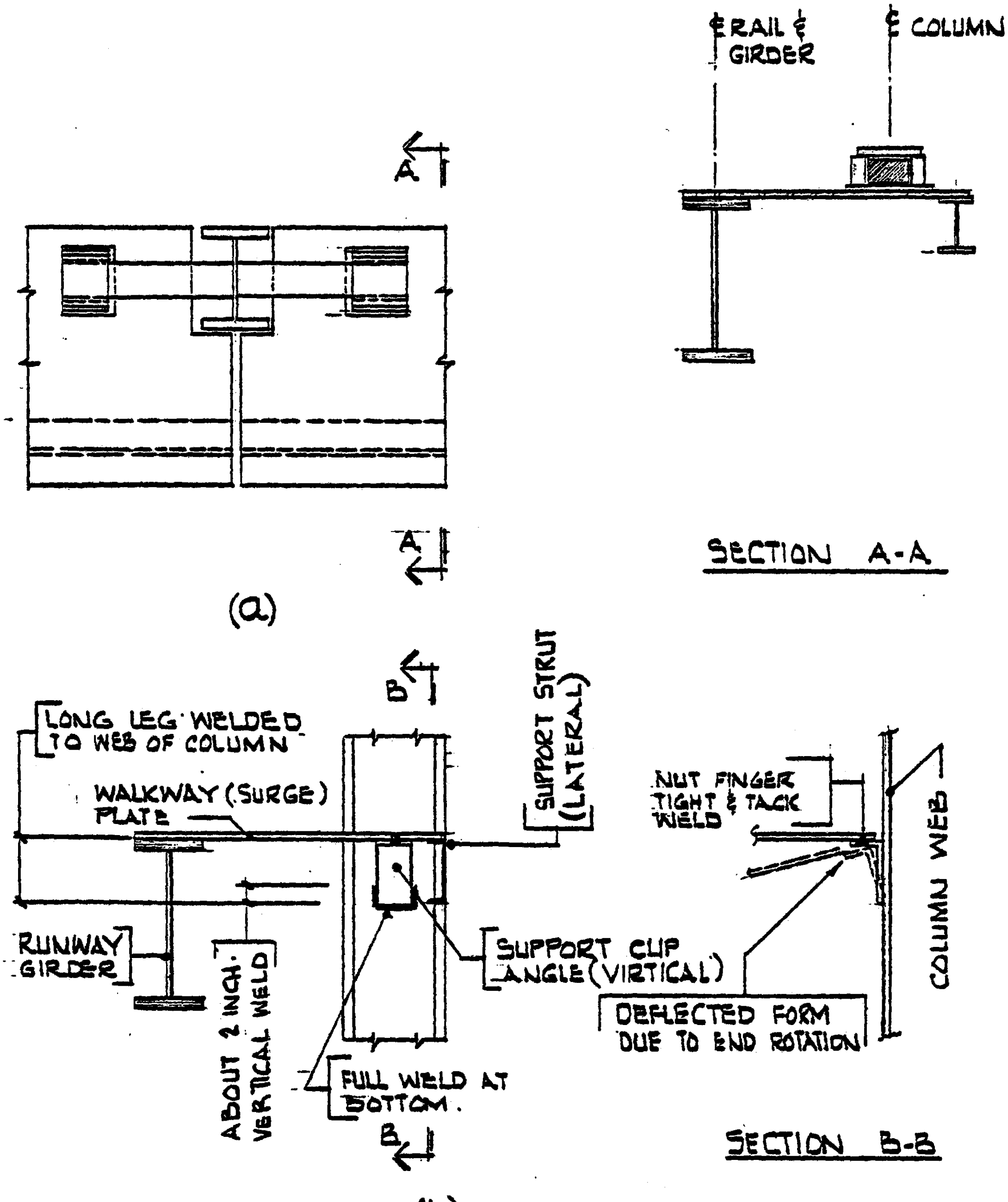


Figure 5.5 Typical Details for the End Support of the Walkway (surge) Plate in the Vertical and Lateral Direction

CHAPTER 6

COLUMNS AND FOUNDATIONS

6.1 COLUMNS

The basic approach for designing a crane column is to consider it as pinned at the top and the bottom, supported laterally against buckling in both axes as required. In most instances, it is designed spanning free in the direction of the major axis, and it is always oriented with its web parallel to that of the girder.

In designing the base of the column, the general practice is to assume that there is no shear transfer except at the braced bay. The column, being pinned at top and bottom, and being slender, will have enough flexibility to prevent the shear transfer. Therefore, shear keys between the base plate and the concrete are often neglected.

Dynamic loading and associated vibration may however cause a certain amount of movement in the base. Bolts lacking proper bearing area might wedge against the concrete, causing eventual break of the concrete bond, resulting in loose anchor bolts. In some existing cases, base movement has been quite visible.

For this reason, it is always a good practice to provide shear keys in all the bases. Their cost relative to that of the structure is negligible, and hence economically justifiable. Figure 6.1 shows some typical types of columns, and Fig. 6.2 shows typical column connections along with potential failures.

6.2 FOUNDATIONS

The runway structure is expected to be stable in all directions, in order for a crane to operate on it efficiently. To maintain this stability, it is imperative that the foundation be stable. Unstable foundations have a profound effect on the structure above them. Due to the height of the structure, a relatively small differential settlement can cause an appreciable horizontal displacement at the top of the rail. This will result in:

a) Damaged rails, wheels and other parts of the crane.

b) Higher lateral forces due to crane wheels bearing against the rail, producing more thrust in that direction.

c) Uneven distribution of load which may cause deformation and failure.

In one known case, the unstable foundation of a gantry crane runway sank whenever the crane operated. This caused the top of the rail to visibly translate laterally (see Fig. 6.3).

In another instance, one side of a runway settled more than the other, causing the crane to be unstable. To stabilize it, guide rollers bearing laterally against the rail were installed. Thus higher lateral forces were induced, causing the foundation to sink further. Finally it had to be jacked up and resupported.

Mill buildings in general are erected on poor soils. For example, one local steel mill is built largely on filled areas. On the other hand many designers, anxious to economize, have a tendency to produce foundations which are at times unstable. Because of this, jacking of mill structures has become an accepted fact of life.

Whenever dynamically loaded structures are involved, the design of the foundations should tend to be conservative. Failures are far too costly, to justfy the short term savings by designing to the absolute limit. Also soil samples from the mill site should be used in order to arrive at a realistic bearing capacity.

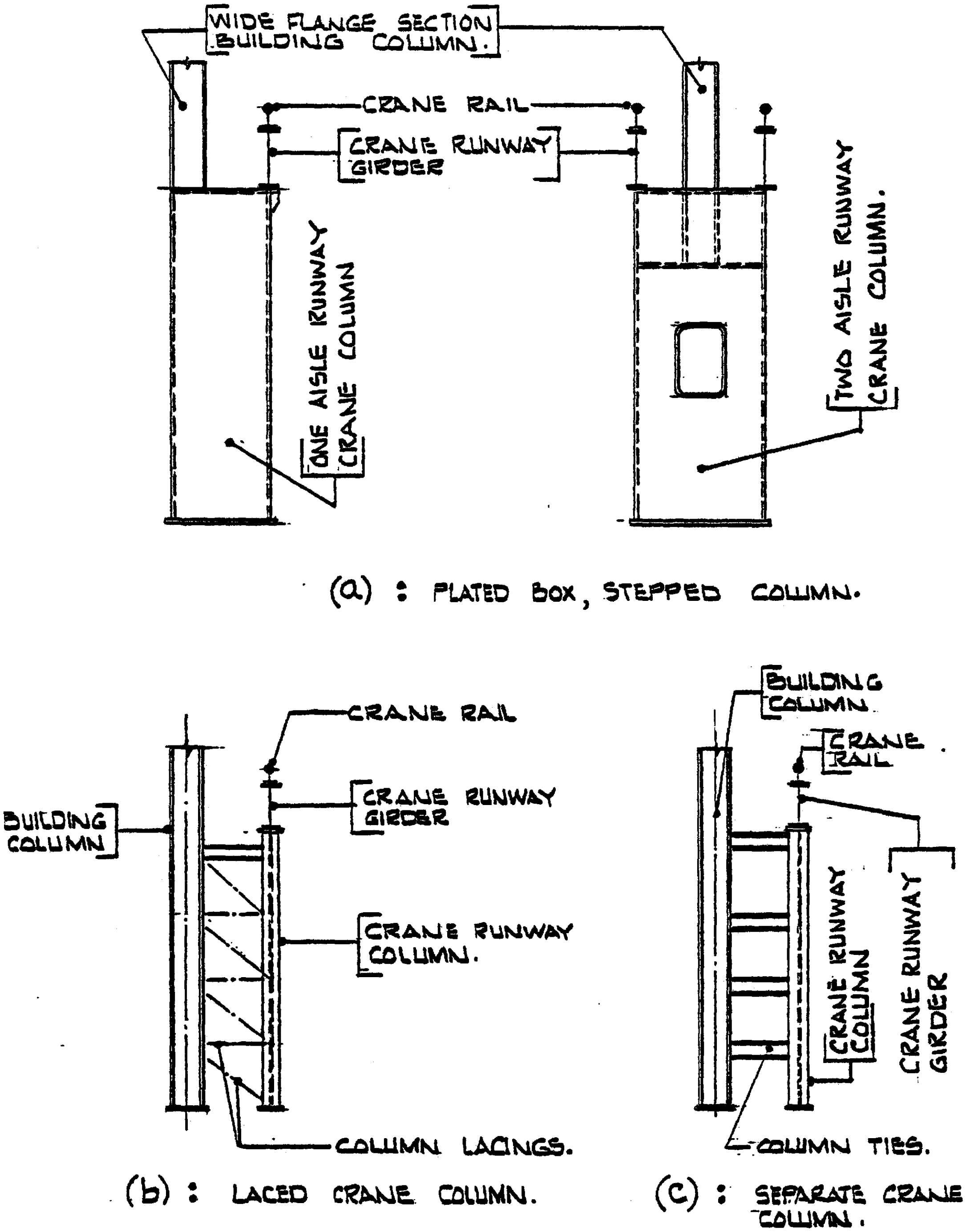


Figure 6.1 Typical Types of Column for Crane Runways

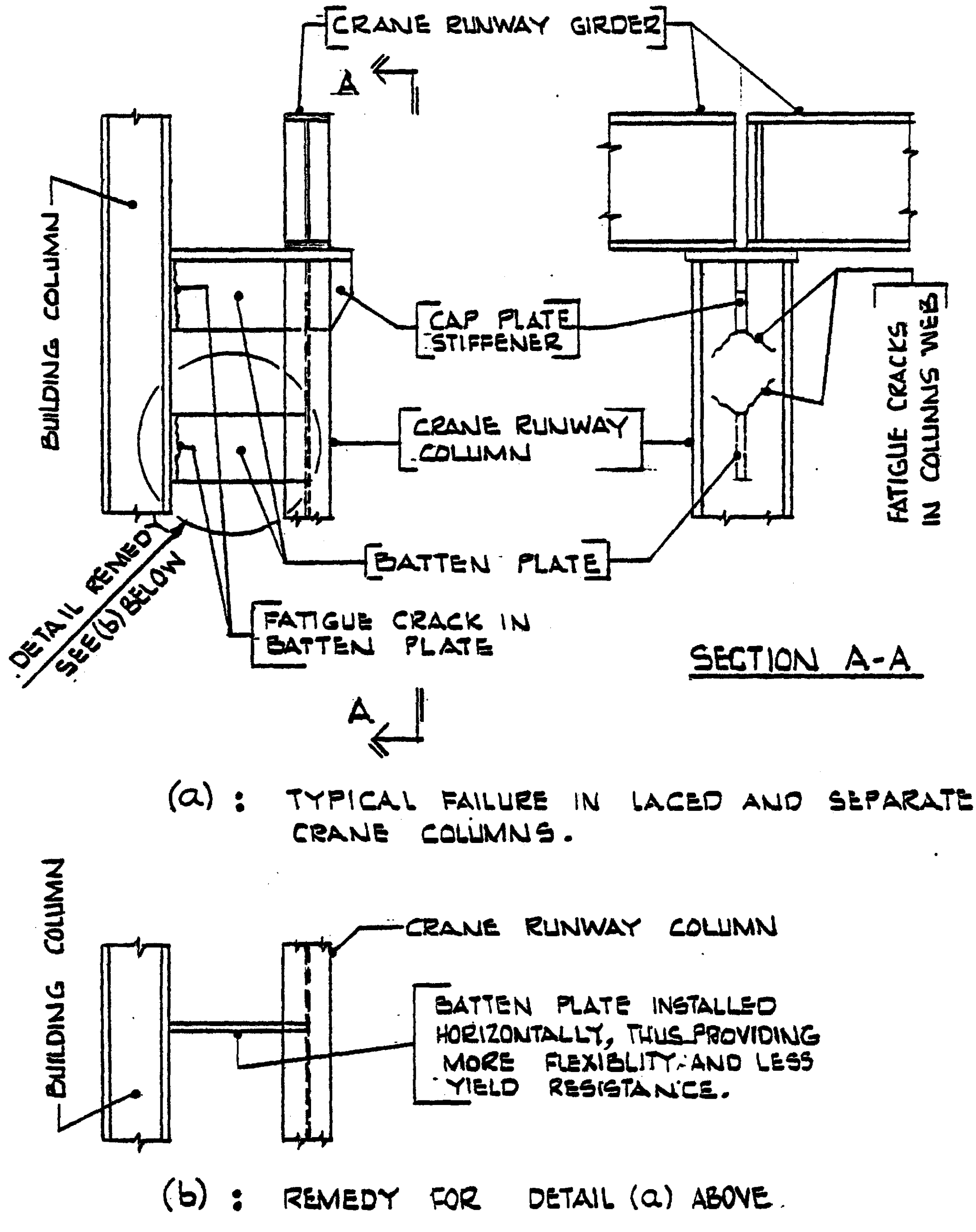


Figure 6.2 Some Typical Details of Connections in Column and Types of Failures

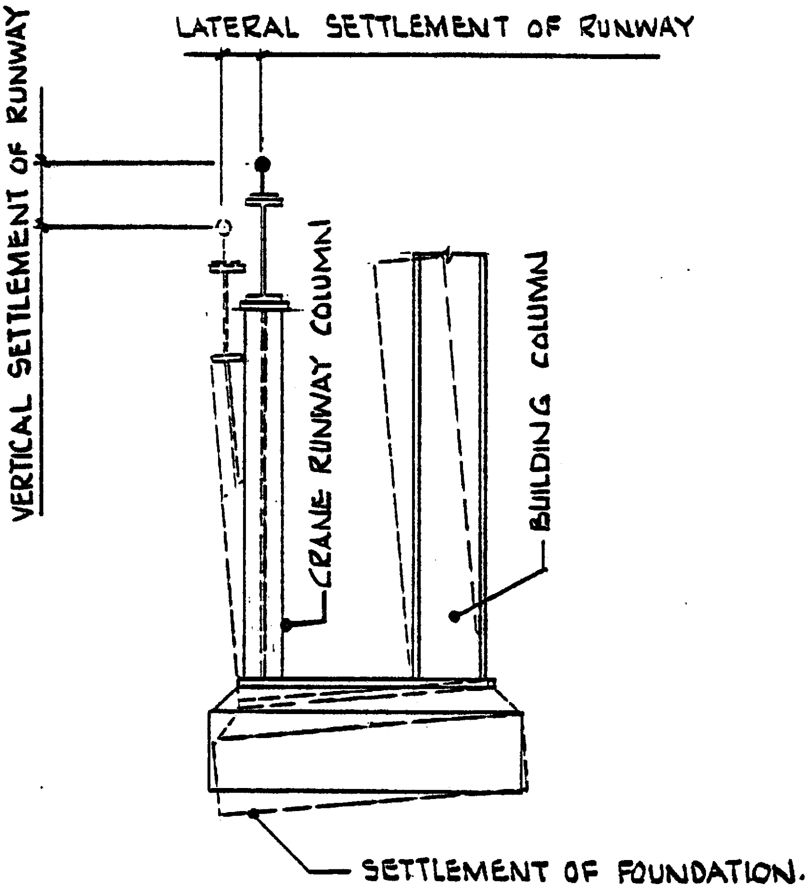


Figure 6.3 Foundation Settlement and its Effect on the Runway

CHAPTER 7

GIRDER SUPPORT SYSTEMS

Girder systems are generally classified according to the method of their support. There are four systems, namely:

- Continuous
- Trussed
- Knee Braced
- Simple

Each system has advantages and disadvantages and in order to arrive at a rational comparison, a computer analysis of each system is needed, using uniformly applied data and criteria.

A computer program written by R. Bent [18] analyzes the structure by moving the wheel loads from the left to the right, calculating the maximum positive and negative moments at one foot intervals. From these a moment envelope diagram is constructed. To illustrate the relative advantages and disadvantages of different girder support systems a typical layout was analyzed using R. Bent's program based on the following data and criteria.

a) Two bays are used for continuous and knee braced system. One bay is used for the trussed and the simple system.

b) The bay span is 50 feet from centre to centre of the columns.

c) The height of the structure from the underside of the base plate to the top of the girder is set at 60 feet.

d) The structure is assumed stable longitudinally. It is analyzed for vertical loads only.

e) The wheel load is assumed at 120 kips total, including impact, with four wheels per side spaced as shown below:

f) The size of the structure is based on the inertia required to limit deflection to:

$$\frac{\text{Span}}{1000} = \frac{50 \text{ feet}}{1000} \times 12 \text{ inches} = 0.60 \text{ inches}$$

7.1 CONTINUOUS GIRDER SUPPORT

The girder is supported continuously over the top of the column. Its connection to the top of the column can either be rigidly fixed or pinned, see Fig. 7.1. If rigidly fixed, the column participates in resisting the moment, thus providing longitudinal support. If it is a continuous pin connection, the column will act as a prop only, with some form of bracing required for longitudinal stability.

The moment envelope diagram for a rigidly fixed structure is given in Fig. 7.2 and for continuous pinned structure in Fig. 7.3.

Advantages

a) For a certain deflection value a smaller inertia will be required, and hence a lighter girder. Analysis shows that an inertia of 100,000 $in.^4$ is sufficient to limit the deflection to 0.60 in.

b) Due to elimination of bracing, accessibility to equipment and machinery will be somewhat improved.

Disadvantages

a) The girder can be prone to fatigue failure. In general, for dynamically loaded structures, the fatigue life expectancy is relatively short when:

- there is stress reversal
- the stress fluctuates over a wide range
- the structure experiences great number of cycles.

Figures 7.2 and 7.3 shows that most of the girder experiences stress reversal over wide ranges, satisfying the first two conditions for fatigue failure. As for number of cycles, the basic design value of over 2 million cycles applied (see Section 3.2.2) may not be sufficient. Mass [12] reports that this value was reached within eighteen months of operation. Therefore, the third condition of failure can be claimed satisfied.

b) The structure, being continuous, has to be transported in segments and field spliced by welding. This can create weakness in fatigue strength. To splice dynamically loaded structures by welding is a very poor practice. The probability of it acting as a stress raiser is very high. If splicing is ever necessary, special caution and care will be required for performing the welding under controlled conditions, in order to reduce the chances for failure. Attempting such welding in the field is even more risky, since its quality can not be ensured. This is more so when the structure is under severe stress conditions. Therefore, there is absolutely no justification for taking such a risk.

c) Being a rigid free system, longitudinal stability of the structure will be provided by the stiffness of the column and the girder respectively through their rigid connection. Therefore, stiff columns and beams are necessary. This in itself will nullify the system's advantage regarding its small inertia requirements.

d) A stiffer column draws heavier moment. These high moments will induce higher shear forces in the base, necessitating unduly heavier foundations.

e) Future repairs and modifications will not be practical nor economical.

7.2 TRUSSED GIRDER SUPPORT

In a trussed system, Fig. 7.4, the main runway girder spans between two columns, and is proped at its midspan by a member called the

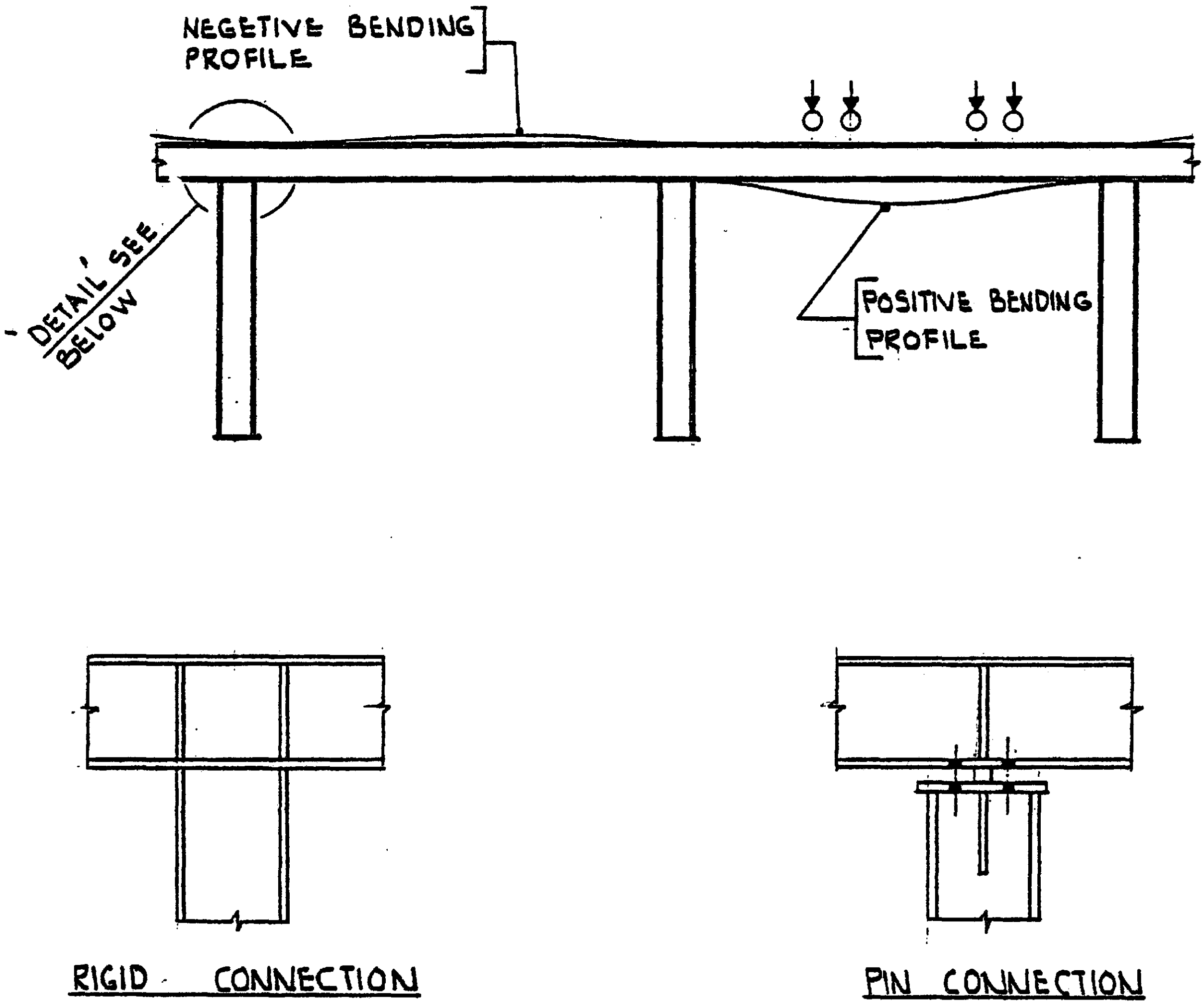


Figure 7.1 Typical Continuously Supported Runway Girder and its Bending Profile with one Span Loaded

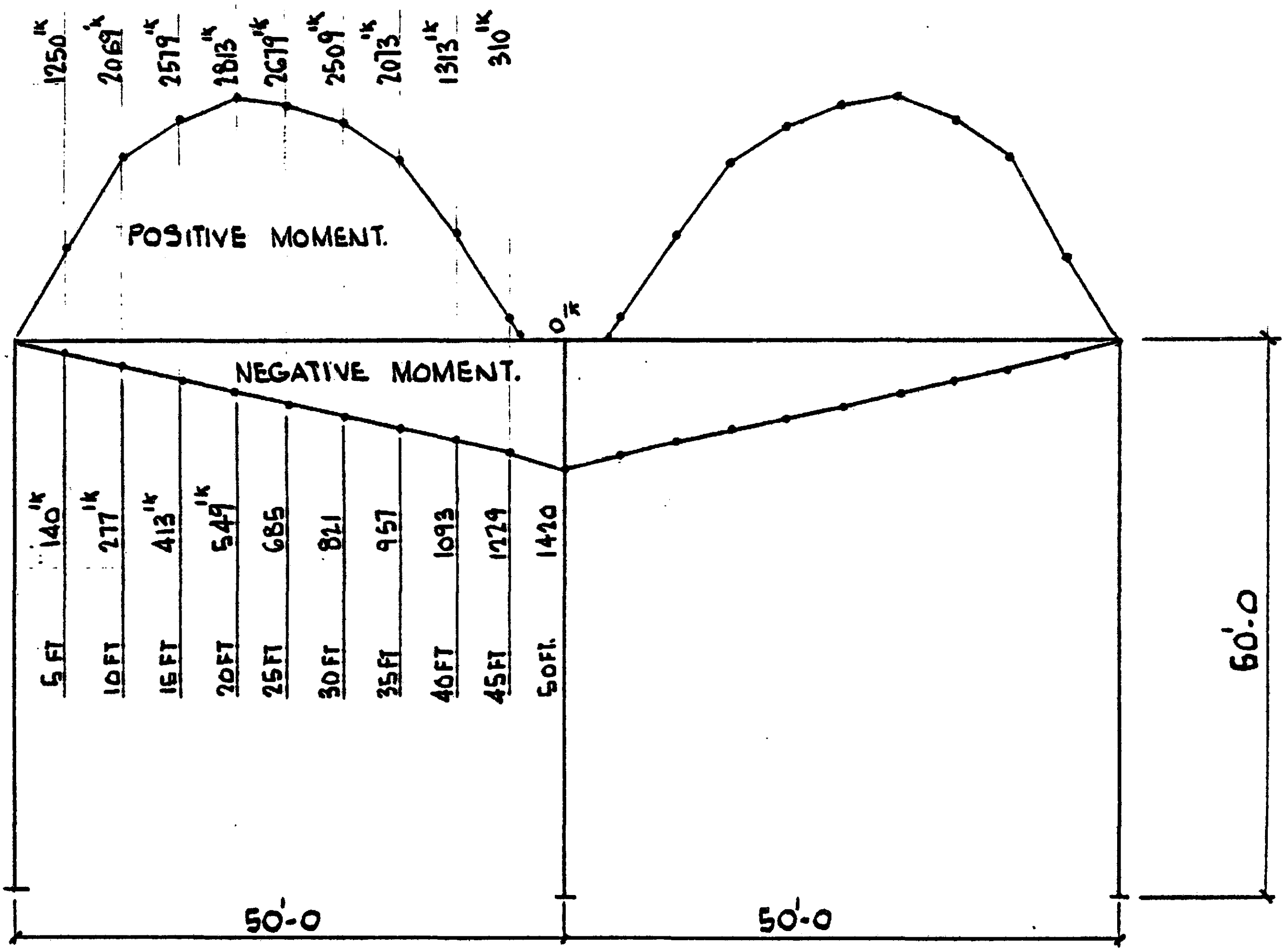


Figure 7.2 Positive and Negative Moment Envelope for the Continuous Rigid Girder Support

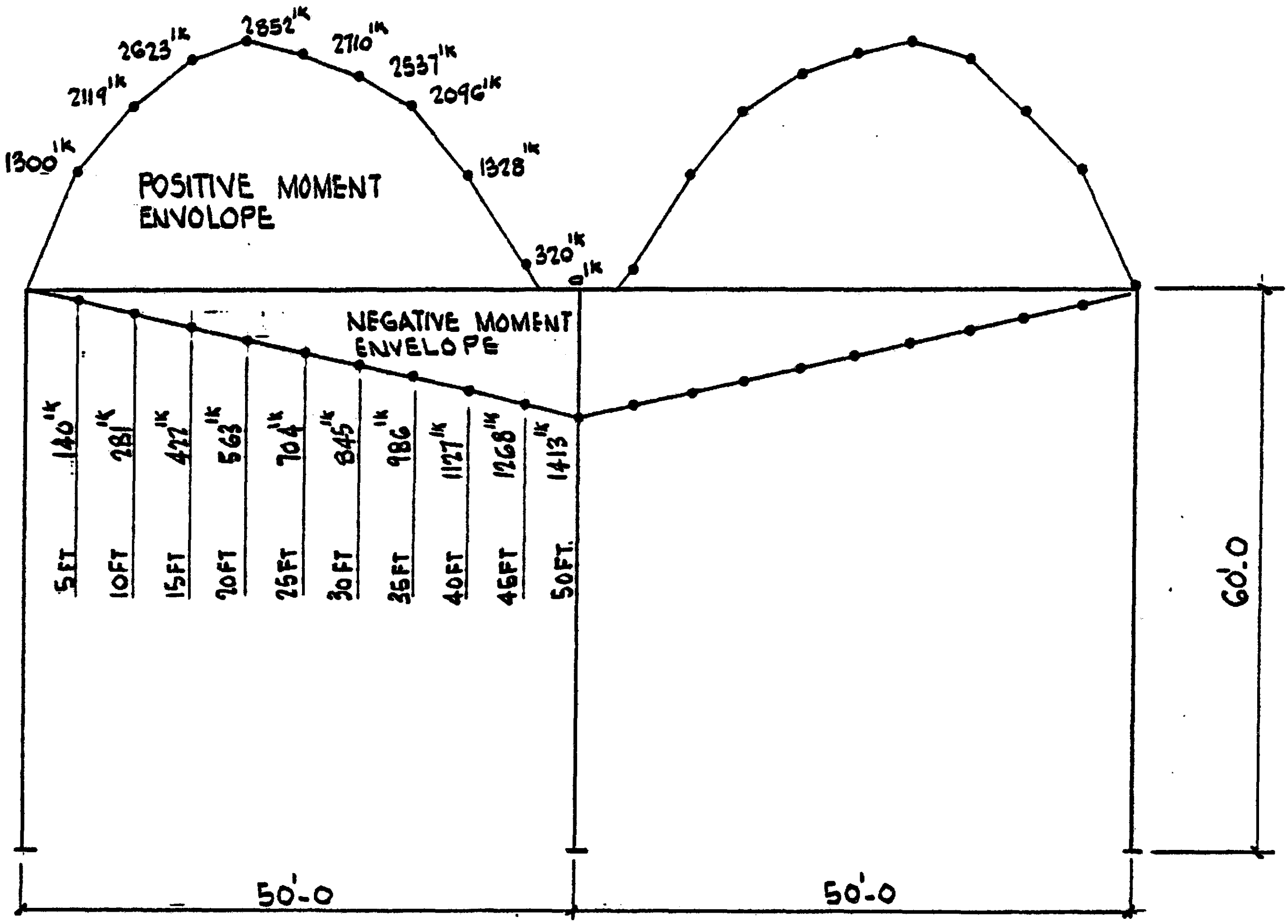


Figure 7.3 Positive and negative moment Envelope for the Continuous Pinned Girder Support

King Post, which is braced back into the girder. This forms a truss with the girder acting as its top chord. The girder is continuous and pin supported at the king post. The pinning action is achieved by providing a rocker bar between its underside and the top of the post. It is simply supported at the column with the provisions made for end rotation. The moment envelopes for positive and negative moments are shown in Fig. 7.5.

Advantages

a) Longer spans are possible with standard rolled wide flange sections as compared to deep built-up plate girders. For example, built-up plate girders will not be necessary for a 50 feet span.

b) It is realtively easy to fabricate and erect.

c) Lighter columns are possible due to the reduction in their unsupported spans by the depth of the truss.

d) The bottom chord can be utilized as a main service walkway. It also provides route for service lines such as piping and electrical conduits.

e) Since one design criteria of the mill structure is for abuse, the trussed system has proven relatively safe. For example, in one known case, one of the bracings interfered with mill activities. Rather than consult the engineering department, the operators removed that bracing completely. The structure remained stable under the usual operation until it was repaired.

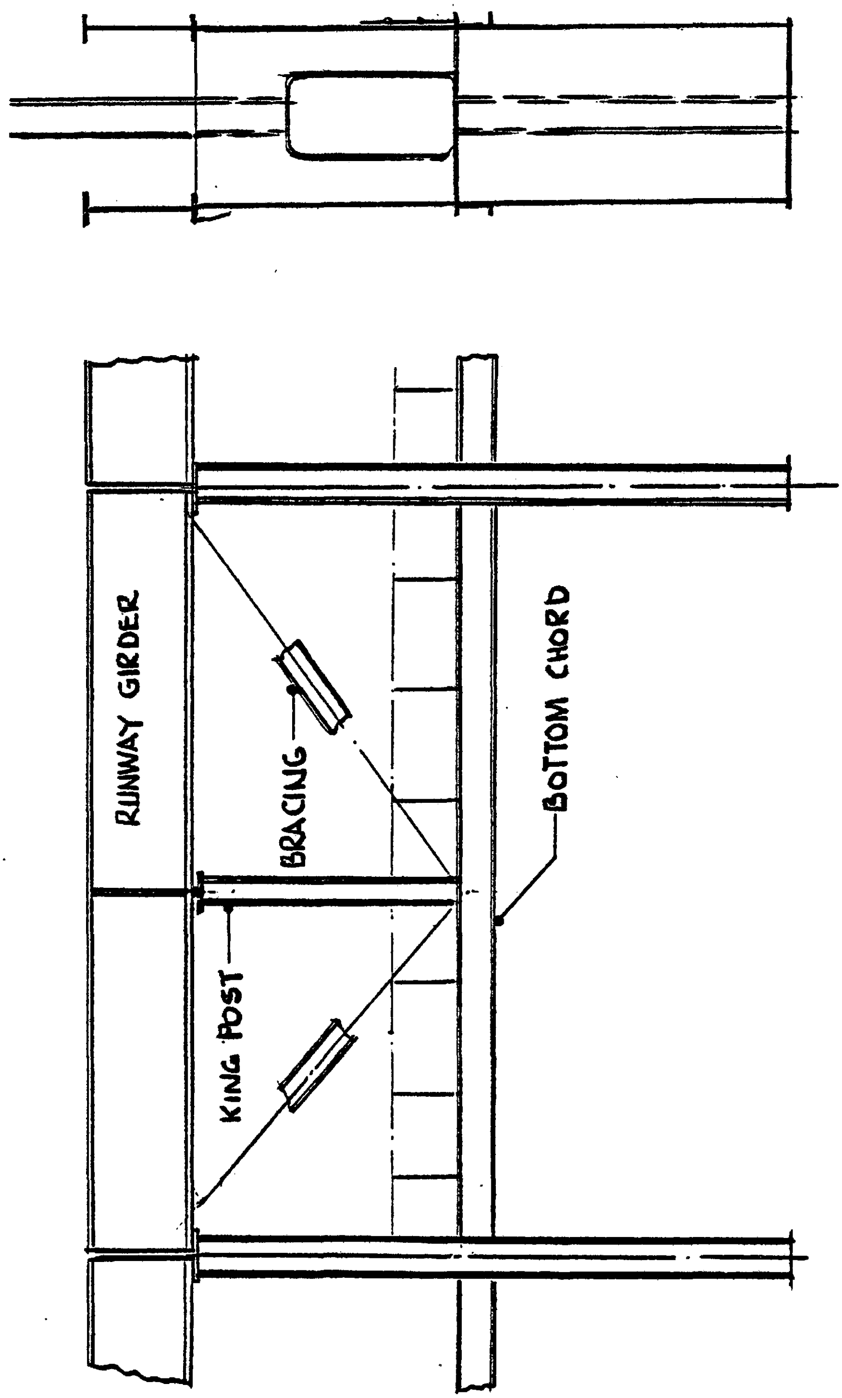


Figure 7.4 Typical Trussed Girder Support System

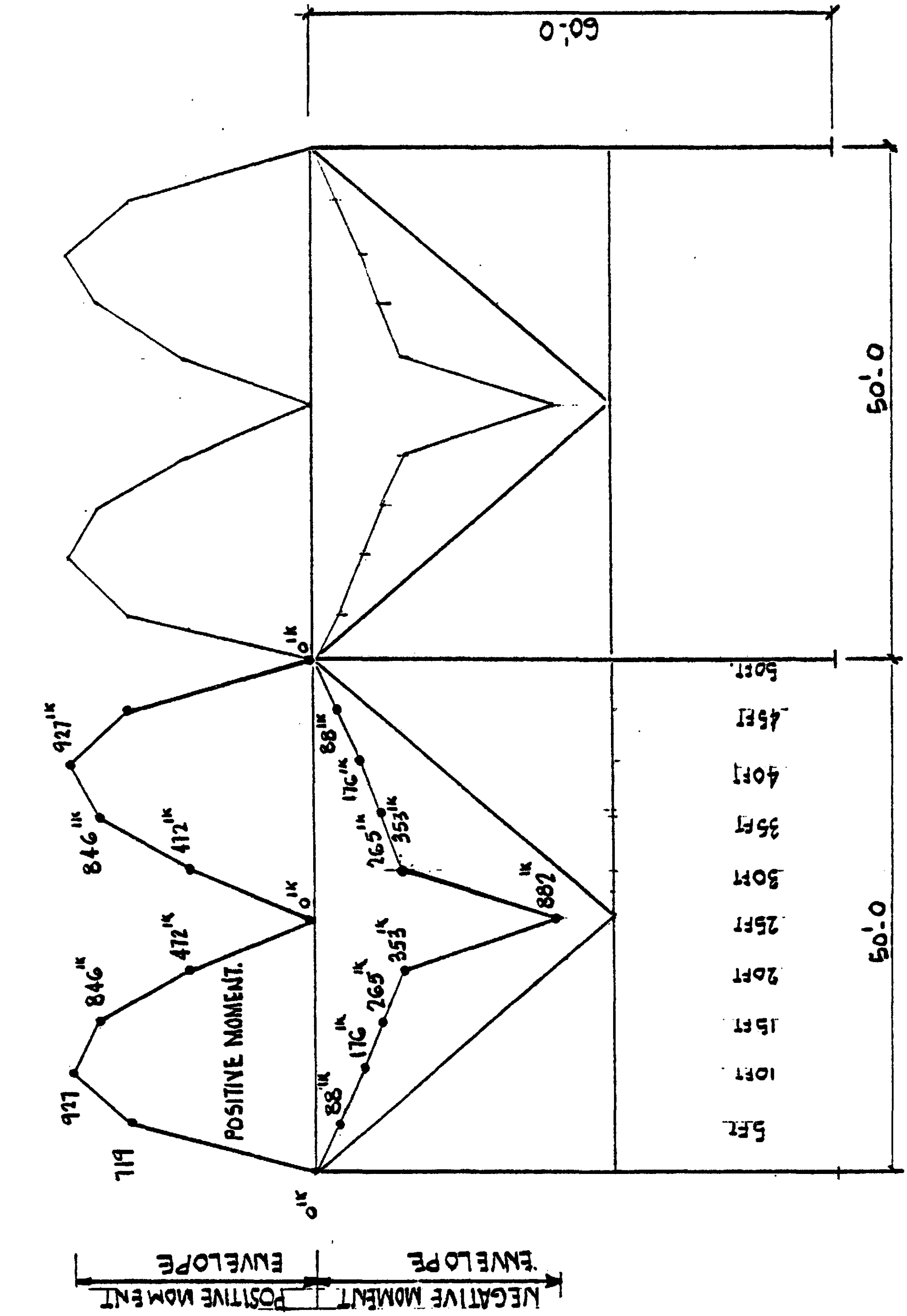


Figure 7.5 Positive and Negative Moment Envelope for The Trussed Girder Support

f) A trussed girder is relatively easy to repair or to modify. Repair in the example cited above did not interfere with the mill operations.

Disadvantages

a) The top chord (main girder) being continuous, will be sensitive to fatigue for the same reasons as those of the continuous system in Section 7.1.

b) Being continuous, special care must be exercised when detailing its connections, specifically splicing, if anny.

c) Headroom may be restricted along the full length of the runway.

7.3 KNEE BRACED GIRDER SUPPORT

Similar to the continous girder support, its advantages are very limited, while the disadvantages are numerous. For a brief discussion of its design approach and the associated problems see Section 5.2 and Fig. 5.1(b). Moment envelopes for positive and negative moment are shown in Fig. 7.7. A typical elevation of a knee braced runway structure is shown in Fig. 7.6.

Advantages

Longer spans are possible with a standard rolled wide flange section versus built-up plate girders due to the reduction in the free span of the girder.

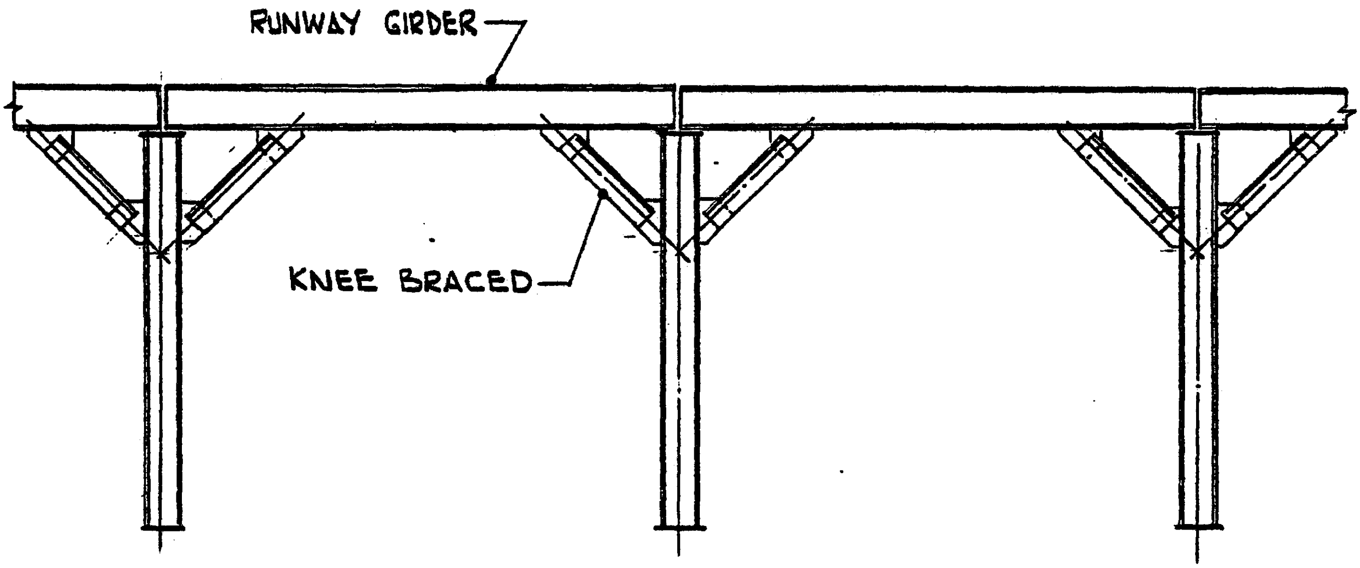


Figure 7.6 Typical Knee Braced Girder Support System

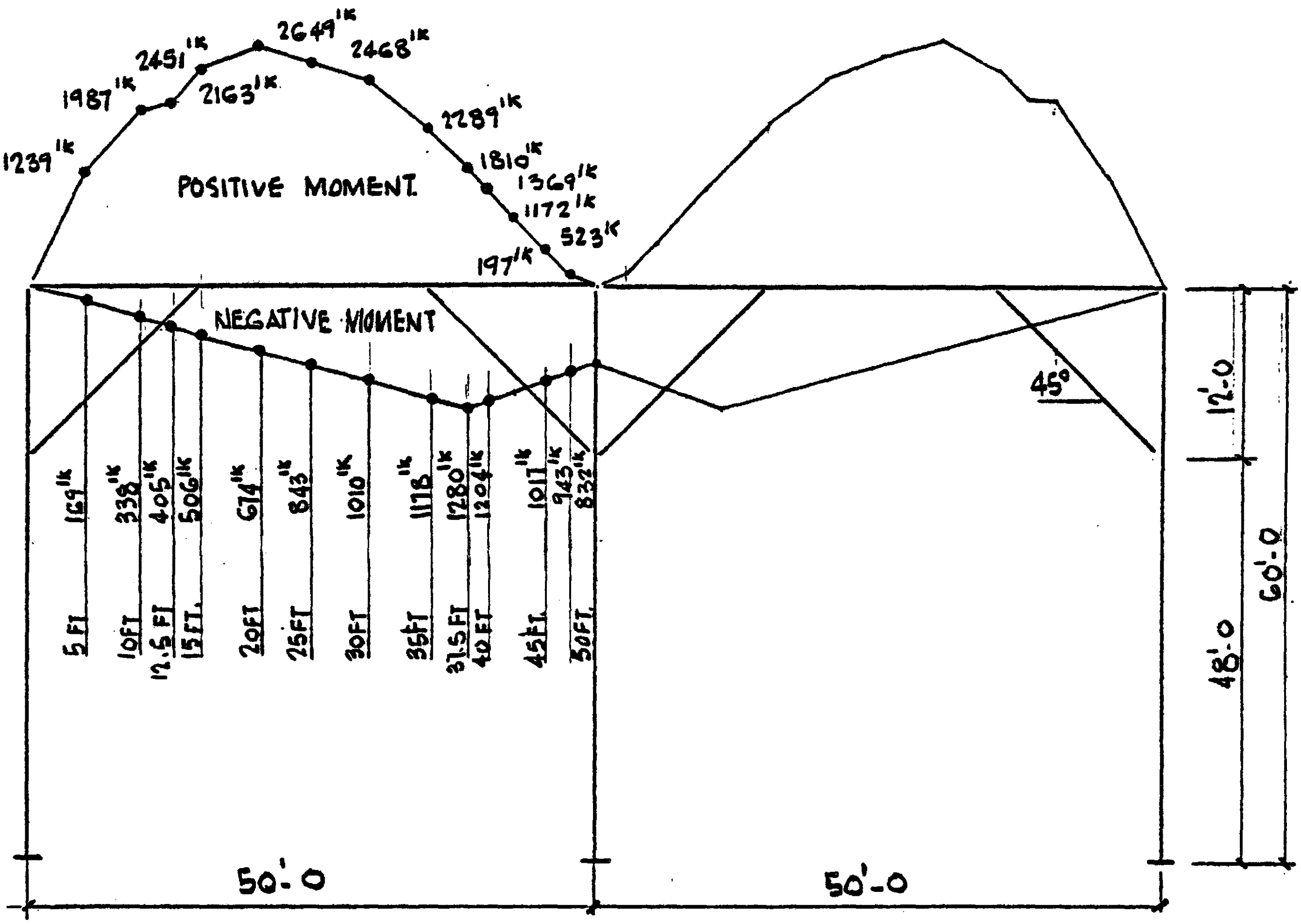


Figure 7.7 Positive and Negative Bending Moment Envelope for the Knee Braced Girder Support

Disadvantages

a) Failures due to fatigue caused by its continuity are frequent. The reasons are the same as those given in Section 7.1.

b) Failure incidence is high in knee braced connections at the column and the girder.

c) Buckling of the knee brace is frequent.

d) Fatigue failure in girder web directly above the knee brace connection is frequent. The same applies to the column web.

e) Moments induced in the columns, result in heavy sections.

f) Heavy foundations result from shear forces induced in the base by these moments.

7.4 SIMPLE SUPPORT GIRDER SYSTEM

The girder spans freely between the two columns, without any intermediate supports. Its ends are independent of the girders in the adjacent bays. It is detailed to perform as close as possible to a pin, in order to allow freedom of end rotation (see Section 5.2). A typical elevation is shown in Fig. 7.8, and the appropriate moment envelope diagram is given in Fig. 7.9.

Referring to Fig. 7.9, the absence of the negative moment envelope will be noticed. This indicates that there are no stress reversals. The fluctuation is between minimum (dead weight) and the maximum (vertical wheel loads). This does not mean that the girder will not experience some reversal of stresses. Studies have indicated the existence of stress reversal caused by impact. The difference though is that the range of the stress reversal if ever present, is very small, and incidence is infrequent.

Advantages

a) A simple support girder system is easier to design, fabricate and erect.

b) It requires fewer welded connections and joints, hence, there are fewer stress raisers to cause fatigue failures.

c) End rotation is possible, thus eliminating end restraints which can cause failure.

d) Through pinning the end, loads will be concentrically induced into the column. This, combined with the freedom to rock, virtually eliminates moments and their consequences in the column.

e) Future repairs and modifications are economical and practical.

Disadvantages

a) For longer spans deeper and heavier girders become necessary.

b) Access is restricted in the braced areas. The number of braced bays will depend on the total length of the runway. One braced bay in each length of up to 600 feet is required.

Conclusions

From the discussion in this chapter and those of preceding chapters, it can be concluded that a simply supported girder system is far superior when used in a crane runway system, when compared to the other girder systems such as the knee braced, and the continuous rigid and pinned. It is simple to fabricate and easy to repair or modify. It has proven very efficient structurally. The consensus among the designers of heavy crane runway structures in particular and of dynamically loaded structures in general is to use simply supported girder systems.

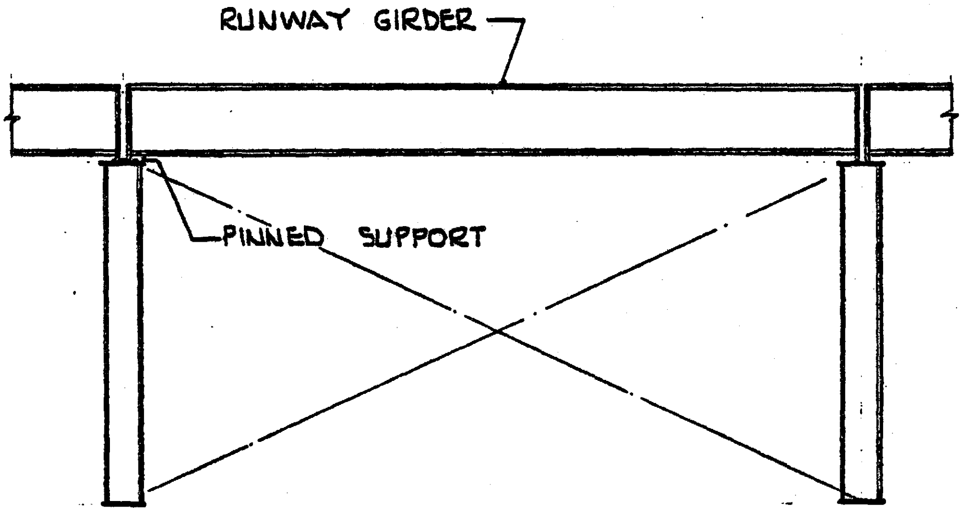


Figure 7.8 Typical Simply Supported Girder System

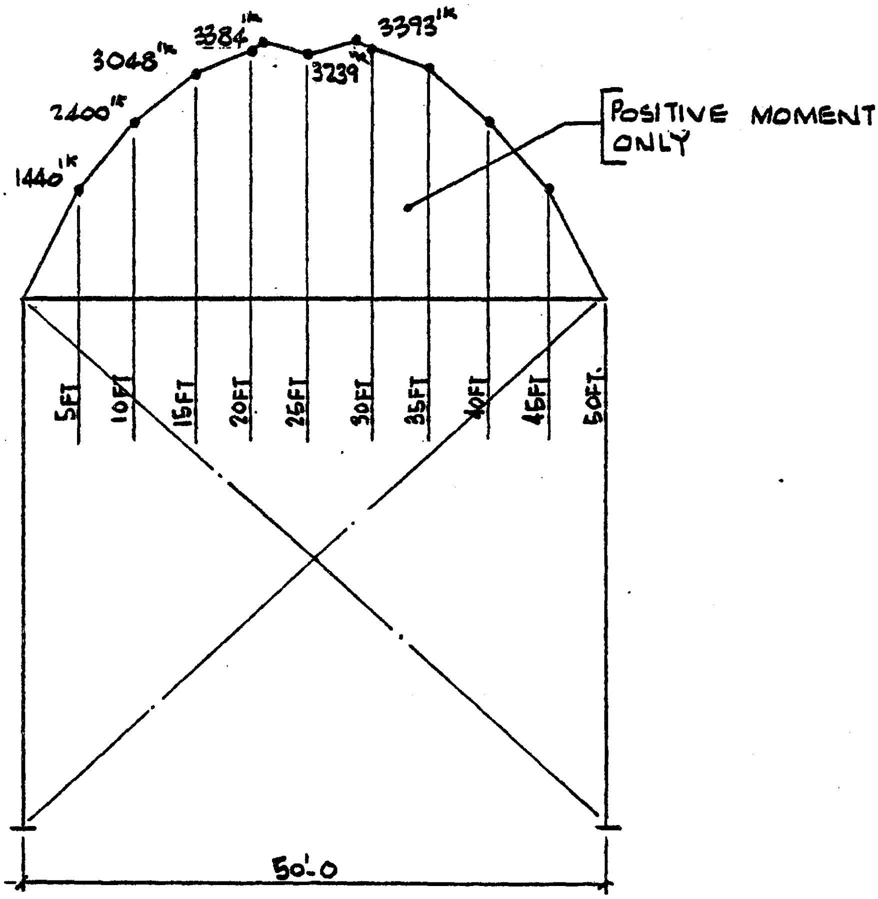


Figure 7.9 Moment Envelope for the Simply Supported Girder

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

Perhaps more than in any other type of steel structure, the successful design of crane gantries depends on the combination of design expertise with the experience of the operator or maintenance engineer. It has been shown in several of the preceding chapters that the guidelines provided in various codes of practice are insufficient in themselves or require to be implemented with considerable thought given to the detailing of the structural elements and connections. The environment and conditions under which crane gantries are operated are extremely severe and subject to considerable uncertainty. As a result it is frequently desirable to make generous allowance for the abuse which may result from operator errors or the effect of wear and tear on members and their connections. In situations where a design is done "in-house" the necessary feed-back from the maintenance engineer to the designer is easily achieved. If, however, designs are sub-contracted to external consultants every effort must be made to ensure that the consultant is aware of the extreme conditions to which the structure may be subjected and also of the considerable expense which may result from failure of a component due to certain "idealized" and assumed conditions not being realized in practice. In general, short term economies at the design stage prove to be expensive over the lifetime of the structure.

A number of specific points may be summarized here relating to the assumptions made at the design stage.

1) Fatigue appears to be the major cause of failures in crane runways.
2) The failure of connections is more common than those of the members.
3) The design assumptions and the criteria in general, are based on the ideal behaviour of the structure, which invariably is not the case in practice.
4) The current code of practice "CISC - S16", does not adequately cover the design of mill buildings in particular crane runways.
5) Current literature on fatigue deals mainly with the design criteria for the members, through formulae, graphs and charts. However, very little information is available for the proper design of efficient connections.
6) Unlike the other types of structures (e.g. highrise, bridges, highways, etc.), mill structures and crane runways have received relatively little attention.

In Chapter 3 it was emphasized that proper detailing of the rail and girder is probably more important than the design. The importance of proper rail location and connection cannot be over-emphasized. The use of stiffeners in the main girder leads to a number of problems and use of stiffeners in the main girder leads to a number of problems and it may frequently prove cost-effective to use a thicker web plate and dispense with stiffeners completely. If stiffeners are employed attention to detailing is once again of extreme importance. The following are the more important aspects:

(i) stiffeners should be terminated one sixth of the girder depth above the bottom flange.

(ii) all welds should be carefully finished and ground.

(iii) stiffeners should be fitted and not welded to the underside of the top flange.

Due to the dynamic nature of the loading crane gantries are subject to fatigue stresses. Further research is required into the following aspects of design.

(i) the number of load cycles or stress-reversals to which the members are subjected.

(ii) the amount and effect of rail eccentricity.

(iii) the local effect on the top flange of the inclusion of web stiffeners.

(iv) the effect of rail condition and method of mounting on girder stresses.

Chapter 4 discusses the resistance of horizontal loads and describes a number of alternative schemes. As a general conclusion, the lateral and vertical support girders should be detailed to allow them to act independently. The only interaction between the vertical and horizontal girders should be to provide lateral stability to the other members.

The use of diaphragms to produce box girders is not recommended since this practice produces more problems in the form of local "stress-raisers", buckling of members and torsional effects than it solves.

Longitudinal forces arise from expansion, tractional forces or crane-stop loads. It is recommended that longitudinal bracing be located at the centre of runways up to 600 ft long thus eliminating the need for built in expansion joints. In longer runways built-in expansion joints may be unavoidable in order to maintain the maximum 300 feet length.

Forces from crane stops are best taken through the runway girder to the centrally located longitudinal bracing. This requires proper detailing of the girder-column connections. Bracing at the end-bays prevents the elimination of expansion joints and it is the latter that contributes more to the fatigue and breakdown of the rail and girder than crane-stop loads.

Finally in Chapter 7 alternative types of support systems were compared with the aid of computer generated analyses. The general conclusion is that simply supported girders are best since they minimize fatigue effects and stress-reversals.

Formulation of guidelines for the design of the mill buildings and crane runways similar to AISE.13 specification but more comprehensive, is recommended. It is known that many industries conduct their own investigations as to the reasons and nature of failures. The continued compilation and publishing of their findings in related journals is recommended.

REFERENCES

[1] CISC: Canadian Institute of Steel Construction, "Handbook of Steel Construction".

[2] NBC: Associate Committee on the National Building Code, National Research Council of Canada, "Candian Structural Design Manual".

[3] AISC: American Institute of Steel Construction, "Manual of Steel Construcction".

[4] B.S.449: British Standards Institution, "Specification for the use of Structural Steel in buildings".

[5] Association of Iron and Steel Engineers Standard #13. "Specifications for the Design and Construction of Mill Buildings", AISE #13.

[6] Association of Iron and Steel Engineers Standard #6. "Specifications for Electrical overhead traveling cranes for Steel Mills Service", AISE #6.

[7] DIN 120: German Specifications, "Bases of calculation for steel parts of cranes and crane tracks".

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[11] A.G. Senior and T.R. Gurney, "The Design and service life of the upper part of welded crane girders", Institute of Structural Engineers, October 1963.

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[14] S16.1: Canadian Standards Association, "Steel Structures for Building".

[15] B.E. Goreng, "Crane Runway Girders", Steel Construction.

[16] John J. Murray, "Impact Investigation Using Strain Gages on a Hot Metal Charging Crane", Iron and Steel Engineer Year Book, 1956.

[17] P.M. Worthington, "The effects of the wheel tread profiles and other factors on the long travel motion of gantry cranes", the Structural Engineer, December 1957.

[18] Ray Bent's computer program for moving loads.

APPENDIX A

SECTION B1 LOADING CONDITIONS AND TYPE AND LOCATION OF MATERIAL

In the design of members and connections subject to repeated variation of live load stress, consideration shall be given to the number of stress cycles, the expected range of stress, and type and location of member or detail.

Loading conditions shall be classified as shown in Table B1.

TABLE B1

Loading Condition	Number of Loading Cycles	
	From	To
1	20,000[1]	100,000[2]
2	100,000	500,000[3]
3	500,000	2,000,000[4]
4	Over 2,000,000	

[1] Approximately equivalent to two applications every day for 25 years.
[2] Approximately equivalent to ten applications every day for 25 years.
[3] Approximately equivalent to fifty applications every day for 25 years.
[4] Approximately equivalent to two hundred applications every day for 25 years.

The type and location of material shall be categorized as shown in Table B2.

SECTION B2 ALLOWABLE STRESSES

The maximum stress shall not exceed the basic allowable stress provided in Sects. 1.5 and 1.6 of this Specification, and the maximum range of stress shall not exceed that given in Table B3 except that, in the case of stress reversal only, the value F'_{sr} given by Formula (B1) may be used as the stress range for those categories marked with an asterisk in Table B2.

$$F'_{sr} = \left(\frac{f_t + f_c}{f_t + 0.6f_c}\right) F_{sr} \qquad \text{(B1)}$$

where f_t and f_c are, respectively, calculated tensile and compressive stresses considered as positive quantities, and F_{sr} is the allowable stress range given in Table B3.

TABLE B2

General Condition	Situation	Kind of Stress[1]	Stress Category (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Plain material	Base metal with rolled or cleaned surfaces.	T or Rev.	A	1, 2
Built-up members	Base metal and weld metal in members, without attachments, built up of plates or shapes connected by continuous full penetration groove welds parallel to the direction of applied stress.	Rev. Rev. T or C	B*[3] B B	3 4 3, 4
	Base metal and weld metal in members, without attachments, built up of plates or shapes connected by continuous fillet welds parallel to the direction of applied stress.	T, C or Rev.	B	4, 5, 6
	Calculated flexural stress, f_b, at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners:			
	When $f_v \leqslant F_v/2$	T or Rev.	C	7
	When $f_v > F_v/2$	T or Rev.	D	7
	where F_v = allowable shear stress.			
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends.	T, C or Rev.	E	5

[1] "T" signifies range in tensile stress only: "C" signifies range in compressive stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear including shear stress reversal.

[2] These examples are provided as guide lines and are not intended to exclude other reasonably similar situations.

[3] Formula (B1) applicable in situations identified by asterisk (*).

[4] Where stress reversal is involved, use of A307 bolts is not recommended.

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[1]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Mechanically fastened connections	Base metal at net section of high-strength-bolted connections, except bearing-type connections subject to stress reversal and axially loaded joints which induce out-of-plane bending in connected material.	T or Rev.	A	8
	Base metal at net section of other mechanically fastened joints.[4]	T or Rev.	B	8, 9
Groove welds	Base metal and weld metal at full penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection.	T or Rev.	A	10
	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles, when welds are ground flush.	T or Rev.	B	10, 11
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection.	T or Rev.	B	12, 13

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[1]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Groove welds (cont'd)	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½, when reinforcement is not removed and/or weld soundness is not established by radiographic or ultrasonic inspection.	T Rev. T or Rev.	C C* C	10 10 11, 12, 13
	Base metal or weld metal in or adjacent to full penetration groove welds in tee or cruciform joints.	T Rev.	D D*	14 14
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading.	T, C or Rev.	E	15
	Weld metal of partial penetration transverse groove welds, based on effective throat area of the weld or welds.	T or Rev.	G	16
Fillet welded connections	Base metal at intermittent fillet welds.	T, C or Rev.	E	
	Base metal at junction of axially loaded members with fillet welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses.	T, C or Rev.	E	17, 18, 19, 20
	Continuous or intermittent longitudinal or transverse fillet welds (except transverse fillet welds in tee joints) and continuous fillet welds	S	F	5, 17, 18, 19, 21

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[1]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Fillet welded connections (cont'd)	subject to shear parallel to the weld axis in combination with shear due to flexure.			
	Transverse fillet welds in tee joints.	S	G	20
Miscellaneous details	Base metal adjacent to short (2 in. maximum length in direction of stress) welded attachments.	C T or Rev.	C D	22, 23, 24 22, 23, 24, 25
	Base metal adjacent to longer fillet welded attachments.	T, C or Rev.	E	26
	Base metal at plug or slot welds.	T, C or Rev.	E	27
	Shear stress on nominal area of stud-type shear connectors.	S	G	22
	Shear on plug or slot welds.	S	G	27

TABLE B3

Category (From Table B2)	Allowable Range of Stress, F_{sr} (ksi)			
	Loading Condition 1 F_{sr1}	Loading Condition 2 F_{sr2}	Loading Condition 3 F_{sr3}	Loading Condition 4 F_{sr4}
A[1]	40	32	24	24
B	33	25	17	15
C	28	21	14	12
D	24	17	10	9
E	17	12	7	6
F	17	14	11	9
G	15	12	9	8

[1] For A514 steels in Category A, substitute the following values: $F_{sr1} = 45$, $F_{sr2} = 35$, $F_{sr3} = 25$ and $F_{sr4} = 25$.

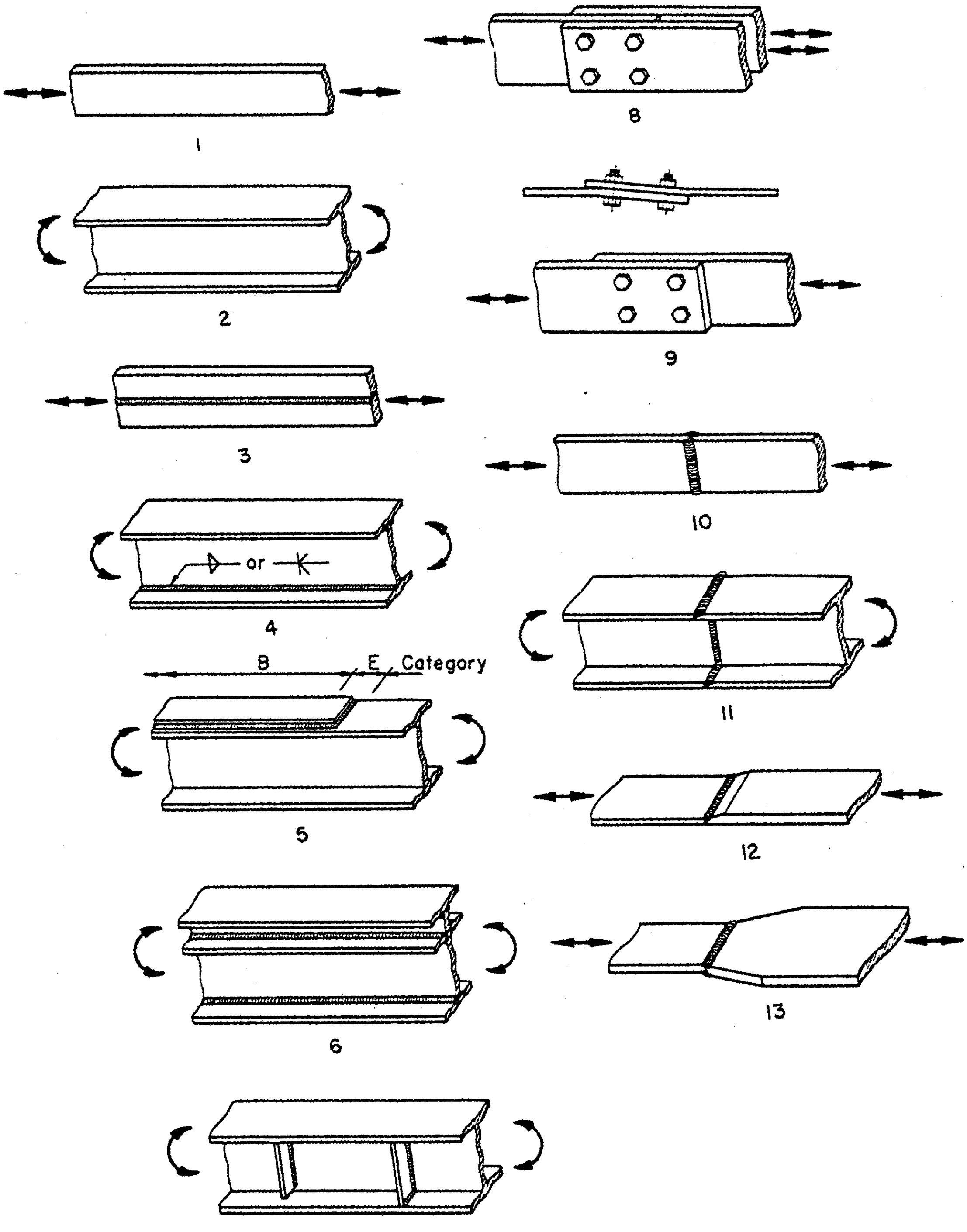

Fig. B1. Illustrative Examples

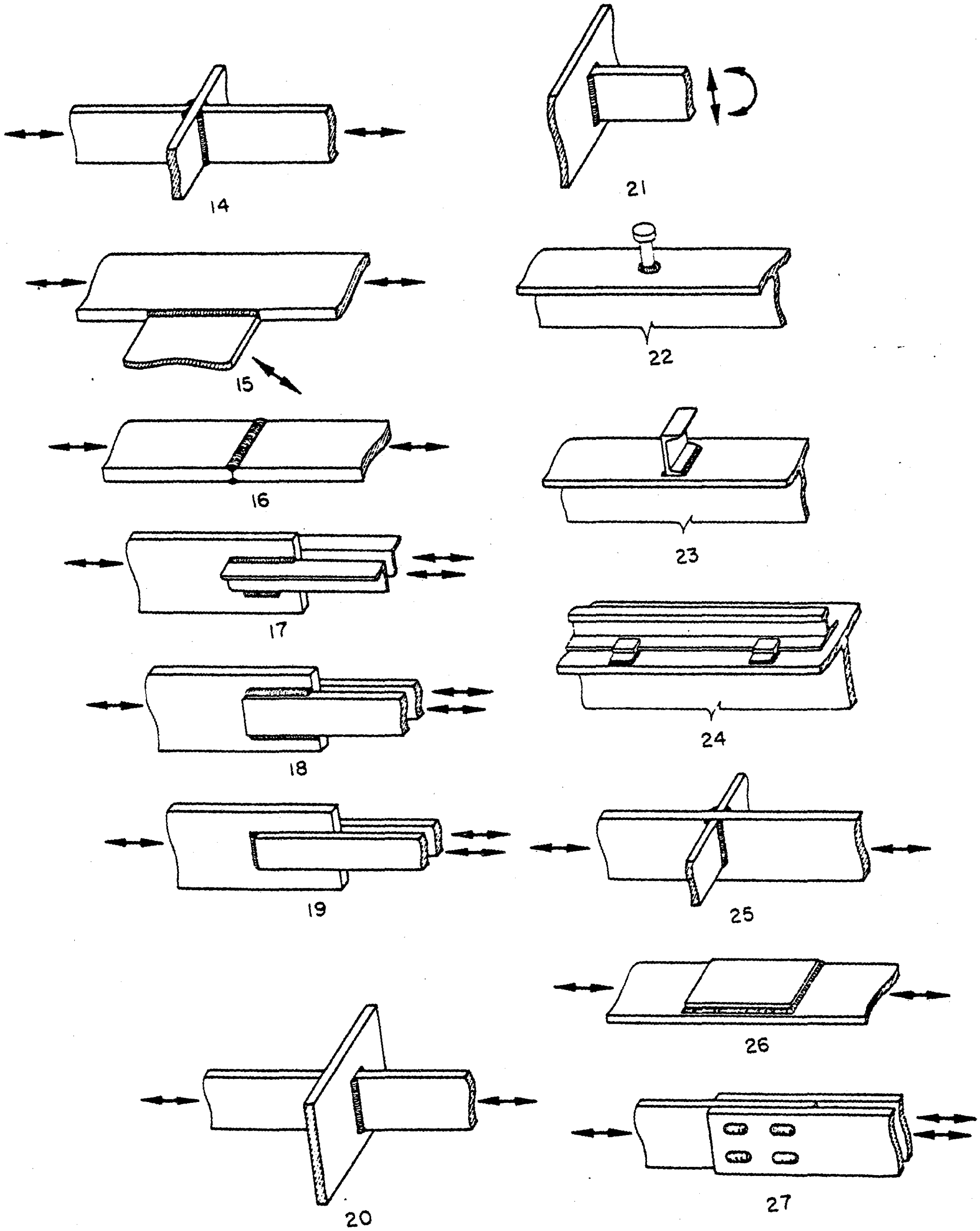

Fig. B1. Illustrative Examples (continued)