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FATIGUE RESISTANCE OF FRANKFORD EL LINE VIADUCT

by

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ABSTRACT

The safety and integrity of the Frankford Elevated Line Viaduct is studied through examination of the fatigue resistance of the superstructure. The cracks in the components of viaduct trusses are caused by fatigue. Restraint at truss ends is a contributing factor to these cracks. Sudden brittle fracture of viaduct component members is unlikely. Field measurements indicate that live load stresses are generally low. Sideways motion of train cars (nosing), however, induce fairly high stresses through the restraints at truss ends. Three major bridges of the line are adequate for use for many more years with proper maintenance. Cracks in some transverse girders of the bents are believed to be fatigue cracks induced by fretting. Deflections of the bent columns are stationary and are expected to recover when connections to the bents are relieved. Replacement of the viaduct spans is recommended.
1. INTRODUCTION

1.1 Background

The Frankford Elevated line is a principal rapid-transit rail-
road in the city of Philadelphia, connecting the center and the north-
east section of the city. The railroad construction began in 1915,
and was open for public service in 1922. The elevated line has about
six miles of viaduct and three bridges, which consist of longitudinal
trusses, floor beams, and concrete decks. The railroad tracks are
supported on the concrete decks which encase the floor beams. The
trusses are supported by rigid frame steel bents.

Since 1952, inspections of the structure have revealed cracks
in the longitudinal trusses, primarily at the end panels of these
trusses. Repairs were made by welding the cracked members, yet cracks
reappeared at the same location. Also, inspections showed that cor-
rosion has taken place in the superstructure, particularly where the
concrete decks encase the truss members. The columns of the rigid
frame steel bents have been detected to deflect in the direction of
the elevated line. A number of studies were made to evaluate these
conditions, but no major repair or correction have been undertaken.

In the 1960's plans were initiated to relocate a section of
the line in conjunction with the construction of the I-95 Expressway.
A segment of the railroad was subsequently replaced by a new structure.
The replaced segment provided an opportunity for an on-site study of the strength of the viaduct. The consultant firm of Wiss, Janney, Elstner and Associates (WJE) conducted the static load carrying capacity testing of two viaduct spans and concluded that the static strength was five and a half to six times the design live load of the current trains.

The tests by WJE were static destructive tests which did not incorporate dynamic loads, and did not address the question of how and why those cracks occurred. The study reported herein was setup to examine the cause and consequence of fatigue cracks related to train loads, and to assess the safety and integrity of the viaduct structures. The objectives and scope of work are briefly summarized below.

1.2 Objectives and Scope of Study

After a field inspection of some viaduct spans with cracks and a brief review of a few reports on the structure, it was preliminarily assessed that the cracks were fatigue in nature. The study was then established with the objectives:

1. To evaluate the fatigue strength of the truss members where fatigue cracks have been detected,

2. To evaluate the safety and life expectancy of the viaduct truss spans with regard to fatigue and fracture, and,

3. To evaluate through structural analysis the feasibility of using existing structural components of the viaduct for renovation of the line.
The outcome of the study was expected to provide information for the determination of a renovation scheme.

To achieve the above objectives, the following phases of study were planned:

1. Analysis of a single truss of the viaduct spans for the evaluation of effects of support restraint;
2. Analysis of a viaduct span, with three trusses and the deck, to estimate stresses in the truss component members;
3. Field measurement of actual stresses in viaduct truss components to correlate with analytical results from computer analyses;
4. Estimation of the fatigue strength of truss members in the end panels, on the basis of available data on fatigue of structural details;
5. Laboratory tests to examine the fracture toughness of the steel of the viaduct;
6. Examining the likelihood of brittle fracture of truss members in viaduct spans;
7. Analysis of the rigid-frame support bents to assess the column deflections and the cracks at the transverse bent-girders; and
8. Assessing the safety and integrity of the viaduct truss spans. Recommendations with regard to renovation of the structure was to be made from the viewpoint of safety and integrity of the structure.
As the intended study was commencing, a possible renovation scheme was being considered which would include new structural connections to the transverse girders of the rigid-frame bents. Also, it was decided that three major bridges in the viaduct line need to be investigated with respect to their life expectancy. Consequently, the scope of the study was enlarged to include additional phases.

9. Review of proposed connection details between longitudinal girders in viaduct spans and transverse girders of the bents;


Adding of these new phases changed the course and depth of the study, but the objectives remained the same.

All phases of the study have been completed. The results are reported in the following chapters. Chapter 2 summarized the findings on the viaduct truss spans, Chapter 3 deals with the major bridges, Chapter 4 examines the rigid frame bents, and Chapter 5 considers the structural connections at transverse girders and other relevant items. Suggestions are made at the end of each chapter with regard to renovation and maintenance. All these are summarized in Chapter 6.
2. VIADUCT TRUSSES

2.1 Existing Conditions

The viaduct is a riveted steel structure with more than six hundred spans of steel trusses. Typically, each span consists of three parallel trusses with the inbound and outbound tracks separated by the center (interior) truss. The ends of the trusses rest on steel rigid frame bents. A schematic plan view is shown in Fig. 2.1, an elevation in Fig. 2.2 and a typical bent in Fig. 2.3.

The special features of the truss spans include the following:

1. The span length and column height vary along the viaduct. The number of panels of trusses also vary, depending on the span length.

2. The trusses were designed for "simply supported" condition at the bents. Connection plates at the top chord of two adjacent trusses and steel links between corresponding end verticals, however, provided continuity from truss to truss. Although expansion joints were installed at every three or four spans, the top chord connection plates also existed at these joints. Through the decades, repairs have been made to some of the connection plates and steel links have been removed at a number of locations. A large majority of the
viaduct spans, the connection plate and link remain in place. Figures 2.4 and 2.5 are (1979) pictures showing an example of these details.

3. The concrete slabs are at about mid-depth of the trusses. The slabs encase the transverse floor beams and all the web members of the center trusses. Figure 2.6 is a sketch showing this condition. Figure 2.7 is a picture taken from below the viaduct showing the encasement.

Earlier inspections by the team from Southeast Pennsylvania Transit Authority (SEPTA) and by others have revealed corroded conditions of the steel truss members (particularly at the center truss at some locations), frozen expansion joints, and cracks at the truss end panels (primarily in the end diagonals and connection plates). Figure 2.8 is a sketch indicating the general area of detected cracks in the diagonals.

Repairs have been made, including welding of reinforcing (doubler) plates at end diagonals, relieving of frozen expansion joints, replacement of end panel top chord connection plates, removal of end panel steel links, and others.

Currently the SEPTA inspection team conducts continuous visual examination from one end to the other of the viaduct along its full length to detect and repair cracks.
2.2 **Analysis by Computer Model**

In order to assess the fatigue strength of the viaduct truss members and the safety of the trusses, accurate analysis of a span is essential. Because of the highly redundant condition of the viaduct spans which include three trusses and the concrete slab with all the special features cited in Section 2.1, it is very cumbersome and time consuming to compute stresses at selected points of a truss member. The outcome can not be expected to be quantitatively accurate.

Based on this crucial condition, it is therefore decided to examine the stresses by analogy so as to gain qualitative insight. The actual stresses at certain members will have to be examined through actual measurement.

Two models are used, the first a single truss and the second three parallel trusses with a concrete slab. The former is for the evaluation of effects due to restraints at ends; the latter provides results for comparison when a concrete deck is an integral part of the span.

2.2.1 **Single Truss**

An eleven panel truss is arbitrarily chosen as shown in Fig. 2.9. The dimensions of the members are equivalent to those from an outside truss of the viaduct and are listed in Table 2.1. All joints at panel points are assumed rigidly connected. This means that all truss members resist bending moment. The left end support is assumed as a roller at the top chord, simulating an expansion joint without the top chord connection plate.
The influence lines for the axial forces and bending moments at the upper end of the end panel diagonal ($U_0 - L_1$) are plotted as Figs. 2.10 and 2.11. The corresponding maximum axial and bending stress at the bottom edge of the upper end of this end diagonal are 0.099 ksi and 0.031 ksi, respectively, with a sum of 0.130 ksi. The maximum axial, bending, and total stress for the end panel top chord ($U_0 - U_1$) are -0.057 ksi, -0.072 ksi and -0.129 ksi, respectively per 1 kip of load on the truss. These and other stresses are listed in Table 2.2.

It is important to note that the stresses due to bending of truss members are of lower magnitude as compared to those due to axial forces, but can not be ignored. For example, for the diagonal of the end panel, the bending stress is approximately 30% of the axial stress.

The primary intent of this single truss analysis is to evaluate the effects of end restraint, such as the top chord connection plates. The same model of Fig. 2.9, but with a horizontal hinge support at the left end of the top chord, is examined. The influence line for the axial force and bending moment at the upper end of the end diagonal are also plotted in Figs. 2.10 and 2.11. The presence of the end restraint changed the forces and moments slightly. The corresponding stresses at the truss component member also changed slightly. On the other hand, the horizontal reaction at the left end of the top chord, representing the force in the connection plate, has a maximum influence line value of 1.153 kips. This influence line is shown as Fig. 2.12. The total force in a connection plate due to train loads on the truss could be high enough to cause concern of the plate.
2.2.2 Three Parallel Trusses

The finite element model of three parallel trusses with a concrete slab is depicted in Fig. 2.13. Some assumptions are listed below:

1. The floor beams and the concrete deck, which encases the floor beams and the web members of the interior truss, are assumed as integral components connecting the three trusses.

2. The transverse floor beams are considered rigidly connected to all three trusses at panel points.

3. Two rails between trusses are also regarded as integral parts of the span. Each rail is equivalent to a continuous beam at the level of the deck.

4. All joints are assumed rigidly connected in the model.

5. The geometrical properties of the truss members are from those of a typical eleven panel span (Table 2.1).

6. The end panel lower chords and verticals are omitted in the model.

7. Hinge supports are assumed at both ends of the trusses, to simulate the existence of the end panel connection plates. For comparison, the support condition of a roller at the left end is also studied.

The forces, stresses and displacements from the analysis of these models are not exactly those of the actual viaduct members.
Nevertheless, the comparison of results is applicable to the viaduct truss spans. Some of these results are summarized here.

1. The influence line coordinates indicate that

   the end diagonal of the interior truss (member 100 of Fig.
   2.13) is subjected to higher axial force and bending moment
   than the end diagonal of the exterior truss (member 21).

   Figure 2.14 and 2.15 compare these influence lines
   corresponding to one pair of unit loads on the two
   rails between the two trusses.

2. For the end diagonals, both at the exterior and

   the interior trusses, the point of maximum stress at the
   upper end of the member is at the bottom edge. This is
   in the general area where the cracks have been detected
   (Fig. 2.8). The axial, bending and combined stresses at
   this point are listed in Table 2.3. The computed stresses
   are higher at the exterior end diagonal because the member
   is smaller. It is important to note, again, that the
   bending stresses could constitute a significant portion
   of the total stress.

3. The edges of the top chord member at the end panels

   are subjected to stress reversal, as is revealed by the
   influence lines in Fig. 2.16 for the bottom fiber near
   the support. This condition is induced by the bending
   moments in the members as a load travels on the rail.

4. The axial forces and bending moments in members

   of other panels of the trusses do not cause stresses as
   high as those in the members of the end panel.
5. No analysis is conducted of the condition where all joints are "pin-connected". For the evaluation of stresses at local areas, "rigid" joints must be used, as it has been pointed out earlier.

6. The effects of the top chord end restraint are not very pronounced on the end diagonals. Figure 2.17 shows the axial force and bending moment influence lines for the upper end of the end diagonal (member 21) with and without horizontal restraint. The most affected member by the end restraint is the top chord at the end panel, where the axial force is reversed from compression to tension as is depicted in Fig. 2.18 for the end panel top chord of the exterior truss.

7. The influence line for the horizontal end reactions at two trusses are plotted in Fig. 2.19. For the model of this analysis in which the deck system is rigidly connected to all three trusses, the horizontal reactions are about the same at the interior and exterior support. At the other exterior truss support, the horizontal reaction is negligible.

The most important results of these analyses are that bending stresses exist in truss members because of the rigid joints at panel points, and that the horizontal restraints at truss ends cause stresses in and near the restraints at the top chord connection plates and the top chord of the end panels.
2.2.3 Estimate of Stress Fluctuations Due to Trains

The train wheels traveling on the viaduct rails cause superimposed stresses in the truss members. The original Frankford Elevated passenger cars had axle loads and total weight heavier than those of the present cars which were put into service around 1960. The axle load spacing and magnitude of the present cars are sketched in Fig. 2.20. There are usually six cars to a train.

By using this loading condition and the stress influence lines, the stress-time relationship for the bottom edge at the upper end of the exterior end diagonal (member 21) is computed and shown as Fig. 2.21. The train speed is assumed at 55 MPH, but no dynamic effect is included in the computation. The stress curve repeats its peaks and valleys till the last car of the train comes along, producing a lower stress peak. Thereafter, the live load stress is reduced to zero.

The corresponding live load stress fluctuation in the bottom edge of the top chord (member 89), near the support of the interior truss, is given in Fig. 2.22. The magnitude of the peak stresses in this member is much lower than that in the end diagonals.

Both these estimates of stress fluctuation, Figs. 2.21 and 2.22, are results of static analysis. The curves serve to give qualitative comparison such that insight can be gained of the normal pattern of stress variation and the order of magnitude of live load stresses. The dynamic effects of the trains may change moderately or drastically the pattern and the magnitude, depending on the dynamic characteristics of the trains, the railroad tracks, and the
viaduct structure. Without knowing the precise conditions of these governing factors, an accurate evaluation of the fluctuation of stresses in truss component members using an elaborate model to simulate the viaduct spans, is not possible. Measurements on stress variations confirm the drastic change of stress fluctuation pattern. This is discussed next.

2.3 End Diagonal Stresses by Measurement

2.3.1 Locations, Setup and Output

Measurements of live load stress fluctuations in viaduct truss members have been made. The southern exterior trusses at bents 490 and 491 were chosen. Span 490 (between bents 490 and 491) has eleven-panel trusses. The support conditions are as shown in Fig. 2.23.

The main members of interest are the end diagonals. Six electrical resistance strain gages were placed at three end diagonals, as indicated in Fig. 2.24. One gage each was mounted on the two end panel top chords adjacent to bent 491. All gages were on the exterior face of the southern trusses so that their installation could be carried out without interrupting the train traffic. Figures 2.25 to 2.27 are sample pictures of the end diagonals and top chords with strain gages. Table 2.4 and Fig. 2.24 summarize the gage numbers and relevant information.

The strain gages were connected to instruments in a vehicle below the viaduct. Four gages were monitored simultaneously. The output from the recording instruments are stress versus time traces. Figures 2.28 to 2.33 are selected portions of these traces.
2.3.2 Results and Discussion

As expected, the stress fluctuations in viaduct truss members are strongly affected by the operating condition of the trains. Strain measurements were taken during a period of time when train loads and frequency of train increased then decreased, providing results for the examination of their effects.

When train loads were light, with only a few riders in some of the six cars, the live load stresses were low. Figure 2.28 shows that the maximum live load stresses in gages 1 to 4 were about 2.5 ksi in the end diagonal (by gage 2). These are "dynamic" stresses generated by the moving train. The fluctuating stresses had a stress-range of approximately 1.5 ksi. It is interesting to note the resemblance of these stress-time traces to the lines of Fig. 2.21 and 2.22, which are "static" curves computed for fully loaded six-car trains on the model span.

The stress fluctuations as shown in Fig. 2.28 were induced by an outward bound train, traveling between the exterior trusses with strain gages and the center trusses. The inward bound trains on the other track did not generate large stress fluctuations in the gages. This condition is evident from Fig. 2.29, which shows stress changes in the two opposing end diagonals at bent 491 due to first an inward bound train and then an outward bound train.

That the inward bound trains do not cause significant live load stresses in the exterior trusses of the outward bound track, or vice versa, is anticipated from the results of the model analysis for
"static" loading conditions. When dynamic characteristics of the trains induce responses of the viaduct to be different from the static ones, trains in both directions may cause significant stresses in all trusses. Figure 2.30 shows recorded stress fluctuations under such a condition.

The stress-time traces in Fig. 2.30 are for the same gages of Fig. 2.29, on the end diagonals at bent 491. The data were recorded at a time when the trains were fully loaded and traveling frequently (every three minutes in both directions). The important results as indicated by these data are:

1. The magnitude of stresses in Fig. 2.30 due to outward bound trains were higher than those of Figs. 2.28 and 2.29, because of heavier train loads.

2. After the train moved away from the span, high magnitude of stress fluctuation continued to occur. The viaduct span was felt to vibrate irregularly (or non-symmetrically).

3. An inward bound train caused stress fluctuations with magnitudes higher than those due to an outward bound train, at least for the case shown in Fig. 2.30.

4. A bus traveled on a street bridge, which is directly below and shares the same piers with the viaduct span 490, caused stress excursions of high magnitude in the viaduct truss members with the gages.
The above results were obtained during the peak period of train operation. When the train load and train frequency decreased, the stress fluctuation magnitudes also decreased. The traces in Fig. 2.31 are recorded after those of Fig. 2.30. The vibration was less and the corresponding stress was smaller. The inward bound train passed the viaduct span simultaneous to an outward bound train. The stress fluctuations differ only very little from those of Fig. 2.28 and 2.29. Finally, all came back to near "static" responses, as it can be noticed by comparing Figs. 2.32 and 2.33 with the others.

That a bus caused stress fluctuations in the viaduct truss members was examined during stress recording. It was found that the columns of bents 490 and 491 share piers with the street bridge directly below the viaduct. When trucks and buses traverse the street bridge, stresses are induced in some members of the trusses through the interactions between the piers and the columns of the bents. This is considered to be a local phenomenon, not expected at bents where columns are anchored on the ground.

It must also be pointed out that, although the stress records such as those presented in Figs. 2.28 to 2.33 are actual data by measurement, these records can only serve as indications of how stresses vary at the points of measurement, not as exact stress data along the elevated line. Local geometry, alterations and repairs render every end diagonal and top chord different from others. The condition to be remembered is that the recorded stress fluctuations do agree qualitatively with computed patterns under nominal train
operating conditions. This agreement permits a rational assessment of the fatigue resistance of the viaduct trusses.

2.4 Fatigue, Brittle Fracture and Safety

There were numerous occasions of detected cracks in the viaduct trusses of the Frankford Elevated. Repairs or corrections were made immediately in all occasions. The question why these cracks develop, the concern about sudden failure of some truss components due to undetected cracks, and the safety and reliability of the viaduct spans are some of the major reasons of this study. The results and conclusions and the rationale behind them are summarized here.

2.4.1 Fatigue Cracks

At the onset of this study, it was preliminarily assessed that the cracks were fatigue in nature.

Fatigue cracks initiate from flaws or defects and propagate under repeated stresses. Flaws or defects are irregularities at the edges of truss members, at rivet holes, and at welded connections. The sizes of the initial flaw which may constitute the early stage of fatigue cracks depend on the magnitude of the repeated stresses. For bridge structures under normal loading conditions, initial flaws or defects a hundredth of an inch in size could propagate as cracks.

The repeated stresses which may cause fatigue crack growth are the stresses due to train loads, wind, and vibrations of the structure. Laboratory and field studies have shown that the higher the range of these repeated stresses, the faster the growth of the cracks.
cracks, and the less the number of stress fluctuations to cause failure. Figure 2.34 is a plot of repeated stress ranges against the number of their applications to cause "failure" of riveted joints in tension. The coordinates of the plot are in log-scale and the straight line can be considered as the lower bound of all data.

The straight line in Fig. 2.34 is the category D allowable stress range line for bridge structures with redundant load path, as specified by AASHTO. The line is one of many in the AASHTO Specifications, and is based on experimental and analytical studies. The stress ranges used for the development of this line are constant in magnitude for each data point. In actual structures such as the Frankford Elevated viaduct trusses, members are subjected to fluctuating stresses which vary in magnitude of range. Consequently, cumulative damage hypothesis must be employed in fatigue life evaluations. Miner's linear relationship and the root-mean-square procedure have been shown to correlate well fatigue damages on highway and railway bridges. To consider cumulative damage, actual stress history at points of possible cracks is needed.

For the Frankford Elevated viaduct trusses, all detected cracks including those developed in the already repaired end diagonals and truss end top chord connection plates, were repaired immediately. Once repaired, either by welding of doubler plates or by replacement of the cracked plates by new ones, the stress fluctuation of the original plate can not be measured. Of the locations at bent 490 and 491 where live load stresses were monitored by strain gages, the end diagonal at bent 490 developed cracks and has been repaired by doubler
plates welded onto the original ones. The measured stresses at the edges of a doubler plate, with gages 7 and 8, do not represent the stress fluctuations before repair. The end diagonals at bent 491 are original and without cracks. Examination of the fatigue strength of the end diagonal west of bent 491 is made below using the measured stresses from gage No. 2.

Strain gage No. 2 was placed near the point at which most likely would develop crack in the diagonal. It is the point with the highest stress by the analytical model, and the point where cracks were detected in other diagonals. From the measured stress fluctuations, the stress range histogram is constructed and is shown as Fig. 2.35. During the period of measuring stresses at this gage, thirty five trains passed by. The trains and vibrations, most frequently induced by the trains, caused 194 stress range occurrences each with a magnitude higher than 1 ksi. The magnitude of stress ranges and their corresponding frequencies of occurrence in percent are plotted as the histogram of Fig. 2.35. The average stress cycles per one train is $194 \div 35 = 5.54$.

By using this histogram, the lower bound line of Fig. 2.34, and Miner's hypothesis, the equivalent constant magnitude stress range is computed to be only 1.1 ksi. By assuming that there have been 100 round trips of trains each day, the total number of stress ranges in the 58 years of viaduct operation are

$$ N = (100 \times 2) \times 365 \times 58 \times (194 \div 35) $$

$$ = 23.5 \times 10^6 $$
A point in Fig. 2.34 with a stress range of 1.1 ksi and 23.5 million cycles, is below the lower bound straight line. No fatigue crack is expected presently at the location of gage No. 2.

To be more conservative, because the train loads were heavier prior to 1960, and more frequent, the equivalent constant magnitude stress range is proportionally raised by the ratio of 106,550 lbs. to 70,550 lbs. and 110 round trips are assumed. These give $S_r = 1.6$ ksi and $N = 25.8 \times 10^6$. The corresponding point in Fig. 2.34 is still below the lower bound fatigue strength line. No fatigue crack is expected at this end diagonal, and none has been detected.

Other strain gages used in the field measurement recorded stress ranges comparable to or lower than those of gage No. 2. Whereas gages 7 and 8 are on a doubler plate as stated before, all other gage points did not have cracks. It is, however, clearly possible that the stress ranges at some end diagonals could have equivalent constant magnitude stress ranges of 4 to 5 ksi, and a higher number of stress ranges per one train. These conditions could produce fatigue cracks many years ago, and did. These conditions will continue to cause fatigue cracks at some end diagonals.

The original end panel top chord connection plates were attached by rivets. The "static" maximum live load stresses computed from the model of Section 2.3 is about 5.5 ksi in these plates. The magnitude of stress ranges evaluated from recorded stress fluctuations and equilibrium condition at bent 491 is of the same order. Therefore, the connection plates are subjected to the stress cycles similar to
those in the end diagonals and, being of the same type of riveted structural detail, could develop fatigue cracks at about the same time as did the end diagonals. This, by and large, is the situation along the viaduct.

During the last few years, many of the replacement connection plates and end diagonal plates which were attached by welding cracked again. Welded connections of this type have a fatigue strength corresponding to the Category E details of AASHTO specifications, which are duplicated as Fig. 2.36. Category E has a fatigue strength lower than Category D. That is to say, the welded replacement plates are less durable against fatigue than the original riveted plates if the stress conditions are the same. Consequently, the welded replacement plates cracked after a relative short period of a few years.

The above procedure of assessing fatigue cracks from accurate stress histogram can also be employed to estimate the life expectancy of viaduct truss components. The difficulty is the development of accurate stress range histograms for the crucial components of the viaduct spans. Extensive field measurements at many locations to acquire long-period stress fluctuation data is beyond the scope of this study. The review of possible fatigue crack development at a few members in truss end panels indicates that crack growth, if occurs, will be slow. This will be discussed later. The important question to be answered is whether the viaduct trusses are safe against sudden failure because of brittle fracture.
2.4.2 **Brittle Fracture**

When fatigue cracks propagate to the extent that the remaining portion of a structural member can not undertake the dynamic load applied to the structure, sudden brittle fracture of the member may occur. Brittle fracture is governed by the maximum stress in the member, the size of the flaw (crack), the fracture toughness of the material, the possibility of load transfer to neighboring members of the structure (redundancy), and other less important factors.

One procedure of avoiding brittle fracture is the simple and traditional method of specifying minimum toughness of steels. The current AASHTO requirement of fracture toughness for bridge steels is tabulated in Table 2.5. This table does not include ASTM A7 steel, to which the steel of the Frankford Elevated viaduct structure is comparable in physical properties and chemical composition. By considering the concept and procedure of the establishment of the AASHTO toughness of Table 2.5, the corresponding toughness requirement for A7 steel is in the order of 15 ft-lb of energy absorption by Charpy V-notch tests at 70°F for the Philadelphia area. The toughness of the Frankford El steel by a few CVN tests is plotted in Fig. 2.37. The results are better than those available from literature, as shown in the figure. At 70°F the energy absorption is higher than 15 ft-lb. The Frankford El steel is adequately tough against sudden brittle fracture of components under anticipated conditions of viaduct operation.

A more rigorous evaluation of brittle fracture is by the procedure of fracture mechanics. The condition under which a structural member may fracture rapidly is when the stress intensity factor, K, at
a crack becomes equal to or higher than the fracture toughness, $K_c$, of the material.

The fracture toughness is affected by temperature and the rate of loading as well as by the thickness of the steel plate. Figure 2.38 is the fracture toughness versus temperature relationship of ASTM A7 steels with thickness comparable to those of Frankford El viaduct steel plates. The results are from tests under dynamic loads which occur within thousandths of a second. Actual loading rates for viaduct components are intermediate between dynamic and static conditions, in the order of half a second, with the fracture toughness curve shifted to the left of the dynamic curve in Fig. 2.38. That is to say, at a specified temperature, a steel has higher fracture toughness against intermediate rate of loading than against dynamic loading. While studies are still in progress to gather more data for A7 steel, it can be conservatively assumed that based on the dynamic rate fracture toughness of approximately 60 ksi $\sqrt{\text{in.}}$ at $0^\circ$F, the fracture toughness of 0.5 inch thick A7 steel under actual viaduct loading is higher than 120 ksi $\sqrt{\text{in.}}$ at $0^\circ$F.

When a crack or a flaw is present at the edge of an end diagonal or a top chord angle, the stress intensity factor may be estimated as $K = 1.2 \sigma \sqrt{\pi a}$, where $\sigma$ is the maximum tensile stress at the point of crack and $a$ is the crack size. For the end diagonals of the viaduct trusses, the actual maximum tensile stress (dead load, live load, and impact stresses) at the edge is not known. If a magnitude of static allowable stress, 18 ksi, is assumed with a crack size of 0.5 in. the resulting $K$ value is 27 ksi $\sqrt{\text{in.}}$. As indicated above the intermediate
loading rate value of $K_c$ at $0^\circ$ F is at least 120 ksi $\sqrt{\text{in}}$. This toughness is sufficient to sustain the stress intensity factor of 27 ksi $\sqrt{\text{in}}$ at that temperature. If local geometry, such as rivet holes or welds, causes stress concentration so that the maximum stress is at the yield point of 33 ksi, the corresponding $K$ value is 50 ksi $\sqrt{\text{in}}$. If a crack is not detected and has grown to 2 inches long, and the corresponding maximum stress is at the static allowable stress, $K$ is about 55 ksi $\sqrt{\text{in}}$. These latter cases are hypothetical extreme conditions, ignoring the influencing factors such as stress gradient due to bending moment and redistribution of forces to other members because of redundancy. Yet the stress intensity factor values are still below the material fracture toughness. Therefore, the condition that component members of the viaduct trusses suddenly fracture is very unlikely.

2.4.3 Safety

It has been discussed that for the viaduct trusses, the cracks were fatigue induced, that fatigue cracks will continue to develop and that sudden brittle fracture of truss component members is not likely. The situations remain to be examined are the behavior of trusses with cracked members and the urgency of inspection.

The behavior of a viaduct span with a cracked member can be deduced from the load-carrying capacity tests by WJE. Those tests showed that stresses and deflections remain practically proportional to the applied load beyond the design live load.

To examine this condition further, the analytical model of this study is employed. The end diagonal of an exterior truss (member
is reduced to half area and then to zero in an analysis. A 50% reduction in area implies complete cracking of one of the two plates of the end diagonal, or one crack each, halfway through both plates. The resulting stress influence lines for some selected points on the most affected truss members are presented in Figs. 2.39 and 2.40. The stress in the remaining portion of the cracked end diagonal almost doubles when 50% cracked, but the change of stresses in nearby members is relatively small, in the order of 10 - 15%. When the exterior diagonal is completely cracked, the adjacent top chord theoretically has to carry all the reaction force at the support and is subject to high bending moment, resulting in high stress at the edge as is shown by the upper portion of Fig. 2.40. The corresponding stresses in the end diagonal and end panel top chord of the center truss are about doubled (see lower part of Figs. 2.39 and 2.40). These local changes of stresses are confined to members near the crack, as it is revealed through examining stresses at other points of the trusses. The analysis shows that there is no drastic change of overall behavior of the trusses.

It must be pointed out again that the results of the computer analysis give only qualitative indication of actual behavior. The actual trusses have an end panel lower chord which is connected to the transverse girder of the supporting bent, providing additional redundant load path in the local area. Actual stresses in the end diagonal and top chords are expected to be smaller than computed. Measured stresses in the end panel diagonals and top chords from the WJE tests confirm this. Furthermore, no visible damage has been observed in members near the cut diagonal in the WJE test at design live load, nor has there been any
reported during all these years of viaduct service. The effects of cracked members, therefore, are confined to local areas.

Under repeated live load, local changes due to large cracks produce changes of stress fluctuations and changes of stress histograms for affected members nearby. If the changes in affected members are unfavorable with regard to fatigue strength, fatigue cracks may develop. To avoid this from happening, cracks need to be detected and repaired. That the detected cracks in the end panel diagonals, top chords, and connection plates were repaired immediately and no new cracks developed in nearby truss members attests to the importance of detection and repairs.

The inspection and detection should be carried out regularly. The maximum time between inspections can be estimated using the rate of crack growth as basis for evaluation. From studies of fatigue crack growth in structural steels and the theory of fracture mechanics, the rate of fatigue crack growth, \( \frac{da}{dN} \), is proportional to the stress intensity factor range, \( \Delta K \),

\[
\frac{da}{dN} = 3.60 \times 10^{-10} (\Delta K)^3
\]

where \( a \), \( N \) and \( K \) are defined before and \( \Delta K \) is calculated using stress range in the expression for \( K \). By integrating this equation from an initial flaw size to a tolerable crack length, the corresponding stress cycle number \( N \) can be computed. The length of time equivalent to this cycle number can be evaluated from the frequency of trains.

As it has been shown earlier, cracks halfway through end diagonals would not cause very large change of stresses in nearby truss members and there is unlikely to be sudden fracture of the members. Therefore, it may be assumed that two-inch long cracks can be tolerated.
The number of stress cycles needed to grow a very small crack to two-inches is about 1.3 million cycles by integration. This corresponds to about three years of time, assuming that the stress histogram condition at bent 491 is prevalent. Thus an inspection interval of three years is adequate for the viaduct truss members at bent 491.

However, stress conditions at other points of the viaduct may be more severe. While there is no crack at bent 491 in 58 years, some cracks developed at points along the viaduct in about 30 years, or about half of 58. If it is assumed that this difference in time is a measure of difference in crack growth, then the corresponding inspection interval is half of three years, that is, one and a half years. To be more conservative, a yearly inspection is recommended.

2.5 Suggestions

Although analysis revealed that end panel components of the viaduct trusses are the most highly stressed members, the actual stress fluctuations differ at these members depending on the local conditions and the dynamic characteristics of the trusses. It is not possible to alter these conditions and characteristics without a total change of the superstructure. While such a change is being considered, some minor adjustments can be made to ensure that the existing viaduct trusses are safe and reliable to the time of change. The suggested actions are listed as follows:
1. Continuation of inspecting the viaduct trusses at regular intervals, particularly the end panel component members. An annual inspection is suggested.

2. When cracks are detected, repairs should be made as soon as possible.

3. Repairs of end panel top chord connection plates are to be made by bolting of replacement plates onto the top chords. Attachment of these connection plates by welding may also be employed but it must be realized that the fatigue strength of the welded plates will be lower than that of the bolted connections, thus cracks may reoccur earlier for the welded plates.

4. The end panel top chord connection plates may be omitted if the plates are not needed functionally for the operation of the trains. Omission of cracked connection plates will affect only a little the overall behavior of the trusses and will change slightly the stress patterns in the neighboring members.

5. Repairs of cracked top chord members may be by bolting of replacement component angles or plates. If the end connection plate is omitted near a cracked top chord member and the crack in the top chord component is small, simple addition by bolting of angle or plate over the crack is sufficient.

6. Cracked end diagonal plates may be replaced by new plates with bolted connections. This procedure of repair
requires removal of the concrete which encases the end diagonal. Where corrosion of the end diagonal at the concrete deck is not serious, an addition of a partial length doubler plate bolted onto the cracked plate may be simpler in procedure.

While repairs by bolting of replacement plates increase the fatigue strength (from Category D to Category C in Fig. 2.36), the stress magnitude and fluctuation are not changed by these repairs. Without making any change to the viaduct superstructure or the passenger trains, the only item which may reduce the live load stresses is the improvement of the rail straightness. As it was observed during measurement of stresses at bents 490 and 491, the trains oscillated sideways at certain spots of the viaduct and the recorded strains showed large fluctuations. Improvement of the rail straightness is then expected to reduce the magnitude of stress range and the number of cycles per train and, in turn, reduce the likelihood of numerous and frequent fatigue cracks in the viaduct truss component members.

In view of the extent of corrosion of the steel members in the viaduct superstructure, the condition that cracks will continue to occur at highly stressed viaduct truss components necessitating more repairs, and of the situation that many of the repairs may need to be repaired again, an overall replacement to the viaduct spans is suggested.
3. BRIDGE TRUSSES

3.1 The Bridges

There are three major bridges in the Frankford Elevated Line: the pony truss bridge over the AMTRAK (Penn Railroad) Line north of Tioga Street, the twin through-bridge (Richmond Bridge) over the CONRAIL (Reading) Line north of Lehigh Avenue, and the truss arch bridge over Lehigh Avenue.

The bridge north of Tioga Street has a skewed span of 143 ft. The lower chords of the Baltimore trusses and the floor beam system are encased in the concrete deck which supports the inward and outward bound tracks. Figure 3.1 is a photograph showing one of the trusses, viewed from the tracks. Figure 3.2 is a half elevation of a truss giving the panel dimensions. The trusses are supported at the end subpanels by steel truss towers. Bents 408 and 409 form the tower at the west end, 410-411 the east. From Figs. 3.1 and 3.3, it can be seen that the verticals at primary panels have buttress type bracing, and the truss members are relatively stocky.

North of Lehigh Avenue the tracks are separated. The inward bound and outward bound track each passes through one of the twin bridges which are interconnected by lateral bracing. Figure 3.4 depicts this condition. Figure 3.5 shows the half elevation of the Pennsylvania trusses with inclined top chords. The lower chords and the floor beam systems are enclosed in the concrete deck which...
extends to outside of the trusses to form a walkway. The span length is 196 ft. with Bents 273 and 274 serving as supports at ends.

The bridge over Lehigh Avenue has truss arches. The arch ribs and the vertical spandrels are all in compression. Since the objective of this study is to evaluate fatigue strength and safety with regard to fatigue and fracture, attention has been directed to the other two bridges instead of this one.

No fatigue crack has been detected in any of the components in these three bridges. The problem appeared to be corrosion rather than fatigue. Where corrosion had caused concern, repairs were made to restore or increase the area of the component members. Some repaired locations can be seen in Fig. 3.3. Repairs usually consist of removal of concrete, attachment of reinforcing plates or angles by welding, injection of bonding grout, casting of new concrete, and painting.

The expansion joint at the west end of the bridge over the AMTRAK Line has been repaired recently to relieve the fixity caused by the relative movement between the bridge and the support. The knee bracing members in the through-truss bridges north of Lehigh Avenue were cut and elevated around 1960 to accommodate the new passenger cars. Besides these repairs and modifications, there were no other changes made to the bridges.

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3.2 Live Load Stresses

3.2.1 Bridge over AMTRAK Line (North of Tioga Street)

To evaluate the fatigue strength of the bridge components, live load stresses need to be examined. The computed design live load stresses for a few members are listed in Table 3.1. These are live load (including impact) stresses based on the original passenger car loading, and are higher than those computed for the viaduct truss components (see Figs. 2.21 and 2.22). Instead of further computation of stresses by the finite element procedure and considering the rigidity of the panel points, actual measurements were made.

The members in Table 3.1 are the highest stressed components of this bridge by computation. Strain gages were therefore placed on these members. Figure 3.6 identifies them by numbers corresponding to those in the table. The gages and setup were all similar to those reported in Chapter 2 on the viaduct trusses. The recording instrument utilized a magnetic disk. The results of measurements were plotted out as strain-time records. Figure 3.7 shows two traces each of such records for two members of the truss. The fluctuation of stresses due to the passing of a six-car train has the same general pattern as for the viaduct truss members (see Fig. 2.21). The maximum stress range is about 1 ksi in this figure. The maximum measured stress ranges of all the members with strain gages were less than 1.5 ksi and are listed in Table 3.1. The measured stress ranges are low when compared to the design values, as it is indicated by the low percentage in the last column.
The reasons that the measured stresses are much lower than the original design stresses include the reduction of train weight when new cars were adopted around 1960, the increase of bridge rigidity by the concrete encasement of the lower chords of the trusses and the floor beams, and the conservativeness of design requirements which apply to all bridges of the era. Also, the trains traveled over the bridge very slowly during the time of strain measurements because of repairing of the expansion joint at the bridge end. Additional measurements were intended using the same strain gages. Unfortunately all these gages were damaged by uninvited pedestrians while the expansion joint repair was being completed.

A second set of strain gages were attached to the bridge, on the eastbound truss. The approximate locations and numbering of gages (11 to 18) are indicated in the lower truss of Fig. 3.6. The same instrument and setup as used for the viaduct trusses were employed. The outcome of the recording were ultraviolet light traces of strain fluctuations. Figures 3.8 and 3.9 are representative pattern and magnitudes of the stress variations at the gages. The measurements were made at peak traffic hours with full passenger loads and normal train speed. The maximum stress range recorded was less than 2 ksi.

Because the bridge is of pony-truss type with two trusses, it is anticipated that both eastbound and westbound trains will induce stresses in both trusses. Figure 3.10 shows the recorded stress changes at four gages when an outward bound train and an inward bound train traveled over the bridge successively. By comparing the duration of the stress fluctuation of the traces with those of Figs. 3.8 and 3.9, which
have the same time and stress scales as for Fig. 3.10, it is evident that the inward (west) bound trains did cause live load stresses in the outward (east) bound truss. Records of stress fluctuations due to westbound trains alone shows the same pattern as eastbound trains in Figs. 3.8 and 3.9, but with lower magnitudes. In fact, when trains from both directions arrived at the bridge simultaneously, the magnitude of live load stresses increased only slightly from that due to the eastbound train alone. The maximum stress ranges recorded were 2.5 and 2 ksi, respectively. Figure 3.11 shows hand traces of ultraviolet light records of four gages when two opposite trains were both on the bridge. At the time of measurement, the eastbound trains were occupied by riders while the westbound trains had only a nominal amount of passengers. This is the normal operating condition for the commuter line. Therefore, the recorded live load stresses are most likely the maximum values.

3.2.2 Bridges over CONRAIL Line (North of Lehigh Avenue)

This bridge was designed by the same company which did the bridge north of Tioga Street, using the same code, identical loading conditions and material. There are no design stresses or computed member forces available for reference. Simple analysis gave live load stresses in members with magnitudes comparable to those computed for the other bridge over the AMTRAK Line. Again, direct measurement of live load stresses were made, employing the same techniques as described before. The south truss of the eastbound bridge was chosen for measurement.
The approximate location of the strain gages on the truss members are marked by numbers in Fig. 3.12. Gage 21 was on a flange of the wide flange shape U16-L16; gage 22 was on the web surface of the built-up member M17-L16; gages 23 and 24 were each on a 4" x 7/16" plate of the sub-vertical M17-L17; etc., etc.

The results of measurements indicate that westbound trains cause little stress in the members of the eastbound bridge, as no strain variation was recorded. The maximum live load stress at peak hour with full passenger load was about 2 ksi. Figure 3.13 and 3.14 are traces of typical records from the eight gages.

Gage 23, which was on a subpanel vertical plate, showed relatively high frequency vibration with correspondingly higher stress ranges. This member, however, was slightly bent and its vibration could be physically felt and seen. This minor condition can be corrected easily. Gage 24, which was on the twin plate of gage 23, showed the normal stress fluctuation pattern due to train load. So did gage 28, which was on a similar subpanel vertical plate. As it has been indicated above, the highest stress range recorded in any member was about 2 ksi.

3.3 Fatigue Strength

Because there are welded repairs and changes with welded attachments in the two bridges, fatigue strength as defined by Category E allowable stress ranges of AASHTO are to be used for the members with welds. For all other members with riveted joints, fatigue strength corresponding to AASHTO Category D is a conservative
estimate. The evaluation of fatigue strength of the members requires the examination of stress ranges at these members.

For the bridges, the maximum measured live load stress was 2.5 ksi under present peak loading conditions. By assuming that the present peak passenger load is 13,000 lbs. average per car at the time of stress measurement, and the cars weigh 53,500 lbs. each, the total weight is 66,500 lbs. Therefore, the maximum live load stress corresponding to the maximum possible passenger live load of 70,550 lbs. per car would be (2.5 ksi) x 70,550 ÷ 66,500 = 2.7 ksi.

Prior to 1960 the old cars were heavier. The design live load was 106,550 lbs. Based on the measured stresses, the maximum live load stresses due to fully loaded trains at that time would be (2.5 ksi) x 106,550 ÷ 66,500 = 4.0 ksi.

The possible highest axial loads are due to the T-1 and T-6 cranes used on this line. The cranes weigh 128 kips with a maximum axle load of 18.8 kips. The maximum axle load of the present passenger trains are 70.6 ÷ 4 = 17.7 kips. Thus, the estimated maximum live load stress range due to the crane loads is (2.7 ksi) x 18.8 ÷ 17.7 = 2.9 ksi. This is less than that due to the fully loaded trains prior to 1960.

For all the original members without welding, the Category D permissible stress range for long life is 7 ksi. For all members with welded attachments or welded repairs, the Category E stress range for high number of stress cycles is 5 ksi. Both of these are higher than the present day and previous maximum live load stresses as well as
the stress ranges induced by the cranes. Therefore, the bridge members
would not be expected to have problems with regard to fatigue.

To evaluate this further, the fracture mechanics procedure of
fatigue crack growth threshold is employed. When the live load stress
intensity factor has a range of \( K \) less than that of the threshold value,
cracks will not propagate. The threshold value depends on the maximum
stress as well as the stress range in a structural member, and may be
conservatively assumed to be \( 3 \text{ ksi} \sqrt{\text{in.}} \) for the bridge components. The
stress intensity factor range for a plate with an edge crack may be
computed as \( \Delta K = 1.2 s_{r} \sqrt{\pi a} \). By using the highest stress range of
4 ksi which is for the fully loaded trains with the original heavy
cars, and assuming that there could be crack-like initial flaws 0.1
inch long, the corresponding stress intensity factor range is \( \Delta K = 1.2
(4 \text{ ksi}) \sqrt{\pi (0.1 \text{ in.})} = 2.7 \text{ ksi} \sqrt{\text{in.}} \). This is below the threshold value.
Therefore, no crack growth would be expected in the bridge members.

Without growth, small initial flaws do not develop into
cracks. Consequently, there would be little concern with regard to
fatigue crack induced conditions of brittle fracture of the bridge
members. Since the bridges and viaduct trusses are constructed of the
same material, which is sufficiently tough, the bridges are safe against
brittle fracture.

3.4 Discussion

As it has been indicated earlier, the problem of the bridges
appeared to be corrosion rather than fatigue, while it has been shown
that fatigue crack propagation is not likely, a brief discussion on
the possible effects of corrosion on the strength of the bridges is necessary.

The most important effects of corrosion on a bridge is the reduction of member cross-sectional area and member strength. In this regard, a loss of ten percent in area of a truss member decreases the load carrying capacity of the member about ten percent. However, the strength of the total truss may have only changed very little, depending on the function of the corroded member in the truss. The underlying factor is that the truss joints are rigid instead of pin-connected, thus there are redundant paths for force transmittal. This is borne out by the results of analysis of the viaduct truss (in Section 2.4.3) when 50% of the area of an end diagonal is assumed lost.

If corrosion takes place in many members of a truss, a condition which actually occurs in the bridges at the locations of concrete deck encasement, the reduction of bridge strength must be evaluated considering the reduction of cross-sectional area of all corroded component members. Generally, the extent of corrosion of the members cannot be determined unless parts of the concrete deck are removed. Exposed truss lower chord and interior of concrete deck at the repair of expansion joint of the bridge over the AMTRAK Line showed that the concrete was clean and the enclosed steel members were corrosion free. Results of inspection by SEPTA crew indicated that the condition of corrosion in the bridge member was less than that in the viaduct trusses, in the order of a few percent loss of area at the line of encasement. Repairs have been made to these corroded areas.
It is important to realize that repairs by welding additional plates to restore or increase the cross-sectional area of bridge component members could actually reduce the fatigue strength. The loss of a few percent in sectional areas of a number of components may reduce the static strength of the bridge by a few percent, but welding changes the fatigue strength from that of AASHTO Category D to E, a reduction by a higher percentage. Repair by appropriate bolted connections would avoid this condition. Judgment must be made to weigh the importance of repairs and their consequences.

The effects of corrosion on fatigue crack propagation are important for some structures. The basic governing factors are identical to those of fatigue under benign environment with the added influence of the corroding agent. Studies have shown that for bridge steels under ordinary environment of bridge operation the effects of corrosion on fatigue are minimal. Since the bridge members have been found safe against fatigue crack growth under ordinary operational conditions, no influence of corrosion on fatigue strength would be expected.

Visual inspection of the bridges indicated that, overall, the bridge superstructures are in fairly good condition. With adequate routine inspection and maintenance in the future, the bridges should be able to carry the present pattern of train traffic for many, many more years.
4. BENTS

4.1 General Conditions

There were 641 bents in the Frankford Elevated Line. Of these, 106 have been rendered obsolete due to realignment of a portion of the line with Interstate Highway I-95, 82 are of single column type, the remaining are two-column rigid frames. Figure 2.3 shows the general configuration of the viaduct at a typical two-column bent. The column height changes throughout the line in accordance with the building and street conditions below. This study briefly reviews the general conditions of existing bents using the two-column rigid frames as examples.

The twin columns are built-up steel members with concrete fills, forming a rectangular cross-sectional shape. The strong direction of the columns are in the plane of the bent. An example is shown in Fig. 4.1. The transverse girders of the bents are I-shaped plate girders with multiple flange plates.

The primary function of the bents is to support the longitudinal trusses of the viaduct. The shortened end verticals of the trusses rest on the top flange of the girder, whereas the end panel bottom chords of the trusses are connected to vertical stiffeners of the girder near its bottom flange (see Fig. 2.6). At every three or four spans of trusses, there is an expansion joint. Spans 490 to 492 in Fig. 2.23
is an example. At the fixed supports, the concrete deck is in contact with the web of the transverse girder. At an expansion joint, the deck and the bottom chords of the trusses can slide on supports. These latter features are modifications from the original design.

Reports of inspections have indicated the following conditions:

1. Frozen and corroded expansion joints were prevalent.
2. There were occasional small cracks at the top flange of the transverse girders of two column bents at the rails.
3. The columns of the bents deflected in the direction of the viaduct, that is, perpendicular to the plane of the columns and the girder. The deflections did not show any pattern, but was rather "inconsistent".

The small cracks at the top flanges have all been repaired. These cracks occurred at the compression flange of the transverse girders and were mostly developed from local stresses under the rails. Repairing of the cracks and adding of shims should prevent reoccurrence of such cracks. A few of the frozen expansion joints have been relieved, but the majority remain corroded and immobile. No correction has been made to the deflected columns of the bents.

In the plans of rehabilitation of the Elevated Line, it is considered that the bents may be retained. Thus, the strength
and behavior of these bents need to be reviewed. Two-column bents are examined below.

4.2 In-Plane Behavior

The bents were designed to carry the dead weight of the viaduct and the train loads. There has been no report of unusual behavior of the bents in the plane of the columns and girder during the past years of viaduct usage. The most often cited need of correction was minor corrosion.

The test results from Wiss, Janney, Elstner and Associates (WJE) indicated that:

1. A two-column bent responded in a linear-elastic manner of behavior when it was subjected to simulated static train loads several times the magnitude of actual train weight.

2. Under equivalent train load, the maximum live load stresses in the girder was less than 3 ksi, and in the columns less than 2 ksi.

3. The vertical deflections of the girders recovered after removal of live loads.

All these indicate that the bents function properly in their plane. If the viaduct trusses and the deck are to be replaced in the rehabilitation, dead loads applied to the bents will be changed somewhat. The resulting changes of stresses and deflections in the plane of the bents are not expected to be large. The live loads (train loads), on the other hand, will remain the same. Therefore,
the overall stresses and behavior in the plane of the bents will be about the same before and after rehabilitation. The adequacy of the bents for rehabilitation is therefore expected with respect to in-plane behavior.

4.3 Transverse Girder Cracks

Those reported and repaired small cracks at transverse girders occurred in a few places, for example, at bents 556, 557, 564, 599, etc. All these cracks were located at the top plate of the top (compression) flange at the rails. During the time of field measurement, however, none were detected and actual measurement of stresses at a crack was not made.

Since the top and bottom flanges of the transverse steel plate girders were subjected to approximately the same magnitudes of primary bending stresses, and the cracks were at the top plate of the compression flange, the cause of these cracks was not the primary bending stresses in the girders. Judging from the condition that the cracks were at the rails, it is believed that local stresses due to passing wheels of the trains induced these cracks.

When the rails were directly or indirectly resting on the top plate of the compression flange, every passage of a train wheel created contact stresses at the top plate. The result was fretting and fatigue cracks could initiate thereon. Such phenomenon has been observed in many structures and test specimens which were subjected to repeated loads. The fatigue strength, that is, the stress versus cycle relationship for this mode of fatigue crack growth has not been
established because the stresses at the contact surface have not been well defined. The principal remedy or correction is to avoid fretting. Repairs by shimming between components at the cracks of the transverse girders is a step in the right direction.

Because the top flange of the transverse girders are made of multilayers of thin plates, fatigue cracks in the top plate cannot propagate across the gap between the plates and into the plate below. Therefore, these cracks at the top plates of the flanges could be left alone without repair. Even when a crack is across the entire top plate, the condition would be similar to that of two butting plates in a multilayered compression flange. The strength of the member is not affected.

Of importance is the fatigue strength of the tension flange of the transverse girders. From the results of testing by WJE, the maximum static live load stresses was less than 3 ksi. The physical arrangement of the viaduct trusses and the bents is such that the impact stresses induced by the vertical loads of the trains are not very high. The maximum live load stresses, including dynamic effects, would be less than 5 ksi at the tension flange of the transverse girders. The high cycle fatigue strength "threshold" of riveted joints in tension, as it is conservatively represented by the Category D allowable stress range (Figs. 2.34 and 2.36), is 7 ksi. With the maximum stress range lower than the "threshold" strength, no fatigue crack would be expected to develop in the bottom flange of the transverse girders.
Care must be taken to re-examine the fatigue strength of the transverse girder when changes are made to the connections between the viaduct superstructure and the bent girders. Further discussion on this point will be made later in the report.

4.4 Discussion on Out-of-Plane Deflections and Stresses

Results of inspections have revealed that the twin columns of the bents deflect without any specific pattern. The deflections are in the direction of the viaduct, thus are out-of-plane deflections of the bents. The magnitudes of deflection are in the order of one or two inches, and are stationary. This situation implies that there are either permanent deflections, or stationary forces acting on the bents perpendicular to their plane, keeping the columns deflected.

Hypothetically, the viaduct trusses are simply supported with hinge and roller bearings. No longitudinal force would be generated if the permissible roller movement and permissible expansion joint movement are larger than the support displacement of the trusses. Actual situation of truss supports differ from the hypothetical condition: bearing plates and rockers are used, the end panel lower chords of the trusses are attached to the transverse girders of the bents, the top chords of two consecutive trusses are joined by a connection plate, and the corresponding end verticals are attached by steel links (see Figs. 2.4 and 2.5). Furthermore, the concrete deck bears on the web of the transverse girders. These features and the frozen expansion joints and rockers render the truss ends "restrained".
probably for many years. Any horizontal movement of the truss ends would cause the bent columns to deflect.

Because of the difference in extent of corrosion and extent of freeze of the truss supports at different locations, as well as the uncertainty of the dynamic characteristics of the trains, complicated dynamic analysis of a bent with adjacent spans is not warranted. In order to examine the possibility of permanent deflections and stationary forces at columns, simplified analytical models are employed.

A simple model assumes that both supports of a truss span are "hinged", representing an upperbound condition for truss end horizontal reactions. The horizontal forces at the truss supports cause deflection of the columns without rotational restraint from the trusses. The second model is a two-dimensional three-bay rigid frame, simulating three viaduct spans with rotational restraint to the columns.

For both models, the possible causes of horizontal forces at the column top include the regular traversing of the trains, acceleration or deceleration of these trains and temperature changes. The highest force probably would be from sudden (emergency) braking of a fully loaded train. These forces induce deflections and stresses.

4.4.1 Estimate of Deflections

The maximum deceleration during an emergency braking is assumed to be about 4 miles per hour per second. For a single car of 106,550 lbs. the longitudinal force generated is $F = ma$ (106,550 lbs. $\div$ 32.2 ft/sec$^2$) x 4 mph/sec = 19.4 kips. Parts of this force are
absorbed by the ties, the ballast, the deck and the components and joints of the truss spans. AREA specified that 15% of the train weight be applied as longitudinal force to the superstructure. For a car weight of 106,550 lbs., it amounts to 16 kips. The horizontal forces which are transmitted to the truss supports and bent columns are lower in magnitude.

If it is assumed that (1) the car wheels do not slide on rails, (2) all braking forces are transmitted through the floor system to the top chords of the trusses, (3) the top chords alone transfer the forces to the "hinged" supports of the trusses, the longitudinal reactions at the supports can be estimated using influence lines. The influence line for the horizontal reactions due to horizontal forces at top chords of a typical eleven panel truss is shown in Fig. 4.2. For a train with fully loaded cars of 106,550 lbs. each and braking at 4 mph/sec, the resulting static longitudinal reactions at the supports on a bent are 8.4 kips and 5.6 kips respectively from two adjacent spans. The total horizontal force at the bent is the sum of these, as is depicted in Fig. 4.3, and is 14 kips.

This magnitude is the estimate of horizontal force at the bent due to a train. The analytical model in Chapter 2 shows that trains on one track generate little reaction on the supports of the third (outside) truss. By assuming that the horizontal force is equally distributed to the two trusses of a track, the out-of-plane deflection at the top of a column is estimated by using the simple model of the bent. A sketch of the model and the results of computation in the form of force-deflection relationship are shown in
Fig. 4.4. For a 14 kip total force, \( P \) is 7 kips and the corresponding deflection would be about 1.6 inches.

When the same computed braking forces of \( P = 7 \) kips are applied at the top of the columns of the three-bay rigid frame model, a more realistic estimate of deflections may result. The model is sketched in Fig. 4.5 with the estimated load-deflection relationship. The stiffness of each column in the model is that of a single column of the two column bent, and the rigidity of the horizontal beams of the rigid frame is converted from that of a truss. The estimate deflections in Fig. 4.5 are smaller than those of Fig. 4.4. For the computed braking force of 14 kips, the column top deflection is estimated to be 0.6 inch.

In comparing the above estimated deflections due to emergency braking, it must be noted that the horizontal support forces are over-estimated (due to the assumption of no sliding of wheels, etc.), the condition of no rotational restraint at column top leads to over-estimate of deflection, and that full rotational restraint at column top is not developed. The combined effect is such that the three-bay model probably provides a better estimate of deflections. It must also be pointed out that the computations have been made assuming that all expansion joints and rockers at truss supports are not functioning. This will be discussed further.

While the horizontal (longitudinal) braking forces induce out-of-plane deflections to columns of the bents, the gravity loads of the trains produce vertical deflections to the trusses and
transverse girders and also out-of-plane deflections to the columns. However, the maximum out-of-plane deflections due to braking forces and gravity loads do not occur at the same instance. From analysis, it is known that the deflection due to unsymmetrical distribution of gravity loads are less than that due to braking forces. Also, for the position of trains that maximum out-of-plane deflection is computed, the gravity load is symmetrical with respect to the spans, thus produce no deflection at the column tops.

Temperature changes also cause deflections. For an assumed condition of continuous symmetrical spans and identical temperature patterns at the hinged or rigid-frame spans, the top of the columns would remain in position.

From the above results of computation and discussions, it can be reasonably expected that the maximum out-of-plane deflections of the columns would be between the computed 0.6 and 1.6 inches, in the order of one inch. This magnitude is in the order of the stationary deflections of the columns. However, the computed maximum deflections are transient, corresponding to a fully loaded train of 106,550 lbs. stopping under emergency conditions. The transient deflections recover totally if no inelastic behavior of the bents has been induced. To examine this latter condition, estimate of column stresses need to be made.

4.4.2 Estimate of Column Stresses

The primary stresses in the twin-columns are compressive in nature. Loads which contribute to column stresses are dead weights,
live loads, out-of-plane forces from the trains to the bents, and temperature changes.

Dead weight stresses in the bent columns are calculated to be about 4 ksi. Live load stresses from actual static tests by WJE were in the order of 2 ksi. Temperature changes do not generate out-of-plane bending stresses in the columns of hypothetically continuous symmetrical viaduct bays. For a two-dimensional three-bay rigid frame model subjected to a 60°F change in temperature at the truss top chords and 20°F change at the deck level, the maximum column bending stresses are computed to be less than 4 ksi. These thermostresses correspond to the condition that the three span rigid frame is not affected by neighboring spans.

The highest column bending stresses would be from the emergency braking of trains. The maximum magnitude estimated from the rigid frame model (of Fig. 4.5) under the computed braking forces (in Fig. 4.3) is 13 ksi. Because the emergency braking condition is dynamic in nature, a dynamic magnification factor must be applied to the computed static stresses. A high factor of 1.5 is assumed from considering the structural configuration of the viaduct spans and the nature of the braking forces. The resulting maximum stress due to sudden braking is around 20 ksi.

The estimated maximum total stress in a twin-column is the sum of all the above: 4 + 2 x 1.5 + 20 = 27 ksi. The dynamic magnification factor 1.5 is also applied to the static live load stress. The effects of temperature change is assumed negligible since the
expansion joints are mostly frozen, making the spans continuous. Where the joints were functioning, the emergency braking force would be reduced and compensates for the temperature effects.

The maximum stress of 27 ksi is within the elastic range of the property of the steel in the viaduct. Therefore, no permanent deflection of the twin-columns would have taken place as the result of emergency braking of fully loaded trains. Furthermore, the maximum load of the current trains is about three-quarters of that for the original trains. The estimate maximum total stress in the columns is about 22 ksi instead of 27 ksi. No permanent deflection of the twin-columns would be expected.

4.4.3 Residual Deflections

If the maximum stresses in the columns did not induce inelastic behavior of the bents but the columns remain deflected when there is no live load on the viaduct near the bents, these deflections must be the response to some stationary forces acting perpendicular to the bents.

Whereas deflections of the columns can be detected, the corresponding stationary forces can not be measured without removal of the restraints to the columns. No attempt was made in this study to do so. The "inconsistency" in, or the lack of pattern of deflections was recognized and the intent has been to explore how and why such deflections occurred.

Based on the results of visual inspections of some truss supports and expansion joints and judged by the prevailing corroded
conditions at these places, it is postulated that the stationary
deflections were "residuals" due to incomplete recovery of the elastic
deflections. The residual deflections were cumulative as the supports
and expansion joints progressively became immobile, and remained
stationary when the supports and joints became frozen by corrosion.

The probable conditions and reasons for the development of
residual deflections are as follows:

1. The structural arrangements at truss ends made
the viaduct spans continuous between expansion joints.
Even at the expansion joints, the top chords of the
adjacent trusses were connected by connection plates.
Without the connection plates, the estimated maximum
horizontal deflections at the expansion joints of a
three-bay model were about 0.6 and 0.3 in., respectively,
for emergency braking and for differential temperature
change in the viaduct trusses. Gravity loads produced some
but negligible amount of deflection. If the thermal and
emergency braking deflections were in the same direction,
the magnitude of maximum deflection would be
\[0.3 + 0.6 = 0.9\] in., less than the expansion joint capacity of one inch.
If the expansion joints were free to move, and the viaduct
structure behaved elastically, no residual deflection
would have developed.

2. The placement of connection plates at the truss top
chords over expansion joints created partial restraint to
the joints. The viaduct spans became continuous with stress
and deflection magnitudes different from those of the free expanding condition. In general, deflections at the expansion joints would be reduced. As long as the viaduct still behaved elastically, no residual deflection would take place. Meanwhile, the relatively higher stresses generated at the connection plates and nearby truss members could cause fatigue cracks, as it has been discussed in Chapter 2.

3. The same condition, but to a less degree, occurred at the non-expansion truss-end rocker supports.

4. When the movement of the expansion joints and rocker supports were hindered but not completely prevented by debris, structural alterations, corrosion, or other reasons, the deflections could take place in one direction, but recover less in the return move, or vice versa. This was particularly possible when the deflections were due to dynamic forces which developed and receded with different rates. (Horizontal forces due to braking and acceleration near train stations and at grades do have this dynamic nature). Also, thermal displacements due to seasonal changes of structural temperature could be accompanied by different frictional coefficients at sliding supports during hot and cold seasons. These conditions caused residual displacements. Repeated occurrence then generated cumulative deflections of the columns. These deflections remained even though there were no
trains on the viaduct. In fact, this phenomenon is most likely what happened to the expansion joint of the truss bridge over the AMTRAK Line.

5. As corrosion became more severe, the character of the expansion joints gradually change and influence the magnitudes of residual deflection. When the joints were corroded "frozen", further movement became impossible and the deflections became stationary. The characters of the structure supports changed from the intended, and stress patterns and magnitudes also changed in the affected structural components.

This development of residual displacement or deflection appears to have occurred at many old bridges, although the primary cause may differ from one case to the other. For the Frankford Elevated Line, no accurate comparison of the contributing causes has been, or can be, made because of the highly redundant nature of the viaduct superstructure arrangement. It is, however, strongly believed that the condition of corrosion of the expansion joints have contributed to a large extent to these out-of-plane deflections of the twin-columns.

By comparing the column deflections estimated under various conditions and the reported deflections of columns at bents with corroded expansion joints and supports, it can be concluded that most of the column deflections are within the elastic range of behavior. If the frozen joints and supports are relieved, most of the deflections would be eliminated.

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4.5 Discussions and Suggestions

From the above evaluation, which are based on estimated stresses and deflections and some results of on-site measurements, the following summary and conclusions can be drawn:

1. The in-plane strength of the two-column bents appears to be adequate.

2. The cracks in the top plate of the compression flanges of some transverse girders were most likely induced by fretting action. The cracks can not grow down to the next layer of plate and should not affect the strength of the transverse girders.

3. Were the expansion joints and truss supports functional, the out-of-plane deflections of the bent columns would be elastic and would not cause permanent deflections.

4. The physical arrangement of structural details at the truss ends restrained the expansion joint and support movement, and corrosion finally made the expansion joints and supports immobile.

5. With the immobile or frozen expansion joints and supports, the viaduct trusses became continuous over the bents.

6. The stationary out-of-plane deflection of the columns appears to be the result of cumulative "residual" deflections due to acceleration and deceleration of trains and thermal expansion and contraction.
7. If the frozen joints and supports are freed, most of the stationary deflections would be eliminated and the columns would move back to or nearly to the undeflected position.

8. The single-column bents were not analyzed in this study. Their behavior and safety, however, were expected to be equivalent to those of the double-column bents.

In accordance with this summary and conclusion, and with the assumption that the bents are retained in the renovation schemes, some suggestions are made.

1. Confirmation of the condition that the stationary deflections of the columns will recover if a bent is freed of longitudinal restraints. This can be achieved through the examination of column profiles, if ever taken, after the demolition of the spans in the relocated portion of the viaduct.

2. Relieving of longitudinal restraints at bents during renovation of the viaduct. This would permit the bents to recover from most of their out-of-plane deflections. However, this suggested correction may be difficult to achieve if the adjacent spans of a bent can not be completely removed because at least one track of the rapid transit line operation must be maintained at all times. In that case, the existing deflections of the columns may be left alone and care must be taken to ensure desired support conditions for the viaduct spans.
3. Realistic evaluation of structural behavior and stresses at the bents. Whether the renovation scheme incorporates the existing deck system or plans to replace the trusses and deck with new arrangements, a realistic analysis of the out-of-plane behavior of the bents and adjacent spans and supports must be made. This not only will provide better understanding of the bent behavior, but also will give information for the evaluation of stresses in the primary members of the viaduct spans.

4. Protection from corrosion of the supports at bents. The existing condition of corroded and frozen expansion joints and supports needs to be corrected when rehabilitation work takes place. This maintenance is as important as, if not more than, the protection of the structural members from corrosion. Frozen expansion joints could induce change of behavior of the viaduct superstructure much more than a single corroded member would, not to mention the influence on the bent behavior.
5. OTHER CONSIDERATIONS

Besides the examinations and results reported in the previous chapters, a brief review was made of some proposed structural details of a renovation scheme. Also, it has been mentioned earlier in this report that sideways oscillation of trains associated with track rail out-of-straightness appeared to accompany large fluctuations of strain in bridge component. Brief discussions on these items are given here.

5.1 Deck Girders

In one of the proposed renovation schemes under consideration, steel plate girders will be placed longitudinally below the existing concrete deck and will be connected to the transverse girders of the bents. The existing viaduct trusses will be cut and removed below the deck and the truss end posts will also be removed. The result will be a deck-girder type of arrangement with the deck supporting the left-over portions of the longitudinal trusses, on which remain the walkways and the support yokes for the energized rail.

Although it is only a renovation scheme, general details of arrangement and connections are given. Examination of these proposed details indicates that caution need to be taken in the final design
of the connections. A number of items are briefly mentioned below as examples.

1. Fixed-end connections between the proposed longitudinal deck girders and the transverse bent girder. The single bay or multiple bays consisting of bents and deck girders must be analyzed according to renovation scheme so as to gain better estimates of stresses at the connections. If expansion ends of deck girders are provided every three or four spans, the girders and bents in between will behave as rigid frames. Consequently, the connections may be subject to stresses higher than those computed from single spans. Live load stresses at the connections due to trains need to be examined against fatigue strength of the final details of the connections.

2. Some expansion ends of the proposed deck girders have abrupt change of depth at the connections. Local bending moment at connection plates could generate high local stresses. Fatigue cracks have often been detected in bridges at such details.

3. Placing of deck girders below the existing concrete deck necessitates bearing support between the lower flanges of the floor beams and the top flanges of the deck girders. The proposed "shim tight and weld" procedure must be preceded by evaluation of the possible movement at these points and the stresses in the floor.
beams. Fretting is to be avoided and the reduced fatigue strength of the floor beams (because of welding) must be sufficient to carry the live load stresses induced in them.

4. Hanger plates at expansion ends of some longitudinal beams at single column bents. The stresses at the pin holes of these plates need to be considered, so as to avoid fatigue crack growth. The toughness of the material must be assured such that failure by brittle fracture will not occur.

These are but some examples of details which need to be examined carefully during the process of detailing. The primary intent of cautioning here is to indicate the importance of knowing the strength of the details so as to avoid future problems.

5.2 Track Conditions

The straightness of track rails and smoothness of rails at curves and at change of grades are essential to the steadiness of trains. The trains at Frankford Elevated Line were observed to swing sideways at a number of places along the line. This motion may be described as a rotation of individual cars with respect to their vertical axes, and appears to be associated with out-of-straightness of the rails.

A sideways swing of train cars generate horizontal forces transverse to the tracks. If the rails are securely fastened, these
forces are transmitted through the intermediate elements such as ties, ballast, deck, and the truss supports to the bents. How much are the forces at the bents depends on the condition of the elements in the path of transmission as well as the horizontal force of the train cars. It is very difficult, if not impossible, to estimate accurately the forces and stresses induced at various parts of the superstructure and the bents.

Qualitatively, however, it is known that the deck with the encased trusses has a large mass and that the lateral stiffness of the truss-end supports are relatively low. The lateral motion of the deck system as induced by the sideways movement of the train cars could generate high forces perpendicular to the trusses at the supports. The consequence is that relative deflections or rotations could be developed at the supports. These displacements cause horizontal in-plane bending of the top-chord connection plates and out-of-plane bending of the end panel diagonal plates. That this condition could be the cause of high stresses has been pointed out in Chapter 2 when discussing the live load stress fluctuations in the end panel diagonals.

For the typical bents in this elevated line, any horizontal transverse force \( H \) (in kips) which is transmitted from the deck-and-truss system to the top of the twin-columns is estimated to generate maximum bending stresses in the columns about \( 0.3 \) \( H \) (in ksi). The magnitude of \( H \) is difficult to estimate. AREA specified static lateral force for the design of lateral bracing as \( 1/4 \) of the maximum axle load, or about 28 kips considering the old cars. From a simple
dynamic analysis, assuming that the dynamic action of the sideways motion of the train is an impulse with a force as serious as that of the emergency braking, the dynamic lateral force is of about the same magnitude. The estimated column bending stress would be around 8 ksi. Adding to this the dead load and live load stresses, the maximum column stress would be in the order of 15 ksi, well below the elastic limit of the column material. No inelastic behavior of the bents would take place.

That the estimated column bending stress is conservative can be assured from examining the deflections. The computed lateral (in-plane) deflection at the column top corresponding to the maximum estimated column stress is about half an inch. Actual movements of bents 490 and 491 appeared to be much less than that. Obviously, the computed deflections and the column stresses are conservative. The bents should have behaved and would continue to behave elastically in the sideways motion of the trains.

On the other hand, the large fluctuation of stresses in viaduct truss components near the supports will continue to occur if the condition of out-of-straightness of the rails remains. Fatigue cracks in those members could continue to propagate, necessitating frequent inspections and repairs.
5.3 Restraints, Redundancy, and Safety

While restraints at truss panel points can cause bending stresses in members, resulting in stress magnitudes higher than those computed from "hinged" truss joints, these "rigid" panels and the concrete deck also make the viaduct spans rigid frames. The transmittal of force from the train to the bents follow complicated paths. There is redundancy. When a component of the structure loses its capability, as in the case of an end panel diagonal with fatigue cracks completely across its width, the paths of force transmittal from the train to the bents are altered but not terminated. This condition assures safety against catastrophic failure. Redundancy is essential for safety.

At the truss ends, the top chord connection plates provide restraint and additional path for transmission of force out of the trusses. However, this restraint causes change of stress patterns in the neighboring members and contributed to the development of fatigue cracks there.

In the case of restraints at the truss panel points and the encasement of the truss chords by the concrete deck, the beneficial effects of multiple load path are dominant. For the end panel top chord connection plates, the detrimental effects of higher stresses prevail. Other examples of these situations are abundant: the continued service of through-truss bridges with broken or buckled major members because the top and bottom lateral bracing provided redundancy; the failure of bridge girders because of unexpected stresses from connecting components, etc. In most cases, the stresses
in the structural components were not computed in the original design for the restrained conditions.

The state-of-art in structural analysis and design has advanced to the stage that stresses and deflections can be calculated sufficiently accurately. Unfavorable stresses at restraints such as lateral bracing connections can be detected and design correspondingly changed. Redundancy such as additional deck girders can be featured in design without loss of efficiency. To dwell on this is beyond the scope of this study. It suffices to point out here the importance of maintaining structural redundancy in the renovation schemes for the Frankford Elevated viaduct so as to ensure continued safety and reliability.
6. SUMMARY AND CONCLUSIONS

The results and conclusions of this study are summarized below.

1. Analysis of single truss showed that the components of the trusses were subjected to bending as well as axial forces. The bending stresses could be of the same order of magnitude as that due to axial forces.

2. Placing of connection plates at end panel top chords induced stresses which could be high enough to cause fatigue cracks in the connection plates.

3. Analysis using a three-dimensional model to simulate the viaduct spans, confirmed the results from the single truss analysis that the end diagonals were the members with highest stresses. The highest computed stresses were at the bottom edge of the upper end of the end panel diagonals in the exterior trusses (Fig. 2.13), where many of the cracks were detected (see Fig. 2.8). The end panel top chord members of the trusses were strongly influenced by the presence of the connection plates. Stress reversals under live load occur in these top chords.

4. The patterns of stress fluctuation in an end panel diagonal and top chord were estimated from static structural analysis of a viaduct span subjected to a six-car train. (Figs. 2.21 and 2.22)
5. Actual measurements of stresses showed strong influence of train load (passenger load) and train frequency on the pattern and magnitudes of stress fluctuation in viaduct truss components. When train loads were light, the measured stress fluctuation patterns were as predicted and the stress magnitudes were relatively low. When trains were fully loaded and traveled frequent curing "rush hours" the measured stresses were higher than expected and fluctuated in an irregular pattern (Fig. 2.30), and the viaduct spans where the stress measurements were made were felt to vibrate. Stress fluctuations returned to the predicted pattern when train loads became light again (Figs. 2.32 and 2.33).

6. The cracks which were detected in the truss end panel components were fatigue cracks, judged from the estimated stresses and the fatigue strength of riveted structural components.

7. Because cracked members had been repaired and there was no new crack detected during the short period of field work in this study, measurements of stresses at cracks could not be made. Results of stress measurements at a few members of viaduct trusses indicated that live-load stress ranges in truss end panel components were high enough to cause fatigue cracks after many years of viaduct operation. Since conditions of the structure and the trains remain the same, the life load stresses will continue to induce fatigue cracks in the truss components.

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8. Repairs by welding to close the cracks in end diagonals did not change the pattern of live load stress fluctuation at the points of crack, and such welding often reduced the fatigue strength of diagonals. Fatigue cracks were observed to reoccur and will continue to do so.

9. Repair of cracks by welding doubler-plates onto original plates of truss components did not necessarily reduce the live load stress ranges at the cracks, depending on the state of stress at the crack and the geometry of the plates. Also, the repaired components could actually have lower fatigue strength than the original riveted member. Fatigue cracks did reoccur at some of these repaired plates.

10. A possible result of fatigue crack growth is sudden brittle fracture of the cracked component if its fracture toughness is not sufficient. The toughness of the viaduct material is better than that of A7 steel in energy absorption by Charpy V-notch tests. A 15 ft-lb toughness at 70°F is judged adequate for this viaduct superstructure in the Philadelphia area. By employing the fracture mechanics concept of evaluation, it was estimated that end panel diagonals with half-inch long cracks at the edge or at rivet holes would not cause sudden brittle fracture. Even if the crack grew to two inches long, fracture of the diagonal was not anticipated.

11. Analysis of a viaduct span by the three-dimensional model showed that if an end panel diagonal lost half of its
area, only the neighboring component members of the truss were affected with changing of stresses in the order of 10-15%. Even when the end panel diagonal lost its area completely, the effects were primarily within the neighborhood without much influence on the overall behavior of the span. This was also borne out from the tests of Wiss, Janney, Elstner and Associates.

12. To avoid possible development of fatigue cracks in nearby members, cracks in end panel diagonals or top chords of viaduct trusses need to be detected and repaired.

13. The maximum time interval of inspection for cracks was established through consideration of crack growth and viaduct truss behavior. A conservative interval of one year is recommended between inspections till replacement of the trusses.

14. Members detected to have cracks should be repaired. Repair by bolting of plates is recommended. Cracked end panel top chord connection plates may be left alone if the crack is in the plate and does not interfere with the operating system of the trains.

15. In view of the condition of continuing occurrence of fatigue cracks necessitating constant repairs, and of the condition that the extent of corrosion of the center trusses in the viaduct spans is not known without removal of the existing concrete in the vicinity, overall replacement of the viaduct trusses is suggested.
16. Live load stresses were measured from members of the bridges over the AMTRAK Line and the CONRAIL Line. The maximum live load stress was about 2.5 ksi measured under the normal peak hour load of the present day cars. Trains from either direction generated live load stresses in truss members of the pony-truss bridge over the AMTRAK Line. The through truss bridges over the CONRAIL Line supported single tracks and each bridge carried its live load without apparent effects on the other although these bridges were connected by a lateral bracing system. Measured live load stresses in the bridges were low in comparing to the original design live load stresses. (See Table 3.1).

17. Fatigue cracks are not likely to develop in the bridge members. The extrapolated maximum live load stress range for the old cars is within the allowable specified by AASHTO for Category D, corresponding to riveted members, and Category E, corresponding to the welded repairs on the bridge members. Analysis by the procedure of fracture mechanics for crack propagation indicates that no crack growth would be expected in the bridge members under the loads of the trains.

18. Repair of bridge members in the future should weigh the loss of member area due to corrosion, against reduction of fatigue strength due to introduction of structural repair details. Control of corrosion by
maintaining good coat of paint is preferable to repair after corrosion has started.

19. Overall, the bridges are judged adequate for retention in a reconstructed Elevated Line. With no anticipated increase of maximum train loads and with maintenance, these bridges should be adequate for 40, 50 years of service.

20. Double-column bents were examined analytically for the evaluation of their behavior. In-plane behavior under train load was estimated to be within the elastic range of response. This was confirmed by the results from the tests of Wiss, Janney, Elstner and Associates.

21. The cracks at the top flange of the transverse girders of the bents occurred in the compression flange and were judged to be caused by fretting under the rails. These cracks can not penetrate down to the next layer of the compression flange, thus the cracks could be ignored. Fretting, if it does occur, should be corrected.

22. The bottom flange of the transverse girders of the bents are not expected to have fatigue cracks if conditions of the viaduct are not altered. If changes are to be made, such as attaching longitudinal girders to the transverse girders of the bents, care must be taken to examine the fatigue strength of the new structural details.
23. The deflections of the double columns in the direction of the viaduct, as reported by others in previous examinations of the structure, are stationary in nature and not influenced by the presence of trains. The estimated maximum out-of-plane deflection of the columns is in the order of an inch under the extreme conditions of emergency braking of the heavy old trains. This estimated deflection is transient, not stationary, and would recover totally if no inelastic behavior of the bents has been induced.

24. Maximum combined axial and bending stresses in the columns of the double-column bents, under the condition of emergency braking of the heavy old trains, plus the effects of vertical live loads of the same trains and the dead weight of the viaduct spans, is estimated to be around 27 ksi. This magnitude of stress is within the elastic range of property of the steel in the viaduct. No permanent deflection of the columns would have taken place.

25. The existence of the out-of-plane deflections of the columns is believed to have been from incomplete recovery of elastic deflections. The incomplete recovery or "residual" deflections could have been caused by debris, corrosion, or other restraints at the supports of the viaduct spans. When the truss supports and expansion joints were corroded "frozen", the residual deflections became stationary. Relieving of the frozen supports and expansion joints at the bents is expected to eliminate the out-of-plane deflections of the columns. Protection of expansion joints and
supporting devices such as bearing plates and rockers from corrosion is considered as important as, if not more important than the protection of the structural members from corrosion.

26. The connections of any proposed reconstruction scheme need to be carefully examined against the fatigue strength of the connections. Adequate and realistic modeling of structure and connections must be made in the structural analysis so as to evaluate accurately stresses in the proposed details. These details include fixed-ended connections between longitudinal and transverse girders, expansion joint connection plates, hanger plates and pins, etc., etc.

27. The track rails of the Elevated Line are out-ofstraight at some locations. Trains were observed to swing sideways at these places. The sideways swing (nosing) of the trains are believed to have caused high fluctuating stresses in the components in the end panels of the viaduct trusses. On the other hand, the sidesway motion of the double column bents, corresponding to the sideways movement of the train cars, is estimated to generate stresses in the columns well below the elastic limit of the column steel. For the reduction of high fluctuating stresses in the viaduct superstructure, maintaining of straight and smooth track rails is considered essential.
It must be pointed out again that the summary and conclusions are more qualitative than quantitative. This is primarily due to the complex nature of the existing viaduct superstructure system with its high degree of indeterminacy for analysis and difficult-to-define support conditions. In spite of this, the safety and reliability of the viaduct has been affirmatively assessed for the interim period before any replacement of the superstructure. The assessment was made considering the most severe condition of train loads on the viaduct. Under these severe conditions, the three major bridges and the double-column bents were found adequate and safe. (The single column bents were not analyzed in this study but their safety and adequacy are expected to be equivalent to those of the double-column bents.) Fatigue cracks in the viaduct trusses, however, are anticipated to continue to develop.

Since fatigue cracks are expected to develop in the trusses, necessitating constant inspection and repair, and since the extent of the corrosion of the center trusses is not known with certainty without removal of the existing concrete in the vicinity of these members, it appears that complete replacement of the superstructure, including the trusses and the deck system, is the best approach of renovation. Alternately, new structural members (plate girders or trusses) may be added below the existing concrete deck to support the viaduct spans and to relieve the structural function of the existing trusses. A less desirable approach is to remove the concrete encasement along the center trusses for repair of corroded truss member, and to replace or repair all the truss supports and expansion joints. Complete
replacement of the superstructure not only is more reliable in assurance of a safe and integral rapid transit line superstructure, but it may turn out to be the least expensive approach in the long run. Complete replacement of the superstructure is therefore recommended.

The major recommendations are summarized again, as the following:

1. The viaduct trusses and decks be replaced by a new superstructure system.
2. The three major bridges need not be replaced.
3. The double-column bents can be retained.
4. Adequate and realistic analysis of the new superstructure and details be conducted to ensure that the stresses in the structural details are within allowable values against fatigue.
5. Protection from corrosion of all structural elements, including all bearings and expansion joints, be considered in the replacement scheme of the new superstructure.

For a time period of a few years while planning and design are carried out for the complete replacement of the superstructure, the viaduct spans of the Elevated Line will remain adequate for use if an uninterrupted comprehensive inspection and repair program is carried out as recommended.
ACKNOWLEDGMENTS

This project entitled "Fatigue Resistance of the Frankford Elevated Line" is sponsored by the City of Philadelphia through the Kaufman Construction Company. Mr. Thomas A. Horvai is the Project Manager for the City of Philadelphia. His constant help is highly appreciated.

The assistance of Mr. James Busch of Southeastern Pennsylvania Transportation Authority (SEPTA) and the help in field work by his inspection crew members are gratefully acknowledged.

The analysis was conducted in Fritz Engineering Laboratory and the Department of Civil Engineering at Lehigh University, Bethlehem, Pennsylvania. Dr. Lynx S. Beedle is the Director of the Laboratory and Dr. David A. VanHorn, the Chairman of the Department.

Thanks are due Mr. Hugh T. Sutherland for his major role in the field study. The constant help and patience of Mrs. Dorothy Fielding in completing and typing this report is sincerely acknowledged.
<table>
<thead>
<tr>
<th>Location</th>
<th>Outside Truss</th>
<th>Inside Truss</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>U₀ - U₁</td>
<td>2Lₛ -6&quot; x 6&quot; x 1/2&quot;</td>
<td>2Lₛ -6&quot; x 6&quot; x 5/8&quot;</td>
<td>Top Chord</td>
</tr>
<tr>
<td>U₁ - U₂</td>
<td>2Lₛ -6&quot; x 6&quot; x 1/2&quot;</td>
<td>2Lₛ -6&quot; x 6&quot; x 5/8&quot;</td>
<td>Top Chord</td>
</tr>
<tr>
<td>U₂ - U₅</td>
<td>2Lₛ -6&quot; x 6&quot; x 1/2&quot;</td>
<td>2Lₛ -6&quot; x 6&quot; x 5/8&quot;</td>
<td>Top Chord</td>
</tr>
<tr>
<td>L₁ - L₂</td>
<td>2Lₛ -6&quot; x 6&quot; x 1/2&quot;</td>
<td>2Lₛ -6&quot; x 6&quot; x 5/8&quot;</td>
<td>Bottom Chord</td>
</tr>
<tr>
<td>L₂ - L₃</td>
<td>2Lₛ -6&quot; x 6&quot; x 1/2&quot;</td>
<td>2Lₛ -6&quot; x 6&quot; x 5/8&quot;</td>
<td>Bottom Chord</td>
</tr>
<tr>
<td>L₃ - L₅</td>
<td>2Lₛ -5&quot; x 3-1/2&quot; x 5/8&quot;</td>
<td>2Lₛ -6&quot; x 6&quot; x 5/8&quot;</td>
<td>Bottom Chord</td>
</tr>
<tr>
<td>U₁ - L₁</td>
<td>2Lₛ -5&quot; x 3-1/2&quot; x 3/8&quot;</td>
<td>2Lₛ -8&quot; x 3-1/2&quot; x 9/16&quot;</td>
<td>Vert. Member</td>
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<tr>
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<td>2Lₛ -8&quot; x 3-1/2&quot; x 7/16&quot;</td>
<td>Vert. Member</td>
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<tr>
<td>U₃ - L₃</td>
<td>2Lₛ -3-1/2&quot; x 3-1/2&quot; x 5/16&quot;</td>
<td>2Lₛ -6&quot; x 3-1/2&quot; x 7/16&quot;</td>
<td>Vert. Member</td>
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<tr>
<td>U₄ - L₄</td>
<td>2Lₛ -3-1/2&quot; x 3-1/2&quot; x 5/16&quot;</td>
<td>2Lₛ -5&quot; x 3-1/2&quot; x 3/8&quot;</td>
<td>Vert. Member</td>
</tr>
<tr>
<td>U₅ - L₅</td>
<td>2Lₛ -3-1/2&quot; x 3-1/2&quot; x 5/16&quot;</td>
<td>2Lₛ -3-1/2&quot; x 3-1/2&quot; x 5/16&quot;</td>
<td>Vert. Member</td>
</tr>
<tr>
<td>U₀ - L₁</td>
<td>2PLₛ -12&quot; x 7/16&quot;</td>
<td>2PLₛ -14&quot; x 5/8&quot;</td>
<td>Diagonal</td>
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<td>U₁ - L₂</td>
<td>2PLₛ -10&quot; x 7/16&quot;</td>
<td>2PLₛ -14&quot; x 9/16&quot;</td>
<td>Diagonal</td>
</tr>
<tr>
<td>U₂ - L₃</td>
<td>2PLₛ -8&quot; x 7/16&quot;</td>
<td>2PLₛ -12&quot; x 1/2&quot;</td>
<td>Diagonal</td>
</tr>
<tr>
<td>U₃ - L₄</td>
<td>2PLₛ -7&quot; x 7/16&quot;</td>
<td>2PLₛ -10&quot; x 1/2&quot;</td>
<td>Diagonal</td>
</tr>
<tr>
<td>U₄ - L₅</td>
<td>2PLₛ -6&quot; x 3/8&quot;</td>
<td>2PLₛ -7&quot; x 9/16&quot;</td>
<td>Diagonal</td>
</tr>
<tr>
<td>U₅ - L₄</td>
<td>2Lₛ -3&quot; x 2&quot; x 5/16&quot;</td>
<td>2Lₛ -2&quot; x 2&quot; x 3/8&quot;</td>
<td>Diagonal</td>
</tr>
<tr>
<td>U₆ - L₅</td>
<td>2Lₛ -3&quot; x 2&quot; x 3/8&quot;</td>
<td>2Lₛ -5&quot; x 3-1/2&quot; x 3/8&quot;</td>
<td>Diagonal</td>
</tr>
<tr>
<td>Member</td>
<td>End Restraint</td>
<td>Axial Stress (ksi)</td>
<td>Bending Stress (ksi)</td>
</tr>
<tr>
<td>------------------------</td>
<td>---------------</td>
<td>--------------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>End Diagonal</td>
<td>1*</td>
<td>0.0986</td>
<td>0.0314</td>
</tr>
<tr>
<td>(U₀ - L₁)</td>
<td>2**</td>
<td>0.0988</td>
<td>0.0311</td>
</tr>
<tr>
<td>End Panel</td>
<td>1</td>
<td>-0.0574</td>
<td>-0.0716</td>
</tr>
<tr>
<td>Top Chord</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(U₀ - U₁)</td>
<td>2</td>
<td>0.0686</td>
<td>0.0248</td>
</tr>
<tr>
<td>2nd Panel</td>
<td>1</td>
<td>-0.1017</td>
<td>-0.0556</td>
</tr>
<tr>
<td>Top Chord</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(U₁ - U₂)</td>
<td>2</td>
<td>-0.0514</td>
<td>-0.0237</td>
</tr>
</tbody>
</table>

1* No longitudinal restraint at top chord

2** With longitudinal restraint at top chord
### TABLE 2.3 MAXIMUM STRESSES IN END DIAGONALS FROM INFLUENCE LINE OF THREE PARALLEL TRUSSES

<table>
<thead>
<tr>
<th>Member End Locations</th>
<th>Member Restraint</th>
<th>Axial Stress (ksi)</th>
<th>Bending Stress (ksi)</th>
<th>Maximum Stress (ksi)</th>
<th>Locations</th>
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<tbody>
<tr>
<td>Exterior End Diagonal</td>
<td>1</td>
<td>0.096</td>
<td>0.025</td>
<td>0.121</td>
<td>Bottom Edge of Upper End</td>
</tr>
<tr>
<td>End Diagonal</td>
<td>2</td>
<td>0.101</td>
<td>0.029</td>
<td>0.130</td>
<td></td>
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<tr>
<td>(Member 21)</td>
<td>2</td>
<td>0.071</td>
<td>0.005</td>
<td>0.076</td>
<td></td>
</tr>
<tr>
<td>Interior End Diagonal</td>
<td>1</td>
<td>0.069</td>
<td>0.005</td>
<td>0.074</td>
<td>Bottom Edge of Upper End</td>
</tr>
<tr>
<td>End Diagonal</td>
<td>2</td>
<td>0.071</td>
<td>0.005</td>
<td>0.076</td>
<td></td>
</tr>
<tr>
<td>(Member 100)</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

1. No longitudinal restraint at top chord

2. With longitudinal restraint at top chord
TABLE 2.4 STRAIN GAGES ON VIADUCT TRUSSES

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Location</th>
<th>Analogous Member in Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Member*</td>
<td>Edge</td>
</tr>
<tr>
<td>1</td>
<td>ED</td>
<td>Upper</td>
</tr>
<tr>
<td>2</td>
<td>ED</td>
<td>Lower</td>
</tr>
<tr>
<td>3</td>
<td>TC</td>
<td>Lower</td>
</tr>
<tr>
<td>4</td>
<td>TC</td>
<td>Lower</td>
</tr>
<tr>
<td>5</td>
<td>ED</td>
<td>Upper</td>
</tr>
<tr>
<td>6</td>
<td>ED</td>
<td>Lower</td>
</tr>
<tr>
<td>7</td>
<td>ED</td>
<td>Upper</td>
</tr>
<tr>
<td>8</td>
<td>ED</td>
<td>Lower</td>
</tr>
</tbody>
</table>

*ED - End Diagonal
TC - Top Chord
**TABLE 2.5 AASHTO FRACTURE-TOUGHNESS SPECIFICATIONS FOR BRIDGE STEELS**

<table>
<thead>
<tr>
<th>ASTM DESIGNATION</th>
<th>THICKNESS</th>
<th>ENERGY ABSORBED, (ft-lb)</th>
</tr>
</thead>
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<tr>
<td></td>
<td>Zone 1*</td>
<td>Zone 2*</td>
</tr>
<tr>
<td>A36</td>
<td>15 @ 70°F</td>
<td>15 @ 40°F</td>
</tr>
<tr>
<td>A572+</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>Up to 4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>mechanically</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fastened</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 2&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A440</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>A441</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>A442</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>A558+</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>Up to 4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>mechanically</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fastened</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 2&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Over 2&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>welded</td>
<td>20 @ 70°F</td>
<td>20 @ 40°F</td>
</tr>
<tr>
<td>A514</td>
<td>25 @ 30°F</td>
<td>25 @ 0°F</td>
</tr>
<tr>
<td>Up to 4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>mechanically</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fastened</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 2-1/2&quot;</td>
<td>25 @ 30°F</td>
<td>25 @ 0°F</td>
</tr>
<tr>
<td>welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Over 2-1/2&quot;</td>
<td>25 @ 30°F</td>
<td>35 @ 0°F</td>
</tr>
<tr>
<td>to 4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>welded</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Zone 1 = Minimum Service Temperature 0°F and above.
Zone 2 = Minimum Service Temperature from -1 to -30°F.
Zone 3 = Minimum Service Temperature from -31 to -60°F.

*If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.*

-80-
<table>
<thead>
<tr>
<th>Member</th>
<th>Original Design Live Load Stress (ksi)</th>
<th>Maximum Measured $S_r$ (ksi)</th>
<th>Measured/Design (%)</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>7.72</td>
<td>1.3</td>
<td>17</td>
</tr>
<tr>
<td>2</td>
<td>6.55</td>
<td>1.0</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>7.01</td>
<td>1.3</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>7.10</td>
<td>1.3</td>
<td>18</td>
</tr>
<tr>
<td>5</td>
<td>5.43</td>
<td>1.3</td>
<td>24</td>
</tr>
<tr>
<td>6</td>
<td>5.39</td>
<td>---</td>
<td>--</td>
</tr>
<tr>
<td>7</td>
<td>3.11</td>
<td>1.0</td>
<td>32</td>
</tr>
<tr>
<td>8</td>
<td>5.57</td>
<td>1.0</td>
<td>18</td>
</tr>
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</table>
Fig. 2.1 Schematic View of Viaduct
Fig. 2.2 Elevation of Viaduct
Fig. 2.3 Typical Section of Viaduct
Fig. 2.4 Joint Detail (continuity from truss-to-truss)

Fig. 2.5 Joint Detail (top chord connection plate)
ENCASED FLOOR BEAM
CONCRETE SLAB
TRANSVERSE GIRDER
SUPPORT COLUMN

5'-3"

Fig. 2.6 Viaduct Concrete Slab
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Fig. 2.34 S-N Curve for A7 Steel Riveted Joints in Tension

A7 Steel Riveted Joints in Tension

\[ \frac{\sigma_b}{\sigma} \text{ (bearing ratio)} \]

- 1.37
- 1.83
- 2.74
- 2.36
- 3.8 (exceeds limit of 1.5 \( \sigma_u P \))
- Full Scale Tests (failure)
- Full Scale Tests (small cracks)
Strain Gage No. 2

35 Trains

194 Stress Ranges

(= 100%)

Fig. 2.35 Histogram of Stress Ranges and their Corresponding Frequencies of Occurrence in Percent, Strain Gage No. 2
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Fig. 4.2 Influence Line for Horizontal Reactions at Left Hinge Support
5.6 + 8.4 = 14 kips

Fig. 4.3 Horizontal Forces at Bents due to Emergency Braking of Trains
Fig. 4.4 Force-Displacement Relationship for Column of Simple Bent

\[ \delta = 0.23 \, P \]
Fig. 4.5  Force-Deflection Relationship for Columns in a Three-Bay Rigid Frame
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