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The design of highway bridges in the U.S. is based on the
"standard" AASHTO H and HS truck and lane loads. (Ref. 1) These
design live loads have remained virtually unchanged in over 40
years and no longer accurately represent the majority of the
heaviest trucks using the highway system. In response, many
states have increased their design live loads. Pennsylvania, for
example, requires that all structures designed using the load
factor design (LFD) method be designed for HS 25 and/or 125% of
AASHTO alternate military loading, and checked for a 204 kip, 8
axle, standardized permit load. (Ref. 2)

The analysis of the majority of typical highway bridges in
the U.S. is based on the empirical AASHTO load distribution
criteria. (Ref. 1) These criteria were developed piece-meal,
with the result that similar criteria appear in different forms.
(Ref. 3) Some criteria have remained virtually unchanged in over
50 years, although radical changes have occurred in bridge types,
in the magnitude and frequency of loading, in the methods of
analysis and design, and in the methods of construction during
those years. Many criteria are unclear and subject to differing
interpretations. Many criteria are being applied to rating al-
though they were developed and intended for design.

Live load stresses will occur in all superstructure compon-
ents when the as-built three-dimensional structure is subjected to
the real dynamic highway traffic loads. However, for a majority
of typical bridges, use of the AASHTO load distribution criteria
will result in approximate live load stresses calculated only for
the main members. For example, live load stresses are calculated
in the stringers and girders of straight bridges but not for
diaphragms and lateral bracing. This situation arises because
the load distribution criteria are based on the static application
of "standard" AASHTO live loads to an over-simplified two-dimen-
sional representation of the superstructure.

In view of the above it should not be surprising that virtu-
ally all stress history studies conducted on in-service bridges
over the past 30 years consistently show that the actual measured
live load stresses in superstructure components are considerably
different from those assumed in design. (Refs. 4,5,6) For example,

field measurements of strain response show that for straight, 22
girder and multiple stringer bridges, vehicles similar to the
AASHTO design truck produce stresses in main members (components
having calculated live load stress) which seldom exceed 50% of the
calculated design stress. On the other hand, measured live load
stresses in diaphragms, lateral bracing and other "secondary"
members and connections (components having no calculated live load
stress) can either be negligible, or very large, depending upon
the geometric configuration and construction of the superstructure.
For example, displacement induced stresses below cut short diaphragm
connection plates (transverse stiffeners) and in tie plates fre-
quently reach the yield stress. (Ref. 6) Obviously the designer
has no control of the strength and serviceability of these secondary
members and connections when the design is based on traditional
simplified and empirical procedures.

From the standpoint of ultimate strength, bridges in the U.S.
have exhibited exemplary behavior. Very few have collapsed as a
result of overload. Field tests show that the typical bridge has
considerable static overload capacity. Indeed, the provisions of
the AASHTO specifications have historically been developed to en-
sure substantial factors of safety against collapse under static
loading. On this basis the use of conservative, simplified analy-
tical models and empirical load distribution criteria are entirely
justified.

From the standpoint of fatigue, however, especially in steel
bridges, the behavior over the last 30 years has not been satis-
factory. Although very few bridges have collapsed as a result of
fatigue crack growth and subsequent fracture, many in-service
bridges exhibit damage due to fatigue cracking. Considerable
research effort has and is being devoted to improve fatigue
specifications and to develop retrofit procedures for extending
the fatigue life of existing bridges. Use of these specifications
and procedures is dependent upon accurate design office estimates
of the stress range spectra in all components and connections of
the three-dimensional superstructure. The required estimates
cannot be obtained from the simplified analytical models and
empirical load distribution criteria in use today.

Although design office procedures for the majority of bridges
should remain as straight forward as possible, a need clearly
exists for improvements in methods of analysis and in bridge
design and rating specifications. (Ref. 7) Analysis should more
accurately reflect the real spectrum of bridge loads and the re-
sponse of all components of the superstructure to those loads.
Design and rating specifications should allow for the vastly

increased analytical capabilities that electronic computation has brought to the modern design office.

In the past decade advances have been made in weigh-in-motion (WIM) technology. (Refs. 8,9,10) A computerized WIM system developed for the USDOT Federal Highway Administration (FHWA) is portable, and by using suitable transducers converts an in-service bridge to an equivalent weigh scale. Reasonably accurate data is obtained on gross vehicle weights (GVW), axle weights, axle spacings and speeds of vehicles crossing the bridge at normal highway speeds. (Ref. 9) Since the weighing operation is not easily detected by truck drivers the results are not subject to the bias associated with traditional truck weighing stations, where, with the aid of the CB network, some trucks, for whatever reason, can easily avoid an operating weigh station. WIM systems have begun to reveal the true spectrum of truck weights, especially the frequency of the high weights which are known to cause significant bridge (and pavement) damage. (Ref. 10)

Much more can be done however. By coupling the WIM system with a system for measuring strains in the components and connections of the superstructure simultaneous load plus bridge response data can be obtained for nearly all vehicles crossing the bridge within an arbitrary period of time. For an evaluation of bridge response the primary information required is the magnitude and variation of stress in the superstructure components and connections during the passage of vehicles of known weights over the bridge. The correlation of GVW, axle weights and frequency of vehicles with stress range and maximum stress is the foundation of the needed improved bridge analysis, design and rating procedures and specifications based on strength and serviceability (such as fatigue) requirements.

This paper briefly describes the results of a recent 30 month FHWA sponsored research investigation at Lehigh University during which an FHWA owned WIM system was redesigned and used during the summer and fall of 1985 to obtain and process simultaneous load plus response data produced by 19,402 trucks crossing four in-service bridges in Pennsylvania. This field study was primarily intended as a "proof of concept"; that is, to use the WIM+RESPONSE system developed at Lehigh University to collect, store and process a large quantity of data from in-service bridges to prove the system concept. The field study was not designed to investigate bridges having suspected structural problems due to overstress or fatigue. However, the brief results presented later in this paper do show the expected behavior of steel and pre-stressed concrete bridges which do not exhibit structural problems.

The Lehigh system is designed the WIM+RESPONSE system. (Refs. 11-17) This portable system has both hardware and software necessary to acquire and store (on floppy disks) data from steel and concrete bridges and also to process that data either in the field during data collection or later in the office. The WIM+RESPONSE system is currently in possession of the owner, FHWA, for further evaluation and data collection.

WIM+RESPONSE SYSTEM CAPABILITIES

Figure 1 shows a typical setup for a WIM+RESPONSE system field study. The installation includes (1) tape switch axle detectors placed in one or both traffic lanes, (2) an optional keypad to record truck body type and other visible hauling information, (3) up to six specially designed strain gage transducers clamped to the bridge girders which provide WIM and/or RESPONSE data, (4) up to ten strain gages, attached to any bridge member or connection, which provide RESPONSE data, and (5) a minicomputer, complemented with dedicated electronic equipment, located in a mobile van parked under the bridge, which receives and stores data coming from the strain transducers and strain gages. The equipment temporarily located in the van during the field study consists of a Digital Equipment Corporation (DEC) MINC 11-23 16 bit word minicomputer with two 8-in. disk drives; a DECLAB-11/MNC integrated system containing 2 clock modules, 1 digital input module and 1 A/D converter module; a VT125 Graphics Terminal; a WIM signal conditioner; and a RESPONSE signal conditioner. Additional equipment used during data processing includes an LA50 dot-matrix, serial printer, for hardcopy tabular and graphical display.

The WIM+RESPONSE system capabilities are briefly summarized as follows:
1. The WIM+RESPONSE system can acquire and store up to 16 channels of simultaneous truck weight plus bridge resonse data. However, the total number of transducers plus gages can greatly exceed the 16 available channels. While WIM data is being acquired the remaining channels can be dedicated to different groups of gages, thus making it possible to obtain a large amount of simultaneous truck weight plus bridge response data.
2. The WIM strain transducers are designed for use on multiple, steel, concrete or prestressed concrete stringers of girders. Thus the WIM+RESPONSE system can acquire data from multiple stringer bridges as well as from 2-girder bridges with floor beams and stringers by attaching the WIM strain transducers to the stringers. The system can be used on right or skew bridges

TAPE SWITCHES

1

2 KEYPAD OPERATOR

3 STRAIN TRANSDUCERS

4 STRAIN GAGES

5

INSTRUMENTATION VAN

-5-
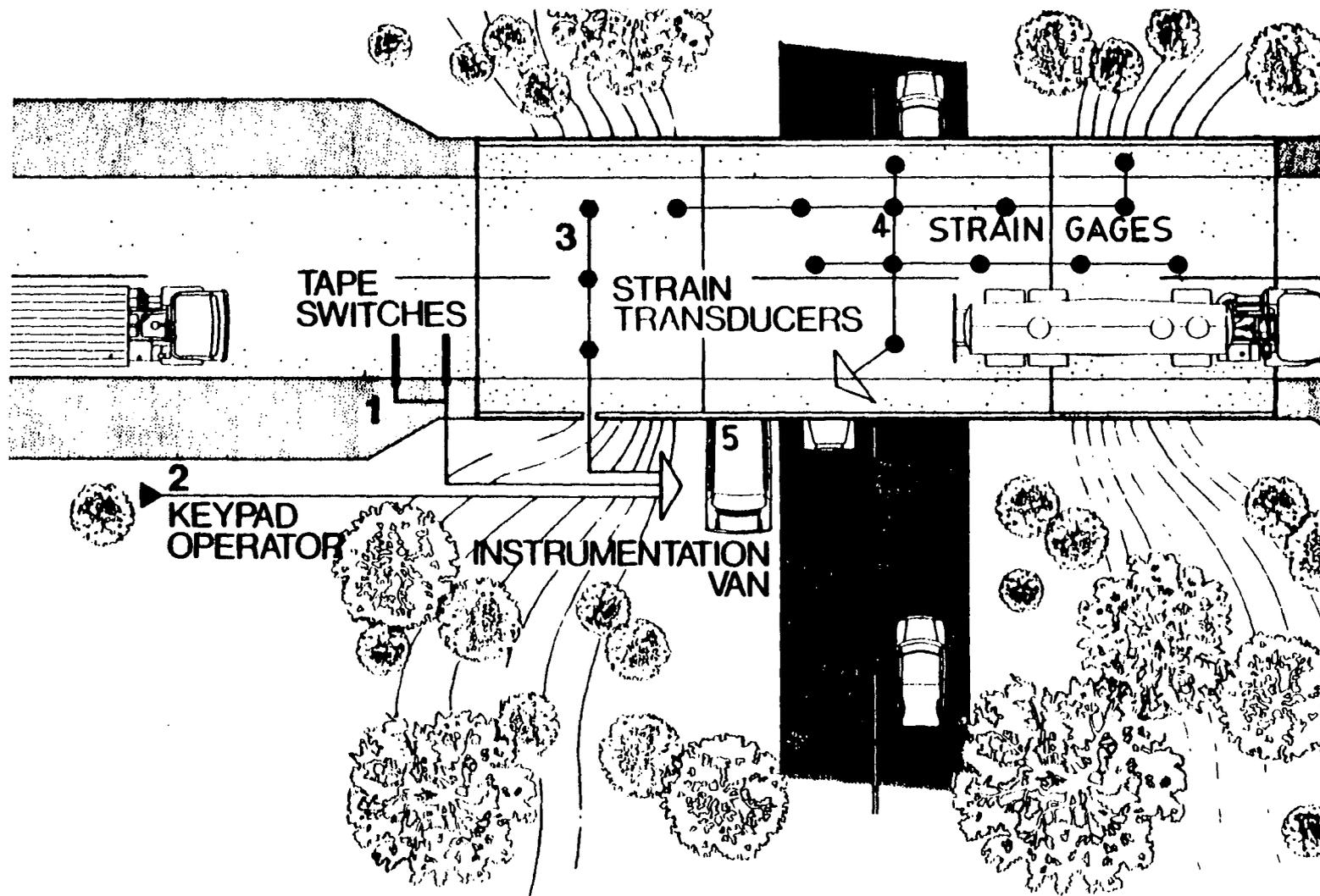
as well as simple or continuous spans and composite and non-composite bridges.

3. The WIM+RESPONSE system can acquire and store simultaneous truck weight plus response data from any combination of simple and/or continuous spans with a total bridge length of 170 feet. For example, weight and response data can be obtained from one span while additional response data is obtained from another span. Alternatively, if strictly simultaneous weight plus response data is not required, WIM data could be obtained from one bridge until a representative GVW histogram is obtained for the route then RESPONSE data obtained from another bridge on the same route shortly after.

4. THE WIM+RESPONSE system software permits data reduction and statistical load plus response information to be processed in the form of, for example, GVW histograms, stress range histograms, relationships between GVW and maximum stress, stress range versus strain rate, and other displays of interest to the bridge engineer, researcher and planner. The system allows the user to provide additional software which would use the weight plus response data for other studies such as load distribution, bridge dynamics and damping characteristics.

FIELD STUDY BRIDGES

The four bridges selected for the field study during the summer of 1985 are briefly described as follows:

Bridge 1 - East bound bridge on PA Route 22 over 19th Street in Allentown, PA. Two lane, composite steel-concrete bridge, with four, right, simple, multiple, riveted steel plate girder spans. Weight plus response data was obtained from an 85-ft. span consisting of 5 parallel plate girders, 57-in. deep and spaced at 8-ft. The K-type diaphragms are spaced at 17-ft. The maximum ADTT is about 3,000.

Bridge 2 - West bound bridge on PA Route 22 over 19th Street in Allentown, PA., constructed the same as Bridge 1. Weight plus response data was obtained from a 125-ft. span consisting of 5 parallel plate girders, 92-in. deep and spaced at 8-ft. The X-type diaphragms are spaced at about 18-ft. The maximum ADTT is about 3,000.

Bridge 3 - North bound bridge on PA Route 33 over Van Buren Road, one mile north of PA Route 248. Two lane, non-composite steel-concrete bridge, with one, skew (53° 29'), simple, multiple, welded steel plate girder span and two, skew, simple, multiple, rolled girder spans. Weight data was obtained from one 40-ft. rolled girder span consisting of 6 parallel W33X130 girders spaced at 7'-4". Response data was obtained from the 40-ft. span and from the 108-ft. plate

girder span consisting of 6 parallel girders, 53-in. deep and
spaced at 7'-4". The X-type diaphragms are spaced at 22-ft.
The maximum ADTT is about 1,000.

Bridge 4 - North bound bridge on PA Route 33 over State Park Road,
    two miles north of the Belfast exit. Two lane, composite
    concrete-concrete bridge, with three skew (48° 47'), simple,
    multiple, prestressed girder spans. Weight data was obtained
    from a 66-ft. span consisting of 6 parallel PADOT Type 24/25
    prestressed girders spaced at 7'-4". Solid 30-in. deep
    concrete diaphragms occur at midspan. Response data was
    obtained from the 66-ft. span and from a 28-ft. span
    consisting of 6 parallel PADOT Type 20/30 prestressed
    girders spaced at 7'-4", with no diaphragms. The maximum
    ADTT is about 1,000.

RESULTS OF FIELD STUDY

      The results of the field study of the four in-service bridges
are briefly summarized in Figs. 2 through 7. All field data were
processed at Lehigh University using the WIM+RESPONSE system MINC
11-23 minicomputer. All of these figures were first displayed on
the VT125 Graphics Terminal, then plotted using the LA50 graphics
printer.

      The GVW histograms were generated from the WIM data and cor-
respond only to single trucks travelling in either the outside
(lane 1) or inside (lane 2) lanes. Stress range histograms were
computed using the reservoir (modified rainflow) cycle counting
method (Ref. 18) and considered all cycles (no lower cutoff) of
the strain history curve for every single and multiple truck event.
Strain rate is computed as the positive chord slope joining con-
secutive valleys and peaks of the strain history curve (similar
to the ascending method). Maximum stress versus GVW results con-
sidered only single trucks travelling in lane 1. For each data
point the maximum stress was determined from the maximum strain
recorded at the particular gage during a single truck event.

GVW DISTRIBUTION

      Figure 2 shows the gross vehicle weight (GVW) distribution
computed for 4,239 single trucks crossing Bridge 1 in lanes 1 and
2. A similar distribution was obtained for the other three
bridges. Characteristic of these distributions is the presence
of two peak values of frequency, the first at about 25 kips, the
second at about 70 kips. The first peak corresponds to a

relatively high percentage of heavy small trucks (3 or 4 axles), the second to a relatively high percentage of heavy large trucks (5 or more axles). This characteristic was also obtained in the 1970 FHWA Nationwide Loadometer Survey, the results of which were used to develop the stress cycles for design against fatigue damage of steel bridges which are incorporated into the AASHTO Specifications. (Refs. 1 and 4) Of particular interest in this field study is the fraction of the GVW distribution corresponding to the high values of GVW. The percentage of trucks exceeding a gross vehicle weight of 80 kips varied from 4.2% for Bridge 1 to 7.0% for Bridge 3. The maximum recorded GVW was about 150 kips for all four bridges.

STRESS RANGE DISTRIBUTION

The stress range distribution in the bottom flange near mid-span of the first interior girder of Bridge 1 and computed for 4,680 trucks, is shown in Fig. 3. Miner's equivalent stress range is 0.7 ksi. The maximum recorded stress range is 5.8 ksi. The shape of the stress range distribution in Fig. 3 is typical for all gage locations on all three steel bridges and does not have the same shape as the GVW distribution as has been assumed. Gages were located near midspan of the bottom flanges of all girders, on selected members of the K and X-type diaphragms, adjacent to a welded flange splice (thickness transition) in Bridge 3, and on the girder web in the coped region at the bottom of a diaphragm connection plate (transverse stiffener) of Bridge 3 which was not welded to the bottom flange of the girder. The maximum values of Miner's equivalent stress range varied from 0.64 ksi for a gage near midspan of the first interior girder of Bridge 2 to 0.85 ksi for a gage near midspan of an interior rolled girder on the 40-ft. span of Bridge 3. The maximum value of stress range was 6.2 ksi for all spans of all three steel bridges and occurred in many of the fascia and interior girders. No stress ranges were calculated for Bridge 4.

Fatigue analyses of details subjected to variable amplitude loading, such as those in highway bridges, can be made directly from measured stress range distributions and are based on the Stress Range versus Cycle (SN) relationships developed at Lehigh University and incorporated into the AASHTO Specifications. (Refs. 1 and 4) Reference 19 indicates that fatigue life is a function primarily of two parameters, the effective stress range (Miner) and the maximum stress range. Three possible situations are encountered:
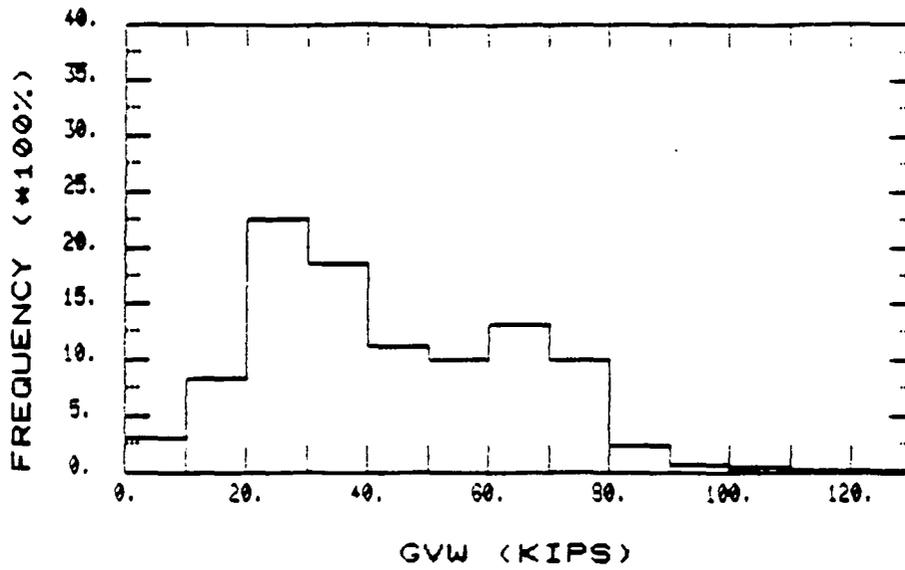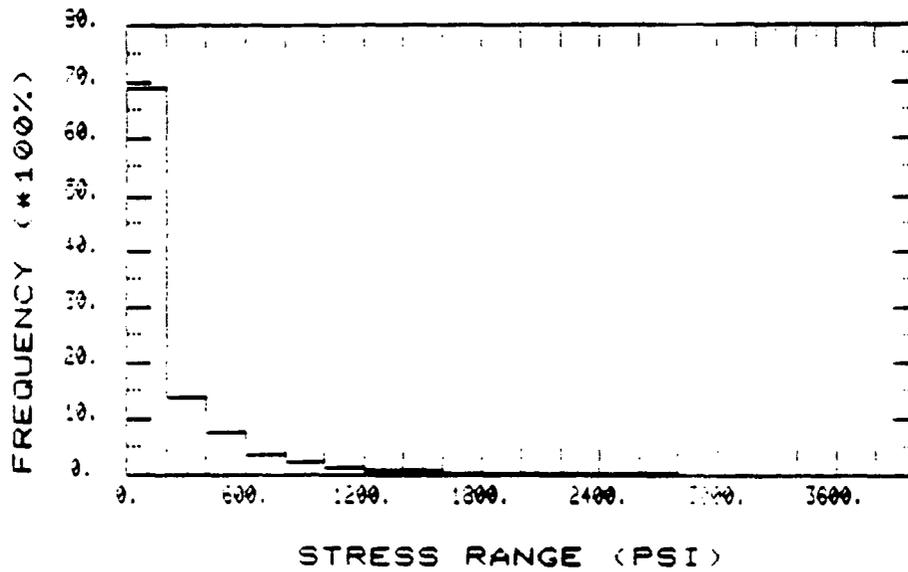
Fig. 2 GVW Distribution for Bridge 1

Fig. 3 Stress Range Distribution - Bottom Flange - Midspan
of First Interior Girder of Bridge 1

1. Effective Stress Range > Constant Amplitude Fatigue Limit

2. Effective Stress Range < Constant Amplitude Fatigue Limit

    Maximum Stress Range   > Constant Amplitude Fatigue Limit

3. Effective Stress Range < Constant Amplitude Fatigue Limit

    Maximum Stress Range   < Constant Amplitude Fatigue Limit

For Case 1, the effective stress range is used as the constant amplitude stress range in conjunction with the constant amplitude SN curves to determine fatigue life.

For Case 2, the effective stress range must be used in conjunction with a straight line extension of the sloping portion of the constant amplitude SN curve to determine fatigue life, since no fatigue limit exists.

For Case 3, since all of the stress range spectrum is below the constant amplitude fatigue limit, none of the stress ranges should be damaging and no fatigue crack propagation is expected.

The above can be used to evaluate the fatigue behavior of the field study bridges. For example, consider a location having a maximum stress range of 6.2 ksi and a Miner's effective stress range of 0.85 ksi. Case 2 would exist for a Category E detail at this location since the fatigue limit is 5 ksi which is below the maximum stress range. On the other hand Case 3 would exist for a Category B detail at this location since the fatigue limit is 16 ksi, well above the maximum stress range.

STRAIN RATE DISTRIBUTION

The strain rate distribution in the bottom flange near midspan of the fascia girder of Bridge 1 and computed for 4,680 trucks, is shown in Fig. 4. The maximum recorded strain rate is 1,700 micro in/in/sec. The shape of the strain rate distribution shown in Fig. 4 is typical for all gage locations on all three steel bridges. The maximum values of strain rate varied from 6,458 micro in/in/sec for Bridge 1 to 8,640 micro in/in/sec for Bridge 3 and always occurred near midspan of an interior girder under lane 1.

MAXIMUM STRESS VERSUS GVW

The relationship between maximum stress and gross vehicle weight near midspan of the first interior girder of Bridge 1 and

computed for 2,861 single trucks is shown in Fig. 5. The absolute
maximum stress recorded at this location is 7.6 ksi. The equation
of the linear regression line through the 2,861 data points is

Max. Stress (psi) = 303.5 + 22.9 GVW (kips)   (1)

and the correlation coefficient is 0.827.

The relationships shown in Fig. 5 are typical for all gage
locations on all four bridges. All tend to show a concentration
of data points extending in an upward sloping manner as shown in
the figure. For the three steel bridges the absolute maximum
recorded stress varied from 9.4 to 9.9 ksi and occurred in the
girders under lane 1. The maximum stress in Bridge 4 was 1.13
ksi and occurred near midspan of the first interior girder of
the 66-ft. span. In computing stress, the flexural modulus of
the prestressed girders was assumed to be 4,500 psi. The maximum
measured live load stresses are consistently below the design
stress.

STRESS RANGE VERSUS GVW

Figures 6 and 7 show the relationships between stress range
and gross vehicle weight near midspan of the first interior
girders of Bridges 1 and 3. In Fig. 6 the maximum stress range
is 5.07 ksi and the equation of the linear regression line
through the 2,861 data points is

Stress Range (psi) = 506.73 + 25.11 GVW (kips)   (2)

and the correlation coefficient is 0.843.

In Fig. 7 the maximum stress range is 4.17 ksi and the
equation of the linear regression line through the 2,856 data
points is

Stress Range (psi) = 417.72 + 24.53 GVW (kips)   (3)

and the correlation coefficient is 0.884.

RESULTS OF ANALYTICAL STUDY

Finite element analyses of each of the three-dimensional
superstructures of the three steel bridges were performed to
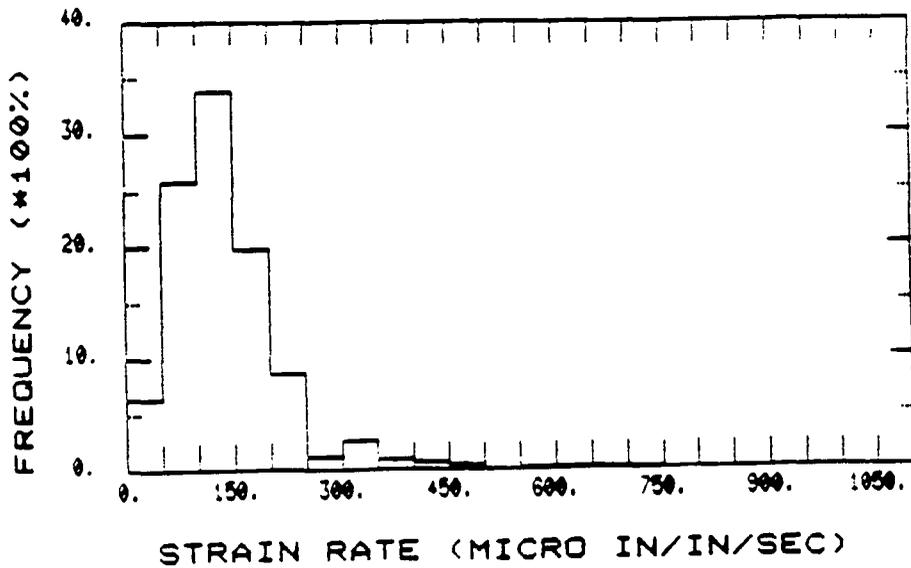determine the stresses near midspan of the girders when each span

Fig. 4 Strain Rate Distribution - Bottom Flange - Midspan
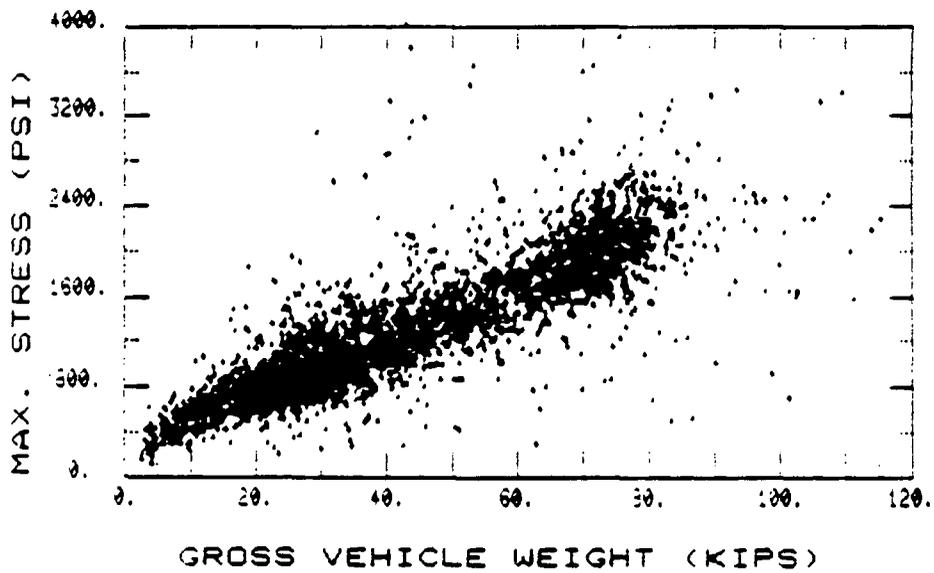of Fascia Girder of Bridge 1

Fig. 5 Maximum Stress vs GVW - Bottom Flange - Midspan of
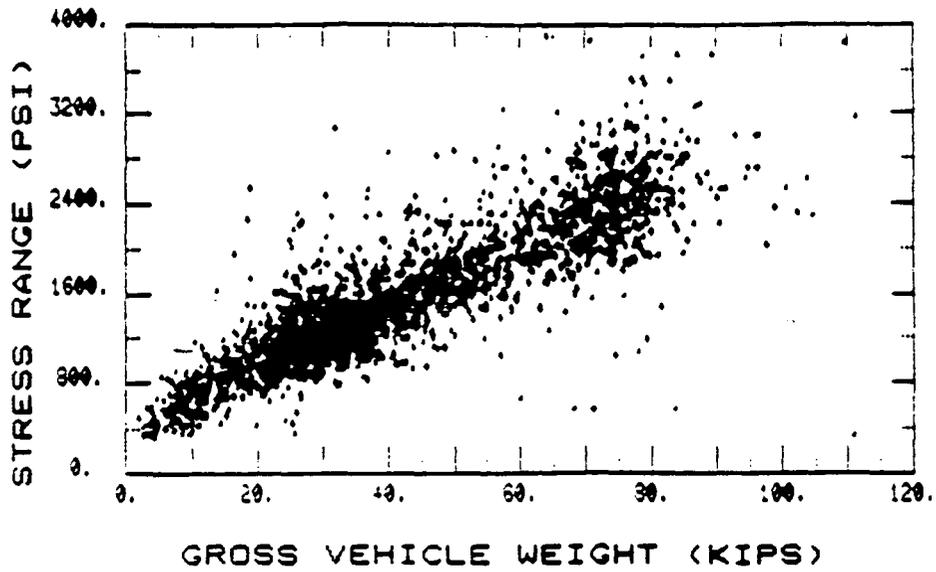First Interior Girder of Bridge 1

Fig. 6 Stress Range vs GVW – Bottom Flange – Midspan of
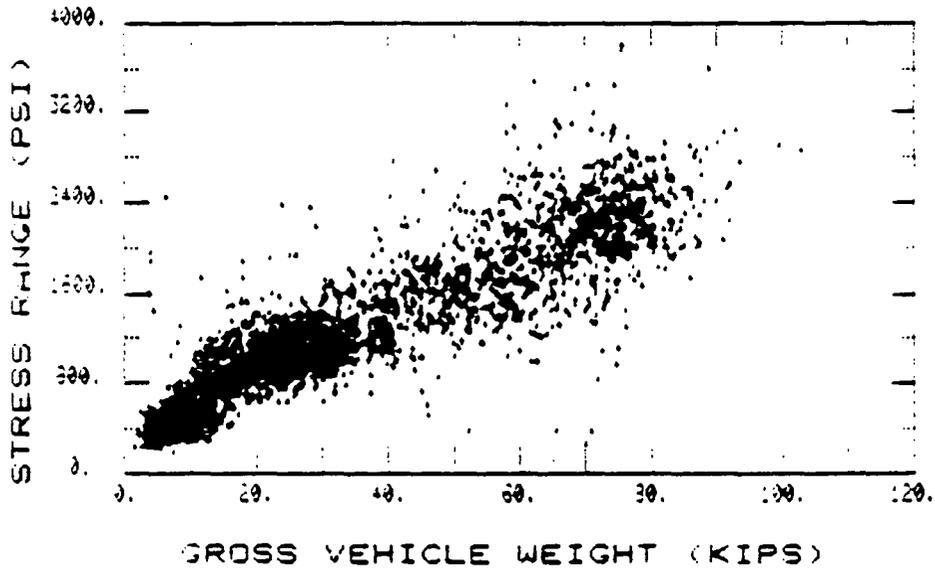First Interior Girder of Bridge 1

Fig. 7 Stress Range vs GVW – Bottom Flange – Midspan of
First Interior Girder of Bridge 3

was statically loaded by one of the approximately 80 kip trucks which crossed the span during the field study. These girder stresses were then compared with the actual girder stresses produced by the truck and with girder stresses computed using the AASHTO load distribution criteria. No finite element analyses of Bridge 4 were performed. However girder stresses were calculated using the AASHTO load distribution criteria and compared with the girder stresses obtained in the field study.

Figure 8 shows a comparison of girder flexural stresses near midspan of the girders of Bridge 2. This comparison is quite typical of the results obtained for all three steel bridges. In the figure the values of girder stresses shown as points on the upper two solid lines are computed in accordance with the 1983 AASHTO specifications for HS 20 loading, but assuming both composite and non-composite construction. Since this bridge was constructed in 1951, fascia girder stresses are also computed using the pre-1957 AASHTO specifications for HS 20 loading and for both composite and non-composite construction. These stresses are shown as the points at the end of the dashed lines in the figure. Values of girder stresses on the lower two solid lines were obtained from the field study and from the finite element analysis of the complete superstructure, assuming composite construction. The axle weights and spacings of an actual 85.2 kip truck producing both the field study stresses and the analytical stresses are shown at the bottom of Fig. 8. The truck traversed the bridge in lane 1 as shown at the top of Fig. 8.

Stress history studies consistently show that for most truck traffic, measured stresses are below AASHTO stresses, considerably so for many bridges. Only a small portion of the truck traffic, that associated with very high GVW and with multiple trucks events, will produce extreme values which may equal or exceed the design stress.

SUMMARY AND CONCLUSIONS

This paper briefly describes the results of a 30 month research investigation conducted at Lehigh University during which an FHWA owned WIM system was redesigned and used to obtain simultaneous load and response data from 19,402 trucks crossing four in-service bridges in Pennsylvania. The Lehigh system is designated the WIM+RESPONSE system throughout the paper.

The WIM+RESPONSE system was designed to obtain simultaneous truck weight and bridge response data which can be used for a
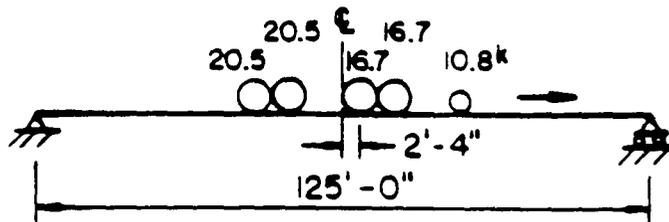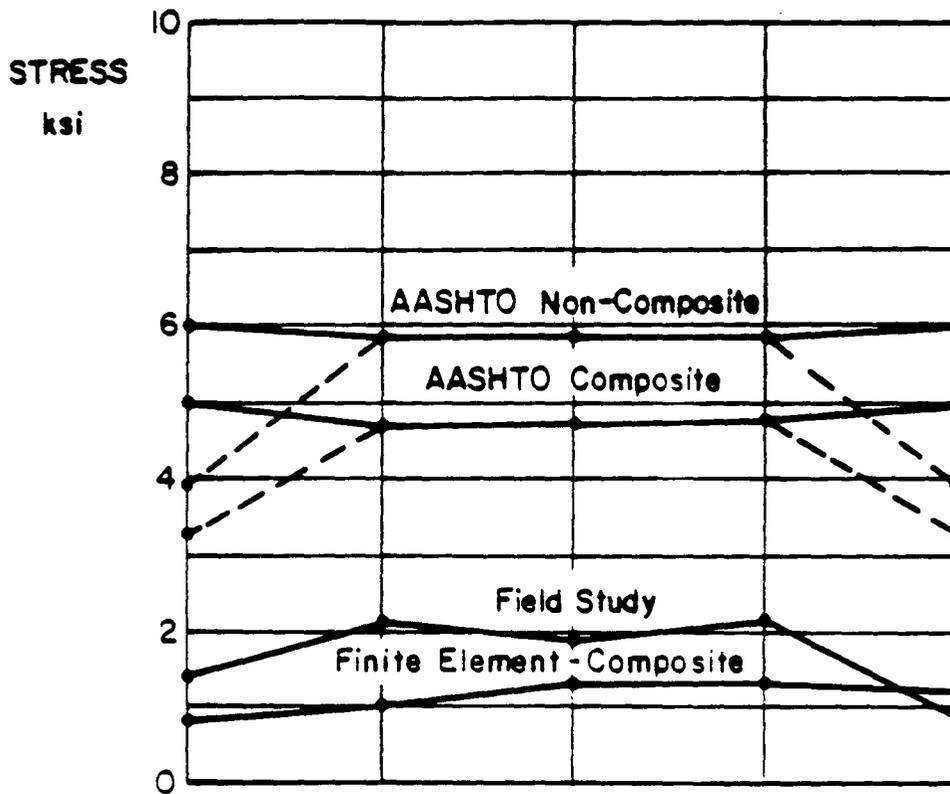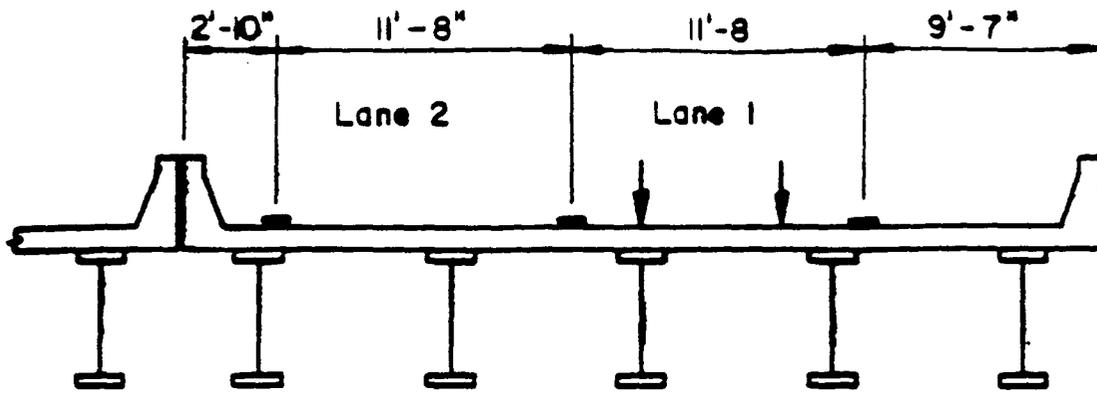
Fig. 8 Comparison of Girder Stresses – Bridge 2

detailed evaluation of the structural performance of in-service bridges as well as for needed improvements in analysis and design procedures and specifications. Specific needs which can be addressed by the WIM+RESPONSE system include GVW distributions, stress range distributions, strain rates, maximum stresses, load distribution and dynamic effects, among others.

The WIM+RESPONSE system was used in a field study to obtain simultaneous truck weight plus bridge response information from four in-service bridges in Pennsylvania. Three of the bridges have rolled, riveted or welded, steel, multiple girder, simple spans and include composite and non-composite construction, both right and skew. The fourth has composite, prestressed, multiple I-girder, simple spans with skew. Information obtained from the field study is briefly summarized in the paper with respect to GVW distributions, stress range distributions, strain rates and maximum stresses. The GVW distributions are in agreement with other studies. The stress range distributions do not have the same shape as the GVW distributions as has been assumed. Maximum live load stresses are consistently below the design stress.

Analysis of the four in-service bridges were also performed in the study. For the steel bridges, girder stresses, computed using the AASHTO load distribution criteria were compared with stresses obtained from a detailed finite element analysis of each complete superstructure and with stresses obtained in the field study. For the prestressed concrete bridge girder stresses are computed by the AASHTO load distribution criteria and compared with stresses obtained in the field study. The paper presents a comparison of girder stresses for one of the steel bridges. As expected, measured stresses compare favorably with those obtained from a finite element analysis of the complete superstructure, while both are somewhat lower than girder stresses computed by the AASHTO load distribution criteria.

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