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C. N. Kostem

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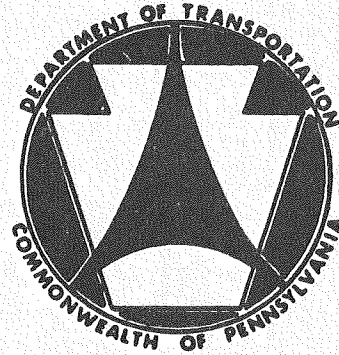
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OVERLOADING OF PRESTRESSED CONCRETE I-BEAM HIGHWAY BRIDGES

FINAL REPORT
Celal N. Kostem

Research Project 77-2: Implementation of Program BOVA

Fritz Engineering Laboratory Report No. 434.3

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Project 77-2: Implementation of Program BOVA

FINAL REPORT

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I-BEAM HIGHWAY BRIDGES

by

Celal N. Kostem

This work was sponsored by the Pennsylvania Department of Transportation, and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the view of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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ABSTRACT

This report presents the summary of the findings of the research program on the overload response of simple span beam-slab highway bridges with reinforced concrete deck and prestressed concrete I-beams. Specific recommendations are provided for bridge engineers, bridge inspectors, and overload permit officers in order to minimize the adverse effects of overloaded vehicles. Guidelines are provided to identify the load levels which can traverse the bridges without violating the serviceability limits.

This report contains the summary of the findings of two extensive research programs, which included parametric studies on overloading. The detailed description of the case studies and the analytical research are referenced in the report for further in-depth review of the investigations.

1. INTRODUCTION

Most bridges are occasionally loaded beyond the load levels for which they were designed. The observations and forecasts made by the bridge engineers and investigators clearly indicate that the magnitude of the overloading, both in terms of the weight of the vehicles involved and the frequency of the occurrence, have increased and will continue to do so (Refs. 3, 4, 7 and 8). It is also recognized that the employment of the "ultimate strength", "load factor," or "load and resistance factor" approaches in the design or rating of the bridges will not alleviate the problems associated with the overloading phenomenon (Ref. 8). The overloading of the bridges and the actions to be taken to limit the vehicular weights, axle weights, or the axle geometry can best be interpreted in light of the serviceability limits that can be adopted for the bridges (Refs. 9 and 11).

A bridge designed for standard HS20-44 design vehicle might be overloaded if the gross weight of the vehicle under consideration is less than the design vehicle, but has closely spaced axles with large axle loads. Conversely, the same bridge will not be adversely overloaded if the new vehicle under consideration is far heavier than the standard design vehicle, but has multiple well-spaced dollies with each dolly having many axles and each axle having many wheels. However, if this vehicle is placed on a very long span bridge then the overloading will again be a

critical issue. In the definition of any "permissible" overloaded vehicle, it is imperative to consider the bridge, the serviceability limits and the vehicle simultaneously. The complexity of this problem inevitably leads, for the sake of simplicity, to some limitations being imposed on the bridges as well as the vehicles, such that the obtained results can be implemented.

1.1 Current Specifications Governing Overloading

The present specifications for the design of highway bridges, i.e. "Standard Specifications for Highway Bridges" (Ref. 1), and the recommended practices for the rating of the highway bridges, i.e. "Manual for Maintenance Inspection of Bridges" (Ref. 2), do not contain specific provisions to consider the very high degree of structural indeterminacy of simple span prestressed concrete I-beam bridges. The omission of the consideration of this indeterminacy leads to the exclusion of the redistribution of the stresses and the loads in a bridge superstructure when it is loaded beyond the linear elastic response range. The research summarized in this report employed the methodology which fully incorporated the structural interaction amongst various components of the superstructure, and considered both the linear elastic and post-linear elastic response characteristics of the superstructure (Refs. 4, 8, 14 and 16). This corresponds to a more realistic assessment of the structural response of the bridge superstructures. It also permits a realistic estimate of the recoverable and nonrecoverable damage to the bridge deck slab as well as the beams.

The current provisions specified by AASHTO (Refs. 1 and 2) and the state agencies in the definition of the rating of the bridges and the activities leading to the issuance of the overload permits are not fully satisfactory. In the absence of more widely adopted guidelines the use of the simplified rules and/or judgmental decisions, though undesirable, is fully understandable. The material contained in this report does not suggest design changes to accommodate heavier overloaded vehicles. The suggestions and recommendations contained herein are primarily for the use of bridge engineers who are involved in the estimation of the strength of the bridges, engineers, and/or other personnel who regulate the overload permit operations. The recommendations regarding the permit operations do not necessarily require major alterations in the current policies and practices. The recommendations could be implemented, where appropriate, for realistic processing of overload permits. The recommendations will provide refined technical tools and guidelines, as compared to the current practice of educated guesses.

1.2 Objectives of the Reported Research

The research project, "Overloading Behavior of Beam-Slab Type Highway Bridges," (Pennsylvania Department of Transportation Research Project 71-12) was aimed at the determination of the overload response of simple span beam slab type highway bridges with reinforced concrete deck slab and prestressed concrete I-beams (Ref. 4). The investigation was to be carried out to predict the bridge response due to the live loading up to

the collapse of the superstructure. The research resulted in a detailed finite element program to simulate the nonlinear behavior of the superstructure, called Program BOVA (Bridge Overload Analysis), and a parametric study on the overload response of select bridge configurations subjected to predefined overload configurations. The findings of this research were presented in a number of reports and publications; the detailed description of each is included in Ref. 8.

At the conclusion of the above referred research it was noted that three specific areas required further work and additional investigations:

- (1) Simplification of the input and output options of Program BOVA,
- (2) Additional parametric studies on the overloading of bridges, and
- (3) Development of recommendations for implementation based on the findings.

The need for the additional investigation led to the initiation and conduct of the research project "Implementation of Program BOVA" (Pennsylvania Department of Transportation Research Project 77-2) (Ref. 7). The results were reported in two interim reports (Refs. 9 and 10), and in this final report.

1.3 Computer Program BOVAC

In accordance with the objectives of the research program extensive modifications were made to the input and output options of computer

program BOVA (Refs. 13 and 15). In view of the extensive changes, and also in view of the heavily prestressed concrete bridge orientation of the modified program, the resulting computer program was acronymed BOVAC (Bridge Overload Analysis-Concrete). All current prestressed concrete I-beam sections employed by the Pennsylvania Department of Transportation were included within the program, thereby eliminating the need for the definition of detailed design parameters to use the program. Furthermore, a number of default options have been built-in to permit the use of the program by individuals with marginal background in bridge engineering. The output options of the program were also custom-tailored to fit the needs of the users such that if the program is used only for the permit application, a few page printout is provided regarding the acceptability of the permit application. Provisions were also made for the detailed computer printout to enable the bridge engineers to have an in-depth study of the stresses and deformations of the superstructure for unusual cases. A detailed user's manual for computer program BOVAC, which also constitutes one of the interim reports of this research project, was prepared (Ref. 9). In the organization of the report, in compliance with the recommendations of the sponsoring agencies, extensive introductory tutorial material were included. This permitted the study of this report alone for the use of the program by those who are not extensively involved with the technical aspects of overloading (Ref. 9).

1.4 Additional Parametric Studies

Earlier parametric studies on the overloading behavior of prestressed

concrete I-beam bridges have indicated the need for additional information on specific issues (Refs. 5 and 7), as follows:

- (1) Use of exterior lane, i.e. right-lane, vs. interior lanes for overload traffic,
- (2) Effects of beam spacing on the overload response of bridges,
- (3) Effects of deck deterioration, in the form of the loss of concrete cover over top deck reinforcing bars, on overload response of bridges,
- (4) Effects of "lower strength" deck concrete on overload response of bridges,
- (5) Combined effects of deck deterioration and lower strength deck slab concrete on overload response of bridges, and
- (6) Overload response of bridges to heavy four wheel construction or mining vehicles.

Detailed investigations of the above referred areas were presented in Ref. 10, thus no attempt will be made to redescribe the pertinent details of the case studies. Highlights of the findings and the specific recommendations based on the parametric investigations will be summarized in the next chapter of this report.

1.5 Organization of the Report

This report provides a summary of the findings of the research programs, "Overloading Behavior of Beam-Slab Type Highway Bridges" and

"Implementation of Program BOVA." Emphasis in the selection of the materials to be included herein is based on their prospective implementability by the Pennsylvania Department of Transportation. Other findings and conclusions, regardless of their possible importance, have not been included in this report, if they were deemed to have less of a chance for possible short term positive impact and immediate implementation. It should, however, be noted that for an in-depth understanding of the two research projects referred to previously, study of the interim reports is of great importance (Refs. 5 - 10, 13 and 14).

1.5.1 Basic Concepts and Terminology

The investigations on the overloading behavior of beam-slab type highway bridges with prestressed concrete I-beams have indicated that the best measure of distress of the superstructure is the cracking of the deck slab concrete and the cracking of the concrete cover of the prestressing strands in the beams (Refs. 5, 8, 9, 13 and 16). The compressive stresses in the deck slab concrete and in the beams were not large enough to cause permanent damage. The changes in the stresses in the deck reinforcing bars and the prestressing strands due to the overloading were not large enough to be used in monitoring the structural "damage." The deflections of the beams or the bridge in general were too small to cause concern in the violation of the live load displacement limits prescribed by AASHTO Specifications (Refs. 1 and 2).

It is observed that in short span bridges, about 40 feet or less in span

length, the interface shear between the beams and the slab can be of concern in a very few cases. The excessive interface shear can be noted near the supports. The scarcity of the "high shear induced" damage to the superstructure, as compared to the flexure induced damage, did not warrant further investigations of interface shear.

The cracking of the deck slab concrete requires additional considerations. Most of the cracking is in the form of "working cracks," that is, as the axles are traversing a certain area cracks develop and open and after the passage of the axles the cracks close. The closing of the cracks is simply due to the elastic rebound of the deck slab and, especially, the prestressed concrete beams. If the slab and the beam(s) are to lose part of their elastic rebound capability, than the cracks will not fully close.

The investigation did not consider the possible cumulative aspect of the crack growth, that is, if, for example, a one inch deep crack develops in the deck slab due to the first passage of the vehicle, what will be the depth of the crack after, for example, the 1,000th passage of the same vehicle. It is expected that there will be a noticeable increase in the crack depth. Due to the absence of universally accepted and universally applicable rules or formulae for concrete at the time of the conduct of the reported research, this issue was not considered. Frequent loading of a highway bridge by a vehicle, which can crack the deck, will cause cumulative effects. Until the AASHTO Specifications can quantify the allowable frequency of overloading, any propositions on the part of the

researcher will be speculative (Ref. 1).

In the definition of "unacceptable damage" to the bridge superstructure the cracking of the concrete cover of the prestressing strands in the beams was taken as the limiting factor. The cracking of the concrete cover is considered to be unacceptable as far as the maintenance of the structural integrity and the serviceability of the prestressed concrete members. All references in the report to "beam cracking" indicate the cracking of the concrete cover of the strands.

All references made to the cracking of the deck slab concrete are indicated by the depth of the crack. The crack initiation refers to cracks having depths less than one third-to-a half the thickness of the concrete cover. In the case of deteriorated decks, since the top concrete cover was already removed, the crack initiation refers to the cracks that have penetrated beyond the reinforcing bars.

Other types of cracks of the deck slab concrete are (1) the cracks that are one third the thickness of the slab, and (2) the cracks that are half the thickness of the slab. The second type of cracks is found to be highly undesirable. The stress blocks in these types of cracks have very high stress gradients through the depth of the uncracked portion of the slab. Any gross material or construction imperfections in the deck slab can easily cause spread of the cracks which are half the depth of the slab.

2. OBSERVATIONS AND FINDINGS

The interim reports of the research projects, "Overloading Behavior of Beam-Slab Type Highway Bridges" and "Implementation of Program BOVA," contained a number of findings, observations and recommendations, each of which was accompanied by detailed discussions and pertinent references to the material which led to the findings in question. In order to relate the recommendations and findings to each other, and also to make this report a summary report for the above referred projects, important findings are summarized in the following section.

2.1 Earlier Research Findings

The observations reported below are for bridges designed in accordance with the AASHTO Standard Specifications for Highway Bridges (Ref. 1) and the prevailing design standards in the Commonwealth of Pennsylvania (e.g. Ref. 12). If the design dimensions of the bridge superstructure are substantially different than those that will be obtained through the application of the "AASHTO Standard Specifications," it is possible that the observations listed herein may not be fully applicable.

Two of the observations listed below should especially be carefully reconsidered if they are to be extended to the bridges that are designed in accord with specifications and provisions that are substantially

different than the "AASHTO Standard Specifications for Highway Bridges" (Ref. 1). They are "the stresses in reinforcing bars of the bridge deck slab" (Item #2) and the "structural response mode and the mode of damage initiation to the bridge deck slab" (Item #10). If the amount of reinforcing steel in the bridge deck slab is substantially less, or if the thickness of the deck slab is substantially less than the values prescribed by the "AASHTO Specifications" (Ref. 1), then the stresses in the reinforcing bars may not be low or the structural response mode will not be primarily flexural, respectively.

(1) Damage to the deck concrete in the form of cracking is the first sign of distress due to the overload. These cracks are roughly parallel to the beams, and can take place at the bottom of the slab near the mid-spacing of the beams and initially occur in the vicinity of the load at the bottom and at the top of the slab near the top flanges of the beams (Refs. 5, 8, 10, 14 and 16).

(2) Stress levels in the reinforcing bars of the slab are low, even after substantial cracking of the concrete of the deck slab (Refs. 14 and 16).

This observation is true for the bridges that are designed in accordance with the prevailing AASHTO Standard Specifications for Highway Bridges (Ref. 1) and the bridge design practice in the Commonwealth of Pennsylvania (e.g. Ref. 12). If the amount of reinforcing bars in the bridge deck is substantially reduced, it is possible that the stresses in the reinforcing bars will be higher, and may need to be considered in

the case of overloading.

(3) Damage to the beams in the form of cracking of the concrete cover of the strands near the midspan initiates only after substantial cracking of the deck concrete (Refs. 5 and 10).

(4) After the initiation of the cracks in the bridge deck the propagation of the cracking is not limited to an area which is immediately under the vehicle. For increased vehicular weights shallow cracks throughout the "unloaded" parts of the deck develop, rather than the deepening of the initial cracks. For further increased overload levels the initial cracks deepen (Refs. 5, 10 and 14).

(5) The overload response of bridges is adversely effected, not necessarily by the gross weight of the vehicle, but by the (a) increase in axle loads, (b) decrease in number of tires per axle, and (c) decrease in axle spacing, and increase in the number of closely spaced axles, as in the case of dollies (Refs. 5 and 10).

(6) Bridge decks are not susceptible to shear punch failure. Prior to the attainment of the load level that can cause shear punch failure, the deck will undergo almost total damage due to flexure (Ref. 6).

The primary structural response mode, as well as the failure initiation mode in the deck slab, is due to the flexure. This observation is applicable to the deck slabs designed in accordance with the current

AASHTO Standard Specifications for Highway Bridges (Ref. 1) and the current bridge design practice in the Commonwealth of Pennsylvania (e.g. Ref. 12). If the bridge decks are designed using provisions that are substantially different in basic design philosophy from that employed in the AASHTO Specifications, it is possible that the mode of failure of the bridge decks could be due to other modes, such as the shear punch failure.

(7) Shear stresses in the beams are not critical. However, their presence may amplify the effects of the flexural stresses - which is the primary source of damage to the beams (Refs. 14 and 16).

(8) Interfacial shear between the beams and the slab may reach critical values near the supports for short span bridges (40 ft. span length or less). Prior to the interfacial shear damage the deck slab undergoes extensive flexural cracking (Refs. 5 and 10).

(9) Crushing of the slab or beam concrete is very unlikely. Through the redistribution of stresses additional concrete cracking takes place rather than the stress block causing crushing (Refs. 5, 10 and 14).

(10) If the bridge deck is permitted to undergo slight cracking the overload vehicle that can cause this damage is far heavier than the "overload" vehicle which will not cause any cracking, with the exception of hairline surface cracks.

It is important to note that bridge deck slab develops shallow hairline cracks even if the bridge is subjected only to the "regular truck traffic." These cracks are essentially surface cracks with depths of about one sixth to one third of the thickness of the concrete cover of the reinforcing bars.

It is essential to recognize the ramifications of permitting the uncontrolled and/or "frequent" passage of the overloaded vehicles which can cause the development of working cracks, not the hairline surface cracks just described. If the frequency of loading, which has not been quantified in the AASHTO Specifications (Ref. 1), is too "high," then there exists a high degree of probability that these cracks will grow and penetrate further into the deck slab. The cracks can even penetrate beyond the reinforcing bars of the deck. Such a phenomenon could be considered unacceptable as far as the serviceability criteria of the bridges (Ref. 11).

2.2 New Research Findings

The major activities conducted within the framework of the research project, "Implementation of Program BOVA," can be broken into three categories: (a) those pertaining to the use of computer program BOVAC, (b) those pertaining to the use of overload directories, and (c) specific recommendations emanating from consideration of the parametric study reported and from the previously conducted parametric study.

2.2.1 Computer Program BOVAC

The user's manual has been written in as simplistic terms as the program and the subject area permit to enable the use of the program by those with no background on "finite element method," which is the basis for the program, and no prior bridge engineering expertise. The computer program BOVAC is extremely easy to input. Some of the input information have been set to default values, thereby simplifying the input of the program even further.

All standard prestressed concrete I-beam shapes, deck reinforcement details, etc. are incorporated into the program. This permits the user to define either the Pennsylvania Department of Transportation or the Standard FHWA sections via simple alphanumeric name. The program proceeds with all internal calculations to define the bridge. If the program's "beam library" is to be expanded, it requires additional, but extremely simple programming. This operation can be undertaken by any computer center personnel with very little effort. The program can accept any form of "solid" beam section, i.e. box-beams are not acceptable, having a vertical axis of symmetry, e.g. I-, T-sections, rectangles. The program can be modified to comply with the bridge design practices of other states. This can be accomplished via simple modifications in the computer program. It is also possible to have the program modified in an all-inclusive mode, such that through the definition of the pertinent design practice, i.e. the design practices of various states, the computer program can pick-up the correct logic path

to identify the beam shapes and design details.

The computer program does not have any limitations as far as the vehicle configuration is concerned, i.e. there are no limitations in terms of number of wheels per axle, and number of axles and their spacing. The only limitation to be recognized is that each loading case for a given bridge corresponds to one case study, i.e. one "computer run." If the bridge is to be analyzed for various vehicles and/or various placement of vehicles on the bridge, then each loading of the bridge requires slight modification of the input data, and resubmission of the problem for execution.

Currently program BOVAC employs the "fixed format" input. This is the type of FORTRAN compiler currently used by most agencies involved with bridge analysis and design. It is also recognized that in the very near future this type of FORTRAN compiler will be abandoned in favor of FORTRAN77 compiler, which usually employs the "list directed input" option. Recompilation of program BOVAC using FORTRAN77 will eliminate the need for careful FORMATTING of the input data. Such simplification will have a great appeal to the users of the program. The needed recompilation is a simple process which can be done by any computer center operations personnel.

Regardless of the simplicity of the input and output of the program, the execution time of the program, especially for wide bridges and complex loading conditions, is unacceptably long for inputting the data from a

remote terminal, execution of the job, and display of the results on the remote terminal during one terminal session. Thus, it is recommended that the input to the program be undertaken via remote terminal, or the batch site if preferred, and sign-off the terminal session. After the completion of the execution of the job, the required time for which varies depending upon the workload of the computer at that given time, the user can later sign-on, and have the results displayed at the remote terminal site. The remote terminal printing should be considered only for the "short printout" option. The "long printout" option requires a long terminal session to print the full output. In the case of the long printout option it is desirable that the results be printed at the central site using a high speed printer, and the output dispatched or mailed to the requester.

For routine overload permit operations for simple span prestressed concrete I-beam bridges the execution time of program BOVAC is still too long to be considered an expeditious tool. The use of overload directories, as described in the following sub-section, will eliminate the need for routine applications of the program. However, if the overload permit application is for a type of bridge not included in the overload directories, or more importantly, if the vehicle in question does not resemble the standard overload configurations included in the overload directories, than the use of computer program BOVAC becomes justifiable.

2.2.2 Overload Directories

The research projects, "Overloading Behavior of Beam-Slab Type Highway Bridges" and "Implementation of Program BOVA," resulted in the detailed investigation of 45 and 28 case studies, respectively. These case studies are presented in a tabular form in two reports (Refs. 5 and 10). These case studies were labeled as "overload directories." Examples of the use of the overload directories for the overload cases where both the vehicle and the bridge are similar to those included in the overload directories were presented in Ref. 5. The same reference also contains examples of the application of overload directories where neither the bridge nor the vehicle is similar to the cases included in the reports. Guidelines for the use of interpolation between the case studies, and limited use of extrapolation, were also included in this reference.

It is recognized that the use of the overload directories, as prescribed in Ref. 5, is superior to any other method, with the exception of the use of computer program BOVAC, in the processing of overload permit applications and in bridge ratings. As has been discussed previously, these directories are applicable only to simple span prestressed concrete I-beam bridges.

In this report simplified guidelines are presented in the definition of the allowable axle weights. However, since these values are based on a statistical regression (an averaging scheme), the use of overload directories is always more accurate than the simplified expressions presented in the later sections of this report.

2.3 Research Findings Based on the Parametric Studies

Seventy-three case studies included in two reports contain extensive information on the linear elastic and post-linear elastic (representing damage initiation and propagation to the bridge superstructure) overload behavior of the types of bridges in question. Attempts have been made to arrive at simplified formulae which will be representative of the case studies. Extensive statistical analyses have indicated that it is not possible to develop simple expressions for various types of bridge vs. loading combinations, which will be applicable to all cases with a high degree of reliability. It was then decided to present the results in the form of simple rules that have higher degrees of reliability. In all the following subsections references have been made regarding the extent of the reliability of the findings and recommendations.

It should also be noted that all results that are quantified either in terms of percentages, or in terms of axle weights, are based on the static loading of the bridges. The "impact factors" were not incorporated into the analyses, because it is assumed that impact loading can be controlled by speed regulation through the permitting process.

2.3.1 Choice of Traffic Lane for Overloaded Vehicles

In the case of the traverse of a bridge by an overloaded vehicle a decision needs to be made: should the vehicle use the slow lane, i.e. rightmost traffic lane over the exterior beam, or should it use one of the interior lanes? The answer to this question was sought, not from a "traffic engineering" standpoint, but from bridge engineering. It was found that if the vehicle uses an interior lane, as opposed to the exterior lane, vehicle weight can be 10% higher for short span bridges (40 ft. span length), and 5% higher for medium-to-long span bridges (70 ft. span length). In arriving at these percentages the crack initiation of the deck slab concrete was used as a measure. If the amount of damage that the slab will have to sustain is neglected, and only the beam cracking is employed as a measure of control, then both for short and longer span bridges the vehicle on the interior lane can be about 15% heavier than the vehicle on the exterior lane to cause a similar damage.

The studies were repeated for bridges where the cylinder strength of the deck slab concrete is 500 psi (about 13%) less than the design value. This case corresponds to one where poor field labor or material were used in the construction of the deck slab. For these types of poorly constructed bridges the selection of the interior or the exterior lane resulted in the same percentages listed above.

It can be concluded that since extensive slab damage will not be acceptable from the standpoint of the serviceability of the bridge, the use of 15% rule does not carry any practical significance. However, the

percentages related to the deck damage initiation do have practical ramifications. Since these percentages are not large enough for routine overload traffic the use of the interior or the exterior lane does not seem critical. The use of interior lanes is always advisable, if the lane selection is not critical. In the case of heavy overloads that will have to traverse the bridge the preference must be given to the interior lanes, thereby providing an additional factor of safety.

2.3.2 Beam Spacing

In the majority of the parametric studies the beam spacing was taken as 7 ft. - 6 in.. It is recognized that the deck slab between the beams undergoes substantial flexure in the lateral distribution of the live load. Thus the effect of the beam spacing needs to be investigated. Additional studies were conducted for two different types of dollies, for 40 ft. and 70 ft. span lengths, for original design of the deck slab and for deck slab with concrete cylinder strength 500 psi (about 13%) less than the original design, i.e. poor field construction, and for two different beam spacings, 7 ft. - 6 in. and 6 ft. - 6 in.. The change in the beam spacing is about 15%. It is noted that the load levels which cause the deck crack initiation for the bridges with closer beam spacing are about 4% less than for bridges with wider beam spacing. In view of this small percentage, it could be assumed that for the bridges built in accordance with the current specifications (Refs. 1 and 12), beam spacing plays a minor role in the load carrying capacity, if the serviceability limits are observed.

If the extent of the damage, i.e. substantial cracking of the deck concrete, to the bridge deck is ignored, then the bridges with closer beam spacing carry about 10% higher load than the bridges with wider beam spacing. The controlling parameter here is the beam cracking. Any recommendations that will be based on beam cracking should not be employed.

2.3.3 Crack Initiation in Deck Slab Concrete

The research have indicated that for static axle weights of about 25 kips the bridge deck should not exhibit any discernible cracking. This observation is true for the cases where there are at least four wheels per axle and no more than four axles per axle group (or dolly). Furthermore the axle spacing should not be less than 4 ft. The standard deviation for the above axle loading is 6 kips.

With the above referred limits on the wheel and axle geometry, it was found that if the axle weight is about 29 kips bridge deck cracking will initiate and will crack the concrete cover at the top of the top reinforcing bars and the bottom cover below the bottom reinforcing bars. Cracking of this magnitude may be considered as acceptable for infrequent overloading of the bridges.

If the axle weights are about 56 kips than the cracks in the deck slab will penetrate half the slab depth. Damage of this magnitude, regardless

of its recoverability, is too severe.

Extensive statistical analyses were conducted to relate the axle weights to the vehicle and bridge geometry. Regardless of the type of independent regression variables chosen, none of the formulae were acceptable, i.e. with very small coefficient of determination.

2.3.4 Beam Cracking

Beam cracking takes place after substantial cracking of the deck slab concrete. To allow beams to crack, or the acceptance of a load level that is just below the load level that causes the beam cracking, implicitly indicates permission to allow substantial deck slab cracking.

The formulae and the axle weights given in this section should not be used as they are for permit operations. These loads cause the cracking of the beams and prior to the cracking of the beams substantial cracks develop in the slab. These formulae could very well be used to identify the axle weights that are totally unacceptable .

The research indicated that for bridges with 100 ft. span length the axle weight that causes the beam cracking is 75.6 kips. This value has a standard deviation of 12 kips. For the bridges with 70 ft. span length the corresponding axle weight is 65.6 kips; with standard deviation of 10 kips. And, for bridges with 40 ft. span length the axle weight in question is 51.7 kips; with standard deviation of 5 kips.

The above observations and the following formulae are applicable to the cases where there are at least four wheels per axle and no more than four axles per axle group (or dolly). Furthermore, the axle spacing should not be less than four feet.

Using the above values for the axle weights a regression analysis was conducted to relate the axle weight to the span length. The following formula was obtained:

$$P(\text{axle}) = 36.4 + 0.4 L$$

where

$P(\text{axle})$ = axle weight that causes the beam cracking (in KIPS.), and

L = span length (in FEET).

The coefficient of determination for the above formula is 0.99, 1.00 being the perfect curve fit. It should be noted that this formula is not as perfect as it looks. In the development of the formula the axle weights had standard deviations that are far less perfect than the coefficient of determination for the formula indicates.

Additional regression analyses have resulted in a number of formulae. The thrust of the additional studies was to use the "raw" data as input to the regression analysis. Some of the obtained results have had totally unacceptable coefficient of determination. These formulae were

rejected. However, one of the formulae with acceptable coefficient of determination, 0.91, was:

$$P(\text{axle}) = 90.0 - 0.17 \text{ NW} - 16.0 \text{ NA} + 0.47 \text{ L}$$

where

$P(\text{axle})$ = axle weight that causes the beam cracking (in KIPS.),

NW = total number of wheels,

NA = total number of axles, and

L = span length (in FEET).

In the above formula the definition of the $P(\text{axle})$ is the same as in the previous formula. For both formulae presented in this subsection the limits regarding the number of axles, wheels per axle and the axle spacing referred to previously should be observed. Additionally, for the second formula special attention should be given to the axle and wheel counting. The counting should be made for the axle group which will be placed at the midspan of the bridge. If a vehicle consists of the front axle, drive axle group, and the rear axle group, and if these axle groups are spaced somewhat wider than one third the span length of the bridge, than the count should be made for only one, preferably the heaviest, axle group.

2.3.5 Reserve Strength

An important concept that the bridge engineer and permit officer can employ in the determination of the existing overload carrying capacity of a given bridge is the reserve strength of the superstructure. The material presented herein assumes that reliable information is obtained from the field inspection of the bridge. Without such information, the employment of the material presented herein will be highly speculative.

If the deck slab exhibits longitudinal cracks at the top and/or bottom of the slab, and also if these cracks are rather fresh, it can be assumed that these cracks have formed due to the transverse bending of the slab. If these cracks are known to be caused by the "overloading" of the bridge, and also if the magnitude of the load levels that caused this cracking is either known or can be reliably estimated, and also if the depth of the cracks is less than about one third to a half of the concrete cover of the reinforcing bars of the deck slab, the following approximate, but practical rules can be used:

(1) If the bridge is to be subjected to new overloads that are about 45-50% higher than those that caused the above referred cracks, then the depth of the new cracks will at least reach and penetrate beyond the reinforcing bars of the slab.

This observation should not be construed as a "permit" to increase the current overload levels by the said percentage. It only relates the vehicular weights that can cause slight cracking to the bridge deck to

those that can cause cracks deep enough possibly to violate the serviceability limits .

(2) If the bridge is to be subjected to new overloads that are about twice the loads that caused the slight cracking, then the new crack depths will be at least half the depth of the slab thickness.

Again this observation only relates the vehicular weights that can cause slight cracking to the bridge deck to those that can cause unacceptably deep cracks.

Based on the investigation of the stress blocks of reinforced concrete slabs having cracks as deep as half the depth, the structural integrity of the slab will be highly questionable. Possible imperfections in the slab concrete can cause the rapid growth of these cracks.

No overloading should be permitted which can cause cracks in the slab which have depths of half the slab thickness.

(3) Observations #1 and #2 should not be employed in a multiplicative manner, i.e. as a chain rule.

If, for example, a 100 kip vehicle caused cracks of about half the depth of the concrete cover of the slab deck reinforcing bars, the load level that can cause the development of the cracks half the depth of the slab thickness is not $P=(100 \text{ kips})(1.45-1.50)(2.) = 290 - 300 \text{ kips}$. The

above guidelines merely mean that if the new load is about 140 to 150 kips the cracks will penetrate beyond the reinforcing bars of the slab. And, if the new load is about 200 kips the cracks will have a depth of half the thickness of the slab. The former may violate the serviceability criteria, and the latter will impair the structural integrity of the deck slab.

2.3.6 Effect of Deck Deterioration

It is well recognized that some bridges, due to heavy traffic, and a number of other causes, show extreme wear on the concrete cover above the top reinforcing bars of the deck slab. This is especially noticeable on the slow lanes, i.e. exterior traffic lane. To obtain an extreme limit to the reduction of the load carrying capacity of the bridges, deck deterioration was simulated by removing all concrete above the top reinforcement for the full bridge. The analyses were conducted and were compared with the same bridges without any imposed damage.

It was found that, as compared to the original bridges, the deck slab of the bridges with the above defined deterioration will start cracking at about 80% of the load. It should be noted that for the bridges without deterioration the cracking barely reaches the reinforcing bars. In the case of bridges with deterioration the reinforcing bars are already exposed. The same percentage holds for the penetration of the cracks through half the depth of the slab thickness.

In the case of bridges with the deterioration described above, the cracking of the beams starts at about 90% of the load level that would have caused the cracking of the beams in intact bridges.

2.3.7 The Effects of Low Strength Deck Slab Concrete

Due to possible poor construction practices it is possible to have deck concrete with strength less than what was assumed and required by the bridge designer. The compressive cylinder strength of the deck concrete was reduced by 500 psi (approximately 13% reduction), and the full series of analyses were repeated. It was found that there will be approximately a 12% reduction in the load levels that will cause the initiation and propagation of the crack in the deck slab, as compared to the bridge with the original design strength.

The corresponding reduction in the load level which will cause the cracking of the beam is 2%. As has been discussed in previous sections, in view of the serviceability criteria, the important factor to be considered is the 12% reduction, and not the percentage corresponding to beam cracking.

2.3.8 The Effect of Deck Deterioration and Low Strength Concrete

The previously presented two different "imperfections," i.e. removal of concrete cover at the top of the reinforcing bars of the slab and the reduction of 13% in the compressive cylinder strength of the slab concrete, were considered simultaneously. For various loadings and span lengths it was found that there will be a 30% reduction in load levels that cause the crack initiation and propagation in the slab, as compared to the bridges without any imperfections.

The reduction corresponding to the beam cracking load was 12%.

2.3.9 Number of Axles and Axle Spacing

In the design of the parametric studies the axle spacing was not taken as an independent variable. Therefore the effects of the axle spacing can not be quantified.

In all dollies considered the axles were spaced four feet apart. Two distinct types of dollies were considered, one with three axles and the other with four axles. It was observed that the total weight transmitted by these dollies was a better measure in the determination of the damage to the bridge components, than the number of axles.

Even though it can not be quantified, it can be qualitatively stated that:

(1) If the axle spacing is greater than four feet, than the vehicular weights that can cause the types of "damage" to the bridge superstructure will be "less" than those reported in this report.

(2) If the number of axles are fewer than those reported in this report, i.e. two axles rather than three or four, than the vehicular weights, or axle weights, that can cause the types of "damage" to the bridge superstructure will be "less" than those reported herein.

The quantification of the above guidelines requires the conduct of additional limited scope parametric studies, similar to those reported in References 5 and 10.

3. SUGGESTIONS FOR IMPLEMENTATION

Detailed research projects on the overloading behavior of simple span beam-slab type highway bridges with prestressed concrete I-beams and reinforced concrete deck slab have indicated that there is little that can be done to improve the overload carrying capacity, while not violating the serviceability limits, of the bridges designed using the prevailing specifications (Refs. 1, 12 and 17). If the bridge superstructures are designed for heavier live loads at the design stage, as in the case of European bridge design practice, than at the time of application for overload permits the discrepancy between the loading for which the bridge was designed and the loading for which the permit application is made will be less pronounced.

Almost all of the findings of the research projects relate to the activities of the offices that deal with bridge inspection, bridge maintenance, and overload permit applications.

The following sections of this chapter are arranged in such a manner that:

The direct findings of the research programs are listed under the heading of "Findings." The list of findings presented herein is

only a fraction of all the detailed findings presented in the earlier research reports (Ref. 10; detailed descriptions of the other reports are found in Ref. 8). The general findings listed herein are those that will be of immediate interest and use to the bridge engineers, inspectors, and overload permit officers.

The section titled "recommendations" contains some of the findings, which were not listed under the previous subsection, which have potential for immediate implementation and incorporation into the bridge design, rehabilitation, inspection, and overload permit practices.

The section titled "Suggestions for Long Range Planning" contains suggestions for the incorporation of the overload permit operations and the rating of bridges into a computerized data base management system for highway bridges. This data base can include information about the design characteristics, bridge inspection and rehabilitation information, average daily truck traffic and the overloading, etc. of the bridges. Such a unified data base will permit the identification of the criticality of any given bridge for overload permit operations as well as the priority assignment for rehabilitation and replacement.

3.1 Findings

(1) The current knowledge on the deterioration of the prestressed concrete beams in the U.S., where the extensive use of these types of beams started in the late 1950s, and the Central and Western European countries, where the extensive use of these types of beams started after the Second World War, is different. In Western European countries it is now recognized that the prestressed concrete beams tend to deteriorate, despite earlier optimistic projections. In the U.S. any serious concern for the deterioration of the prestressed concrete beams has not surfaced as yet, with the exception of a few pioneering technical papers. Considering the European experience it would be highly advisable that in any and all overload permit applications no compromise should be made for the structural integrity and the serviceability of prestressed concrete beams. No overstressing of these beams should be permitted. A prestressed concrete bridge that shows any "aging" or "deterioration" in the beams should not be subjected to substantial overloading without careful examination.

(2) The weakest link in the overload response of prestressed concrete I-beam bridges is the deck slab. It is noted that any decrease in the cylinder strength of the deck slab concrete results in an almost proportional decrease in the live loads that cause the initiation of the damage to the bridge deck slab. Even though no major actions can be taken for the existing bridges, it is possible, and highly recommended, that deck slab concrete with the highest possible concrete cylinder strength be used in the deck rehabilitation. This strength is related to

the rupture strength of the concrete, thereby effecting the cracking strength of the deck slab.

(3) The stress levels in the reinforcing bars of the bridge deck slab are quite low.

(4) The primary mode of structural response of the slab is the transverse bending. Even if there may be no load on the slab between the beams, the transverse bending of the slab, due to the differential deflection of the beams, is large enough to cause concern.

Shear punch failure of the deck slab is highly improbable. This is due to the transportation industry's approach to the increased vehicular weights, at least in the case of special hauling equipment. Rather than using tires with high internal pressure, i.e. greater than 100 psi, low pressure tires, i.e. less than 80-100 psi internal pressure, are used. Increased vehicular loads in the new hauling equipments are handled through the increased number of axles and increased number of wheels per axle.

(5) The primary mode of damage initiation to the prestressed concrete I-beams is due to the flexure of the beams. Prior to the initiation of any discernible damage to the beams, the bridge deck undergoes substantial cracking. Any prestressed concrete I-beam with cracks at or near the mid-span that are essentially perpendicular to the axis of the beam and at the bottom of the beam requires in-depth assessment of the

causes of such cracking. Any bridge having beams with such cracking without accompanying damage to the bridge deck should be studied to assure that the "quality" of the precast and prestressed beams is not substandard.

(6) Any diagonal cracks at quarter span or near the supports of prestressed concrete I-beams of a bridge should be studied to identify the cause. This issue especially becomes more pronounced if the bridge in question does not have any cracking in the bridge deck slab. The research have showed that prior to the formation of any diagonal cracks in I-beams, there must be substantial cracking in the bridge deck slab. The research showed that cracks of this nature are not encountered, without the accompanying deck cracking, in bridges with span lengths greater than 40 ft. If the span length is about 30 ft., or less, the possibility of cracks as such are theoretically possible, but have not been verified.

(7) The use of exterior lane (right lane or slow lane) versus an interior lane does not substantially change the adverse effect of the overload vehicle to the bridge superstructure. If an additional margin of safety is required the use of an interior lane is preferable.

A vehicle on an exterior lane causes 5-10% higher stresses in the bridge deck slab as compared to the same vehicle on the interior lane. In the case of "two lane twin bridges" both lanes in each direction are essentially "exterior lanes," and the above percentages do not apply.

The vehicle needs to straddle the centerline of the bridge in a given direction in order to qualify for the "interior lane" margin of safety.

(8) Beam spacing does not make an appreciable contribution to the increase or decrease of the load levels which will cause damage to the bridge deck. Closer beam spacings slightly reduce deck slab stresses. Within practical ranges, through the use of the closer beam spacing, as compared to the current design practice (e.g. Ref. 12), 10% additional vehicle load can be accommodated before the cracking of the concrete cover of the strands of the beams. It should be noted that this increase in load tacitly leads to the cracking of the deck slab concrete.

(9) In the overload permit applications the following approximate rules can be used. In all cases the "dolly" under consideration should not have more than four axles per dolly, no less than four wheels per axle, and an axle spacing of no less than four feet.

If the axle weight is 25 kips or less the deck may not exhibit any cracking.

If the axle weight is 29 kips, or more, but much less than 56 kips, the concrete cover of the reinforcing bars of the deck slab will start cracking. Depending upon the magnitude of the axle weight and the imperfections in the deck slab concrete, the cracks may reach the reinforcing bars.

If the axle weights are 56 kips, or more but less than the value given in recommendation #10 in Section 3.2, the cracks in the bridge deck slab will reach half the depth of the slab. This is a totally unacceptable damage.

(10) If the concrete cover of the reinforcing bars of the deck slab already exhibits longitudinal cracks due to the overloading that are about one third to half the depth of the thickness of the cover, and also if the overloading history of the bridge can be estimated, than the new additional overloading will exhibit the following damage:

If the new overload levels are about 45-50% higher than the previously recorded overloading, the new cracking will at least penetrate the full thickness of the concrete cover.

If the new overload levels are about twice the value of the previously recorded overloading, the new cracking will at least penetrate at least half the depth of the bridge deck slab. A damage as such is unacceptable.

(11) If the reinforced concrete bridge deck shows extreme deterioration, such that the concrete cover over the reinforcing bars is essentially "removed" or "ineffective," the load levels which can cause cracks in the deck concrete with depths about one quarter to one third the thickness of the concrete between the top and bottom reinforcing bars are about 80% of the load levels that would have caused crack initiation

in bridges without any deck deterioration.

(12) If due to various reasons the cylinder strength of the bridge deck slab concrete is less than what was required in the design process, the reduction in the load levels to cause cracking in the deck slab is roughly proportional to the reduction in the concrete cylinder strength.

(13) If a bridge deck slab shows extreme deterioration, in the form of the " loss " of the top concrete cover of the reinforcing bars, and also if the quality of the deck slab concrete is poor, about 13% less than the design value, the load levels that will cause crack initiation and propagation to the existing concrete core are about 30% less than the load levels that would have caused recoverable damage to the deck.

3.2 Recommendations

(1) All bridges, especially the bridge deck slabs, should be designed and/or rehabilitated for load levels higher than HS20-44, HS15-44, etc. (whichever is applicable).

(2) In the bridge deck replacement program it is strongly recommended that high quality and high strength concrete be employed. The concern should be directed to the quality of the deck concrete, rather than the increased percentage of deck slab reinforcement.

The bridges designed and built in accordance with the current AASHTO

Standard Specifications for Highway Bridges (Ref. 1) and the prevailing bridge design provisions of the Commonwealth of Pennsylvania (e.g. Ref. 12), when subjected to various realistic loading conditions, indicate that (a) tensile stresses in the deck concrete can be high enough to cause cracking, (b) tensile stresses in the reinforcing bars are far below the values predicted by the designers, and (c) the primary structural response mode of the bridge deck is transverse flexure. Any increase of the "rupture strength" of the deck concrete will improve the serviceability characteristics of the bridge decks.

(3) Bridge inspection programs should be closely linked with the overload permit application processing activities. Any bridge with longitudinal cracks at the top and/or bottom of the deck slab should be closely inspected to identify the source of the cracking. The research clearly indicated that this type of cracking is quite common in the case of the overloading of bridges. If sources of such cracks can not be explained, then the overload permits for the traverse of vehicles on bridges with such "damage" should not be issued without prior "investigation." If these cracks are due to the overloading, and if large numbers of overloaded vehicles traverse this bridge, then the cracks will grow to a depth that will be totally unacceptable to the bridge engineers.

(4) In the rating and overload permit application of bridges the current simple "s/5.5" lateral live load distribution factor should either be discontinued in favor of a more refined expression, or if the

expression of "s/5.5" is to be used, great care must be exerted. This is due to the fact that the "s/5.5" expression is far from being accurate. If the stresses in the superstructure in the rating or overload permit process are close to the "permissible stresses," there exists a possibility that under the actual loading conditions the actual stresses might be higher than the predicted stresses.

(5) Overload directories can be put into immediate use in the overload permit applications (Refs. 5 and 10).

(6) Both in the processing of overload permit applications and also in bridge inspection programs the working cracks in the reinforced concrete deck slab due to vehicles may be permitted. These cracks do not exceed the depths about one third to one half the thickness of the concrete cover of the reinforcing bars.

(7) The cracking of the reinforced concrete deck slab due to overloads should not be permitted to reach a magnitude such that the crack depths are about half the depth of the slab thickness. Only under rare and controlled conditions, if at all, i.e. extreme heavy loads with special permits, should the crack depths be permitted to reach about one third the slab thickness. Increased frequency of the passage of the vehicles that can cause damage as such to the bridge deck will gradually lead to these cracks performing as "very deep working cracks." Increased frequency of such a loading will lead to the deepening of the cracks. Both from the serviceability and the maintenance of the structural

integrity of the deck slab standpoint, this is unacceptable.

(8) Under no circumstances should any form of damage, or "overstressing" of the prestressed concrete I-beams, be permitted. In some "oversimplified" engineering computations - using the "s/5.5" distribution factor and working stress approach - only the beams are checked. It is not uncommon to request an overload permit using these computations. "A few psi tension" at the bottom of the prestressed beam is an excellent indicator of substantial damage to the bridge deck slab. As far as the serviceability limits of the prestressed concrete beams are concerned tensile stresses even below the rupture strength of the concrete could and should be considered as overstressing.

(9) In all overload permit applications, if due to the complexity or the criticality of the loading, engineering computations are made and submitted to the transportation agencies, then careful study of the slab stresses must be required. With the current technology there exists sophisticated hauling equipment which can move the "wheel groups" laterally. The assumption of having the wheel groups coincide with the axes of the beams does not solve the problems emanating from the overloading of the bridge. In a loading like this the differential deflection of the beams may still cause large stresses and possible damage to the bridge deck slab.

(10) The axle weights that can cause the cracking of the concrete cover of the prestressed concrete beams can be estimated by the following

formula:

$$P(\text{axle}) = 36.4 + 0.4 L$$

where

$P(\text{axle})$ = axle weight that causes the beam cracking (in KIPS), and

L = span length of the bridge (in FEET).

This load can be used to determine if the axle load in question requires any further consideration. If the proposed axle load is higher, or just below this value, than the deck slab under this loading will crack to unacceptable depths.

The above formula was developed for "dollies" with a maximum of four axles per axle group and a minimum of four wheels per axle.

(11) This and the following two recommendations pertain to the computer activities. Computer program BOVA, though impractical for permit operations, can be of value to the research and development activities in the future. It is strongly believed that this program was ahead of its time. If a need arises for a tool for the fully nonlinear response of simple span bridge superstructures, then BOVA can be used. The use of this program will reduce the additional new investments that need to be made. It is recommended that computer program BOVA be maintained by the Pennsylvania Department of Transportation for possible future use. This does not correspond to the daily maintenance of the program by the

computer services personnel. It merely corresponds to keeping the program operational in the main computer of the Pennsylvania Department of Transportation.

(12) Computer program BOVAC should be maintained by the computer services personnel of the Pennsylvania Department of Transportation. The program should preferably be recompiled using the FORTRAN77 compiler, and all fixed format options be eliminated. For further assistance to the users, it could be assumed that the inputting of the program could be modified to the "interactive conversational mode," thereby eliminating the need for the use of a reference manual for every input of every case study.

The computer program could and should be used for bridge rating and overload permit applications, if the guidelines needed can not be obtained through the overload directories and/or other accepted methods.

(13) Both computer programs BOVA and BOVAC can be transmitted to the Federal Highway Administration, U.S. Department of Transportation and to the National Technical Information Service (NTIS) for the dissemination and distribution of the programs.

3.3 Suggestions for Long Range Planning

(1) The information on the overloading behavior of bridges, correlation of actual field measurements and the analytical predictions, and serviceability limits to govern the overloading practices is still far from being complete (Ref. 18). It is recommended that all the available information be processed through a "national clearinghouse" to share the pertinent experiences. Such an approach will permit the future directions to be taken more realistically.

(2) In view of the extensive computerization that is taking place throughout bridge engineering and transportation, it will not be unrealistic to expect the following scenario. Key structural features of a bridge can be stored in a general purpose data base. Through the bridge inspection program this data base can be continually updated. In the case of an overload application, once the routing of the vehicle is defined, than another computer program, using another data base, can identify all the bridges that will be traversed. These bridges can be related to the earlier data base, and the pertinent data can be "fed" into another computer program.

This program can contain all the needed analyses and checks for the overload response of the bridge superstructure. The analysis results can then be displayed and summarized for the permit application. Either program BOVAC can be used for all the bridge checks referred to above, or another approach could be used. The proposed approach could be done through the establishment of a data base using the overload directories.

Through the interpolation between the data points using pre-defined acceptable "recoverable damage," the acceptability of the application load could be verified. Even though the above suggested scenario may be considered a very long term project, either the suggested approach or similar ones will substantially reduce the problems encountered in processing overload permit applications.

(3) In interactions with engineers at the "district" level, it was noted that the information exchange among the bridge inspection personnel, permit operations officers, and other interested parties is slow. Due to the limited time available to the permit officers, it would be highly desirable if they tapped the information and knowledge-base of the bridge inspectors and district and central bridge office personnel. Such wide-scale interaction can be attained if all the recent data can be transferred immediately to a data base that can easily be accessed. The recommendations made in this report can easily be interfaced with any data base. This will permit the transfer of knowledge and the experience gained in these research programs to the permit officers through bridge engineers.

(4) The overload directories could be incorporated into appropriate data bases to be used in conjunction with the overload permit operation and/or bridge rating. The type and form of the inclusion into the data base should be decided by the Pennsylvania Department of Transportation such that the material can and will easily interface with the existing and projected data bases and future plans for computerization.

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