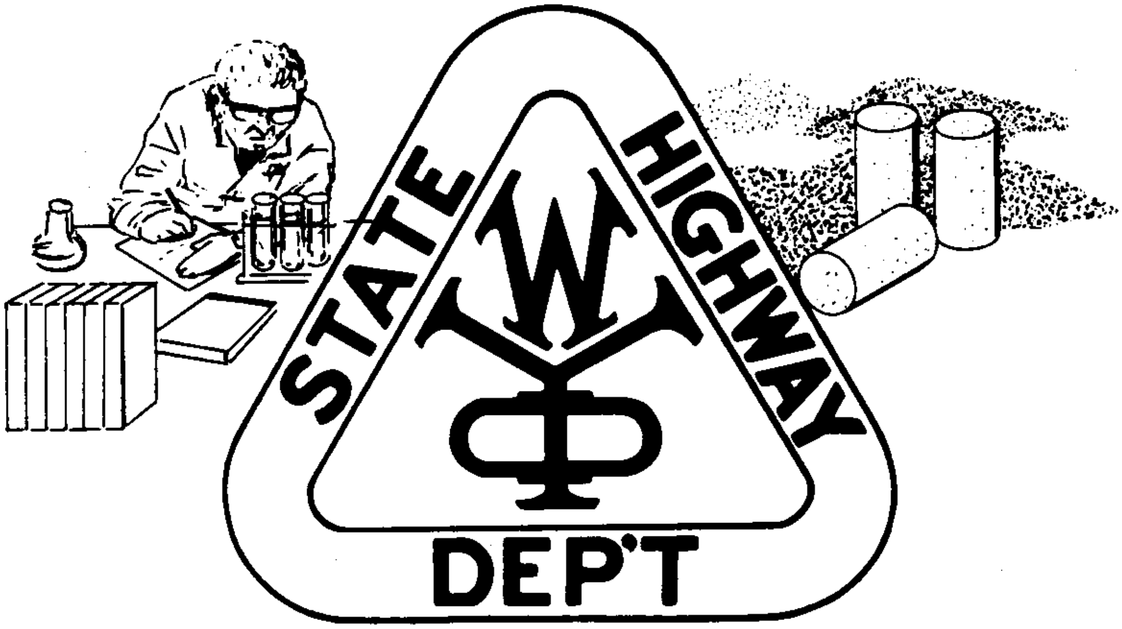
FHWA-WY-89-002

EVALUATION OF BRIDGE DECK UTILIZING ONTARIO BRIDGE DECK DESIGN METHOD

Volume No.1

Review of Experimental and Analytical Research in Lightly Reinforced Bridge Decks



WYOMING HIGHWAY DEPARTMENT
RESEARCH

BRIDGE BRANCH

JULY 1989

Technical Report Documentation Page

1. Report No. FHWA - WY - 89 - 002	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle

Evaluation of Bridge Deck Utilizing Ontario
Bridge Deck Design Method
Volume 1 of 3

5. Report Date
July 1989

6. Performing Organization Code

7. Author(s)

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8. Performing Organization Report No.

9. Performing Organization Name and Address

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10. Work Unit No. (TRAIS)

11. Contract or Grant No.
HPR-PR-PL-1(20) 302

12. Sponsoring Agency Name and Address

Wyoming State Highway Department
P.O.Box 1708
Cheyenne, WY 82002-9019

13. Type of Report and Period Covered
Final Report
May 1985 -July 1989

14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the US Department of Transportation,
Federal Highway Administration

16. Abstract

Investigators have consistently found that the traditional deck is an extremely conservative design, utilizing significantly more reinforcement than required. Furthermore, the additional reinforce may have an adverse effect on long-term serviceability. This led the Ontario Ministry of Transportation (OMT) to undertake an extensive research program to evaluate the effect of significantly decreasing the reinforcement content in bridge decks. As a result of many years of investigation, the OMT has codified an empirical procedure requiring 0.3% reinforcement in an isotropic configuration (OHBDC,1983). The research findings are extremely consistent, closely parallel the OMT work, and support the position that the 0.3% isotropic deck is safe for the present AASHTO loading and exhibits serviceability characteristics that meet or exceed the AASHTO deck. The intent of this report is to review and summarize the research on the lightly reinforced decks. The research has included all the important behavioral characteristics such as the inherit arching action, the punching shear failure mode, the serviceability behavior, fatigue testing, prototype testing, and long term performance of decks in service.

The relevant research is divided into three groups:

1) Works related to the arching in concrete slabs,
2) Research undertaken by the OMT for bridge deck slabs in the development current OHBDC(1983), and
3) Research undertaken in the US to investigate the lightly reinforced bridge deck design.

17. Key Words

18. Distribution Statement

No Restriction. This document is available through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages 48	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

EVALUATION OF BRIDGE DECK UTILIZING ONTARIO BRIDGE DECK DESIGN METHOD

Volume No. 1

Review of Experimental and Analytical Research in Lightly Reinforced Bridge Decks

FINAL REPORT

Project No. 38717

by

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January 1988

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DISCLAIMER

UNIT CONVERSIONS

Because of the nature of the subject matter, research results are often described with qualitive results and hence require the appropriate units. Such results are given in the units used by the original author(s). To enhance readability, the unit conversions are not given in the body of the test. The table below is provided as an aid.

Multiply	**By**	**To Obtain**
inch	25.4	millimeter
foot	0.304800	meter
$inch^2$	645.160	millimeter
pound	4.44822	newton
psi	0.112985	kilopascal

EXECUTIVE SUMMARY

Reinforced concrete bridge decks in the United States and elsewhere are designed using conventional flexural theory which results in an orthotropic reinforcement scheme (AASHTO, 1983). This deck design has been extensively evaluated with experimental methods in the laboratory and in prototype structures, and by analytical and numerical procedures. Investigators have consistently found that the traditional deck is an extremely conservative design, utilizing significantly more reinforcement than required. Furthermore, the additional reinforcement may have an adverse effect on long-term serviceability. This led the Ontario Ministry of Transportation (OMT) to undertake an extensive research program to evaluate the effect of significantly decreasing the reinforcement content in bridge decks. As a result of many years of investigation, the OMT has codified an empirical procedure requiring 0.3 percent reinforcement in an isotropic configuration (OHBDC, 1983).

Since the Ontario investigations, several agencies within the United States (US) have initiated research to further study the behavior of the lightly reinforced decks. Ultimate strength and serviceability concerns have been addressed using experimental and numerical methods. The research findings are extremely consistent, closely parallel the OMT work, and support the position that the 0.3 percent isotropic deck is safe for the present AASHTO loading and exhibits serviceability characteristics that meet or exceed the AASHTO deck.

The intent of this report is to review and summarize the research on the lightly reinforced decks. The research has included all the important behavioral characteristics such as the inherit arching action, the punching shear failure mode, the serviceability behavior, fatigue testing, prototype testing, and long term performance of decks in service.

The relevant research is divided into three groups:

1) Works related to the arching action in concrete slabs,

2) Research undertaken by the OMT for bridge deck slabs in the development of the current OHBDC (1983), and

3) Research undertaken in the US to investigate the lightly reinforced bridge deck design.

It is important to reiterate that the major findings from the diverse work performed under varied laboratory and field conditions by independent investigators have shown remarkable consistency. The major findings are summarized:

1) The failure mode is punching shear regardless of the reinforcement content (above a nominal amount). Only unreinforced decks failed in a flexural mode.

2) The compressive membrane action contributes significantly in enhancing the capacity of the deck slab.

3) The isotropic reinforcement (0.3 percent) is adequate for normal field conditions and results in significant savings for bridge deck design.

4) The compressive membrane arching action increases for increasing edge restraint.

5) The compressive membrane action decreases for increasing span-to-depth ratios.

6) The diaphragms, cantilever overhangs, barrier stiffness, and skew did not significantly affect the isotropic deck design.

7) The isotropic deck design was found to behave satisfactorily for static loads, dynamic concentrated cyclic loads as well as for moving load tests.

8) The AASHTO code was found to be conservative in regard to strength.

Although the research is very consistent, it is important to note the two findings that were inconsistent with the consensus findings:

1. In a trapezoidal box girder bridge, a longitudinal crack developed along the top of an interior flange three months after casting (Holowka, 1979). Holowka attributed this to the removal of temporary bracing and recommended leaving the bracing in-place. This does not agree with item 7 above.

2. In the University of Texas research, Kueng (1988) concluded that the transverse membrane forces increase with increasing span-to depth ratios. This contradicts numerous independent studies and item 5 above.

In summary, the work related to lightly reinforced concrete bridge decks has been reviewed and summarized. The results are consistent and without exception, the investigators report adequate factors of safety and favorable behavioral characteristics of the lightly reinforced decks.

CHAPTER 1

INTRODUCTION

A review of literature pertaining to the development and research of concrete bridge decks with low percentages of isotropic reinforcement is given. The literature is segmented into five areas:

1. A brief review of the current design procedures as specified by the American Association of State Highway Officials (1983) (AASHTO) and the Ontario Highway Bridge Design Code (1983) (OHBDC),
2. Works related to arching action in concrete slabs,
3. Experimental and analytical work conducted by the Ontario Ministry of Transportation,
4. Experimental and analytical work conducted in the United States, and
5. Research in progress or scheduled at the time of this writing.

1.1 Review of the AASHTO and OHBDC Specifications for Deck Design

a) Summary of the current AASHTO bridge deck design provisions:

As per the current practice (AASHTO, 1983), deck slabs of highway bridges must be designed to resist wheel loads plus self-weight. For slab and girder bridges, the deck slab is considered a one-way slab spanning transversely and supported by girders. The main reinforcement is placed transversely. The maximum transverse design moment is $M = P(s+2)/32$ where s is the effective span in ft. or the distance between edges of flanges plus one-half of the stringer flange width for slabs supported on steel stringers, and P is the concentrated wheel load. An impact factor of 1.3 is used and the design moments are reduced by 0.8 for slabs continuous over three or more supports.

b) Summary of the empirical method of current Ontario Highway Bridge Design Code (OHBDC):

As per the current OHBDC (1983), concrete deck slabs shall be designed with 0.3 percent reinforcement in both the transverse and longitudinal directions. The design must satisfy the following restrictions: 1) Maximum girder spacing is 3.7 m.; 2) The cantilever portion of the slab should extend at least 1 m. beyond the exterior girder with the cross-sectional area of the integral curb, plus the slab beyond the centerline of the external girder should not be less than the cross-section of 1 m. length of the deck slab; 3) The span length (skew or straight) to depth ratio should not exceed 15, 4) For skew angles greater than 20 degrees, the end portions shall be provided with 0.6% isotropic reinforcement; 4) Slab thickness should not be less than 9 in. and the isotropic reinforcement spacing should not exceed 12 in.; 5) Diaphragms shall extend throughout the transverse cross-section of the bridge between the external girders and must be provided at the supports for reinforced and prestressed concrete girders and at a maximum spacing of 26 ft. for steel I-beams and box girders.

CHAPTER 2

WORKS RELATED TO ARCHING ACTION IN SLABS

2.1 Early Work

In the early sixties, Christiansen (1963) developed a method of analysis that considers the arching action in beams and one way slabs. Four simply supported beams were loaded with a concentrated load and the load-deflection response was studied. The load due to arching was 1.3 to 1.5 times those predicted by bending theory and about 8% greater than values calculated by a proposed analytical procedure. A method to estimate the combined ultimate strength of one way slabs due to bending and membrane stresses were recommended.

Christiansen, et al. (1982) tested eleven model rectangular slabs subjected to concentrated loadings. The slabs had different amounts of reinforcements and different ratios of side lengths. In addition, two full scale model tests were conducted on a demolition relegated reinforced concrete building by means of lead blocks hoisted by a crane. One of the slabs tested had an interior slab with a horizontal restraint along all four sides and the other was an edge slab with horizontal restraint along three sides. The contribution of bending to the capacity was determined by using Johansen yield line theory (YL).

Christiansen combined the laboratory results with data from 76 slabs previously tested by other investigators to arrive at a simplified expression for the capacity of rectangular concrete slabs with horizontal restraints on all four sides. For commonly used edge restraints, the concentrated load capacity, P, of rectangular concrete slabs was estimated as $P = YL + h^2 \times f_c$ where h = slab thickness, f_c = concrete cylinder strength. With increasing rigidity of horizontal restraints the load due to membrane action increases from $h^2 \times f_c$ to $3 \times (h^2 \times f_c)$.

Park (1964a) developed a yield line theory to determine the ultimate strength of uniformly loaded rectangular concrete slabs with three or four edges restrained against lateral movement under short term uniform loading. Due consideration was given to the resulting compressive stresses induced by lateral restraints. The numerous model tests were conducted by varying aspect ratios, edge restraints and reinforcement content which ranged from no reinforcement to heavy reinforcement.

Cracking was observed at 32% of ultimate load for slabs with three sides restrained and at 42% for slabs with all side restrained. Hence cracking was negligible. At 33% ultimate load, the central deflections were always less than 1/500 of short span. Therefore, deflections were not of concern. The ultimate loads were observed to be far in excess of the conventional yield line theory (10 times more for fully restrained cases). The proposed analytical procedure described compared well to the experimental values. Park recommended further testing to establish creep and shrinkage effects, for determination of the stiffness parameters to limit the lateral edge movements for the development arching action and to establish load factors against the collapse of the entire floor.

Continuing from previous work, Park (1964b) has extended the theory for the ultimate strength of uniformly loaded, laterally restrained two way concrete slabs to include its effect on the compressive membrane action of partial edge restraint. A long term load was considered to include the axial shrinkage and creep strains effects. Eight unreinforced slabs with clear spans of 60 by 40 in. and with span-to-depth ratios of 20, 30 and 40 were tested under sustained loading. Some of the slabs were restrained at 10 days while the rest were restrained at 147 days. The slabs were first loaded with a sustained load held constant for 42 days, which was considered adequate for creep & shrinkage effects to be appreciable. The load was then completely removed and then incremented to failure within 15-55 minutes. Elastic and creep strains were measured on axially loaded cylinders while shrinkage strains were measured from time of clamping the slabs to the end of sustained loads. Lateral movements of the long sides were measured by dial gages.

The theoretical analysis showed that the reduction in strength due to partial lateral restraint at the boundaries is significant, especially for thin slabs. Even so, the ultimate load was still observed to be in excess of yield line theory. Park recommended incorporating reduction factors in the analytical model (1964a) to account for the effects of axial strains and boundary flexibility. Experimental results showed that creep strains at working loads were negligible and only elastic and shrinkage axial strains need to be considered in strength calculations.

Park (1965) furthered the experimental program by providing surrounding beams and panels which provided the stiffness required to limit the lateral translation, with the intent of permitting the membrane action to be considered in the design of slab-and-beam floors. Two series of small scale sand-cement mortar slabs (F1-F14, with span-to-depth ratios (l/d) > 29 and G1-G6 with l/d between 17 and 19) were constructed. Each of these consisted of a nine panel continuous slab and beam floor system, three panels wide in each direction. The panels were supported by rollers bearing against the test frame along the lines of the edges of the interior panel and along the outside edges of the floor. The exterior panels in series F were square while those for series G were rectangular. Tie reinforcement was placed around the edges of the center panel to simulate reinforcement in support beams. The center panel was hydrostatically loaded to failure. Cracking and deflections were measured.

It was noted that the tie reinforcement was utilized only after the exterior panels had cracked. The narrow exterior panels of some of the G-series slabs illustrated that lateral bowing considerably reduces membrane action. Park concluded that the slab-and-beam floors designed to include the compressive membrane action require more reinforcement as ties in the beams than can be saved in the panels. Also the experimental results on small scale models showed good agreement with theoretical analysis including axial strains and lateral edge displacements for the case in which the exterior panel had cracked.

2.2 Recent Work

A reinforced concrete bridge of length 11.815 m. and 8 degree skew with six concrete beams was tested for the serviceability requirements of deflection and cracking (Kirkpatrick, 1986). Four edge panels were designed with varying transverse reinforcements of 1.7%, 1.1%, 0.6% and 0.25%. A maximum load of 400 Kn, well over the service load of 112.5 Kn. wheel load, was applied. After gradually decreasing to zero, the load was again increased to 400 Kn. to check progressive widening of cracks which were measured with feeler gages followed by strain gages with 62.5 mm. gauge length. Deflections were measured for service load requirements of 112.5 Kn. and then for 400 Kn. for all reinforcement ratios. Maximum slab deflection for a 0.25% reinforcement ratio was 1.16 mm. The slab was visibly uncracked at 112.5 Kn. service load. Initial cracks occurred at 160 Kn. A punching failure was observed. Even for relatively low levels of loadings between 1 to 3.5 times the service loads, compressive membrane forces were observed. The increased capacity due to membrane action was recommended to be included in the design codes. The cracks were within the serviceability limits currently imposed. A 0.6% isotropic reinforcement mesh of 12 mm diameter high yield bars at 150 mm spacing was recommended for beam and slab reinforcement. This would control the cracking due to early thermal movement and shrinkage.

2.3 Summary

The works by Park and Christiansen clearly indicated that the effect of arching action was significant for uniformly loaded panels in building systems. The empirical models were developed that reliably predicts the favorable in-plane effects. Although the early works cited herein are certainly not comprehensive and generally relate to uniform loads, the scope of the citations clearly illustrates that the arching behavior is related to increased slab strength.

CHAPTER 3

DEVELOPMENT OF OHBDC PROVISIONS FOR CONCRETE BRIDGE DECKS

The OHBDC specifies a minimum of 0.3% isotropic reinforcement for concrete decks. This criterion is principally empirical based combined with an analytical analysis that includes compressive membrane action. This analytical model was used to determine the ultimate strength of restrained slabs. This model and the verification research are described.

3.1 Mathematical Model

Hewitt, et al. (1975) and Batchelor et al. (1978) first developed a mathematically based (calibrated with empirical results) method to describe the behavioral characteristics of restrained slabs specifically related to bridge decks. Due to the continuity of a slab system across the girders, both in-plane and out-of-plane stiffness is present around an individual panel. As the slab cracks, the in-plane or compressive membrane actions can develop and significantly effect the serviceability and strength behavior. This action can be visualized by in-plane compressive forces from the applied load to the boundary edges in the shape of an arch. Hewitt described these actions at the panel edge as a compressive force and a moment at the level of tension reinforcing. These actions combine to enhance the strength of the slab and this enhancement allows punching shear to be the ultimate failure mode.

Kinnunen theorized a conical shell of concrete extending diagonally from the load edge to the bottom of the slab along a shear crack to predict when failure of a simply supported circular slab occurs (Hewitt, et al., 1975). The Hewitt model extended Kinnunen's theory to include boundary restraining forces acting at the tensile reinforcing. Failure occurs when the tangential strain exceeds the ultimate concrete strain, impairing the strength of the conical shell. This strain is located at the top surface near the diagonal shear crack. The punching load is readily determined using a computer program based on the Hewitt model.

A hypothetical slab was analyzed to determine the effect of boundary forces. The calculated punching strength, using the Hewitt model, increased considerably as boundary forces rose. This load was adjusted for dowel effect (found to be approximately 20% of the simply supported slab strength) to give an ultimate punching load. The specifics of the Hewitt model are given in more detail elsewhere (Hewitt, et al., 1975).

Because actual boundary forces are unknown for in-service bridge decks, a restraint factor is used. Boundary forces are accounted for by adjusting the idealized fixed boundary restraint by this factor. A restraint factor of 0.0 is used for simply supported slabs and 1.0 for totally fixed restraint. Restraint factors are derived empirically from tests on in-service bridges (described later). The typical slab previously analyzed for various boundary forces was analyzed for variation in restraint factors. The punching strength of the slab increased 200% as the restraint factor increased from zero to one.

3.1.1 Verification Research

The Ontario Ministry studied the Kinnunen model to determine its accuracy. Next the mathematical model by Hewitt was verified by comparing it to slab tests of known restraint. Restraint factors were also calculated for slabs of unknown restraint. Each of these are discussed.

3.1.2 Accuracy of Kinnunen Model

The Kinnunen model was studied for 165 simply supported slabs to verify its accuracy (Hewitt, et al., 1975). The strengths of the slabs were found to be approximately 20% greater than predicted by the Kinnunen model. A factor of 1.2 was therefore used to include this effect and was assumed to be attributed to dowel effects. Parametric studies showed the Kinnunen model was applicable only to slabs within certain geometrical and material limits. These limits were a span-to-thickness ratio of four to 17, a modulus of elasticity for steel reinforcing of 27,000 to 33,000 ksi. and a reinforcing index of 0.05 and 0.45. The reinforcing index is defined as the percentage of steel reinforcing multiplied by its yield stress then divided by the concrete compressive strength. Based on 137 slab tests, the mean value of the calculated load was 0.978 of the average test load. It was concluded that the Kinnunen model including the dowel factor produced satisfactory results.

3.1.3 Slab Tests of Known Restraint

Membrane forces were measured for 27 circular slabs (Dorton, et al. 1977; Csagoly, 1979). The objectives were to provide validation of the Hewitt model and determine if a restraint factor of 0.5 used in the design of the Conestogo River bridge deck was adequate. Various slab thickness and reinforcing ratios of 0.2, 0.3 and 1.0 percent were used. The mathematical model derived for circular slabs was used for comparison to the experimental tests. Strain gages, bonded to a steel restraining ring, measured the membrane forces. The ring, located at the tension reinforcement, provides restraint corresponding to factors of 0.5 to 0.75. The results showed membrane forces varied linearly as the concentrated load was increased. The slabs all failed by punching shear at load levels slightly higher than predicted by the Hewitt model. The results also showed a 0.5 restraint factor was very safe for the Conestogo River bridge design.

The modified model by Hewitt was also used to analyze the punching strength of restraint slabs reported in various literature. The investigation was hampered because the boundary restraints were difficult to accurately determine.

The theoretical results were also compared to prestressed concrete slabs. In one case, the prestressing cable acting as bonded reinforcement (i.e. the circular slab reinforcing in the radial direction had not yielded) and dowel action was included. In the second case, the cables acted as unbonded reinforcement and dowel action was not included. In each case the contribution of the prestressing cable was estimated and the punching load strength adjusted accordingly. The mean value comparison (theoretical/experimental) punching load was 1.02. A value of 1.10 of the test load was the poorest comparison reported.

3.1.4 Restraint Factors for Slabs of Unknown Restraint

Restraint factors were determined for slabs from previous research using the Hewitt model. The slabs were either supported within a steel frame or by concrete edge beams. Considerable scatter was reported. The author tentatively recommended a restraint factor for the following parameters (Hewitt, et al., 1975) :

$C/c < 6.0$ and $\omega < 0.1$: Restraint Factor > 0.5

$6.0 < C/c < 9.0$ and $\omega < 0.2$: Restraint Factor > 0.25

where C is the diameter or equivalent diameter of slab, c is the diameter or equivalent diameter of loaded area and ω is the reinforcement index. Restraint factors were also determined for ultimate strength tests of composite steel bridges by Hewitt. The following recommendations for these slabs (span/depth < 20) were:

$3.7 < C/c < 5.6$ and $\omega > 0.02$: Restraint Factor > 0.5

It was concluded that a restraint factor of 0.5 was applicable for rectangular slabs of composite steel/concrete bridges reinforced with 0.2% isotropic steel.

3.2 Ontario Ministry of Transportation Testing

By far the most extensive testing of minimally reinforced decks has been performed by the OMT. Ultimate strength tests of model bridges and field tests on in-service bridges proceeded simultaneously. Afterwards, prototype testing on concrete decks with various percentages of isotropic reinforcement further verified model and field tests results. All model and prototype bridge tests were composite decks while field testing included both composite and non-composite

3.2.1 Model Tests on Restrained Slabs

Models of concrete slabs with various amounts and patterns of reinforcing were tested to determine scale effects and ultimate strengths for static and dynamic loadings. The results of each test are discussed.

3.2.1.1 Scale Effects

Model tests performed in 1971 at Queen's University at Kingston, Ontario in cooperation with the Ontario Ministry, established the effect of model scale on the punching strength of slabs (Batchelor and Tissington, 1972). An 80 foot span bridge composed of square slab panels was modeled. Square panels were used so the behavior was a two-way slab. Models of 1/15th, 1/12th, 1/10th, 1/8th and 1/6th scale were subjected to concentrated loads. All slabs were 0.33% isotropically reinforced and failed in punching shear which led the authors to conclude that model scale had little effect on the failure mode.

3.2.1.2 Ultimate Strength

Model tests by Batchelor, et al. (1978b), at Queen's University were more extensive than previous research. The ultimate strength of conventional reinforced and various percentages of isotropic reinforcement were established. Nine 1/8th scale model concrete/steel I-beam bridges of which eight were supported by four beams and one on three beams were tested. The bridges were 80 feet between supports (prototype dimensions) and were composite with a seven inch concrete deck. Overhangs of four feet were on either side of the exterior beams. The three and four beam models provided a span-to-thickness ratios of 20.6 and 13.7, respectively. Span refers to the transverse spacing of longitudinal beams. Lateral bracing was spaced every 20 feet in both bridges. The portion of the deck bound by beams and bracing is defined as a deck panel and all deck panels in the transverse direction were referred to as a strip. Four of the models used conventional orthotropic reinforcing while the remaining five were isotropically reinforced. Reinforcing steel percentages of 0.6, 0.4, 0.2, as well as plain concrete (0.0%), were used. The influences of span-to-slab thickness ratio, load position, dead load stresses and concrete strength on the punching strength of each deck panel were studied. Punching shear failure for the orthotropic panels occurred at various concentrated load magnitudes. The orthotropic bridges were estimated to have a factor of safety (F.S.) of 16 for a wheel load of 16 kips and impact factor of 0.3. Cracks began appearing, regardless of reinforcing percentage, at 25 to 50% of the failure load characterized by an elliptical cracking pattern. Location of the panel on the bridge, failures in adjacent panels, and concrete strength did not influence the punching strength of the deck.

The isotropically reinforced bridge model tests indicated that punching strength rose as the reinforcing percentage increased and as the span-to-slab thickness ratio decreased. Flexural failures in panels with 0.2% reinforcement and unreinforced panels were either assumed to occur simultaneously with the punching shear failure or were excluded from the results due to extenuating circumstances. Panels of plain concrete had a F.S. of 10 for 16 kip wheel load and 0.3 impact factor. A F.S. of 12 was determined for 0.2% reinforcing using the same parameters. Batchelor recommended 0.2% isotropic reinforcing be used for temperature and shrinkage effects. Inducement of hogging (negative) moments into the deck had little effect on the punching strength. In all bridges, cracking was observed to be acceptable for service level loads and confined to the panel being tested. The authors concluded that conventional reinforcing was excessive due to its high factors of safety.

3.2.1.3 Fatigue Strength

As a continuation of the above research, 1/8th scale bridge models were subjected to cyclic concentrated loads (Batchelor, 1978a). Prototype dimensions were the same as described in the previous section. The reinforcing percentage and ratios of fatigue load-to-static failure load were varied. These tests provided insight into fatigue life of deck slabs with reduced reinforcing and verified static test conclusions. Five model bridges spanning 80 feet, simply supported on four beams were tested to failure or to a minimum of 100,000 cycles of loading. Dynamic loads were applied at a frequency of 1 to 5 hz. Crack patterns occurring after a few cycles were similar to static tests but continued to spread and widen as cycles increased. Panels failed by punching below a fatigue load-to-static failure load ratio of 0.5 for orthotropic reinforcing. Using this ratio, the endurance limit, a 16 kip wheel load and 0.3 impact factor produced a F.S. of eight. This result supports the static load testing by indicating conventionally reinforced decks are conservatively designed. Isotropic panels had an endurance limit of 0.5 for 0.6% and 0.4% reinforcing but a lower bound of 0.4 was recommended to include 0.2 and 0.0 percentages. Failure occurred in all panels by punching except for a few very lightly reinforced panels. The 0.2% panels indicated a F.S. of approximately four for the same wheel load and impact factor as used for orthotropic panels. Panels that did not fail in fatigue were tested to failure by static loads to determine the reserve strength. All panels after a minimum of two million cycles of dynamic loading showed large reserves of strength for various reinforcing percentages. The investigators recommended endurance limits of 0.5 and 0.4 of the ultimate static failure loads be used in design of the orthotropic and the 0.2% isotropically reinforced decks, respectively. Based on these limits it was concluded that decks of adequate lateral restraint were unlikely to fail due to fatigue.

3.2.2 Field Tests on Existing Bridges

The Ontario Ministry has conducted testing of existing bridges designed by AASHTO specifications (Csagoly, et al., 1978). The objectives were to observe if any slab failures occurred and determine restraint factors (Hewitt, et al., 1975) for different types of bridge support structures. A lower bound restraint factor was also estimated for use in design. The first field test occurred in 1971 on a bridge which has been out of service for 20 years (Csagoly, et al. 1979a). Reinforcing bars were exposed and bonded with strain gages. The bars were then monitored during a concentrated load of 125 kips. The results indicated only 1/6th of the load was carried by direct flexure for the given span-to- depth ratio and

AASHTO reinforcing. The author did not elaborate on how the direct flexure load was established. The results encouraged the Ontario Ministry to investigate minimally reinforced decks with model tests as previously described and with more field testing. As model research was proceeding simultaneously, a total of 40 bridges were tested. Each bridge was grouped into composite and non-composite decks for various states of deterioration.

Concentrated loads of 100 kips ensured a multiple of five over a maximum observed design wheel load of 20 kips. Loads were applied and the deflection was measured using a displacement transducer at the load point. The transducer was mounted on a 12 foot bar aligned parallel to traffic flow. Bar ends were assumed to be remote from the load location so deflection at the bar ends could be neglected. The load was cycled to determine the behavior of the slab. If the hysteresis curve was divergent, further cycling would likely cause slab failure and loading was stopped. Only a few bridge decks indicated slab failure may be eminent. Core samples were taken to determine concrete strength so it could be included in the evaluation. No failures occurred except in an isolated case in a portion of a deteriorated cantilever. These tests confirmed model test results and indicated AASHTO design procedures to be conservative.

Deflections from 32 of the 40 bridges were compared to deflections obtained with the Hewitt model to determine restraint factors. This analysis was done by determining theoretical deflections for restraint factors ranging from 0.0 to 1.0. These values were compared to experimental deflection allowing interpolation of restraint factors for each bridge. Lower bound restraint factors were estimated for each category of bridge. These lower bounds were 0.25 for non-composite bridges and 0.5 for composite bridges (Csagoly, et al., 1978). Based on these restraint factors, 0.3% isotropic reinforcing was recommended. This reinforcement percentage provides adequate cracking control due to temperature and shrinkage effects.

3.2.3 Prototype Bridge Tests

Prototype testing of various percentages of isotropic reinforced decks was performed on three bridges by the Ontario Ministry. The discussion is limited to the instrumentation and results pertaining to the concrete deck.

3.2.3.1 Conestogo River Bridge

The Conestogo River bridge was designed to verify a number of innovations used in the OHBDC (Brown, et al., 1976, Dorton, et al., 1977). For simplicity, only those associated with the deck reinforcing are discussed. A continuous steel girder bridge with spans of 114, 145 and 114 ft. was constructed. The superstructure has four haunched welded girders spaced every 8 ft. - 8 in. with lateral bracing every 24 ft. - 6 in. The concrete deck acts compositely over the entire length and was longitudinal prestressed. Twelve test panels, defined as the area bound by adjacent girders and bracing, had transverse reinforcement percentages of 0.95, 0.6, 0.3 and 0.2 for each slab thickness of 8, 7.5 and 7 inches. Initial testing of the deck occurred in 1975 but other tests continued for several years. The test panels were instrumented to determine deck reinforcing strains, slab deflections and concrete strains when subjected to a concentrated load. Electrical resistance strain gages, bonded to steel the rebar, were used to monitor reinforcing strains. Gages were located at the center of each panel and over an interior girder. Longitudinal and transverse bars in the top and bottom layers were instrumented at these locations. During installation, gage locations were ground down changing the cross-sectional area. A plot of load vs. strain for each instrumented bar established calibration constants used to account for variability in the bar area. Displacements in a grid pattern around the load were monitored with 40 linear displacement transducers. Finally, concrete strains were monitored by laser gages. The laser gages were mounted on the bottom surface of the deck in the longitudinal direction near the girders and in the transverse direction approximately midway between girders. Monitoring and installation problems associated with the laser gages hindered accurate results.

As part of the ongoing bridge testing program employed by the Ontario Research Division, two test vehicles each with a maximum gross weight of approximately 200 kips were used. Each vehicle was a five-axle, semitrailer truck loaded with concrete ballast blocks. The number and position of the blocks could be varied to produce different axle weights. A hydraulic actuator mounted under the trailer applied a simulated concentrated wheel load. Two neoprene pads were used to approximate the footprint of a dual tire wheel. Loads were monitored with a load cell then plotted vs. deflection as described earlier. This test allowed the restraint behavior to be determined. Strain readings were taken at loads of 0, 35 , 60 and 95 kips. The stresses in the deck reinforcing were found to be acceptable for each test panel and successfully withstood the 95 kip load (King, et al., 1978). A maximum stress of 31.2 ksi. occurred in the bottom transverse reinforcing steel of the 0.2% test panel. For a 16 kip wheel load and 0.3 impact factor an equivalent stress of 6.84 ksi. was determined. The maximum deflection recorded by linear displacement transducers was 0.122 inches for the 0.2% reinforcement. Measured deflections were less than the theoretical value predicted using a restraint factor of 0.5 in the Hewitt model except for two tests. Previous verification determined that the 0.5 value was justified. Thus, the authors concluded the restraint factor was adequate for a deck designed with 0.2% reinforcing. Excessive cracking however remained in all 0.2% reinforced panels after removal of load. This behavior might be attributed to the large spacing between reinforcing bars (no. 4 bars at 16 in.). Test panels of 0.3% reinforcement in comparison showed fine cracks that disappeared upon load removal. This study and other research (Csagoly, et al., 1977, 1978) led to the minimum specification of 0.3% isotropic reinforcement to control cracking. Some crack widths were measured by laser gages on the underside of the deck but major problems with the gages hindered conclusions. The Conestogo bridge prototype test confirmed model research that internal arching action was present and has a significant influence on the load carrying capacity of the deck.

3.2.3.2 Trapezoidal Box Girder Bridge

The bridge, located in Ontario, spans 140 feet with two trapezoidal steel box girders (Holowka, 1979). The girders were spaced 8 ft. - 4 in. apart for a total width of 34 feet. Temporary bracing between the two box girders was located every 23 ft. to provide dimensional stability during pouring of the deck slab. These braces were removed for appearance reasons afterward. The concrete bridge deck is 8 inches thick with 0.3% isotropic reinforcement. The reinforcement consisted of no. 4 rebar placed 10 inches on center, top and bottom, in the longitudinal and transverse directions. Instrumentation of the bridge deck included monitoring of steel reinforcing strains.

Significant stresses occurred when the loads were on the same transverse line as the bars (Holowka, 1979). For point loads of 100 kips, maximum stresses of 18.64 ksi. and 14.5 ksi. were recorded for the transverse and longitudinal bars, respectively. These stresses were less than permitted by AASHTO while using approximately three times less steel reinforcing and a load 4.8 times greater than the AASHTO design wheel load. The deck also experienced similar responses to axle loads and positioning of two 200 kip trucks on the bridge simultaneously. Reinforcement stresses of 10 ksi. tension and 3 ksi. compression for the latter case were observed. Obviously, concentrated point loads caused the critical stresses in the deck reinforcing. Three months after the initial load test, a longitudinal crack 65 feet long developed in the deck along the interior top flange of one of the box girders. Analysis revealed transverse bending stresses became exceedingly high when the temporary bracing was removed. To prevent similar cracking, it was recommended the bracing be left in place when a 0.3% isotropic reinforced deck is used.

3.2.3.3 Composite Concrete AASHTO Girder Bridge

Incorporating four AASHTO type III girders spaced 7 ft. - 10 in., this bridge has spans of 60, 70 and 60 feet supported by two piers (Holowka, 1980). The seven and one-half inch composite deck is reinforced with no. 4 bars, top and bottom, spaced 11 inches o.c. in both orthogonal directions (0.3% isotropic reinforcement). Additional reinforcing in the form of no. 9 bars were used in the negative moment region over the piers. Intermediate diaphragms between the concrete girders are usually not used due to construction difficulties. The main objective was to monitor any adverse effects because of the absence of diaphragms during testing in 1978.

Forty-two gage installations located in one of the 60 foot spans were used. Ten locations used two uniaxial gages each with dummy gages to complete the Wheatstone bridge. These gages could measure the axial and bending strains in the bars. The remaining gages used two "tee" rosette gages which allowed only axial strains to be measured. The no. 4 bars were calibrated while for the no. 9 bars were not as it was felt the variability due to grinding of cross-sectional area was negligible. Twelve of the 42 locations were on no. 9 bars in the negative moment area. Six locations were on longitudinal bars in the positive moment regions. The rest were on transverse bars at the midspan, between the girders, and over the girders in the positive and negative moment regions.

Simulated wheel loads of 100 kips were located over the centerline of the pier, at the centerline of the bridge and midway between the exterior and interior girders. Also the two test trucks were positioned at different locations on the bridge. From the behavior of the deck slab, it was estimated the slab had an ultimate strength of 245 kips (11 times greater than an OHBDC design load of 22 kips). The hysteresis curve from the load vs. deflection plot was convergent during cycling of the load. Maximum steel stresses always occurred in the bottom transverse reinforcing, the largest of which was approximately 20 ksi. The reinforcement design for top transverse steel bars over the longitudinal girders is a critical area in AASHTO's design specifications. The stresses in these bars were insignificant, indicating the conservativeness of AASHTO design. Maximum stresses in the no. 9 bars over the pier were approximately 2.9 ksi. Based of these results, bridge decks supported by AASHTO girders, without intermediate diaphragms were observed to develop sufficient lateral restraint.

CHAPTER 4

TESTING IN THE UNITED STATES

Model and prototype testing similar to the Canadian research was conducted by the New York State Department of Transportation, the University of Texas, Case Western University, and the University of Wyoming. The model tests were designed to address ultimate strength and serviceability. Prototype and field tests used load magnitudes that indicated the deck service behavior.

4.1 Initial Modeling Concrete Bridge Decks

The objectives were to determine the ultimate capacity and distribution of bending moment in the model bridge deck (Beal, 1981). Two models of approximately 1/6th scale were built. Both modeled a five girder, 72 ft. simple span bridge. One was reinforced per AASHTO specifications and the other using four different isotropic reinforcing schemes. Four isotropic reinforcing schemes were used as follows, the minimal AASHTO reinforcing permitted, typical reinforcing used by Ontario (0.3%), a single layer of reinforcing located at three quarter depth of the slab and plain concrete (0.0%).

The AASHTO model was subjected to cracked and uncracked slab tests. Uncracked testing allowed stresses to be compared to a similar in-service bridge. Model reinforcement strains were measured by electrical resistance strain gages. Actual strains on the in-service bridge were found by removing the concrete cover and attaching strain gages to the bottom reinforcement. The stresses compared well for the transverse reinforcing, but less favorably for longitudinal reinforcement. Beal concluded the slab and beam interaction caused the differences in longitudinal values. For the cracked section, the bending moments in facia panels and interior panels were 60% and 65% of the theoretical values, respectively. The maximum stress recorded was 58% of the allowable working stress for a load 67% greater than design (0.58/1.67 = 0.34). Failure in the orthotropic model occurred by punching at all locations except cantilever portions of the deck.

The isotropically reinforced model was subjected to cracked and failure tests. Crack testing of the slabs indicated the moment coefficient and stresses were greater for the heavily reinforced panels. The maximum moment induced was 62% of the theoretical. This observation is consistent with the AASHTO model and is believed to be due to the increased reinforcement attracting more of the induced moment (Beal, 1981). Flexural failures occurred at the overhangs and a portion of the deck containing a single layer of reinforcing. This flexural failure in the facia panel occurred because the reinforcement was accidentally placed near the bottom instead at mid-depth. Thus no reinforcement was present in the negative moment regions. The rest of the deck panels failed in punching at loads six times greater than a design wheel load of 16 kips.

An analytical calculation based on theory developed for restrained slabs by Hewitt was performed. A restraint factor of 0.5 was used in the calculations. The analysis predicted loads less than the average test load and deflections less than observed during loading. Beal concluded the Hewitt model is adequate for predicting punching load but before it can be considered an accurate mathematical model more rigorous verification is required. This verification would include controlling and measuring the boundary forces during load tests using circular slab tests as earlier described.

4.2 Prototype and Model Concrete Bridge Decks

To further the previous investigation, another series of model tests with the addition of a prototype bridge test were performed under the direction of Beal (1983). Strength and serviceability evaluation of isotropically reinforced decks were the main objectives.

Ultimate strength and endurance limit states were used to determine the strength criteria for the deck design. Ultimate strength limit state employed design load of the

AASHTO wheel load (16 kips) plus a 0.3 impact value, dead and live load factors of 1.3 and 2.08, respectively, and a performance factor of 0.85 (ACI value for shear) consistent with New York practices. The factored design load was calculated to be 66 kips. Endurance limit state used the AASHTO design load with a performance factor of 0.4 (endurance limit) and no load factors. The endurance failure load was 52 kips, thus the ultimate strength design load of 66 kips controlled. For simplicity, the author used an upper strength criterion of 70 kips. Serviceability criterion was much more difficult to evaluate. Cracking as a criterion was dismissed as unreliable, thus limiting serviceability stress in the reinforcement was based on an allowable working stress of 24 ksi. for grade 60 bars. The test decks described below were designed using these criteria.

4.2.1 Model Tests

Four model bridges simulated girder spacing of 10 ft. - 2.5 in. and a 61 feet span. Model reinforcement strains measured by electrical resistance strain gages. One reinforcement layer at mid-depth and 0.3% isotropic reinforcement were used.

Mid-depth reinforcing failed in flexure suggesting reinforcement placement as well as slab restraint determined the failure mode. Converted loads were 83 to 138% of the strength of the 70 kip design criterion described earlier. This behavior indicates some of the test panels were marginally acceptable while others were not acceptable. Stresses in the model reinforcing all exceeded the serviceability stress criterion. Beal concluded a single layer of mid-depth isotropic reinforcing was not sufficient.

Isotropic 0.3% reinforced panels constructed per the OHBDC all failed by punching shear. The lowest capacity was in the model deck having no positive connection to the girders (i.e.. non-composite deck) but was still 76% larger than the 70 kip criteria. The stresses in the reinforcement were within 50% of the serviceability criteria except for one case. This case was theorized by Beal to be caused by alignment of the simulated wheel load in the longitudinal instead of transverse direction. This distributed the load over a smaller area in the transverse direction causing the high stress.

4.2.2 Prototype Tests

A bridge in Rochester, New York was instrumented in three of the four simply supported spans (Beal, 1983). The bridge had two spans each of approximately 60 and 38 feet carried by five girders spaced at 8 ft. - 9 inches. The deck panels were alternated between various patterns of isotropic and AASHTO reinforcing. The reinforcement designs were located in both composite and non-composite regions of the bridge structure. Four foot bar lengths were gaged with electrical resistance strain gages and were calibrated before installation. Gage location were over the second interior girder in the transverse and longitudinal directions for the top and bottom layers.

Loads were applied using a two-axle dump truck with a rear axle weight of 33 kips and gross weight of 48.8 kips. A load referred to as a crawl run and a simulated concentrated wheel load were performed. A truck straddling an interior girder with the rear tires over the centerline of the deck panel composed the crawl runs. An 8 in. by 20 in. plate simulated a dual wheel and a maximum load of 27 kips was applied. The results showed the stresses were small (2 ksi. or less) for the crawl runs and decreased as load became further away. Simulated wheel loads produced maximum stresses in the AASHTO reinforcing of 3.6 and 5.7 ksi. for the composite and non-composite portions of the deck, respectively. The largest stress in the isotropic reinforcing was 6.9 ksi. in the composite portion of the deck. Maximum stress in the longitudinal bar over the girder was 2.4 ksi. and the top transverse bar stress was negligible at the girder. No cracking was observed in the bridge deck.

The results for the model testing showed punching failure loads six times greater than the design wheel load plus impact. For both model and prototype service loads the stresses in the reinforcing were small. Mid-depth isotropic reinforcing was found to be unsuitable for bridge decks because of the service level stresses and failure loads. The OHBDC specifies the bridge deck must be composite to use an 0.3% isotropic design. The results show that non-composite isotropically reinforced decks tested were also sufficient.

4.3 Reinforced Concrete Bridge Decks Under Pulsating And Moving Loads

Almost all fatigue studies on model and prototype concrete decks supported on steel girders have been performed under a concentrated pulsating load applied at a fixed point. Sonoda and Horikawa (1982) simulated the effects of a traffic wheel load by applying a stepwise moving pulsating load on 1/3 scale model tests. The authors concluded that the fatigue life of deck slabs were remarkably reduced under the stepwise moving load compared to that under pulsating load applied at a fixed point. One passage of the stepwise moving load was equivalent in damage to a range of 80-600 cycles of a fixed pulsating load. An extensive research program was conducted to evaluate the effect of a moving wheel load on isotropically and orthotropically reinforced concrete bridge decks (Perdikaris, 1986).

Small scale model concrete deck slabs reinforced according to the current AASHTO Code and Ontario Code provisions were subjected to a moving constant load in the direction of traffic (Perdikaris , 1988). Fatigue tests under a pulsating load applied to a fixed point were also conducted to determine the effect of load movement on the fatigue strength and level of deterioration of the deck slabs.

The prototype structure represents a typical 50 feet long simply supported bridge with an 8.5 inch reinforced concrete deck supported on four W36 x 150 steel girders spaced at 7 ft. A 1/6.6 scale model was constructed. A total of five models were tested. One specimen (BUSP-1) was unreinforced and was subjected to a static and fixed pulsating load applied at fixed panels (the underline denotes the nomenclature used). Of the remaining four specimens, two specimens (BISP-1 and BIM-1) were reinforced with 0.3% isotropic top and bottom reinforcement and the other two (BOSP-1 and BOM-1) were reinforced with orthotropic reinforcement, 0.7% and 0.35% top and bottom in the transverse and longitudinal direction, respectively. Specimens BISP-1 and BOSP-1 were subjected to static and fixed pulsating load, while the specimens BIM-1 and BOM-1 were subjected to a moving constant wheel load applied to each of the three lanes.

For the tests under static and fixed pulsating load, an MTS hydraulic actuator was used to apply a simulated concentrated load. The custom built moving constant wheel load setup used a 40 kip double acting hydraulic cylinder through which a constant wheel load is applied. The hydraulic cylinder is attached to a steel trailer that rolls at a constant speed on a hardened steel plate attached to a steel beam. For the fixed pulsating as well as moving wheel load tests, the maximum applied load was kept at 60% of the static ultimate strength of the deck.

Under static loads the average static ultimate strength of 13.3 kips for specimen BOSP-1 was 59% higher than 8.38 kips for specimen BISP-1 (BISP-1 has 43% less reinforcement than BOSP-1). At failure BISP-1 showed three times the deflection shown by BOSP-1. The higher reinforcement in the transverse direction for specimen BOSP-1 and the resulting higher stiffness resulted in higher transverse bending moments and steel strains. The reinforcing wires in specimen BISP-1 yielded at 75% of ultimate strength. The unreinforced BUSP-1 failed in flexure while all the remaining specimens failed in a punching shear mode revealing a fan shaped crack pattern.

For a repeated pulsating load applied at a fixed point, the orthotropic reinforcement results in a higher fatigue strength (365,000 vs. 136,000 load cycles to failure at peak load of 60% of ultimate strength). For BOSP-1, the steel strains in the transverse direction are about twice as high as those in the longitudinal direction. For the BOSP-1 specimen the steel strains did not change after 160,000 cycles while those for BISP-1 changed by 60%. The failure mode in both the cases was punching shear.

The reverse phenomena was observed under constant moving load tests in specimens BOM-1 (orthotropic) and BIM-1 (isotropic) in that the load passages to failure is 200 and 4000, respectively. Only about 5% stiffness loss was observed in BIM-1 against 25% for BOM-1. Specimen BOM-1 under moving wheel load showed 40% increase in transverse steel strains in contrast to specimen BOSP-1 subjected to pulsating loads. BIM-1 transverse steel stresses increased by 40% to failure while longitudinal stresses remained unchanged. The failure mode was again by punching shear and the cracking was more extensive.

The orthotropic decks have greater stiffness than the isotropic decks and exhibit more effective arching action and consequently lower steel strains and less loss of stiffness at failure under pulsating loads. In the case of the moving wheel loads, a grid like cracking pattern is seen. Instead of localized steel strains as in the case of pulsating load tests, each one of the transverse reinforcement is stressed under the moving wheel loads. The orthotropic decks illustrated a lesser fatigue life.

A comparison between the behavior of specimens under moving and fixed pulsating loads shows that, in terms of number of load cycles or passages to failure, one passage of moving wheel load corresponds to 34 and 1800 load cycles for the isotropically and orthotropically reinforced deck slabs, respectively. Clearly, the moving load test is significantly different and perhaps more appropriate than the fixed-point pulsating load.

The fatigue life for isotropic deck was observed to be 20 times the orthotropic deck. In terms of static ultimate strength, under a design load of 20.8 kips with an impact factor of 1.3, the factor of safety for orthotropic deck is 23 and for isotropic deck is 14 with a factor of safety against cracking for both the decks being 4. This design load resulted in a maximum steel stress of 3 ksi. and a maximum deflection of 0.15 in.

The fracture process for a moving wheel load test is different from pulsating load tests, the cracking is more segmental, grid-like and extensive. The Ontario design resulted in 43% economy in steel, 37% reduced static ultimate strength (but still conservative) and 20 times higher fatigue strength. In short, it exhibited adequate strength and serviceability characteristics.

4.4 Experimental and Analytical Modeling in Texas

The University of Texas at Austin has conducted extensive studies on the Ontario method of bridge deck design. The experimental and analytical investigations are described below.

4.4.1 Experimental Program

A 20 x 50 ft. full-scale composite bridge (concrete deck on steel girders) was built and tested (Burns, 1986). One end of the 7-1/2 in. thick bridge was made of cast-in-place concrete (CIP), and the other end, of prestressed panels (4 in. thick) with a cast-in-place topping (3-1/2 in. thick). The deck was supported by three 36 in. deep steel girders. The bridge was simply supported on a 49 ft. span and loaded vertically at four locations. The bridge was first loaded statically to 60 kips per actuator (about three times the current AASHTO design wheel load) to study the service and overload response, after which it was subjected to sinusoidal fatigue loading with a maximum of 26 kips and a minimum of 5 kips per actuator. During the fatigue loading, the bridge was tested statically at intervals of about 1 million cycles to assess possible deterioration of the deck due to fatigue. After 5 million cycles of fatigue loading, the bridge was loaded statically to a maximum of 40 kips per actuator to study its service and overload behavior after fatigue cracking.

4.4.2 Analytical Program

To permit the extension of the experimental results to bridge decks other than the one studied experimentally, detailed finite element models of the specimen were developed using SAPIV. The reinforced concrete bridge deck was modeled using two layers of thick shell elements and the steel girder was modeled using three dimensional beam elements and connected at the appropriate nodal points of the shell element using rigid links.

Under the range of loads applied during the testing program, experimental results showed that stresses and deflections of the deck were small, and that the bridge behavior could be numerically predicted. Cracking of the deck was modeled using a smeared crack model.

4.4.3 Behavior of the Bridge Deck

The behavior of the bridge decks is characterized by several parameters as described below.

Load-Deflection Relationships: The experimental load-deflection behavior at the exterior and interior girder, across the loaded section, is essentially linear up to approximately three times the design wheel load. Cracking of the CIP deck did not significantly change its stiffness at the loaded points, even after fatigue loading. Linear load deflection behavior was also observed in the precast panels. Experimentally measured and analytically predicted load-deflection relationships compared well.

Deck Cracking: The CIP deck first cracked at an applied load of 38 kips per actuator, close to the analytically predicted value. The precast panel developed very minor cracks at 60 kips. After 5 million cycles of fatigue loading, the reduced amount of transverse reinforcement (compared to AASHTO design) did not cause excessive cracking. After initial cracking, propagation of minute cracks did occur on the bottom of the (CIP) deck. On the precast panels, cracks did not propagate significantly. The cracks above the panels gap widened slightly, but crack widths were very small throughout all the tests.

Localized Stresses: Strain gages were installed on the bridge deck near the loaded points and along the steel girders. Under the design load of 20.8 kips per actuator, the maximum concrete stress was only about 0.4 ksi. in both the longitudinal and transverse directions, in both types of bridge decks. Maximum stress in the transverse reinforcement near the load in the CIP deck reached about 1.8 ksi. at the design load level. Fatigue loading did not significantly change the local stresses in the deck. Stresses in concrete and reinforcement at the other locations were relatively small compared to those near the loaded point.

Bending Moments: Three strain gages were installed to measure the strain gradient at several locations. Using an assumed linear strain gradient, the bending moments in the deck were obtained in both the transverse and longitudinal directions. At the design load level of 20.8 kips per actuator, peak transverse and longitudinal moments were about 2 kip-ft./ft., less than the current AASHTO design value. After the 5 million cycles of fatigue loading, peak moments increased only slightly. The maximum longitudinal moment occurred in a very small region near the load.

Transverse Membrane Forces: From the strain gradient at each gaged location, transverse membrane forces were calculated for every load stage. At one location near the load in the CIP deck, transverse membrane forces were all tensile before the deck was significantly cracked. After the deck was significantly cracked, the transverse membrane forces near the load were all compressive, and increased as the load increased. The analytically predicted membrane force distribution agreed reasonably well with the experimental results.

Effect of Intermediate Diaphragms: The effect of intermediate diaphragms on the bridge deck behavior was studied using the measured test results with and without midspan diaphragms, together with the verified analytical model. The presence of midspan diaphragms did not significantly change the local stiffness, local stresses, moment distribution, nor compressive membrane force of the deck. However, intermediate diaphragms might still be necessary for lateral load distribution, construction purposes or overall stability.

Effects of Arching Action: Significant membrane forces did not exist before the start of flexural cracking in the deck. Because significant cracking occurred only at the CIP end, compressive membrane forces existed only at that end. Even without taking arching action into account, current AASHTO requirements for slab flexural reinforcement were conservative in this case.

4.4.4 Conclusion and Recommendations (Burns, 1986a)

> "1.) A full-scale, cast-in-place bridge deck on steel girders, detailed in accordance with the Ontario Highway Bridge Design provisions, performed satisfactorily under current AASHTO design load levels, with respect to overall behavior of the bridge specimen, the local stiffness of the deck at the loaded point, crack widths, and bending moments in the deck.
>
> 2.) A similarly detailed deck with precast, prestressed panels also performed satisfactorily.
>
> 3.) Under overload conditions (about three times the current AASHTO design wheel load), the behavior of the deck slab was essentially linear, except for some non-linearity due to minute tensile cracking of concrete. Fatigue loading did not significantly change the behavior of the deck under service loads nor under overloads.
>
> 4.) These bridge decks had about 60 percent of the reinforcement required by the current AASHTO code.
>
> 5.) Analytical predictions and experimental results agreed closely, showing that the analytical models of the bridge specimen are satisfactory, and can be extended to other bridge configurations.
>
> 6.) The presence of midspan diaphragms or additional diaphragms did not significantly change the local stiffness, local stresses, moment distribution, nor compressive membrane forces of the deck.
>
> 7.) Compressive membrane forces did not significantly affect the performance of the bridge at loads below cracking.
>
> 8.) Based on the results of this research, field use of the Ontario-type decks, similar to the one tested in this investigation, seems fully justified."

4.5 Behavior of Ontario-Type Bridge in the Negative Moment Region

The specific objectives of this study were (Burns, 1986b):

> "1) To study the pre- and post-fatigue behavior of the negative moment region of the cast-in-place and panel decks under service load and overload conditions;
>
> 2) To test previously developed analytical models against the observed behavior of the bridge; and
>
> 3) To study the ultimate capacity and behavior of the cast-in-place and panel decks under single and double concentrated loads."

4.5.1 Test Specimen, Loading and Analytical Models

The test specimen was a full-size composite bridge with 7 1/2 in. thick concrete deck supported by three 36 in. deep, W-shape steel girders, spaced at 7 ft. Half of the deck had two layers of reinforcement, designed in accordance with the provisions of the OHBDC. The other half had 4 in. thick precast, prestressed panels which replaced the lower grid of reinforcement in the cast-in-place deck. The span was 40 ft.

A negative moment was induced and then a fatigue test consisting of 5 million cycles (load range of 5 kips to 26 kips per actuator) was conducted and then followed by a static test. Finally, the test specimen was subjected to concentrated load tests involving single and tandem loads. The vertical deflections were measured by dial gages placed at select locations underneath the bottom flange of the girders, while the reinforcement strains were measured using paper backed gages and the concrete strains using surface mounted strain gages.

The analytical finite element model used in the previous test was modified slightly and only the cast-in-place deck was modeled.

4.5.2 Observations and Conclusions

The investigator made the following observations:

1) Both the cast-in-place and the panel halves of the full-scale bridge performed satisfactorily at the support region when subjected to negative moment levels consistent with AASHTO design loads.

2) Satisfactory behavior was seen at midspan level under static tandem loads of 2.5 times the current AASHTO design level spaced 4 feet apart for both the cast-in-place and the precast prestressed panel decks.

3) Five million cycles of fatigue loading (5-26 kips) did not significantly affect the deck as was observed from static tests before and after the fatigue loading.

4) The finite element analytical model which included the revised stiffness in the cracked regions agreed well with the experimental results and thus can be extended to other bridge configurations.

5) Punching shear failure was observed under single tandem as well as under tandem loads.

6) The load capacity was controlled by the punching shear mode.

7) The ACI as well as the AASHTO formulae for punching shear were found to be conservative in estimating the load capacity of the deck.

8) The precast, prestressed panel deck was experimentally found to be stronger, stiffer and more crack resistant than the cast-in-place deck.

In summary, the investigator concluded that cast-in-place and precast, prestressed panel bridge decks similar to the one tested, and detailed with the Ontario-type reinforcement, can be safely used.

4.6 Behavior of Skewed Ontario-Type Bridge Decks

The objectives of this research were (Kueng,1988):

1) to verify, experimentally and analytically the effect of skew on bridge performance,

2) to analytically determine the effects of span-to-depth ratio, longitudinal spacing of live loads, width of cantilever overhangs, and the presence of integral barriers on the bridge performance, and

3) to comment on the acceptability of the Ontario method of design based on the above mentioned criteria and suggest any modifications.

4.6.1 Development of the Experimental Model

Based on earlier work, a modified test specimen which would represent one quarter of a skew bridge was used. Several analytical trials were conducted. A quarter rectangular model was studied to obtain appropriate boundary conditions. The deck reinforcement for the model was of the Ontario type with 0.3% reinforcement, no. 5 bars, both ways except for the transverse edge reinforcement placed parallel to the skew edge where 0.5% isotropic reinforcement was used. The deck thickness was increased from 7.5 in. to 9.5 in. up to four feet from the skew edge, measured perpendicular to the skew edge.

4.6.2 Test Procedure

The 45 degree skew edge was first overloaded until visible cracks occurred at 51 kips. The 20 degree skew edge and then the center was overloaded to 60 kips. Then the 45 degree skew edge was loaded to failure following which the 20 degree skew edge was loaded to failure and then the center was tested to failure.

In tests conducted at the edge, the first bottom cracks occurred at 25 kips and the top cracks at 35 kips. At the center, first cracks occurred at 30 kips on the bottom surface and at 50 kips at the top. Bottom cracks at the 45 degree skew edge were perpendicular to the edge and for the center and 20 degree skew edge were fan shaped. Top cracks propagated in circular arcs around each edge loading point but no significant top cracking occurred for the center tests. Crack widths were smaller than the implied maximum allowable crack widths. The specimen behaved linearly beyond the service live load level. The deck failed by shear near the wall at each skew edge and by punching shear at the center. All failures were sudden.

4.6.3 Test Results and Analysis

Computation of transverse and longitudinal moments showed that experimental and analytical values were in agreement. The negative transverse moment at the interior support was small as compared to the positive transverse moment at midspan of the deck before cracking. The longitudinal moment decreased quickly away from the load and was negligible at a distance of 5 ft. Comparing the results of non-skew and skew bridges, the maximum intensities of the compressive membrane forces are approximately equal. At the 45 degree skew edge, the failure mode was a combination of shear and torsion near the interior support at a load of 97 kips. Considering torsion, the shear capacity was 58 kips. Torsion should be considered in the design because the width of the thickened region at the edge, is much greater than the loading length of the wheel.

Various methods of estimating the shear stress were investigated including one-way shear, shear-torsion interaction, shear-flexural-compressive interaction, etc. The estimates were all conservative with respect to the AASHTO wheel loads.

4.6.4 Parametric Studies

Span-to-depth ratio: The bridge model was analyzed for span-to-depth ratios of 11.2, 15.3 and 21.0. With decreasing deck thickness the in-plane membrane forces increased. This is because a shallow arch member develops larger lateral reactions than a deep member, though it was seen that span-to-depth ratios over 15 still developed adequate arching actions. This upper limit was imposed by the OHBDC code to meet requirements such as deflection control, shear strength and serviceability. While showing little difference in the basic flexural behavior, negative transverse moments at the interior girder increased with decreasing span-to-depth ratio.

Effect Of Overhang: The bridge model was modified to omit the overhang present in the original model and was analyzed. The transverse membrane forces and transverse forces were identical to the original model. Thus contrary to the OHBDC requirement of minimum overhang width, no overhang is required to develop sufficient in plane stiffness to develop arching action.

Effect Of Barrier Stiffness: The bridge model was analyzed for three different cases: no barrier, Texas T5 concrete barrier and a very stiff barrier. It was observed that though changing the barrier stiffness did affect the distribution of the tensile membrane forces, it did not affect the magnitude of the compressive membrane forces and thus do not warrant design consideration.

Effect Of Line Load: Four cases involving line loads were studied: Two loading points spaced at 20 ft., six loading points spaced at eight ft., eight loading points spaced at four ft. and twelve loading points spaced at four ft. It was determined that the development of membrane forces requires that the wheel spacing should be at least four ft., or the total loaded length should be shorter than half the length of the bridge. The eight ft. spacing is better than the four ft. in all cases except for the case when the wheel load exceeds the punching shear capacity.

4.6.5 Conclusions and Recommendations (Kueng, 1988)

Kueng made the following observations:

1) Ontario deck design was found to be satisfactory with regard to overall behavior, crack widths, local stiffness near the loaded points and bending moments.

2) Under 3 times overload conditions the behavior of the deck was fairly linear.

3) The flexural capacity of the skew bridge was significantly increased by the compressive membrane forces.

4) After cracking, arching action can be developed even for span-to-depth ratios greater than 15.

5) Increasing span-to-depth ratios increased the transverse membrane forces.

6) Neither the overhang width nor the integral barrier stiffness affected the membrane forces significantly.

7) The wheel spacing should be at least eight ft. or loaded length should be less than half the length of the bridge.

8) Similar skew bridges to those used in this study may be used in practice.

9) For further study, the effects of different edge stiffening and strength should be examined experimentally. The effects of arching action on the serviceability of minimally reinforced concrete bridges needs to be examined.

4.7 Research in Wyoming

The University of Wyoming, in conjunction with the Wyoming Highway Department and the FHWA, has instrumented two non-composite steel girder bridges on an interstate highway (Puckett, 1988a; Puckett, 1988b). One deck was built per the AASHTO specification and the other was built per the OHBDC. Sixty-four gages were installed in each deck to monitor the reinforcement strains at service load levels under static and moving load conditions.

The maximum observe reinforcement stresses were observed to be in the bottom transverse bars. The maximum stress observed in the AASHTO and Ontario designs were 6.6 ksi. and 7.9 ksi., respectively. The applied load was 30,000 lbs. Scaled to the AASHTO design load these values become 4.5 ksi. and 5.4 ksi.

In the moving load tests, a dump truck with a weight of approximately 54,000 lbs. was driven at various speeds along several paths across the test section. The maximum observed stress in these tests was also in the bottom transverse bars and was observed to be 3.2 ksi. and 6.5 ksi. for the AASHTO and Ontario designs, respectively.

Although bars oriented both longitudinally and transversely in the top and bottom were instrumented, only the bottom transverse steel bars were stressed significantly. The stresses in the other bars were under 1.0 ksi.

4.8 Long Term Testing in New York

Loftus (1988) studied several in-service isotropically reinforced concrete decks and summarized five years of data from load tests and visual inspections performed annually. The load tests consisted of static and moving load situations. The reinforcement stresses were the greatest in the bottom transverse reinforcement and the maximum observed stress due to the overload was 13.4 ksi. This data was converted to the AASHTO design wheel load effect and the maximum was 6.5 ksi. The top transverse steel over the girder was observed to be quite low with a maximum of approximately 200 psi. The longitudinal bars were also minimally stressed with the maximum observed stress of approximately 400 psi. under the single AASHTO wheel load and approximately 1000 psi. due to test vehicle (considering the influence of both axles). Loftus noted that a load did not stress gages on longitudinal bars until within approximately five feet of the gage. Hence, the load effect is localized and the effect of numerous axles is not a concern.

The inspection reports indicated that minor transverse cracking occurred in the test decks and others built with isotropic reinforcement. Loftus noted that the extent of the cracks was no more significant than conventionally reinforced decks.

CHAPTER 5

RESEARCH IN PROGRESS OR SCHEDULED

5.1 Research in Ohio

The Case Western University, in conjunction with Ohio DOT and FHWA, is conducting 1/6.6 and 1/3 scale model tests on the AASHTO and the Ontario bridge decks to determine (Perdikaris, 1988):

a) the effect of loads on the fatigue life of the deck and to determine the endurance limit for fixed pulsating loads and moving load tests,

b) the magnitude of compressive membrane forces and the effect of cracking, damage accumulation and failure modes on these forces under increasing fixed pulsating or moving load passages,

c) the safe strength related to the punching shear failure mode and to recommend minimum reinforcement ratio for serviceability, and

d) the procedures for more economical yet safe design.

5.2 Research in Florida

The University of Florida, in conjunction with the Florida Department of Transportation is conducting a series of load tests on seven approximately half-scale concrete bridge decks. The focus is to extend the Ontario empirical design approach to the use of higher span-to-depth ratios (18 to 22), greater overhang to thickness ratios. The possibility of using bulb tee girders is also being investigated.

CHAPTER 6

CONCLUSIONS

The state-of-the-art in the field of compressive membrane action related to lightly reinforced concrete decks has been summarized. Three main areas of research were addressed:

1) work related to the compressive membrane action theory as applied to any concrete slab,

2) work conducted by the Ontario Ministry of Transportation upon which the current OHBDC is based, and

3) research conducted in the U.S. to confirm and expand upon the earlier work and to possibly recommend the incorporation of results into the AASHTO specification.

A short section on current work being conducted in the U.S. is also included.

In summary, the work related to lightly reinforced concrete bridge decks has been reviewed and summarized. The results are consistent and without exception, the investigators report adequate factors of safety and favorable behavioral characteristics of the lightly reinforced decks.

REFERENCES

Batchelor, B. deV., Hewitt, B. E., and Csagoly, P., "An Investigation of the Fatigue Strength of Deck Slabs of Composite Steel/Concrete Bridges", *Transportation Research Record* 664, Transportation Research Board, 1978, Vol. 1, pp. 153-161.

Batchelor, B. deV., Hewitt, B. E., Csagoly, P., and Holowaka. M., "Investigation of the Ultimate Strength of Deck Slabs of Composite Steel/Concrete Bridges", *Transportation Research Record* 664, Transportation Research Board, 1978, Vol. 1, pp.162-170.

Batchelor, B. deV., and Tissington, I. R., Study of Partially Restrained Two-way Bridge Slabs Subjected to Concentrated Loads, Report RR 178, Ontario Department of Transportation and Communication, March 1972.

Beal, D. B., Strength of Concrete Bridge Decks, Research 89, Engineering Research and Development Bureau, New York State Department of Transportation, July 1981.

Beal, D. B., Reinforcement for Concrete Bridge Decks, Research Report 105, Engineering Research and Development Bureau, New York, (need date???).

Brown, L. M., Wagner, W., Instrumentation of the Conestogo River Test Bridge, Report 76-TIL-19, Research and Development Division, Ministry of Transportation and Communication, July 1976.

Burns, N.H., Klingner, R.E., Fang, I. K., Worley J.A., Behavior of Ontario-type Bridge Decks on Steel Girders, Research report FHWA/TX-86/78+350-1, project 3-5-83-350, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, January 1986.

Burns, N.H., Klingner, R.E., Tsui,C.K., Behavior of Ontario-type Bridge Decks on Steel Girders: Negative Moment Region and Load Capacity, Research report FHWA/TX-86/78+350-3, Project 3-5-83-350, Center for Transportation Research, Bureau of Engineering Research, The university of Texas at Austin, January 1986.

Christiansen, K.P., "The Effect of Membrane Stresses on the Ultimate Stresses on the Ultimate Strength of Interior Panel in a Reinforced Concrete Slab", *The Structural Engineer*, August 1963, No.8 VOL 41.

Christiansen, K.P, and Frederiksen, V.T.,"Experimental Investigation of Rectangular Concrete Slabs with Horizontal Restraints", *Material and Structures*, Vol. 16, No. 93, May-June 1982, pp. 178-192.

Csagoly, P., Design of Thin Concrete Deck Slabs by the Ontario Bridge Design Code, Report SRR-79-11, Ontario Ministry of Transportation and Communication, May 1979.

Csagoly, P., Holowka, M., and Dorton, R., "The True Behavior of Thin Concrete Bridge Slabs", *Transportation Research Record* 664, Transportation Research Board, 1978, pp. 171-179.

Csagoly, P., and Dorton, R. A., "The Development of the Ontario Highway Bridge Design Code", *Transportation Research Record* 665, Transportation Research Board, 1980, pp. 1-12.

Dorton, R. A., Holowka, M., and King, J. P. C., "The Conestogo River Bridge - Design and Testing, "*Canadian Journal of Civil Engineering*, Vol. 4, No. 1, 1977, pp. 18-39.

Dorton, R. A., "The Ontario Bridge Code - Development and Implementation", Canadian *Journal of Civil Engineering*, Vol. 11 No. 4, 1984, pp. 824 - 832.

Hewitt, B. E. and Batchelor, B., "Punching Shear Strength of Restrained Slabs," *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 101, No. ST 9, Proc. Paper 11545, (September 1975), pp. 837-853.

Holowka, M., Csagoly, P., Testing of a Composite Prestressed Concrete AASHTO Girder Bridge, Report RR 222, Ontario Ministry of Transportation and Communication, July 1980.

Holowka, M., Testing of a Trapezoidal Box Girder Bridge, Report RR 221, Ontario Ministry of Transportation and Communication, November 1979.

King, J. P. C., Holowka, M., Dorton, R. A. and Agarwal, "Test Results from the Conestogo River Bridge", *Proceedings Bridge Engineering Conf.*, Transportation Research Board, 1978.

Kirkpatrick, J., Rankin, G.I.B., and Long, A.E.," The Influence of Compressive Membrane Action on Serviceability of Beam and Slab Bridge Decks", *The Structural Engineer*, Vol. 64B, No. 1 March 1986.

Kueng, H.W., Behavior of Ontario-type Bridge Decks, with and without Skew, Dissertation presented to the Faculty of the Graduate School of the University of Texas at Austin in partial fulfillment of the Requirements for the Degree of Doctor of Philosophy, The University of Texas at Austin, May 1988.

Ontario Highway Bridge Design Code, Ontario Ministry of Transportation and Communications, 1983.

Park, R., "Ultimate Strength of Rectangular Concrete Slabs under Short Term Loading with Edges Restrained Against Lateral Movement", *Proceedings of the Institution of Civil Engineers*, No. 6705, Vol. 28, June 1964, pp 125-150.

Park, R., "The Ultimate Strength and Long-Term Behavior of Uniformly Loaded, Two-Way Concrete Slabs with Partial Restraint at All Edges", *Magazine of Concrete Research*, Vol. 16, No. 48, September 1964 ,pp 139-152.

Park, R.,"The Lateral Stiffness and Strength Required to Ensure Membrane Action at the Ultimate Load of a Reinforced Concrete Slab-and-Beam Floor", *Magazine of Concrete Research*, Vol 17, No. 50, March 1965.

Perdikaris, P.C., Beim S., RC Bridge Decks Under Pulsating and Moving Load, *ASCE Journal of the Structural Division*, Vol. 114, No. 3, March 1988.

Puckett, J.A., "Comparison of the AASHTO and Ontario Deck Designs", *Proceedings of Symposium on Research in Progress*, Des Moine, IA, Sept, 1988.

Puckett, J.A., Personal Communications, 1988.

Sonoda, K., Horikawa, T., Fatigue Strength of Reinforced Concrete Slabs under Moving Loads , *Proceedings of the IABSE Colloquium*, Fatigue of Steel and Concrete Structures, Laussane, Switzerland, 37, 455-462.

Standard Specifications for Highway Bridges, The American Association of State Highway and Transportation Officials, Washington D.C, 1983.