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Nonlinear Finite Element Analysis of Deteriorated RC Slab Bridge

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ABSTRACT: Applications of nonlinear finite element analysis (NLFEA) to complete structures have been limited. The study reported in this paper examined the reliability of NLFEA to assess strength and stiffness of a three-span reinforced concrete slab bridge that was loaded to failure in the field. The researchers at the University of Cincinnati and Delft University of Technology, in The Netherlands, conducted preliminary analyses that were then compared to the measured responses. These analyses indicate a significant influence of tensile behavior of concrete in the postcracking range, and the level of slab membrane force that is directly affected by the assumed horizontal support conditions at the slab-abutment connections. Reasonable correlation of the measured responses was possible by removing the horizontal restraints at the slab-abutment connections. However, such models do not simulate the observed behavior at the abutments. The shear keys at the slab-abutment connections would not permit free horizontal movements, yet the slab can rotate about the shear keys. The resulting rotation would reduce the membrane force that can be developed. An improved model incorporating this behavior produced better results than the original model assuming full horizontal restraints at the abutments.

INTRODUCTION

Motivated by the need to better understand seismic behavior of reinforced concrete frames, nonlinear analysis of such structures has gone through significant improvements in the past three decades, and correspondingly many analysis codes have been developed (e.g., NONSAP, DRAIN-2D, ANSR). These codes were largely "calibrated" based on observations made on simple elements and/or structural systems. In the realm of inelastic response of simple but complete reinforced concrete frame structures, severe shortcomings have been observed. Examples abound where seemingly reasonable modeling assumptions regarding formulation of hysteresis behavior, element behavior in the inelastic range, kinematic interaction between var-

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422

ious elements, and material interaction between different orthogonal phenomena (among others) have provided inadequate response correlation (Bertero et al. 1984; Charney and Bertero 1982; Shahrooz and Moehle 1987). Through continued efforts to overcome these difficulties, techniques for proper nonlinear analysis of complete reinforced concrete frame structures appear to be well established now.

Most efforts in nonlinear finite element analysis (NLFEA) have focused on simulating response of individual elements. Applications of NLFEA to complete structures have been limited to design of special structures (van Mier 1987; Muller 1985; Milford and Schnobrich 1984) or to analytical studies of large structures (Huria et al. 1993; Meschke et al. 1991). Only recently have there been some attempts to calibrate NLFEA by correlating experimental data from tests on complete structures, e.g. correlation studies of data from tests conducted on a containment vessel (Clauss 1989) and simulation of response of the reinforced concrete frame–wall model in the U.S.-Japan cooperative study (Chesi and Schnobrich 1991). As a result, reliability of NLFEA for complete structures has not been fully explored. Continued correlation studies of complete structures are needed to calibrate nonlinear finite element analysis, and to verify that NLFEA can be used as a reliable tool if NLFEA is to become of practical use.

A study focused on behavior and NLFEA of deteriorated reinforced concrete slab bridges was completed recently (Aktan et al. 1992; Zwick et al. 1992). The study revolved around destructive testing of a 38-year-old-three-span slab bridge that had been decommissioned because of its deteriorated state. One emphasis of the research was to evaluate the reliability of NLFEA in conjunction with system identification to assess strength and stiffness characteristics of aged reinforced concrete bridges. Prior to the destructive testing of the bridge, several predictive analyses were performed by the researchers at the University of Cincinnati and Delft University of Technology, in The Netherlands. A primary objective of these analyses was to establish the expected bounds of strength and stiffness that were essential for proper design of the test setup and instrumentation. However, these analyses also led to an investigation of other issues. First, some of the engineering aspects of modeling of complete structures could be studied, e.g. the significance of assumed boundary conditions, and the effects of assumed material properties such as concrete compression and tensile properties, constitutive relation for reinforcing steel, and postcracking behavior of concrete. Second, it became possible to gauge the effectiveness of two different NLFEA softwares for predicting response of complete structures at various limit states. Third, sensitivity of results due to differences in modeling of complete structures could be explored.

A main focus of this paper is to present the predictive analyses carried out at the University of Cincinnati and Delft University of Technology. The predicted overall responses are compared with the corresponding experimental results to discuss the aforementioned objectives.

DESCRIPTION OF TEST BRIDGE

The test specimen was a three-span, reinforced concrete, skewed slab bridge that was constructed in 1953, see Fig. 1. The 438-mm deck was supported on two rows of piers and on two abutments. As seen from Fig. 1, the slab and pier caps were of monolithic construction, with shear keys between pier caps and piers. The connection between the slab and abutments comprised standard shear keys (Fig. 1). The piers were set on footings cast
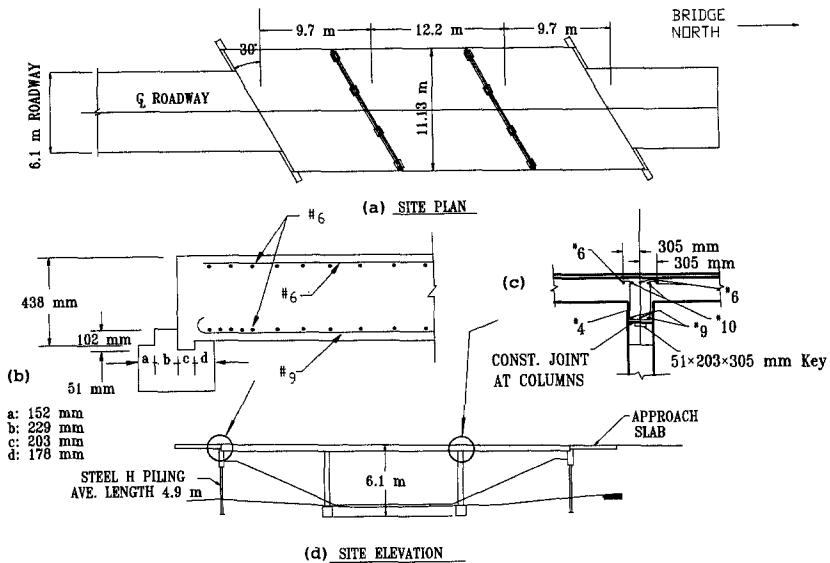
FIG. 1.   Test Bridge

on the bedrock, and the abutments were placed on six steel piles driven to the bedrock. The reinforcement layout is illustrated in Fig. 2. The bottom steel ratio ranged from 0.0038 (near the pier line) to 0.0076 (near the midspan). The top steel ratio was between 0.0011 (near the midspan) and 0.0095 (at the pier line).

The condition survey of the bottom surface of the slab revealed light deterioration, comprising of small cracks and minor spalling. Occasional rust cracks could be seen on the bottom side. The top surface had experienced significant deterioration. Approximately 76 mm of the concrete had either deteriorated severely or spalled off completely along both shoulders over a 1,830 mm to 2,440 mm width. Several of the rusted top reinforcing bars had been exposed on the shoulders. The driving lanes were in reasonably good condition. Furthermore, the concrete quality and strength were much poorer on the shoulders than the driving lanes. The details of bridge condition are provided elsewhere (Aktan et al. 1992; Zwick et al. 1992).

Standard tests conducted on cores indicated an average compressive strength of 52 MPa, tensile strength of 4.5 MPa, and modulus of elasticity of 34,000 MPa. Based on ASTM standard tensile tests, the properties of the reinforcing steel was determined. The bars were apparently Grade 40 steel with yield and ultimate stresses of 345 MPa and 680 MPa, respectively. The modulus of elasticity was 199,800 MPa, and the strain hardening modulus was approximately 3.3% of the elastic modulus.

SUMMARY OF EXPERIMENTAL PROGRAM AND RESULTS

The experimental program consisted of nondestructive (modal and truck load tests) and destructive tests (Aktan et al. 1992; Zwick et al. 1992). Using the modal test results, approximate elastic stiffness characteristics of the abutments were identified. Three loaded dump trucks (each weighing 142
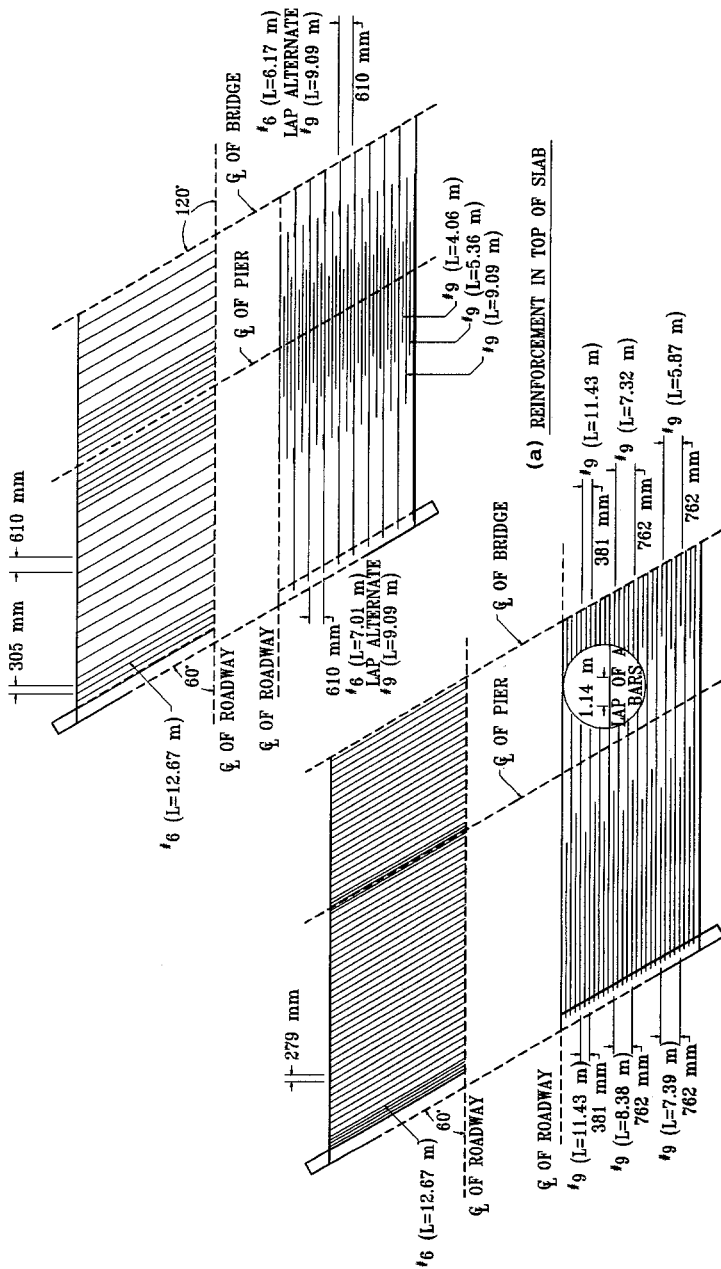
424

FIG. 2. Slab Reinforcement

(a) REINFORCEMENT IN TOP OF SLAB

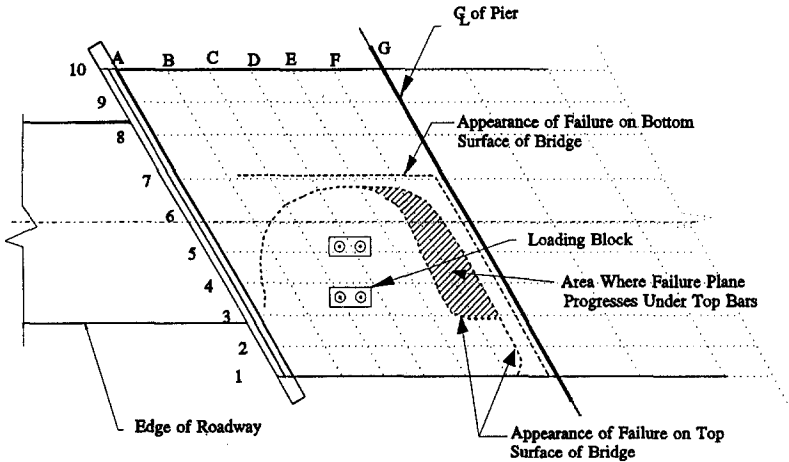(b) REINFORCEMENT IN BOTTOM OF SLAB

425

FIG. 3. Failure Pattern at Conclusion of Destructive Testing

kN) were placed on the bridge deck in six different configurations. The vertical deflection profiles of the slab were measured.

The destructive tests were carried out by placing four, hydraulic servo-controlled actuators on the south-east quadrant of the bridge. To distribute the load and simulate the "footprints" of a tandem trailer, the actuators were placed on two 610 mm × 1,830 mm concrete blocks. The reaction required to load the bridge was provided by rock anchors attached to the actuators. The load on each block was controlled to be equal.

The bridge was loaded at equal increments of 142 kN (71 kN on each block). The total load on the bridge reached 3,200 kN beyond which the bridge failed in a brittle manner. The vertical deflection of the deck under the reaction block closer to the edge was 69 mm prior to failure. The topside view of the failure pattern is shown in Fig. 3. It consisted of a diagonal-tension failure at the edge of the pier-slab connection in the damaged shoulder. It progressed along the pier line until approximately the centerline of the deck at which it arched back towards the abutment. The failure pattern suggests a flexure-shear-type mode of failure.

Using the readings from the strain gages, which had been attached to the slab reinforcing bars, the first yielding of the reinforcing bars was determined to occur when the load on the bridge reached 2,893 kN. These readings also indicated that the first yield occurred in two bottom longitudinal bars (one bar was adjacent to the loading block closer to the edge and the other bar was on the south-west quadrant of the bridge approximately 2,134 mm from the other loading block) and one bottom transverse bar (between the two loading blocks). A comparison between this load and the ultimate load suggests that the mode of failure was not significantly influenced by the flexural behavior of the bridge. This observation may also be drawn from the damage pattern, Fig. 3.

ANALYSES CONDUCTED AT UNIVERSITY OF CINCINNATI

Prior to the destructive testing phase of the research program, a number of analyses were conducted at the University of Cincinnati. These analyses

426

were conducted primarily for establishing the expected strength and stiffness, which were necessary for design of the loading setup and instrumentation.

The level of complexity in performing the predictive analyses ranged from simple yield line analysis to linear and nonlinear finite element analyses. Considering the time constraints, it was decided not to incorporate the observed damage, most notably near the shoulders. The nonlinear analyses were aided by system identification, and were conducted in the context of establishing probable upper bounds of response. Attempts were made to limit typical wide variations of predicted response. For example, the measured material properties were used, and the results from nondestructive modal tests were utilized to "calibrate" support conditions.

YIELD LINE ANALYSIS

Simple yield line analyses were carried out in reference to the loaded span. The boundary conditions were assumed to be: (1) Simple supports at both the abutment and pier cap; and (2) fixed at the abutment and simple support at the pier cap. The effects of strain hardening in the reinforcing bars were approximated by setting the available ultimate moment strength equal to 1.25 times the nominal strength computed in accordance with the ACI 318 building code (*Building* 1989). The loading blocks were simulated by using two concentrated loads located at 1,830 mm apart and perpendicular to the center line of the bridge. Four yield-line patterns were considered, as shown in Fig. 4. Yield line pattern 3 produced the lowest values, which were 2,105 kN for simple-simple supports and 2,950 kN for simple-fixed supports.

The yield-line analyses correlated reasonably well with the ultimate load resisted by the bridge (3,200 kN). In general, such analyses are expected to produce a good estimate of ultimate load-carrying capacity if yield lines are formed at failure. However, in this case the good correlation appears to be coincidental, because neither yielding of the reinforcing bars nor the damage pattern suggests initiation of yield lines.
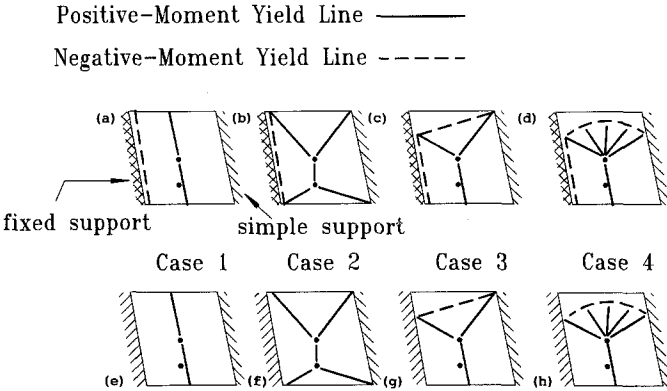

FIG. 4. Yield Line Patterns

427

NONLINEAR FINITE ELEMENT ANALYSIS

Modeling

The nonlinear analyses were conducted using a microcomputer-based software named 3DSCAS (Lee et al. 1991). The architecture and numerical algorithms of the program are based on ANSR-III (Ougourlian and Powell 1982), and it includes several linear and nonlinear elements. The particular elements used in this study were a five-spring RC beam-column element (Ghusn and Saiidi 1986), a linear spring (Huria et al. 1991), and a RC nine-node degenerated isoparametric shell element (Milford and Schnobrich 1984). The shell element is based on the layering concept, by which up to 10 layers of concrete and up to four layers of steel bars can be simulated. Different properties can be assigned to each concrete or steel layer.

To preserve the continuity between adjacent spans, it was decided to
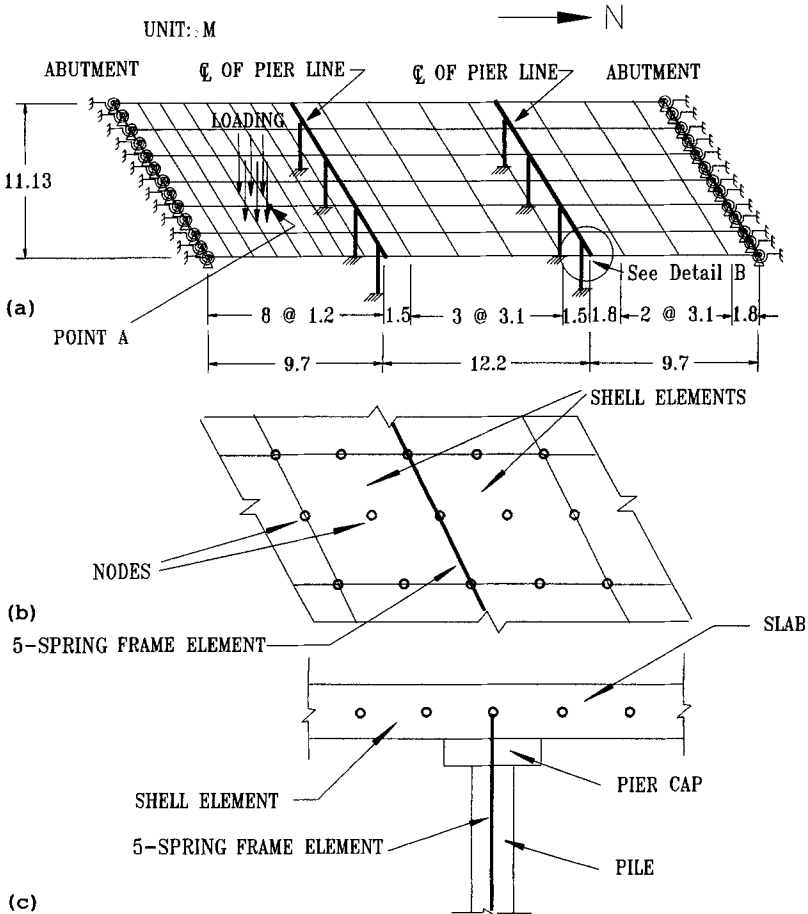
Detail B :   ANALYTICAL MODEL FOR PIER AND SLAB CONNECTION

FIG. 5.   Finite Element Model of Test Bridge for Predictive Analyses

428

model the entire slab-pier-abutment system. For this purpose, 102 RC shell elements (located at the mid-depth of the physical slab) were used to model the bridge deck, as shown in Fig. 5. Additional refinement of the mesh size and layout was not carried out as mesh sensitivity studies did not indicate significant improvements beyond the illustrated mesh. The piers and pier caps were modeled by using 32, five-spring RC beam-column elements. The connections between piers, pier caps, and bridge deck were modeled as shown in Fig. 5. The two loading concrete blocks were simulated by several concentrated loads acting on the nodes covered by the blocks (refer to Fig. 5).

Considering that the deck was connected to the abutments by shear keys (Fig. 1), the horizontal movement of the bridge deck at the abutments was restrained in the analytical model. To simulate the rotational stiffness at the abutments which was observed during the modal tests, a number of linear rotational springs were placed at each abutment (Fig. 5). Based on the modal-test results, appropriate spring stiffness constants were identified such that the measured and computed modal characteristics of the bridge matched closely. The assumptions regarding modeling of the boundary conditions and stiffness at the abutments were further checked using the results from the truck load tests during which loads were several times larger than those used in the modal tests or the service level loads. The experimental and analytical deflection profiles were found to be reasonably similar. Small deviations from the experimental data were attributed to ignoring the observed damage in the analytical model. However, the differences were deemed negligible for the initial predictive analyses.

The concrete and steel constitutive relationships are shown in Fig. 6. The material properties, shown in this figure, were selected according to the test results. The tensile behavior of the concrete before and after cracking was considered. The postcracking participation of the concrete was assumed to diminish at a strain corresponding to 10 times the cracking strain, i.e. $k = \varepsilon_t/\varepsilon_{cr} = 10$, where ε_t is the strain at which tensile strength diminishes, and ε_{cr} is the cracking strain. This model is referred to as model A. This value is within the range expected for typical reinforced concrete slabs with no or little confinement (Gilbert and Warner 1978). A higher participation of concrete beyond cracking was also considered to obtain an upper-bound estimate of strength and stiffness to ensure that the test apparatus would be adequate. For this purpose, the concrete was assumed to provide tensile resistance until a strain of 20 times the cracking strain ($k = 20$). This model is referred to as model B. The failure envelope for concrete was based on the envelope proposed by Kupfer and Gerstle (1973). Depending on the ratio of maximum to minimum principal stresses, four different failure zones may occur (Darwin and Pecknold 1974), i.e.: (1) Yielding and crushing of concrete under biaxial compression; (2) biaxial tension-compression causing yielding and crushing of concrete; (3) cracking in tension under biaxial tension-compression; and (4) biaxial tension causing cracking in tension. The values of Poisson's ratio and shear retention factor are predefined in 3DSCAS as 0.2 and 0.25, respectively. Other values could not be specified, but variations in modeling of local responses (e.g., transfer of shear stresses across cracks, or Poisson's ratio) was found to produce little difference in the global responses (Ho and Shahrooz 1993).

The longitudinal and transverse deck reinforcement (Fig. 2) was simulated with four layers of steel. The steel was assumed to be smeared, defined by the direction of reinforcing bars and reinforcement ratio at each integration

429
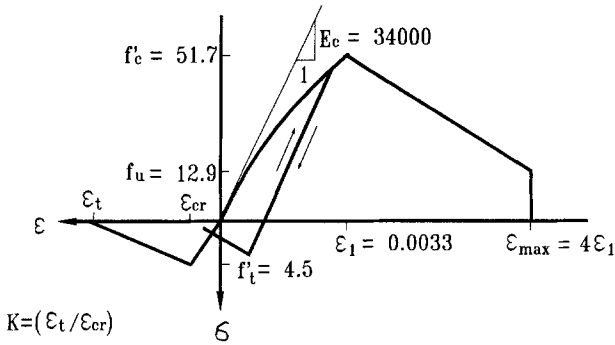
(a) MATERIAL MODEL FOR CONCRETE
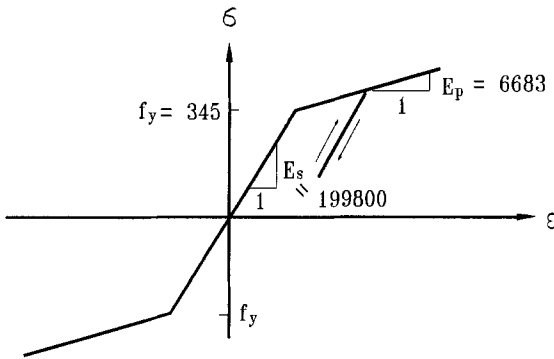
(b) MATERIAL MODEL FOR STEEL

All Stress Units: MPa

FIG. 6. Material Models

point. The slab was divided into eight concrete layers each having identical material properties. An orthogonal smeared crack model was utilized, and the directions of cracks were allowed to rotate. Potential dowel action of reinforcing bars across cracks could not be modeled. Perfect bond between reinforcing bars and concrete, and between various concrete layers is assumed in the shell element.

Results

A Newton-Raphson iteration scheme with automatic stiffness updates every five steps was used. The predicted overall load-deflection curves of the bridge at point A (refer to Fig. 5 for the location of this point) are illustrated in Fig. 7. A larger participation of the concrete beyond the initial cracking (model B) resulted in slightly larger strength and stiffness. The computed peak load was estimated to range between 7,700 kN and 8,160 kN from models A and model B, respectively, with a corresponding vertical deflection of 94 mm at point A. The first yielding of the reinforcing bars,
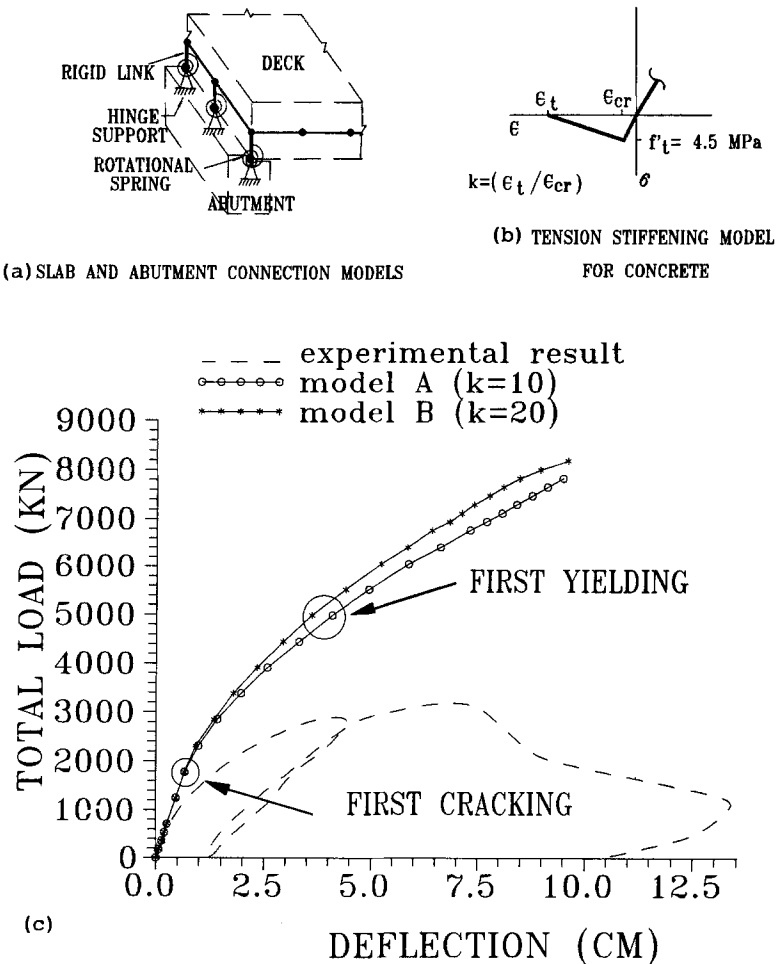
430

(a) SLAB AND ABUTMENT CONNECTION MODELS

(b) TENSION STIFFENING MODEL FOR CONCRETE

(c)

FIG. 7. Predicted Load-Deflection Curves (University of Cincinnati's)

as computed from either model, occurred at 4,980 kN. This load is considerably larger than its experimental counterpart (2,890 kN).

On the same graph, the experimental load-deflection is also plotted. Beyond 710 kN a distinct difference between the analytical and experimental overall stiffness is observed. The vertical deflection of the deck at point A at the measured ultimate load was approximately four times larger than the computed deflection at the same load. The total load on the bridge at failure corresponded to approximately 40% of the predicted strength. Therefore, the stiffness and strength were clearly overestimated.

POSTTEST ANALYSES

Lower than the computed stiffness and strength may be attributed to the existing damage which was not taken into account in the predictive analyses. Of equal importance are assumptions regarding modeling of the boundary

431

conditions at the abutments, modeling of pier–pier cap–deck connection, and material constitutive relationships. The modal tests were useful in identifying the initial support conditions at the abutments. It is questionable whether the initial boundary stiffness would remain valid under large loads causing significant inelastic action. For example, the support stiffness at the abutments was observed to change significantly as the testing progressed (Zwick et al. 1992).

A number of analyses were conducted to determine how the computed response is influenced by various critical assumptions. For each analysis, a combination of horizontal boundary conditions, tensile strength, and post-cracking behavior of concrete was considered. As seen from Fig. 8, the response of the test bridge is sensitive to the tensile response of concrete, and particularly to the assumed horizontal boundary conditions at the abutments. This finding is in accord with other observations that membrane force could double flexural strengths of reinforced concrete slabs (Park and Gamble 1980). The level of membrane force depends on whether the boundary conditions are restrained horizontally to develop such force. Further-
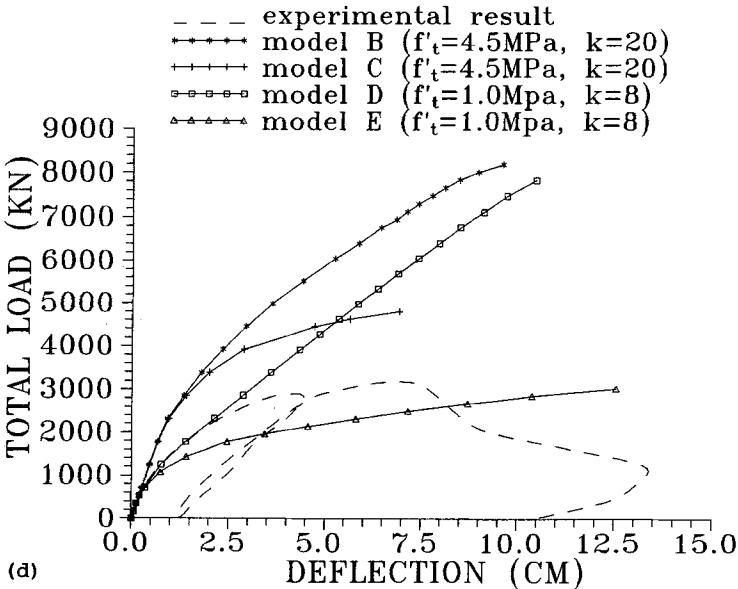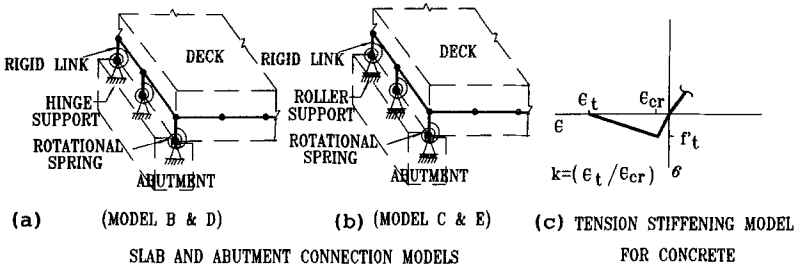

FIG. 8. Results from Sensitivity Studies

432

more, it is apparent that the assumed horizontal restraint affects the response more significantly when the concrete tensile strength is smaller (model C or E).

The previous observations might imply that the large discrepancies between the computed and experimental strength and stiffness could be remedied by releasing the horizontal restraint at the abutments. However, this solution is in conflict with the presence of the shear keys at the abutments (Fig. 1). This apparent contradiction will be addressed later in this paper.

ANALYSES CONDUCTED AT DELFT UNIVERSITY OF TECHNOLOGY

As discussed above, the researchers at the University of Cincinnati utilized the results from the modal tests to calibrate the support conditions at the abutments, and used experimentally obtained material properties. Hence, the expected response of the test bridge was predicted based on a single geometric model, and by using the measured material properties. The analysts at Delft used the modal test results to assess whether the boundary conditions at the abutments were fixed or rollers and hinges. A proper judgment whether the supports at the abutments were rollers or hinges cannot be made on the basis of model test results. For this reason, bounds of strength and stiffness were predicted.

Modeling

The nonlinear finite element analyses at Delft were conducted by using DIANA (van Mier 1987). The bridge deck was discretized with 144, eight-node degenerated plate/shell elements, as shown in Fig. 9. Each element consists of a 2×2 Gauss integration in the plane and a nine-point Simpson integration through the depth. The slab reinforcement was modeled using an embedded approach which uses identical interpolation functions for concrete and steel. Due to time constraints and expected behavior, the illustrated mesh was considered to be sufficiently refined. The loading blocks were modeled as two line loads placed on the edges of two adjacent elements under the blocks (refer to Fig. 9). All the nodes beneath a line load were constrained to have the same vertical displacements.

The bridge deck was modeled only, excluding the abutments and piers. This simplification was justified by recognizing that the final failure load or failure pattern would not be affected by the piers, and the axial shortening of the piers was found negligible. To establish reasonable boundary con-
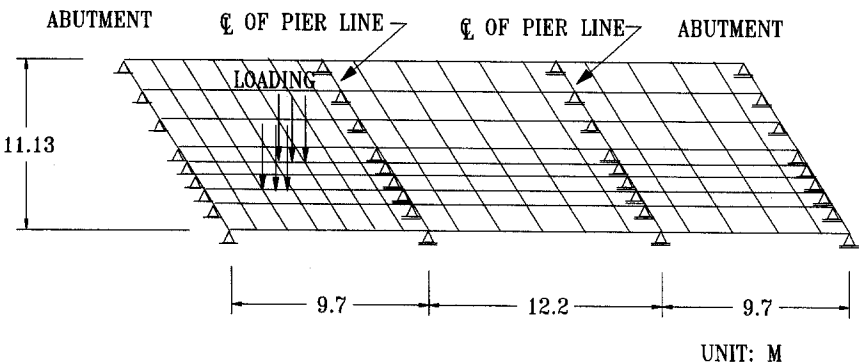
FIG. 9. Finite Element Model for Delft Analyses

433

ditions at the piers and at the abutments, a number of simple analyses were conducted. The first-mode frequency of the bridge was computed based on : (1) Hinges at the supports corresponding to the abutments and piers; and (2) fixed ends at the abutments and hinges at the piers. The computed first-mode frequency for case 1 was 7.1 Hz and for case 2 it was 22.7 Hz. In comparison with the measured first-mode frequency of 8.3 Hz, it is apparent that the supports at the abutments were not clamped and the bending stiffness of the piers did not influence the responses. Consequently, the supports for the slab at the abutments and piers may be assumed to be hinges or rollers. Separate analyses were conducted for each case, as will be discussed later.

A bilinear stress-strain relationship was selected for the reinforcing bars, with identical properties as those used in the analyses carried out by the University of Cincinnati (Fig. 6). The inelastic behavior of the concrete in tension was modeled by the multiple fixed crack model proposed by de Borst and Nauta (1985) and Rots (1988). To account for the stiffness of the concrete between the smeared cracks, a tension stiffening model was used with a linear softening branch beyond first cracking. The residual load-carrying capacity was assumed to diminish at a strain coresponding to one-half yield strain of the reinforcing bars. Previous studies (van Mier 1987) have indicated that this value gives a reasonable prediction of the structural behavior. The concrete stresses in biaxial compression were limited by a Drucker-Prager yield contour (1952), which was fitted such that the pure biaxial compression strength equals 1.16 times the uniaxial compressive strength. Perfect plastic behavior was assumed thereafter. The uniaxial compressive strength was taken approximately as 28 MPa, which is lower than the experimental data. A lower value was intended to account for the observed damage. The tensile strength of the concrete was taken as 1.8 MPa. This value is also less than the experimentally obtained value. The tensile strength was calculated based on the Dutch Codes of Practice for reinforced concrete structures.

Results

Two separate analyses were conducted to establish the lower and upper bounds of the expected response. In one analysis all the supports (at the abutments and piers) were taken as hinges, and for the other analysis all the supports were assumed rollers except for one of the abutments. These analyses were carried out under arc-length control with a method proposed by Schellekens (1992) for estimating the load increment in a step. The solution method was based on a Newton-Raphson scheme, in which the stiffness matrix was updated at the beginning of each load step.

From the results are shown in Fig. 10 it is apparent that the boundary conditions have a tremendous impact on the structural response. The membrane forces that can develop because of the hinges at the abutments and piers effectively prevent collapse of the bridge. For example, at a displacement of 198 mm. (not shown in Fig. 10) a large number of the reinforcing bars were at or beyond yield. At this deflection, no real collapse mechanism could be identified although the load was several times larger than the failure loads predicted by yield line solutions. This behavior was attributed to unrealistically high levels of membrane forces that could develop due to the assumed horizontal restraints at the abutments and piers. When the effect of membrane force was relaxed by assuming rollers at all supports except for one of the abutments, a significant reduction of load-carrying capacity
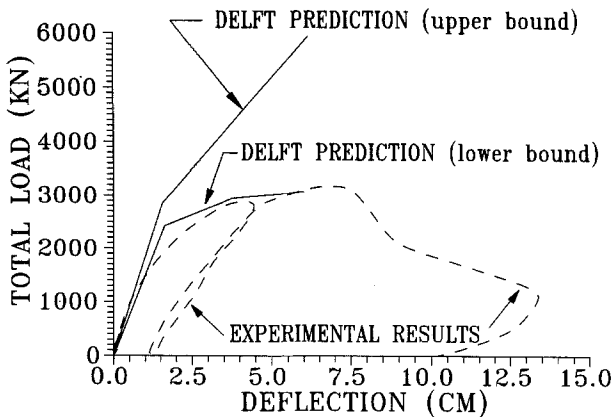
434

FIG. 10. **Predicted Load-Deflection Curves (Delft's)**

is observed. A combination of reduced concrete compressive and tensile strengths (28 MPa and 1.8 MPa versus the measured values of 52 MPa and 4.5 MPa), and removal of horizontal restraint apparently resulted in a better prediction of the measured load-carrying capacity.

EVALUATION OF PREDICTIVE ANALYSES

From the analyses conducted at the University of Cincinnati and Delft University of Technology, it is clear that the assumed support conditions at the abutments were by far the most significant parameter influencing the computed response of the test bridge. It was found that if the horizontal restraint at one of the abutments is totally removed, the overall load-deflection response is closer to the experimental result. This observation is in contrast to the connection between the deck and abutments, which comprised standard shear keys (Fig. 1). Such shear keys are expected to provide some degree of restraint against horizontal movement.

This paradox is due to idealizations of geometry in simulating the connection between the bridge deck to the abutments. At the abutments, the rotational springs, hinges, or rollers were attached to the nodes of the shell elements (representing the slab) located at the mid-depth of the bridge deck. Even if the bridge deck is restrained horizontally by the shear keys at the abutment level, the rotation of the bridge deck could result in an apparent horizontal movement at the mid-depth as illustrated in Fig. 11. Experimental data suggest slippage and rotation of the slab at the abutments (Zwick et al. 1992). Hence, when the horizontal restraint at the abutments is relaxed, a better correlation of the measured response is possible because the movement of the bridge deck at its mid-depth is indirectly taken into account. It should be noted that this approach is overly simplistic, and does not consider the real behavior shown in Fig. 11.

An attempt was made to model the expected behavior (i.e. the observed "rocking" of the slab) at the abutments more reasonably. The model consists of rigid links attaching the shell elements which represent the bridge deck to hinges simulating the shear keys at each abutment, as illustrated in Fig. 12. Employing this model, the response of the bridge was reevaluated at the University of Cincinnati (model F). The concrete tensile characteristics
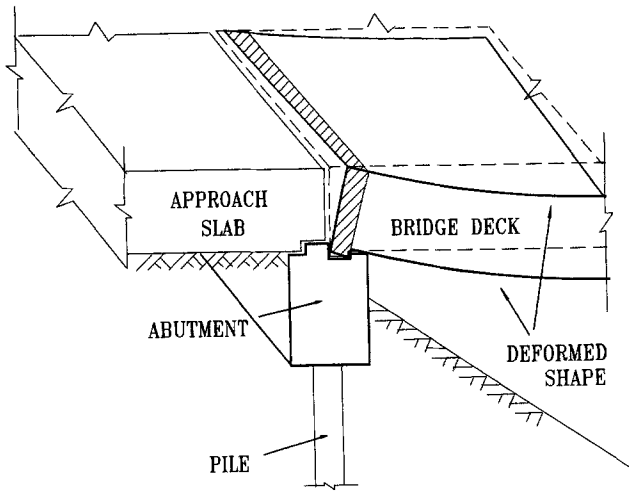
435

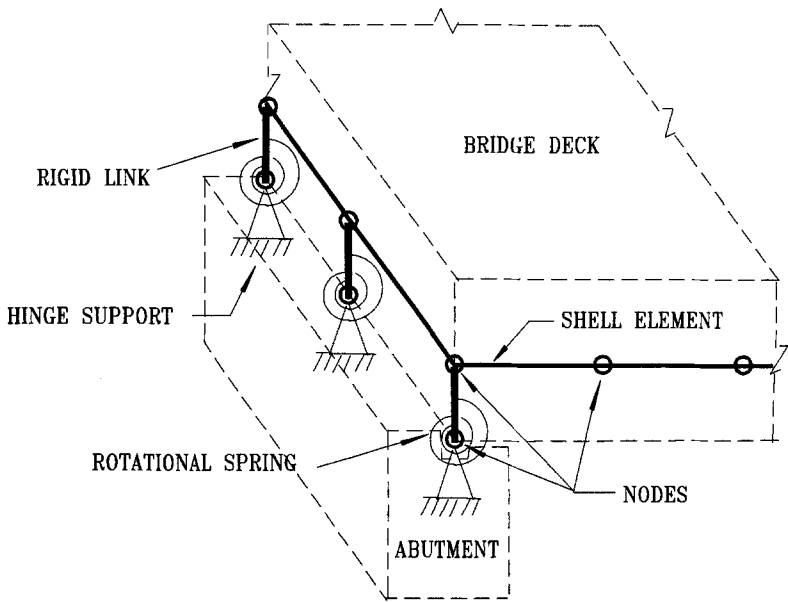**FIG. 11. Behavior of Slab Rotation at Abutments**

FIG. 12. Analytical Model of Slab-Abutment Connection

were set equal to those used in model D and model E, i.e., $f_t = 1.0$ MPa and $k = 8$ ($k = \varepsilon_t/\varepsilon_{cr}$). As seen from the load-deflection relationship shown in Fig. 13, the stiffness and strength are reduced, yet remain larger than the corresponding values if the horizontal restraint at the abutments are fully released. Relaxation of the horizontal restraint at the abutments reduces the membrane force that could otherwise be developed in the slab, making the strength and stiffness smaller. The experimental data suggest
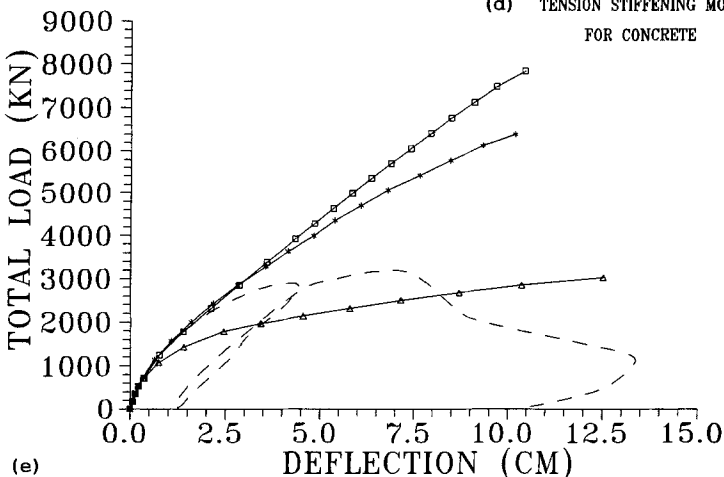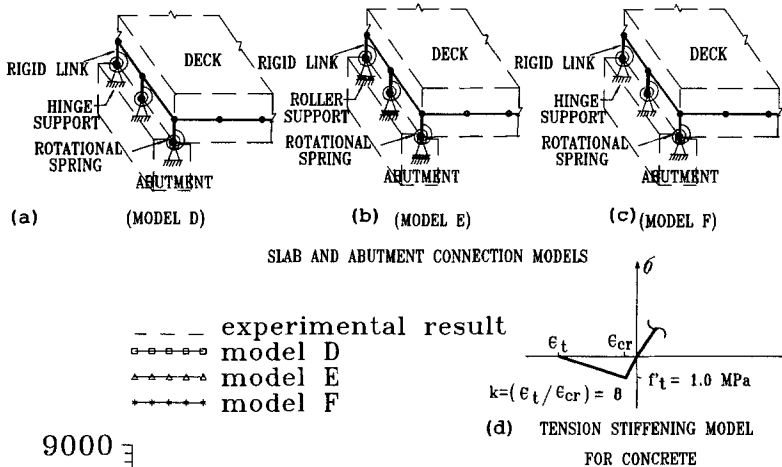
436

FIG. 13. Influence of Modeling of Slab-Abutment Connection

that the level of horizontal restraint provided by the abutments was gradually reduced under larger loads (Zwick et al. 1992). That is, in the analytical model the level of membrane force in the slab should be "regulated" depending on the level of total load on the bridge. Improved modeling techniques have been developed to overcome the deficiencies encountered herein (Ho and Shahrooz 1993), and will be reported in another paper.

SUMMARY AND CONCLUSIONS

A coordinated experimental and analytical study was conducted to understand capacity and load-resisting mechanism of deteriorated reinforced concrete slab bridges. A test bridge was loaded to failure, and the response at different limit states was measured and documented. The failure was not significantly affected by the flexural behavior of the slab. The reinforcing bars yielded at 90% of the load causing failure. Furthermore, the failure plane suggests a flexure-shear type of failure.

The predictive analyses conducted at the University of Cincinnati and

437

Delft University of Technology differed in matching the experimental results. The analyses carried out at the University of Cincinnati indicate a significant influence of tensile behavior of concrete in the postcracking range. Both analyses show the important role of the level of slab membrane force, which is directly affected by the assumed horizontal support conditions. The measured strength and stiffness are matched more closely by reducing or eliminating the horizontal restraint provided by the abutments and piers. This apparent contradiction with the resistance provided by the shear keys is due to modeling of the slab-abutment connection. A simple attachment of the nodes of the shell elements to rollers, hinges, or springs does not simulate the observed behavior. When the slab rotates, an apparent horizontal movement at the mid-depth of the slab is developed even though the deck is restrained at the shear keys. The experimental results indicated that this horizontal movement changed throughout the test, increasing as loads were increased in a nonlinear fashion. A complete or partial removal of the horizontal restraints tends to model this phenomenon approximately, but violates the observed behavior at the abutments.

An attempt was made to develop an improved model of the abutment-slab connection. The computed response was found to be larger than the measured strength and stiffness, but closer to the experimental results than the results obtained from the original model assuming full horizontal restraints at the abutments. The observed and computed modes of failure are also substantially different. While a significant amount of yielding in the reinforcing bars is indicated by the nonlinear finite element analyses (NLFEA), only a limited number of bars were at or slightly beyond yield when the bridge failed. The close match between the measured capacity and that predicted by the yield line analysis appears to be coincidental as the bridge did not fail in a flexural mode. The failure plane suggests a significant influence of shear, which was evidently accentuated by the presence of damage near the east shoulder where the failure was originated. Among plausible reasons leading to the observed differences are: (1) The shell element cannot simulate transversal shear failure; (2) the observed damage is not incorporated; and (3) modeling of the slab-abutment connection needs to be improved. Since the studies reported in this paper, additional analyses have been carried out to incorporate the damage and to enhance modeling of the slab-abutment connection. These analyses have led to very good correlation of not only the global responses (load-deflection responses and deflection profiles) but also regional (slab rotation) and local responses (yielding sequence of reinforcing bars). These studies will be reported in another paper.

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439

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