Research Report
UKTRP-86-27

INVESTIGATION OF COLLAPSE OF LONG-SPAN STRUCTURE UNDER KY 80 IN FLOYD COUNTY (Station 1001+31.30, KY 80-114, SP 36-61-2L)

by

Ronald D. Hughes
Deputy Director

David L. Allen
Chief Research Engineer

and

Robert C. Deen
Director

Transportation Research Program
College of Engineering
University of Kentucky

in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

November 1986

EXECUTIVE SUMMARY

The Kentucky Transportation Research Program (KTRP) was requested to obtain and review design, construction, and post construction documents relative to the long-span structure under KY 80 in Floyd County. Telephone and in-person contacts were made with various agency officials involved with the structure in an endeavor to collect pertinent data. Documents that were initially presented to KTRP staff were thoroughly reviewed and a draft report based upon information submitted was prepared. The draft report included a scenario relating to a probable sequence of events that could have been significant in the ultimate collapse of the structure.

Soon after distribution of the draft report and during the time of its review, additional information was forthcoming, and an Addendum was prepared and is included herein. Another probable cause of failure is included in the Addendum.

ACKNOWLEDGEMENT

During the course of the investigation reported herein, the authors interviewed by phone or in person and/or received background documentation from officials of the following agencies:

Kentucky Department of Highways
Haworth & Associates, Inc.
J. M. Crawford & Associates, Inc.
Bowser-Morner, Inc.
ARMCO Inc.
Brighton Engineering Company
Fuller, Mossbarger, Scott & May
G&G Coal and Energy, Inc.

Principals of the several agencies were extremely cooperative and provided valuable documents, information, and data pertinent to design, construction, and in-service monitoring phases for the long-span structure investigated. Phone or personal contacts were made with the following individuals: C. S. Layson, B. L. Wheat, A. R. Romine, J. M. McChord, F. Goble, R. D. Evans, J. P. Driskell, H. A. Mathis, and B. P. Kelly of the Kentucky Department of Highways; J. E. Haworth, F. A. Meyer, and P. Boleyn of Haworth & Associates,Inc.; J. M. Crawford of J. M. Crawford & Associates, Inc.; D. C. Cowherd and J. H. Wathen of Bowser-Morner, Inc.; D. R. Houchin and F. H. Miller of ARMCO Inc.; M. F. Rudloff of Brighton Engineering Company; J. W. Scott of Fuller, Mossbarger, Scott & May; and T. Caudill of G&G Construction Company, Inc.

TABLE OF CONTENTS

INTRODUCTION

A 28′-1" (span) by 27′-10" (rise) by 267′-0" (bottom centerline length) corrugated metal Multi-Plate, Super-Span structure was designed, manufactured, and constructed to support the embankment and vehicles for KY 80, Hazard-Watergap Road, Station 1001+31.30 in Floyd County over the Chessie System, C&O spur line track 1222 at the valuation 9+53.02. The Chessie System designated that structure as Railway Bridge No. FH-4.7.

The structure was assembled and the associated soil envelop was placed in April, May, and June 1980. Roadway surfacing for the project section including the long-span structure was completed August 4, 1981.

November 1980 correspondence indicated the structure did not meet specified clearances for the railway track and the cross-sectional shape was outside the specified tolerances. Change Order No. 17 was issued in July 1981 and provided for installation of tension ties in a designated portion of the top of the super span. Relatively close surveillance was maintained thereafter.

In April and May 1985, settlement was detected and wedging of the overlying pavement was necessary. Settlement monitoring was initiated in June 1985 and expert professional assistance was requested by Department of Highways, District 12 personnel. On March 5, 1986, a dimple was discovered in the top portion of the structure under the eastbound lanes. A 24-hour watch was implemented March 6 and efforts were undertaken to reduce dead load over the structure by removal of a portion of the embankment on the eastbound lanes side.

Twenty-seven officials representing agencies involved in design, manufacture, assembly, and construction of the structure met March 13, 1986, to develop remedial plans. Traffic was diverted from the eastbound lanes and one-lane, two-directional traffic was effected to the westbound lanes. Traffic control devices were located appropriately and around-the-clock flagging was in effect. Plans for repair were developed and later approved September 12, 1986. A letting date of November 14, 1986, was indicated on those plans.

Sometime prior to about 2:15 am on October 26, 1986, a major portion of the super span under the westbound lanes collapsed. Soon thereafter two vehicles plunged into the chasm. Reportedly, the sole occupant of

each vehicle did not sustain major injuries.

Kentucky Transportation Research Program (KTRP) was requested to collect relevant information and data, review design and construction documents, and determine the probable cause(s) of failure.

AGENCIES INVOLVED

Through a lease agreement with the Turnpike Authority of Kentucky, the Kentucky Transportation Cabinet, Department of Highways (KYDOH), was responsible for design and construction activities associated with the KY-80 relocation of the Hazard-Watergap Road. Brighton Engineering Company (BEC), by contract with KYDOH, acted as general consultants for engineering design and construction phases for the entire KY-80 project.

Haworth & Associates, Inc. (HAI) was responsible for design and construction engineering supervision of Section 4 under the general supervision of BEC. HAI had the section more specifically designated 12-705.0(04), KYRR 80-114, SP 36-61-2L between Stations 812+16 and 1232+85. J. M. Crawford & Associates, Inc. (CAI) acted as subconsultants to HAI for structures on HAI's section. CAI coordinated design and construction engineering supervisor activities for all structures other than the long span structure. Fuller, Mossbarger, Scott & May (FMSM) acted as subconsultants to HAI and performed subsurface investigations, soils investigations and analyses, and presented design recommendations.

Construction of Section 4003 between Stations 960+00 and 1036+00 was awarded to G&G Coal & Energy, Inc.; G&G Kentucky Construction Company, Inc.; and Elmo Greer (G&G) the composite of which is often referred to as Tri-Venture. Design, manufacture, erection and construction supervision of the long-span structure were performed by ARMCO, Inc. under a contract with G&G. Apparently Bowser-Morner, Inc. (BMI) acted as consultants to ARMCO, Inc. during placement of the earth envelop for the structure. The Chessie System owns and maintains the rail spur.

DESIGN FOR MULTI-PLATE, SUPER-SPAN

Pertinent design information contained in documents submitted by officials of various agencies to KTRP principal investigators follows.

J. M. Crawford transmitted an Advance Situation Folder for Bridge at Station 1001+31.30 to BEC and HAI by way of a document dated September 3, 1976. M. F. Rudloff's, BEC, letter of October 21, 1976, to E. V. Hilton, KYDOH, noted the crossroad was not acceptable for the bridge under current policy. For that reason, BEC proposed the subject structure.

J. W. Scott's, FMSM, January 26, 1977, letter to HAI contained information relative to a soils meeting attended by K. Jewell; W. A. Mossbarger, Jr.; and J. W. Scott. Item D of that letter stated:

> Station 1000+00 to 1010+00 (Refer to Note 14 of Brighton's letter of 11-22-76): With 12-inch vertical sand drains spaced on 16-foot centers, 90 percent of consolidation will occur in 200 days. Estimated cost is $130,000 to $175,000. The use of sand drains was not recommended.

The fourth paragraph of M. F. Rudloff's February 14, 1977, letter to E. V. Hilton follows.

> The Soils Engineer recommended against the use of sand drains from Station 1000+00 to 1010+00 in Item D but does not make a positive recommendation for treatment of embankment areas where a settlement in the range of 22 inches is anticipated in a 2 1/2 year period. For this reason we are requesting further comments from the Section Engineer.

It appears a Maxi-Span Corrugated Metal Structure, manufactured by Republic Steel Corporation or Super-Span Corrugated Metal Structure, manufactured by Armco Steel Corporation, was then designated for use at Station 1001+31.30. The structure was to be designed in accordance with requirements of 1973 AASHTO specifications including interim specifications. The live load used was to be the HS 20-44 live load as specified in 1973 AASHTO specifications or an alternate loading of two 24-kip axles spaced 4 feet apart, whichever produced the greater stress.

Design was to include the effect of forces due to settlement. The magnitudes of anticipated settlement, 90 percent of which was to occur during the first 30 days, were:

Along centerline of Underpass - Approximately 2-1/2" at the centerline of proposed KY 80 and 1-1/2" at the ends of Underpass.

Along the outer edges of Underpass - Approximately 3-1/2" at centerline of proposed KY 80.

Anticipated settlement data were included in a July 13, 1977, letter by J. W. Scott of FMSM to HAI.

After preliminary discussions and transmittals between various design agencies and Chessie System officials, requirements of 1976 AASHTO specifications, including interim specifications if any, were referenced on plan notes. Chessie System personnel requested use of an ARMCO super span. Chessie correspondence dated May 4, 1978, indicated reasons for preference of the super span were: 1) had experience with three super spans constructed at various locations and 2) favored concrete thrust beams used by ARMCO rather than metal compaction wings used by Republic. A February 20, 1978, letter by M. F. Rudloff of BEC to E. V. Hilton (KYDOH) noted E-80 loading had been specified.

J. W. Brent's (Chessie System) letter of May 26, 1978, to C. D. Powers (KYDOH) referenced the importance of proper bedding and backfill and noted that a small variation in the quality of bedding and backfill could cause the structure to collapse. The statement, "There have been instances where the structures have failed," was included in the second paragraph of that letter. Mr. Brent stated that 1 gage is justified and alluded to the possibility a lighter gage might be standard.

J. W. Brent's November 16, 1978, letter to L. W. Pike (KYDOH) noted the Chessie System review of Stage 1 Final Plans for the project. The fact that gage (thickness) for the multi-plate superspan was not shown on the drawing was noted. Mr. Brent noted the superspan should be 1 gage and it should be shown on the plans. He also stated that the final paragraph of the construction sequence indicated the Railroad would remove the existing rail and reconstruct the rail when required. Rewording was requested because it had been agreed the State's contractor would perform the work.

CAI's plans for the structure were dated as having been checked June 1978 and were labeled Drawing No. 19782. Sheet 1 of 6 indicated a Multi-Plate Super-Span (1 Gage) Structure No. 81P24-66-75. Notes contained on Sheet 2 specified that the structure be designed in accordance with requirements of 1976 AASHTO specifications, including interim specifications, if any. The live load was to be HS 20-44 or an alternate loading of two 24-kip axles spaced 4 feet apart, whichever produced the greater stress. Design was also to include the effect of forces of settlement. Approximate settlement values were listed and corresponded to those listed in the July 13, 1977, FMSM letter.

The plans designated erection to be in accordance with the manufacturer's drawings and recommendations. The design bearing pressure was not to exceed the recommended allowable of 1,600 psf. Distortion monitoring and control during backfilling were specified. Compaction methods or equipment producing unequal soil pressures or structure distortion were not to be permitted.

The bedding designated was concrete sand or No. 11 crushed limestone at a minimum 6 inches thickness. Only material meeting the requirements of AASHTO M145 for Type A-1 or A-3 could be used in the envelope. Six-inch loose layers compacted to not less than 95 percent of maximum density as determined by KM64-511 were designated. The contractor was to be responsible to remove the existing rail and to reconstruct the railroad when required.

Sheet 3 contained a note requiring the initial roadway embankment to be constructed in advance of the construction of the underpass in order to minimize the differential settlement in the vicinity of the structure. The following were designated for live load:

HS-20-44A - A.A.S.H.T.O.

Cooper E_{80} - A.R.E.A.

An envelope extending beyond the structure for a distance of 12'-0" on each side was shown on the typical section. Select backfill material meeting AASHTO A-1 classification was shown.

A concrete thrust beam was shown on the section along the structure centerline. Also, the proposed roadway and elevations were shown.

Sheet 4 contained subsurface exploration data. Distance from centerline hole surface elevation to refusal was shown as 47.0 feet

(649.9 - 602.9). Distance to refusal at 100.0 feet left of proposed KY 80 and near the track centerline was 24.3 feet. At 110.0 feet right of proposed KY 80 centerline and near the track centerline, distance to refusal was 55.0 feet.

Sheet 5 contained details for the inlet and outlet concrete collars. Sheet 6 provided information for the thrust beams. Detail A specified that reinforced concrete thrust beams be poured in such a manner as to maintain a balanced loading on each side of the structure.

A letter dated January 8, 1980, from F. H. Miller of ARMCO to J. M. Crawford stated ARMCO had revised and was resubmitting SUPER-SPAN drawings. They had been asked to comment on the effect of settlement on the structure. It was noted the SUPER-SPAN is a flexible structure designed to deform rather than fail under moderate external load. Deformation up to 2 percent of its geometrical shape could be accommodated without structural harm. An office copy sketch dated 10-17-79 of the cross section for the structure was attached to that letter. Also attached were drawings for movement control hooks, typical monitoring ideas, text relative to movement control, typical backfill plans for pear shape structure, backfill requirements, and super span structural check sheets.

The first structural check sheet noted dimensions for the structure and indicated the maximum and minimum heights of cover as 10'-0" and 2.5', respectively. Computations indicated backfill was assumed to be 120 pounds per cubic foot and live load was negligible on the high side. Check computations were noted for No. 1 gage. Page 2 contained minimum slope collar details. The third page contained additional computations. Pages 1 and 3 contained: compute ring compression, compute wall stress, check wall stress, check seam strength, and check flexibility factor.

The fourth and last page of the structural check contained computations relative to live loads from 1) HS 20-44 and 2) two 24-kip axles, 4'-0" apart at the guardrail where the height of fill proposed would be 2.3'. The notation at the end of that sheet stated that in either case the high fill condition governs the design as P_T = 1200. No evidence of consideration of the Cooper E_{80} AREA loading was discovered.

A February 11, 1980, letter from E. Q. Johnson of the Chessie System to the attention of D. R. Houchin of ARMCO indicated railway personnel

had reviewed drawings, movement control procedure, and the plan view of the proposed structure. Drawings had been reviewed from a structural standpoint and Chessie personnel had no objection. He also noted that calculations showed greater clearances available than shown on Drawings 103581 A and B.

CONSTRUCTION

The letting date for the section within which the structure was located was May 18, 1979. The project was awarded May 22, 1979, and the contract date was June 8, 1979. A Contractor's Pay Estimate form shows a Notice to Begin Work date of 07-13-1979 and lists the corrugated metal underpass as a lump sum item at a unit price of $600,000. G&G Coal & Energy Corp. Inc. & G&G KY Const. Co Inc. & Elmo Greer, Vendor No. 0577-6125 was noted as the contractor.

B. L. Miller of ARMCO and Tom Caudill of G&G verified that erection of the super span was performed by the Construction Division of ARMCO and associated bedding and backfill was placed by G&G. Records indicate BMI personnel monitored erection activities and obtained samples and tested bedding and backfill materials.

Daily Inspector's Reports for the period March 26, 1980, through June 20, 1980, were provided to KTRP investigators for cursory review. The majority of those reports indicated James H. Singleton as the inspector completing the forms and James E. Tramel as the Project Engineer representing HAI. Dates for staking, track removal, footer placement, cradle excavation, placement of bedding sand, super span erection, 8-inch pipe placement, track reconstruction, backfilling, and placement of materials for the thrust beams were noted.

The Daily Inspector's Reports did not reveal major problems with erection and construction associated with the super-span structure. The report dated 5-29-1980 noted: "BK. Fill on super span was stopped - Material unfit." The report dated 5-30-1980 noted: "BK. Filling super span."

A May 7, 1980, structural inspection report by BEC, CAI, and HAI personnel indicated concern about horizontal alignment of each plate.

It was stated that "The Contractor, ARMCO, assured us the multi plate would align itself after the ring was closed and he would make necessary adjustments then." July 1980 letters indicated concern about the proposed height of cover at the west edge of the pavement. ARMCO personnel apparently thought it was to be about 1.5 feet. J. M. Crawford talked (phone) to F. H. Miller of ARMCO on July 24, 1980, and Mr. Miller forwarded a letter to Mr. Crawford on that date stating the fill height at the pavement edge = 3.5′, at the guardrail = 2.5′, and at edge of shoulder = 2.3′. Mr. Miller urged that, during the construction period, a temporary cushion be maintained to insure that heavy construction loads do not damage the structure.

A November 25, 1980, letter by J. E. Tramel, HAI, to L. Anderson, G&G, stated that measurements by HAI indicated the structure (super span) did not meet specified clearances, relative to the C&O Railway track. Surveys and drawings were made November 6, 1980 showing deformation of the structure and profiling the structure settlement. Maximum settlement was in the order of six inches. It also noted the cross-sectional shape of the structure to be outside specified tolerances. G&G was requested to advise HAI of G&G′s proposed remedial program.

F. Samani, BEC, corresponded with F. H. Miller by a letter dated November 26, 1980. Mr. Samani stated that Mr. Miller had indicated that a 2 percent change in shape of the structure should be a cause for concern. Samani noted that a preliminary investigation of the present cross section at the mid-length of the structure based on the data furnished by the Section Engineer revealed that the structure had gone through deformations equaling approximately 4.5 percent of its overall design depth. Results of ARMCO′s survey, structural analysis of its present shape, and limits of acceptable deformation were requested.

F. Samani′s December 5, 1980, Memorandum to File documented that F. H. Miller had collected survey data. Data were to be studied and results of findings and recommendations (from ARMCO) were expected the following week.

F. H. Miller documented survey data in his December 12, 1980, letter to F. Samani. The following were noted:

1. The structure has settled in a relatively smooth

curve from essentially 0˝ at the ends to 1.4˝ at or near the centerline of KY 80.

2. This "bending of the structure˝s back" has caused an increase in the top radius, an increase in the bottom radius, and has led to a decrease in total rise of approximately 4.4% (exact measurements were impossible because of presence of track and ballast).

3. Maximum span measurements have not increased proportionately (28˝-1" per plan and a maximum of 28˝-4" recorded), so the sides of the structure and the thrust beams are still "locked in".

The cause of the settlements was attributed to the foundation on which the structure and surrounding backfill were placed. Settlements that had already taken place were stated as being about five times the magnitude of predicted settlements. The observed shape of the cross section coupled with ARMCO calculations indicated the structure was sound and still had an adequate, although diminished, safety factor.

Mr. Miller pointed out they could not be certain as to the structure˝s ability to withstand further settlement. The fact that slight movements had taken place between readings on November 13 and December 2 indicated it was possible further settlement would take place. It was not within the realm of their expertise to determine the probability or magnitude of future settlements, according to Mr. Miller. He said it would be prudent for Brighton to undertake an immediate and thorough investigation of the foundations of both the structure and its soil envelop to determine if the settlement had ceased and/or if stabilization were required.

J. W. Scott˝s, FMSM, February 10, 1981, letter to F. Meyer, HAI, stated they had been asked to make an inspection of the Super Span on KY 80 and to determine if the foundation soils beneath the structure were likely to undergo settlement in the future. The following information was presented in Mr. Scott˝s letter.

On December 23, 1980, we met with Jim Tramel at the site and reviewed the construction history of the Super Span. Inasmuch as we were interested in monitoring any possible foundation settlement, we selected the eyebolts protruding

from the structural plates along both sides of the structure and lying below the railroad grade as being the lowest readily accessible point to monitor. We uncovered most of these eyebolts and determined the elevation of each on this date.

On January 23, 1981, the elevations of the eyebolts were checked and no detectable difference from the December 23 elevations was found. Additionally, Mr. Tramel sent us elevations of these points that he took on November 11, December 29, January 16 and February 2 and a review of this information indicates no settlement taking place.

It is our conclusion that the foundation soils beneath the Super Span are presently undergoing no detectable settlement and that there should be no significant settlement of the foundation soils in the future.

The following two paragraphs were taken from J. E. Tramel's, HAI, letter of February 19, 1981, to R. J. Noon, BEC.

We understand the manufacturer is currently analyzing the structure as it exists relative to structural adequacy. Should the manufacturer certify to us that the structure in its present shape has an acceptable factor of safety and the contractor provide us with correspondence from the Railroad agreeing to accept the clearances as they exist and that the Railroad withhold KYDOH harmless from any clearance problems they may have with the structure as it exists, then we recommend that the structure be accepted subject to penalty.

Should these conditions not be achievable, we would recommend expeditious undertaking of a construction program designed to eliminate the current problems with the structure. This will be necessary to eliminate or minimize delay to the projected paving operations on the project.

An April 1, 1981, letter from R. Greer, G&G, to J. E. Tramel stated it was evident dimensional errors were due to foundation settlement rather than deflection of the structure. He said the structure was constructed in strict compliance with specifications in all phases. Mr. Greer pointed out that G&G were not required to conduct a subsurface

investigation nor were they responsible for subsurface conditions or reactions. G&G planned to finish the project within the specified time and needed to begin constructing the super-span headwalls immediately. A response in writing was requested in the event headwalls were not desired. Specific instructions of what was to be done were also requested if headwalls were not to be constructed.

The contents of J. E. Tramel's April 7, 1981, letter to R. Greer follows.

> Your letter of April 1, concerning subject, requested specific instructions concerning the Super Span. Mr. Lee Anderson of your office received a copy of our latest correspondence to Mr. Noon dated February 19, 1981, copy attached. We have not received a response to that letter. We also did not receive a response to our letter of November 25, 1980, to Mr. Anderson of your office advising him of deficiencies in the structure as it exists and requesting advice relative to your proposed remedial program.
>
> We understand the vendor will be in contact with the railroad relative to their acceptance of the structure as it exists. We cannot accept the structure until the clearance problem has been resolved. Any work performed in the immediate area of the Super Span would be at your risk and predicated on your conception of the probability of eventual acceptance of the structure as it exists. Hopefully the matter may be resolved in the immediate future.

F. H. Miller provided information to R. Kendall, Chessie System, in a May 15, 1981, letter. Therein, settlement and track shimming were identified as having trimmed several inches from the top corners of the original rectangular clearance diagram. FMSM's letter and movement control hook reading information were referenced for determining that settlement had ceased. A ring compression safety factor greater than 2.0 was cited. R. E. Weiford was quoted as "There's strength in flexibility." An early review of material was requested along with a response to the acceptability of the structure to the Chessie System.

Mr. Miller's May 15, 1981, letter to F. Samani stated it would be prudent to install some tension ties in the top of the structure. The

ties were to provide some added reinforcing of the top arc in the area of the longest radius about 16 feet long - 8 feet either side of Station 50+12. Suggested details were attached.

Contents of E. Q. Johnson's, Chessie System, letter of June 9, 1981, to F. H. Miller follow.

> This has reference to your letter of May 15, 1981, relative to the above structure constructed by the Commonwealth of Kentucky to carry side track 1222 under Route 80 relocated in Floyd County.
>
> We will accept the structure as it now stands with 22'-0" vertical clearance for a width of six feet each side of centerline of track. Existing clearance in each of the upper quadrants has been reduced below the design clearance of 22'-0" for the width from six to eight feet beyond centerline. This acceptance of the clearance is based on your assurance that settlement of the structure has ceased. In addition, we recommend that the strut shown on your drawing 103674 be installed.

An Advance Approval for Change Order for a change order was initiated by B. L. Wheat, KYDOH, on June 26, 1981. The request was for $6,000.00 for 18 struts to be placed in the superspan at Sta. 1001+31. Change Order No. 17 indicated Struts for the Superspan lump sum for $6,000.00. First and last signature dates on that Change Order were 7/11 1981 (Rex Greer) and 7/31 1981 (Dean Huff, KYDOH), respectively.

The effective date of formal acceptance for the contract section within which the super span was located was November 5, 1981.

POST CONSTRUCTION EVENTS

The complete project was inspected, including structures, and was accepted November 5, 1981. An initial structural inventory and Appraisal Report was completed for the super span on June 16, 1983. Condition ratings for Item 62, Culverts and Retaining Walls, and Item 65, Alignment, were good and no defects were noted. The next scheduled inspection was to be in June 1985. That schedule is in conformance with

normal practices.

A June 6, 1985, memorandum from F. Goble to A. B. Blankenship and attention W. A. Crace related recent experience of a settlement condition over the tunnel for C&O Railroad access to the Bucks Branch area. The existing dip across all four lanes, median and shoulder was 12 to 14 inches and would be leveled by Floyd County Maintenance forces. Mr. Goble said recent meetings with construction and maintenance personnel from the Central office and Pikeville were held to study the course of action needed to be taken.

That memorandum noted that a meeting with ARMCO engineers on the problem was pending. He said their office had set up a weekly monitoring system by the bridge inspectors for comparison with the original shop drawings. It was recommended they take into consideration Transportation plans that utilize coal haul over the large structure. Mr. Goble said the plans should be limited to weights posted by Blankenship's office.

F. Goble's June 7, 1985, memorandum to files referenced a meeting on that date in D. Biliter's office. Mr. Biliter asked H. Reed to contact ARMCO and set an appointment with an engineering expert on the construction of the structure. H. Reed called T. Wederman (ARMCO) who said their engineers had been monitoring settlement and seemed to think the whole fill, railroad and all, were settling; not just the pipe.

Mr. Goble's June 24, 1985, memorandum to file made record of a meeting in Mr. Ison's (KYDOH) office attended by referenced District 12 personnel. F. Goble said he contacted Mr. Wederman and ARMCO personnel had measured the structure last October 1984. Mr. Biliter said District 12 needed to start their own monitoring procedures now. L. Hampton and T. Frazier were to start that day (June 24, 1985). They were to monitor and record in bridge files every two weeks on Mondays. A meeting with ARMCO was scheduled July 8, 1985. A page following the June 24 memorandum showed sketches of the Martin end, center, and Bucks Branch end. Vertical dimensions from top of rail to top of conduit were 22'-10", 22'-7", and 22'-6" for the three locations cited in the previous sentence, respectively. Two horizontal dimensions for each location, respectively, totaled 18'-4", 17'-9", and 17'-2".

D. A. Ream's, BMI, September 19, 1985, letter to F. Goble said top

cord and rise measurements Goble made were being forwarded to J. Noll, ARMCO, for his evaluation. Mr. Ream stated that the rise measurements had not been tied into the bench mark and they, BMI, therefore were unable to determine if any settlement had occurred since the last rise measurements made in 1980. He cited the reason for not using the rails as a reference -- tracks would also settle if structure as a whole was settling. He said the manhole at the east end of the structure had been used as a bench mark and the paint marks were still visible in August (1985).

F. Goble submitted to D. A. Ream additional field measurements with a letter of October 7, 1985. Mr. Goble requested a professional opinion on the problem as soon as possible. In an October 10, 1985, letter from F. Goble to D. A. Ream, Mr. Goble stated he had noticed more settlement over the structure when traveling east on KY 80 on October 9, 1985. He said heavy trucks were causing severe impact once again.

An October 10, 1985, memorandum from H. F. Reed, by F. Goble, to G. Asbury, attention H. Evans or R. Sutherland cited a meeting with ARMCO for expert assistance on steps that should be taken and to determine cause for the settlement. Field measurements had been furnished by Bridge Inspectors to D. Ream and C. Bishop of BMI and their (BMI) report was pending review of those field measurements. A request for assistance in determining procedures that should be taken to correct the settlement condition was included in the memorandum. It was recommended that a contract be awarded rather than using state forces to make the corrections once they were determined.

D. A. Ream's October 15, 1985, letter to F. Goble noted Goble's October 2, 1985, elevations had been compared to measurements made February 23, 1981. Measurements showed the span had settled at all locations and was most severe at points C, D, J, K, and L. The cause of settlement was noted as undetermined. Measurements were forwarded to J. Noll, ARMCO, for his evaluation. Mr. Ream referenced receipt of F. Goble's October 10, 1985, letter noting observation of additional settlement. It was their, BMI, strong recommendation that you (Goble) DO NOT PLACE additional asphalt on the structure until the structural integrity of the span could be evaluated and the cause of settlement determined. Mr. Ream said BMI could provide a complete evaluation

service and would provide a proposal for the work upon request.

An October 25, 1985, letter by F. Goble, from H. F. Reed to G. Asbury, attention R. Sutherland transmitted the BMI October 15, 1985, letter. It was noted that a dip condition over the structure was due to settlement and the traffic crew had installed a warning sign. A request for assistance on possible corrective measures was included in Mr. Goble's letter. He said the District's expertise on a project of that nature was limited. Mr. Goble cited the need for a special task force to complete the investigation and recommended that contract services be issued to BMI for the proposed project. He also recommended that the project be let to contract for repairs. He requested a decision on that very important matter as soon as possible. Field information supplied by L. Hampton was attached.

In response to F. Goble's October 25, 1985, memorandum, R. K. Sutherland's November 5, 1985, memo to K. Ison, Jr., attention F. Goble, requested additional information not included in the October 25 memorandum. Mr. Sutherland stated that any wedging or patching necessary on the roadway should be done as soon as possible. He was of the opinion that increased impact loading due to the dip would be more harmful than some additional dead load.

D. Ream forwarded control hook elevations made during construction to F. Goble by way of form PK111R-2 dated December 5, 1985.

A January 7, 1986, memorandum to R. K. Sutherland, from K. Ison, Jr. and by F. Goble transmitted requested additional information. Under Item 4 of that memorandum, it was noted that visual inspection of the inside walls of the structure reflected no serious distortion problem. Under Item 5, it was stated that any wedging or patching necessary had not been accomplished due to work schedules and availability of hot mix. Dip signs had been installed.

Field book notations and Bridge Inspection Reports for the period of October 2, 1985, through October 2, 1986, were supplied to KTRP personnel for review. Elevations through the structure were referenced to a bench mark on a water main lid on October 2 and again on December 16, 1985. On December 16, it was noted no additional deformation was visible in the top of the structure. The next field book page supplied was dated March 6, 1986.

On March 5, 1986, F. H. Miller of ARMCO stopped by the structure about 4:00 pm and what he saw was not good. Mr. Miller called F. Goble at 5:35 pm to report his observations. Mr. Goble documented events thereafter extensively and most thoroughly. The following is a synopsis of significant events extracted from Mr. Goble's penciled notes for the period 5:35 pm March 5, 1986, through 4:15 pm March 26, 1986.

F. H. Miller told Mr. Goble that a condition such as he had observed March 5 was very serious and that such conditions had led to snap failures in the past. F. Goble called L. Hampton to see when Hampton had last looked at the structure. Hampton said about three weeks ago and it looked OK inside the arch, and inspection notes reflect this. Goble called J. Wright for possible traffic control, and he called D. Biliter to see if he should call Frankfort. Biliter said yes. Various calls were made until 10:20 pm. Mr. Goble met others at the structure at 11:30 pm and trucks with flashing lights, barrels, sign posts, and flashing bar lights were put in place.

On March 6, G. Asbury was informed of the situation and he was to have someone there about noon. Miller and Goble met at the site at 8:30 am and decided to partially unload a portion of the structure. Appropriate contacts were made to start that operation. It was decided to make KY 80 one lane eastbound. Measurements were obtained on the inside walls (D. Ream, BMI). They continued removing earth and making measurements until 6:00 pm.

A boom truck was obtained March 7 for use in obtaining measurements. K. Ison, Jr. arrived at site at 2:00 pm and a Channel 57 (Hazard TV) crew arrived at 2:30 pm. D. Ream called at 4:50 pm and said they had the computer analysis of measurements and reported it as very critical and could possibly fail. Removal of a lot of load over the structure should make it OK; however, Ream advised to continue monitoring. Goble immediately called Roberts to establish 24-hour watch. He radioed for necessary items to be ready to close KY 80 quickly. Mr. Goble conferred with ARMCO personnel and discussed the idea of materials exploration. Yes -- good idea was the response.

On March 9, L. Roberts reported he had noticed that some wires that had been placed for measurements had broken. There were also some more cracks in the concrete parapet wall. Cracks in the parapet and road

were painted and marked and men were asked to keep an eye on them and call Goble if they noticed any change. F. H. Miller called and requested measurements soon.

March 10 discussions involved reducing the weight limit and posting as soon as possible. A meeting was held to discuss alternatives. C&O was asked to remove rail cars and inside measurements were obtained.

Milling operations were begun on March 11. Measurements were relayed to D. Ream and the BMI computer analysis revealed Station 0+60 as 50 percent distorted and worse than at Station 0+48. Station 0+60 was noted as being under the eastbound lanes. A noted entry for that date was "CLOSE 2E bound lanes KY 80. Measurements showed sides of structure were OK."

On March 12, there were more milling operations. D. Ream was contacted and he said movement was not critical enough to close KY 80. It was decided to remove the concrete parapet wall and footer to remove more weight. They were removed and hauled away.

A meeting was held March 13 and was attended by 27 people. Recommendations were: 1) close two eastbound lanes of KY 80, 2) unload more, 3) get soils survey, 4) install temporary rings, 5) make pavement repairs, and 6) get more measurements. The question of safety of personnel within the structure was raised. Some felt uneasy being inside. It was noted that the same type arch collapsed last year in Ohio and killed five to seven persons. The 24-hour watch was to be continued until more work was done.

More measurements were obtained March 14 and relayed to D. Ream. Ream conferred with D. Cowherd and they thought it should be OK.

Goble contacted J. Plummer on March 19 relative to closing the two eastbound lanes, and Plummer said the crash cushions had not arrived yet.

R. Sutherland contacted K. Ison, Jr. on March 20 and requested that earth be removed over the outlet end, shoulders and part of the eastbound lanes. Goble and Biliter arrived at the site at 11:50 am and soil was being removed.

J. Wright helped take measurements on March 21 to determine exactly where the tension ties were located. By 4:30 pm, most of the fill had been removed from pipe back about 60 feet and conditions were the same

as before inside the structure.

More fill was removed March 22. Inside was checked again, and it was noticed that a large sag had appeared in the under side of the top 72 feet back from the Martin end. This was caused by dropping a large section of asphalt when removing fill material. All work was stopped at 10:30 am. Contact was made with F. H. Miller and he agreed work should be stopped. He said the situation was very serious and two men should check for change every 30 minutes -- be ready to close KY 80 quickly. Numerous calls were noted relative to the situation. At 8:45 pm, F. H. Miller called to say he would be flying in at 9:00 am on March 23.

On Sunday, March 23, a meeting was held at the site. It was noted that huge chunks of pavement broken off and dropped by the Bantam had caused a dimple or bulge in the structure. F. H. Miller recommended placing a jacking post under bulge -- Denton said not yet. They decided to try something different. Drill holes on 2-foot centers and then lift pavement from top with Bantam.

The Floyd County Crew was drilling holes at 7:00 am on March 24. At 9:30 am, the pavement was still coming up in big chunks. Structure bulge was still same as before at 11:45 am. The Materials people and drill rig from Frankfort arrived at 1:31 pm. At 4:05 pm, Mr. Biliter stated he wanted photos showing subgrade material and backfill with sandstone.

On March 25, a check was made to see if they could rent a Cat-235 Trac Hoe on an emergency basis. At 9:15 am, Goble checked the bulge (due to dropping asphalt) in the structure -- no change. It was decided to pull pavement from top, peel back, and then load out. It was finally agreed to pull out KYDOH staff and get something in that could do the job quickly. Approval was received from P. Wolf at 4:20 pm.

The structure was checked on March 26 and photos of subgrade materials and fill around the structure were obtained. A Case 580D backhoe with ram arrived at 9:00 am. A 4:15 pm entry noted a night watchman should remain at the site until further notice.

This ends the synopsis extracted from Mr. Goble's notes, and information gained from other documents follows.

G. W. Asbury's March 17, 1986, memo to A. R. Romine contained information relative to the March 13 meeting as supplied by R. K.

Sutherland and J. D. Wood. A rough estimate for repair was $300,000. Mr. Asbury recommended that his office be allowed to negotiate a contract to construct a temporary HS-20 structure at the site in case deformation continued under the westbound lanes. He also noted that a slab was being designed to be placed on beams that were on the bridge yard. That plan would allow them to react immediately to an emergency situation. G. W. Asbury listed the following things the Department was doing.

1. Traffic is being removed from both eastbound lanes to the westbound side. The fill will be excavated on the eastbound side. The possibility exists that the westbound side could also fail. In this event, we would need to provide a temporary bridge to maintain a normal flow of traffic. This could cost as much as $175,000.

2. The Division of Materials is conducting tests to determine the content of the existing fill. Also, a recommendation will be made as to stabilizing existing material.

3. The Division of Bridges is estimating various alternatives to a ring beam retrofit. The cost of these alternatives will be weighed against ARMCO's estimate for ring beam retrofit combined with fill stabilization costs.

A TD 10-1 for $500,000 will be submitted under a separate cover. Account 210 will be shown on the TD 10-1. Bennie Wheat advises that there may be original construction funds still available.

Please advise as quickly as possible.

By way of a March 19, 1986, memo, G. W. Asbury transmitted a Project Authorization to be executed to A. R. Romine. It was noted that funds were available in the KY 80 Project Account 850. It was recommended that engineers in the Department determine a cure for the problems of the structure independent of ARMCO or others.

Mr. Miller forwarded drawings showing possible repair details to K. Ison, Jr. by way of a March 27, 1986, letter. H. F. Miller stated repairs would simply reshape and stiffen the top arc and the long-term solution to the problem must involve a complete soils study and the

appropriate remedial measures.

G. W. Asbury's March 28, 1986, memo to R. K. Capito forwarded copies of letters sent out in an attempt to secure proposals for emergency procedures. Mr. Asbury stated the three contractors represented the best potential based on KYDOH experience and a cursory review of their current situation relative to rapid mobilization and completion. The letters were dated March 17, 1986, and referenced an April 3, 1986, meeting at the site for interested personnel. Mr. Asbury's memo of April 10, 1986, to A. R. Romine noted receipt of two bids -- one for $85,350 and one for $82,400. Insurance and Type A end treatment deducts were cited.

An April 16, 1986, memo from H. Mathis, by D. Smith, to K. Ison, Jr. noted that the requested subsurface investigation had been completed. Nine of ten samples tested from the embankment classified as A-4 by the AASHTO system. The majority of samples indicated the embankment was constructed of shale. A bag sample of the existing subgrade had a CBR of 7.3. The material classified as A-4(0) with approximately 50 percent silt and clay. It was their understanding the project was designed for sandstone subgrade.

G. W. Asbury's April 30, 1986, memo to A. R. Romine forwarded copies of a proposed contract for a temporary bridge in the event the existing structure continued to deteriorate. It was noted as being generally agreed that processing would proceed to the point of obtaining the Secretary's signature, and that would occur only if it was determined the temporary bridge was needed to maintain the flow of traffic on KY 80.

R. K. Sutherland's May 18, 1986, letter to F. H. Miller forwarded a preliminary drawing for retrofit of the KY 80 superspan for Mr. Miller's review and comments.

In June 1986, R. Sutherland forwarded potential remedial plans to F. H. Miller for review. Mr. Miller phoned Mr. Sutherland and stated that plans appeared suitable providing soils had stabilized and no more settlements were anticipated. On August 8, 1986, minor plan revisions were made and were forwarded to District 12 personnel for review. On August 19, 1986, Division of Maintenance transmitted the plans to the Division of Bridges. The Traffic Control and Construction Phasing plans

were to be effected.

Plans for repairs were developed and approved by G. W. Asbury 9-12-1986. A letting date of 11-14-86 was indicated. KTRP personnel photographed the structure on October 21, 1986 and signs of eminent failure were not evident.

At approximately 2:00 am EST on October 26, 1986, the structure collapsed. Contents of Mr. Goble's October 28, 1986, memorandum to K. Ison, Jr. follows.

> On Sunday, October 26, 1986, at approximately 2:00 am, Eastern Standard Time, the large super span arch pipe carrying a C & O Railroad spur under four-lane KY 80 at Martin collapsed.
>
> At 2:40 am, Mr. Denton Biliter called the writer at home and informed him of the condition. He mentioned that two cars had already fallen into the large cavity due to the collapsed pipe. Arrangements had already been made to remove the vehicles.
>
> At 3:15 am, I arrived at the site. The cars were being removed at this time. I checked all traffic control devices and they were still intact, except the signs or barrels that had fallen into the cave-in. The flashing arrows on the west end of the channelization were still flashing. Mr. Biliter and Leo Roberts, County Foreman, had contacted Mr. Jim Wright for quick placement of a detour around the closure. Flagmen were at both ends of the project stopping traffic. I also learned that Mr. Dennis Huff of Crew 036 had been on duty that night and assisted the people in the accident. He got hold of the proper authorities as soon as possible and continued to flag other traffic before they, too, had an accident.
>
> At 4:45 am, I called Mr. Richard Sutherland at Frankfort to inform him of this condition.
>
> As soon as it became daylight, photographs (both polaroid and 35 mm) were taken of the pipe failure.
>
> The detour was being marked by the traffic crews and KY 80 was barricaded at both ends of the project. Flagmen was

placed at each end of the job.

ANALYSES

AASHTO design standards that were current and available to designers during the period of design of the super-span structure were:

1. Standard Specifications for Highway Bridges, Twelfth Edition, 1977:
2. Interim Specifications - Bridges, 1978; and
3. Interim Specifications - Bridges, 1979.

Section 9 of the twelfth edition, 1977, is titled: Soil-Metal Plate Structural Interaction Systems - Corrugated Metal and Structural Plate Pipes and Pipe-Arches. Article 1.9.10 is titled: Long Span Structural Plate Structures. Modifications for Section 9 were included in the 1978 Interim Specifications. No revisions to Section 9 were included in the 1979 Interim Specifications.

Cross-sectional requirements of the structure under design are established. For the structure under KY 80, internal height and width requirements were primarily governed by dimensions necessary to accommodate the rails, ties, ballast, engine(s), and cars (hoppers, etc.). Total lengths (top and bottom) for the structure would be dependent upon height of fill to be placed above the structure; embankment slopes; longitudinal grade to which the structure would be constructed; widths of roadway(s), median, and shoulder(s); and possibly other considerations.

Structural design may commence once cross-section dimensional requirements have been established and soil parameters are known. Realistic, probable expected values for soil parameters are vital because the design criteria consider the mutual function of the metal ring and the soil envelope surrounding it. Interaction of those two materials produces a composite structure. Expected live loads and dead loads to be supported are selected. Design computations are then made for a hypothetical structure one foot in length. The structure is not designed to act as a longitudinal beam. Sectional requirements for an entire structure could theoretically vary throughout its length if each

one-foot segment were to be considered. Some structures have been designed sectionally -- one section for the high fill portion of the embankment and one or more sections under each embankment slope.

Design steps include computations for thrust in the pipe wall, buckling stress, handling and installation strength, seam strength, and deflection or flattening. Values required may be determined and required wall thickness and configuration (pitch, depth, and inside forming radius) may be ascertained. A minimum factor of safety of 2.0 is common. The selected structure is checked and essentially design is complete.

The AASHTO specifications contain recommendations for vehicular live loads and soil parameters for consideration. AASHTO design specifications do not contain any reference to considerations of railway loadings within a corrugated metal structure. The AASHTO specifications do not present structural design considerations for situations of differential settlement throughout the length of the structure wherein it might be necessary to explore the longitudinal beam concept.

Section 23 of the twelfth edition, 1977, is titled: Construction and Installation of Soil Metal Plate Structure Interaction Systems. The 1978 interim contains additions or revisions to Section 23. The 1979 interim did not contain additions or revisions to Section 23. The section presents relevant information pertaining to good construction practices essential to obtaining the soil-metal plate structure interaction. The contents of Article 2.23.8 - Camber follow.

> The invert grade of the pipe shall be cambered, when required, by an amount sufficient to prevent the development of a sag or back slope in the flow line as the foundation under the pipe settles under the weight of embankment. The amount of camber shall be based on consideration of the flow-line gradient, height of fill, compressive characteristics of the supporting soil, and depth of supporting soil stratum to rock.
>
> When specified on the plans, long-span structures shall be vertically elongated approximately 2 percent during installation to provide for compression of the backfill under higher fills.

Information contained in documents presented for review indicates AASHTO design and construction requirements were met or exceeded for the KY 80 structure, except for the consideration of camber. Anticipated settlements, 90 percent of which were expected to occur the first 30 days, were 2 1/2, 1 1/2, and 3 1/2 inches at locations noted under the design section of this report. The absence of camber to accommodate anticipated settlements of those magnitudes for a structure 267'-0" in length is not considered as a significant factor leading to ultimate collapse.

A-1 and/or A-3 class soils were designated for use in the soil envelope for the structure. Construction records indicate specification requirements were met. Specimens obtained from the soil envelope after construction indicated much of that material classified as A-4. In accordance with requirements of AASHTO M 145, soils and soil-aggregate mixtures are classified based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. A requirement for the envelope was that it be compacted in 6-inch loose layers to not less than 95 percent of maximum density as determined by KM 64-511. Materials initially classifying as either A-1 or A-3 may later, after compaction, classify as A-4.

Because of reported large settlements in the foundation, a finite element analysis was performed in an attempt to determine the magnitude and combination of loads that may have been necessary to produce such reported movements. It must be noted that little soils information was available and many parameters had to be assumed. The support provided to the structure by the soil envelope was applied as a concentrated load at each nodal point around the structure. The magnitudes of these loads were assumed from a typical distribution published in Chapter 23 (entitled "Buried Structures," by R. K. Watkins) of the Foundation Engineering Handbook by Winterhorn. The foundation was assumed to have a CBR of 7.0. The modulus of the foundation soil was assumed to be 7,500 psi and it was assumed to have a Poisson's ratio of 0.40. The steel of the super span was given an assumed modulus of 30,000,000 psi and a Poisson's ratio of 0.30. An assumed 1-foot section was analyzed under the 10-foot portion of the embankment. Two load cases were analyzed.

1. The weight of the fill above and around the culvert and the weight of the culvert itself were the only loads considered on the foundation. The influence of the settlement of the foundation and fill adjacent to the culvert (soil arching) was not considered.

2. The weight of the fill above and around the culvert, the weight of the culvert, the weight of an 82,000-pound tridem (the authors are aware that the tridem is a dynamic load, however, for the purposes of a cursory analysis, it was assumed to be static), the weight of a loaded gondola car on the tracks, and any additional dead load that may have been caused by differential settlement between exterior soil prisms and the interior soil prism were considered.

In the first load case, the largest calculated settlement in the foundation was 1.6 inches. The largest deflection in the top of the culvert was calculated to be 2.6 inches. However for Load Case 2, the largest calculated settlement in the foundation was 18 inches. The calculated deflection at the top of the barrel was 40 inches.

As previously stated, all of the original soils and subsurface data were not available to the investigators. However, settlement calculations and some void ratio versus pressure curves were available. The original soil stress values were calculated using the computer program ICES-SEPOL written at Massachusetts Institute of Technology. Although all of the soils data were not available, it was possible to approximate the soil stresses calculated in the original settlement analysis, using the same computer program (ICES-SEPOL). A check of the original settlement calculations using the stresses from ICES-SEPOL indicated proper procedures were followed. Although the original settlement calculations apparently (as best as could be determined) did not include train loads, the resulting difference in settlement would have been less than two inches. The reason for the large difference in calculated settlement between the finite element method and that used in the original settlement calculation is in the magnitude of stresses calculated by the two computer programs.

The following scenario is envisioned as a possible explanation of events leading to ultimate collapse of the structure. Design and construction were basically in accordance with conventional national standards. During construction phases or soon thereafter, settlements

exceeding those anticipated occurred. The structure and adjacent embankments may all have settled. Exact settlement (referenced to a reliable permanent bench mark) data were not supplied. The magnitude of settlements, measured and reported by separate agencies on different dates, varies.

Differential settlement may impose forces upon the structure for which it was not designed to accommodate. Transformations and/or signs of distress that may have been taking place longitudinally escaped detection -- ballast covered the invert sections and upper plates and bolts were beyond the range of normal vision for detecting defects.

Initial distress was observed within that portion of the structure under the eastbound lanes. Embankment height above the top of the structure at that location was in the order of 10 feet. It was reported that the majority of heavily loaded trucks travelled the eastbound lanes. Many trucks returned unloaded in the westbound lanes.

Gross vehicle weights of 150,000 pounds are reported as being frequent occurrences -- some are reported as being 170,000 pounds. Rear triaxle loads for six-axle vehicles having gross loads of 150,000 and 170,000 pounds would be in the order of 82,800 and 94,800 pounds, respectively. Rear dual-axle loads for five-axle vehicles having the cited gross loads would be approximately 69,000 and 79,000, respectively. There is a vast difference in those loads and that expected for the HS 20-44 (32,000 pounds on one rear axle) loading condition. Pavement deformities (dip) magnify the effect of live loads through impact. Live loads and impact loads above the structure along with railway loads (ballast, track, engine, cars) within the structure could foreseeably have been sufficient to cause distress within that portion under the eastbound lanes.

The eastbound lanes were closed to traffic, one-way two-directional traffic was effected within the westbound lanes, and work commenced on unloading the structure under the eastbound lanes. For the westbound lanes, there was approximately a 2.5-foot fill above the top of the structure. Live loading is more significant for shallow fills than for deeper fills. Removal of the soil envelope, or portions thereof, voided the metal structure-soil interaction.

The effects of removal of the soil overburden (eastbound lanes) upon

the metal structure-soil interaction where materials remain in place (westbound lanes) are highly speculative. Areas were created where water could pond for long times as evidenced by the presence of cattails. Water may have seeped into and through the remaining soil envelope and weakened the composite structure.

There are other plausible scenarios. Differential settlement between the structure and adjoining soil prisms could have transferred additional dead loads to be supported by the structure. Some designs strive to insure a reverse of that situation -- conditions are created to transfer a portion of the dead load weight from above the structure to the adjoining soil prisms. That methodology is often referred to as the imperfect trench condition and is applicable to rigid underground conduit but not flexible conduit.

Post construction documents indicate Department of Highways personnel made extensive efforts to determine the cause(s) of distress, monitor the situation, and ensure that precautionary measures were implemented. Frequent inspections were made during much of 1985 and until March 6, 1986. After March 6, a 24-hour watch was effected at the site. Transportation Cabinet officials took immediate actions in development of remedial plans. Sound engineering judgment was exercised in an unusual and difficult situation. Activities undertaken by Department of Highways' engineers in analyses, structural monitoring, and traffic control procedures were all conducted in a professional manner and in accordance with prudent engineering principles. The eastbound lanes were eventually closed and all traffic was routed to the westbound lanes. Closure of the eastbound lanes was prudent since traffic volumes were sufficiently low that diversion of eastbound traffic to one westbound lane would not create delays or backups.

RECOMMENDATIONS

The following recommendations are based upon discoveries made during the course of the investigation reported.

Portions of specifications relevant to requirements for soils or soil-aggregate mixtures should be redrafted. Those specifications

should be worded to insure that soils or soil-aggregate mixtures, after placement and compaction, classify as being one of those currently required prior to placement. Material placed within the envelope for the KY 80 structure was probably relatively soft and particles broke during compaction. Harder materials could be specified to minimize particle breakage during compaction as well as deterioration due to weathering over a long period.

The Department of Highways' provisions for materials that may be used for backfill (soil envelope) appear most suitable. Materials specified would not degrade under normal placement or compactive efforts. It appears that materials placed as backfill for the KY-80 structure did not conform to those designated in Departmental provisions. It is recommended that materials meeting Departmental requirements for backfill be used for future construction.

AASHTO officials should be solicited and requested to develop guidelines relative to design considerations for internal loadings for conduit. Some design texts contain design guides for buoyant forces and dead loads resulting from water within structures. Those officials also should be requested to provide rational guidelines relative to either consideration of longitudinal beam action or the establishment of limiting differential settlements permissible to insure that beam action could not be a failing mode.

ADDENDUM

After a draft of the major portion of this report was submitted for review, additional documents relating to events during construction and soon thereafter were delivered to the KTRP. Information contained in those documents was considered to be significantly important to prepare this Addendum and to suggest an additional scenario. Following are a synopsis of additional information received and a scenario that presents other possible causes for the ultimate collapse of the long-span structure.

On November 17, 1986, the following were received from BMI:

1) initial inspection reports,

2) elevation measurements made on hooks at various times during installation,

3) computer run on MULTSPAN computer program utilizing readings from early 1986, and

4) results of classification tests on soil obtained at the site on October 28, 1986.

The computer-run sheets were based upon field data obtained March 6, 1986. The Findings and Recommendations sheet contained an entry under type of deformation "Case 1 - Symmetrical flattening of crown." One recommendation was as follows: "Pipe deformations are critical - at least one top midordinate deflection is out 30% - close road and do a detailed evaluation, including soil borings to estimate any additional movement and determine needed remedial measures." In response to those recommendations, the eastbound lanes were closed, the Division of Materials performed materials explorations, and development of plans for remedial actions were initiated.

Laboratory Report No. 808124 dated July 28, 1980, contained a discussion under Section I wherein it was stated that the fill was placed in accordance with specifications as modified and approved by Armco. It also was stated that no problems, which would affect this structure due to the backfill operations, were observed.

Section II of the laboratory report contained technician's daily reports. The 5-27-80 report noted the railroad tracks were to have been placed 3' 10" above the bottom of the span, but no measurements were observed being made. It could not be determined if the tracks were at the design elevation and the tracks could not be used as a reference point to determine the span rise. The Report of Density Determination Tests dated 5-17-80 indicated specifications for density as being 90 percent. Percentage compaction from two tests were 98 and 96.

The 5-28-80 report noted that Mr. Frank Miller, Mr. Lee Anderson, and Mr. Sam Green were notified that the backfill material may not be the same gradation as the material sampled in April 1980. The technician thought degradation of the shale might be due to compaction, blading, and tracking operations.

A report indicating services for June 4, 5, 10, and 11, 1980, stated soils classified as A-2-4 after it had been placed and compacted (for

Wednesday, June 4, 1980). It was recommended that the backfill be compacted to 93 percent of the modified Proctor density at a moisture content no greater than three percent above optimum moisture. That was recommended to reduce degradation of the shale. The recommendation was approved by Mr. Miller and Mr. Green. Six percentage compactions reported 6-4-80 were 89, 92, 90, 91, 96, and 93, with 93 being listed as required by specifications. Retests 1A and 2A were reported as 90 and 91 percent of maximum dry density after recompaction.

On 6-5-80, the percentage compacted ranged from 89 to 96; the range was 90 to 98 according to reports for 6-10-80.

D. A. Ream's June 17, 1980, letter to F. Miller noted that fresh and reworked shale sampled May 28, 1980, classified as A-4 and out of Armco and KDOT project specifications. Shale was sampled again on May 30, 1980, after being compacted and the soil type then was A-2-4. Unconfined compressive tests made on the second set of samples showed the material to have approximately 4,000 psf bearing capacity when compacted to 93 percent of the maximum density established by the modified Proctor tests. It was opinioned that the soil was suitable for use around the Super Span and recommended that the moisture content should not be greater than three percent above optimum moisture when placed.

The June 20, 1980, report stated that measurements were made with all fill in place to determine if additional fill would have any effect on the span. No unusual movement was noted, except for a slight movement downward. All movements were well within specifications.

On November 18, 1986, FMSM and HAI personnel delivered to KTRP cross-section sheets containing field measurements obtained November 6, 1980. One sheet depicts design and actual crown elevations and contains a notation -- Max. Error (-1.84'). That same sheet depicts design and actual rail elevations with the notation -- Max. Error (-0.48'). Cross-sectional dimensions are depicted on one sheet for Stations 50 + 97 to 51 + 37. Design, tolerance, and actual dimensions are shown.

At Station 51 + 12, it appears the structure moved outward approximately 1.25 feet from the design position to the left of centerline and inward 0.15 foot to the right of centerline. At Station 50 + 97, indications are the structure was 0.98 foot outward of design

on the left of centerline and 0.68 foot inward of design on the right of centerline. Actual sections were depicted as being within tolerances between Stations 51 + 30 and 51 + 37. Between Stations 51 + 21.5 and 51 + 30, the structure was shown to have moved outward of tolerance both right and left of centerline.

AASHTO specifications for both design and construction guidelines stress the soil-metal plate structure interaction concept. Section 1.9.10 - Long-Span Structural Plate Structures of the 1977 AASHTO specifications designates use of A-1, A-3, A-2-4, or A-2-5 soils for the envelope when the height of fill is less than 12 feet. Material reportedly met classification requirements prior to placement and compaction and later degraded during compaction, blading, and tracking operations; further degradation of the shaley backfill may have resulted from weathering. Properties of the in-place soil may have been unsuitable for the necessary soil-metal plate structure interaction.

Finer-grained soils do not drain readily and are ordinarily more plastic than coarser-grained soils. Fine-grained soils may have very low bearing values when their moisture content is appreciably above optimum. The shale may have been subjected to excess moisture and consequent softening and may have decreased the effectiveness of the soil-metal plate structure interaction.

A decrease or loss of passive resistance pressure, for whatever reason, may lead to distress and eventual collapse of the metal structure.