Investigation of East Chicago Ramp Collapse

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Abstract: A summary is presented of the investigation performed by the National Bureau of Standards (NBS), at the request of the Occupational Safety and Health Administration, to determine the most likely cause of the collapse of a portion of a highway ramp in East Chicago, Indiana. The investigative effort included an extensive field study to ascertain the conditions prior to and after the accident. In addition, the NBS performed physical tests on key components of the temporary support system used to build the ramp. A structural analysis was performed to compute the magnitude of the forces acting in various components of the support system prior to the failure. The calculated forces were compared with the expected strengths of the structural components. It was concluded that the most likely triggering mechanism of the collapse was the cracking of concrete pads supporting a shoring tower. It was further concluded that there were four deficiencies that contributed directly to the collapse. Had any of these deficiencies not existed, it is unlikely that the collapse would have occurred.

Introduction

On April 15, 1982, portions of a highway ramp under construction in East Chicago, Indiana, collapsed, killing 13 and injuring 15 workers (Fig. 1). This was the worst construction accident in Indiana's history. Immediately after the collapse, the Occupational Safety and Health Administration, Department of Labor, requested the assistance of the National Bureau of Standards (NBS) to determine the technical reasons for the incident. A team of structural engineers arrived in East Chicago on April 16, 1982, and was briefed by personnel of the Indiana Occupational Safety and Health Administration (IOSHA), which had jurisdiction over the collapse site. Based on initial field observations the following day, the NBS team formulated a comprehensive investigative plan which included securing construction documents and material samples, interviewing workers on the project, performing detailed field investigations, performing physical tests on materials and components, and carrying out structural analyses of the partially completed structure.

Based on the results of the structural analysis and the physical tests on key components, it was concluded that the most likely triggering mechanism of the collapse was the cracking of concrete pads supporting a shoring tower that was part of the temporary support system. It was further concluded that this initial failure could have led to failure of additional components and resulted in the total collapse of the support

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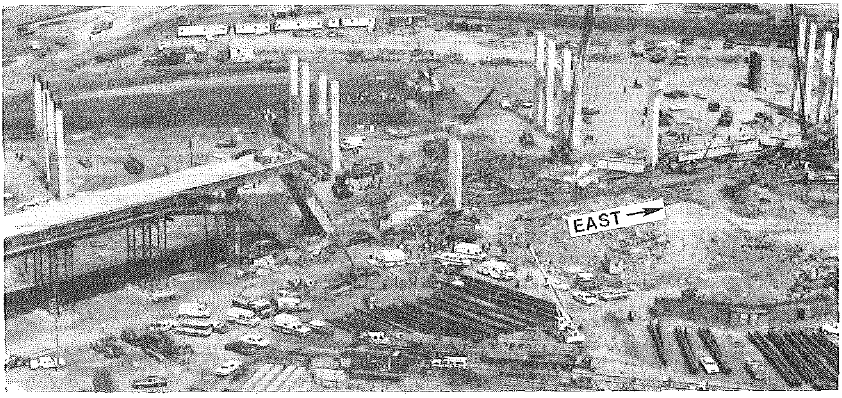

FIG. 1.—Overall View of Collapsed Portion of Ramp; View Is Approximately Toward North (Courtesy of Lake County Coroner's Office)

system. In addition, it was concluded that four deficiencies existed which contributed to the collapse. Had any of these deficiencies not existed, it is unlikely that the collapse would have occurred.

This paper summarizes the NBS investigative report (1), which is available from the National Technical Information Service [National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (order No. PB 83-124800)].

Description of Structure

The structure is known as Ramp C of the Riley Road Interchange and is intended to move traffic from Riley Road to the eastbound lanes of Cline Avenue in East Chicago, Indiana. Fig. 2 is a view looking approximately east that was taken shortly after the collapse, and shows the portion of Ramp C that remained standing. The ramp was to be built in two portions. In the first phase, the portion from pier 404 to Cline Ave. would be built; and in the second phase, the western portion connecting to Riley Road would be built. At the time of the collapse, work was in progress east of pier 407.

An elevation view of the ramp and the falsework system is shown in Fig. 3. The ramp has a cast-in-place, post-tensioned, concrete superstructure, which was constructed in sections referred to as "units." The units were typically 180-ft (55-m) long, and terminated approximately 42-ft (13-m) beyond the piers. On the day of the collapse, work was being performed on units IV and V, and the collapsed portion included these two units. A typical cross section of the ramp is shown in Fig. 4. Each unit was cast in two stages: the bottom slab and webs were cast first, and then the top slab was cast. After the prescribed concrete strength had been reached, the post-tensioning tendons within the webs were stressed.

Prior to post-tensioning a unit, all construction loads (including self-weight) were carried by the falsework system shown schematically in

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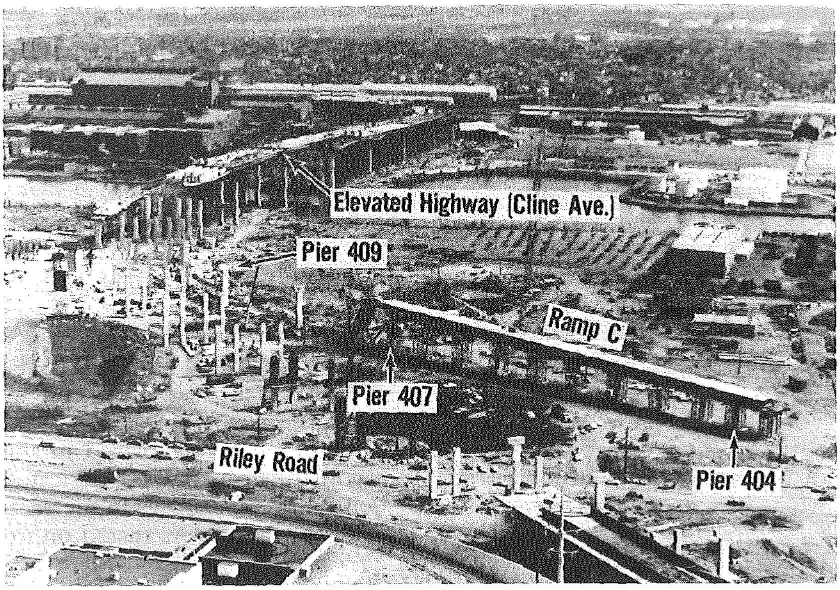

FIG. 2.—Overall View of Riley Road Interchange Site; View is Approximately Toward East (Courtesy of Post-Tribune, Gary, Indiana)

Figs. 3 and 5. The formwork rested on longitudinal stringers which were supported by transverse crossbeams (Fig. 5). The crossbeams were in turn supported by shoring towers, located at the piers and at the third-points of the spans (Fig. 3). The concrete piers did not carry any con-

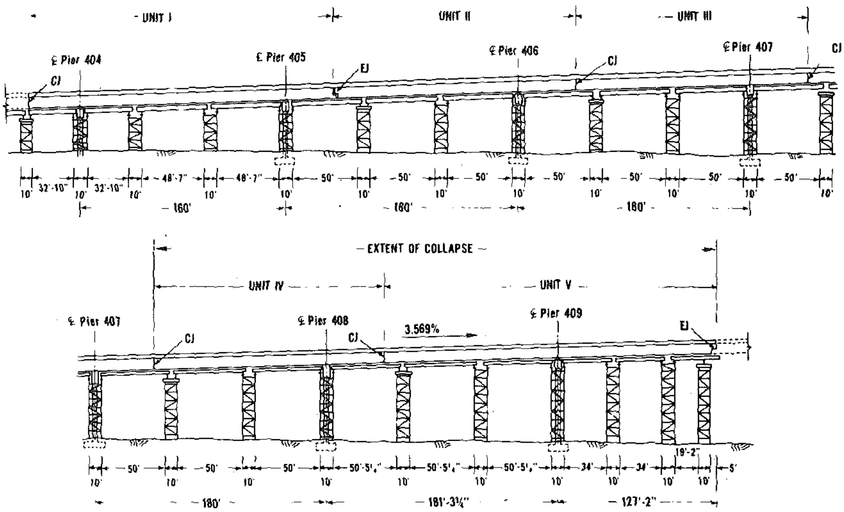

FIG. 3.—Elevation View of Ramp C and Falsework System

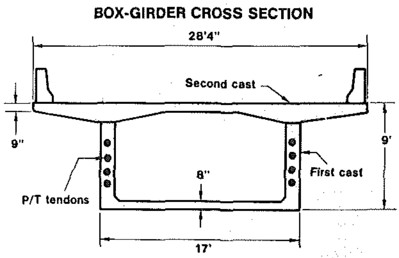

FIG. 4.—Typical Cross Section of Ramp C

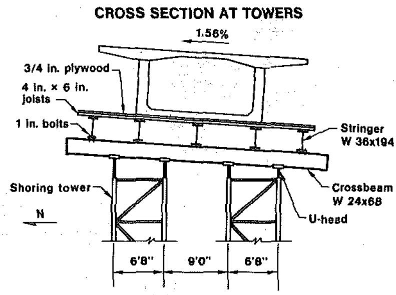

FIG. 5.—Cross Section of Top Portion of Falsework System at Shoring Tower Location

struction loads other than the weight of fresh concrete placed in the diaphragms. The shoring towers were constructed from prefabricated welded frames, which were joined by bolt-on tubular braces in the east-west direction to form the towers. The legs of the towers rested on individual concrete pads with dimensions 5 × 5 × 1-ft (1.5 × 1.5 × 0.3-m). The pads had only nominal wire mesh reinforcement, and in subsequent strength analyses they were considered to be plain concrete. The tower legs did not bear directly on the pads; rather, the legs rested in sand jacks which provided for uniform distribution of the leg forces to the pads and were used to unload the towers during the lowering of the falsework. The sand jacks were 16-in. (410-mm) square boxes, constructed from 2 × 4 lumber and a sheet metal base. These boxes were filled with fine slag.

At each support point shown in Fig. 3, there were two shoring towers: a north tower and a south tower. Fig. 6 shows the numbering system adopted by NBS for the tower locations between piers 407 and 408, and

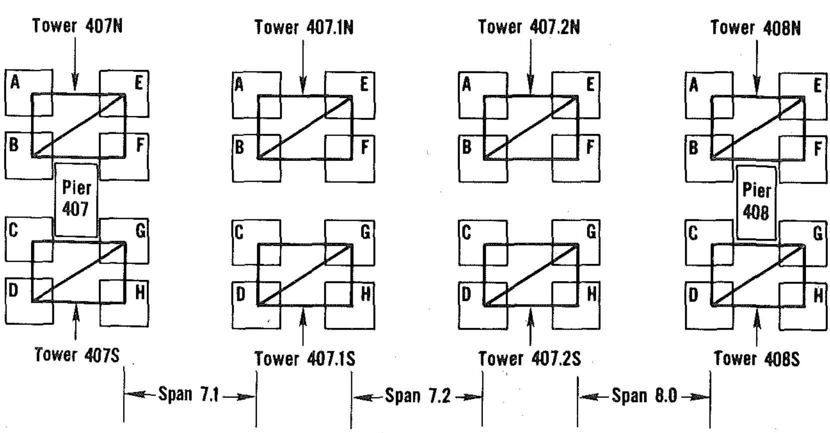

FIG. 6.—Plan View of Shoring Towers and Supporting Concrete Pads, Indicating Identifying Nomenclature

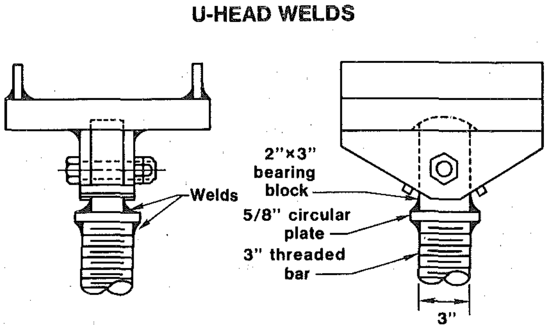

FIG. 7.—Front and Side Views of U-Heads Used to Support Crossbeams at Top of Shoring Towers

also indicates the notation for the tower legs and the concrete pads.

At the tops of the shoring towers, "U-heads" were used to support the crossbeams. Fig. 7 shows a front and side view of the upper portion of a U-head. Of special significance are the two welds used to connect the threaded bar to the rectangular bearing block. One weld joined the bearing block to a circular plate and the other weld joined the plate to the threaded bar. During the field investigation it was noted that many of the U-heads within the collapse debris had severed at either one of these welds. As can be seen in Fig. 7, a U-head can rotate about one axis as a result of the cylindrical fit between the bearing block and the top plate.

As indicated in Figs. 3 and 5, Ramp C has an uphill slope of about 3.6% and a superelevation of about 1.6%, increasing to a higher value beyond pier 408. Because the U-heads could rotate only in one direction, wedges were specified between stringers and crossbeams to accommodate for the slope [Fig. 8(*a*)]. However, during the field investigation of

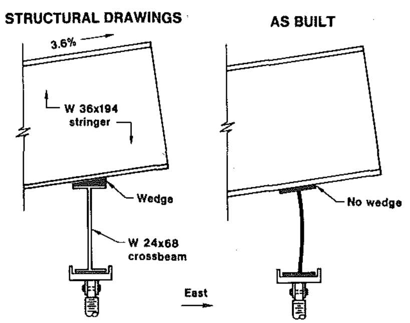

FIG. 8.—Comparison of Specified and As-Built Construction Detail Between Stringers and Crossbeams (Schematic, Not to Scale)

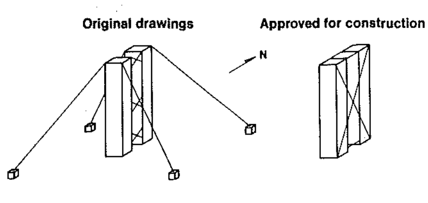

FIG. 9.—Comparison of Guying System in Original Drawings With System Approved and Used in Construction

the standing portion of the ramp, no wedges were observed nor were any found within the collapse debris. The consequences of the missing wedges [Fig. 8(*b*)] were investigated by means of physical tests. The structural drawings also indicated that 1-in. (25-mm) bolts were to be used to connect the outer stringers to the cross-beams (see Fig. 5). There was no evidence that bolts had been used in ramp C; instead, friction clips (similar to those used to hold down railroad tracks) were substituted for the bolts.

The original structural drawings called for a tower guying system as indicated in Fig. 9(*a*). However, a substitute system was proposed by the contractor and approved by the Indiana State Highway Commission for construction [Fig. 9(*b*)]. The substitute X-bracing system—using 5/8-in. (16-mm) steel cables—did not provide the same degree of longitudinal stability to the top of the shoring towers as would have been provided by the external guying system.

Description of Collapse

On the day of the collapse, concrete was being placed for the top slab of unit IV. Based on the statements of workers, concrete delivery tickets and other physical evidence, it was concluded that the top slab had been placed up to about 20-ft (6-m) from pier 408, and the diaphragm form over pier 408 was about half filled. Fig. 10 shows the state of construction prior to the accident. Unit III had previously been post-tensioned, and post-tensioning tendons were in place in unit IV. The unit-IV tendons were attached to unit III with coupler hardware, and they were wedged into place at the end of the portion of unit IV extending beyond pier 408. A gap existed between the bottom portions of units IV and V so that unit IV could be post-tensioned after the top-slab concrete had attained the required strength.

At about 10:40 a.m. on April 15, 1982, unit IV collapsed suddenly. According to eyewitnesses, concrete was not being discharged onto the formwork at this time. Several workers provided descriptions of the failure. Perhaps the most informative description was given by a worker who was descending the scaffolding stairway located on the south side

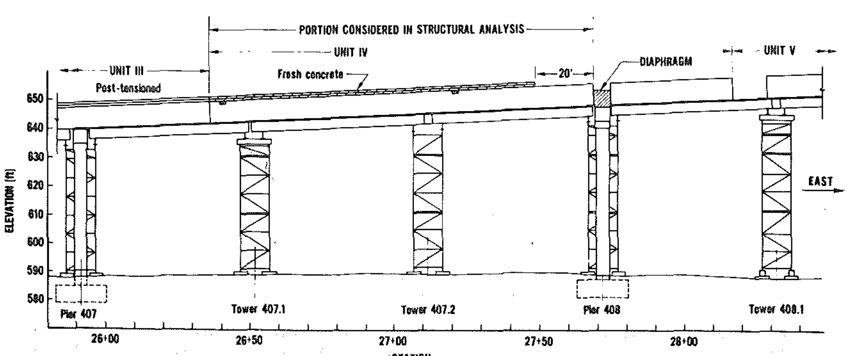

FIG. 10.—State of Construction at Time of Collapse

of unit IV, east of tower location 407.2. He was about 35–40-ft (11–12-m) from the ground [the shoring towers at 407.2 were about 45-ft (14-m) high], facing east and looking toward the north. The first indication this witness had of the start of the collapse was a loud pop, and he saw the longitudinal stringers on the north side of the ramp going down first, followed by a progression of stringers collapsing from the north to the south. Another worker, standing on the south side of the ramp between tower locations 407.1 and 407.2 heard a crack followed by a bass sound. This worker stated that the structure began coming down between tower location 407.2 and pier 408. The statements of these two workers and of others indicate that the collapse appeared to initiate in the vicinity of tower location 407.2. Statements further indicated that the collapse was sudden and without distress warnings. After the collapse started, the portion of unit IV between 407.1 and pier 408 came down in a V-shape. The portion of unit IV extending beyond pier 408 was dragged across the pier because it was attached to the rest of the unit by the post-tensioning tendons. At some point this segment flipped over, was propelled westward, and landed on top of the portion of unit IV already on the ground. Fig. 11 is a view as seen from the northeast of unit IV after the collapse.

Following the failure of unit IV, unit V (Fig. 3) remained standing, but eyewitnesses reported that it was left in a "teetering" condition. About 5 min after the initial failure, unit V also collapsed. However, it did so in a gradual "domino" fashion, progressing from west to east. The NBS investigation concentrated its effort in determining the cause of the initial failure of unit IV, and did not attempt to determine how failure propagated from unit IV to unit V.

After the collapsed structure had been documented by the various investigative groups, the contractor was asked to carefully remove the concrete debris and leave the falsework components as undisturbed as pos-

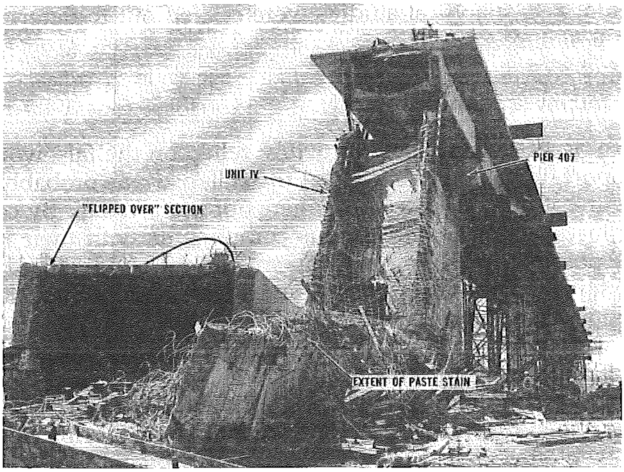

FIG. 11.—View of Unit IV Looking Toward Southwest

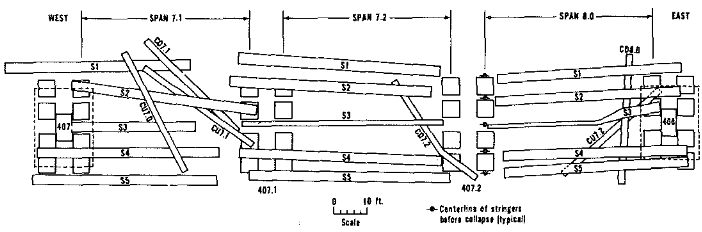

FIG. 12.—Location of Stringers and Crossbeams After Collapse

sible. Some of the components lying east of pier 408 had been moved during the rescue operations to make way for emergency vehicles. After the concrete debris had been removed, a careful inspection of the falsework components was carried out and their positions were documented. Fig. 12 shows the locations of the stringers and crossbeams between piers 407 and 408. Of special significance are the locations of the two crossbeams originally at tower location 407.2. It can be seen that the eastside crossbeam (CU7.2) was hurled a considerable distance toward pier 408. The other crossbeam (CD7.2) underwent a rotation of its north end as it fell almost straight downward. The locations of the stringers in span 7.2 and 8.0 (Fig. 6) indicate that they first lost their supports at tower location 407.2. Thus, the locations of the stringers and crossbeams lend support to the statements of workers indicating the collapse appeared to have initiated in the vicinity of tower location 407.2. The collapse conditions of the shoring towers were also documented carefully. It was concluded that the towers at 407.2 buckled in the east-west direction in a jackknife mode about the bolted connections used to join the welded frames.

While the field investigations resulted in a clear understanding of the conditions existing prior to the accident and of the manner by which the structure collapsed, the triggering mechanism was not revealed. Thus, experimental and analytical studies were carried out to determine what initiated the failure.

Experimental Investigations

Since the partially completed structure relied on the falsework system for its support, it was postulated that the collapse was probably initiated by the failure of one or more components of that system. By comparing the likely loads acting in the various components with their respective strengths, possible triggering mechanisms might be identified. Thus, it was necessary to know the strength and deformation characteristics of key components, and this was the motivation for the experimental studies. Tests were performed on components for which strength could not be predicted reliably with analytical models.

The concrete pads supporting the shoring towers rested directly on ground, so the subsurface conditions were studied. Boring logs and jar samples taken after the collapse were examined by an NBS geotechnical engineer. It was concluded that, while the soils were stratified, there were no marked differences between the soils supporting the shoring towers at 407.1 and 407.2. The areas around the piers, however, were backfilled with boiler slag having generally loose compaction as indicated by low blow-count values in the Standard Penetration Test. Thus, the towers adjacent to the piers had different support conditions from the other towers.

Because the strength of the concrete pads in resisting the tower leg loads would depend on the soil pressure distribution, and because there are no reliable means for predicting this distribution, full-scale loads tests were performed at the site. These tests were performed as a cooperative effort among several investigators, including NBS. Three tests were conducted using fourpad groups having the same layout as used in the ramp construction. The pads used in the tests were obtained from the collapsed structure east of pier 408, and they all appeared to be crack-free. Load was applied to the pads through sand jacks similar to those used in construction. All four pads were loaded simultaneously. Test 1 was performed at the original location of the north tower at 407.2, test 3 was carried out at the location of the north tower near pier 408, and test 2 was located midway between the aforementioned two.

From these tests, information on the foundation stiffnesses and cracking loads was obtained. The pads failed due to the formation of flexural cracks, either along a pad-diagonal or at the middle approximately parallel to an edge. Pad cracking was accompanied by a sudden, downward displacement of the loading leg. Following tests 1 and 3, full-depth cores were taken from the pads. The cores were trimmed and tested for compressive strength using procedures given in ASTM C 42 (2). Table 1 summarizes the results of the full-scale load tests. The foundation stiffness values include the settlements of the soil plus those of the slag filler in the sand jacks.

TABLE 1.—Summary of Full-Scale Load Tests on Concrete Pads

Test number (1)	Pad number (2)	Cracking load, in thousands of pounds (3)	Foundation stiffness, in thousands of pounds per inch (4)	Core compressive strength, in pounds per square inch (5)	Core length, in inches (6)
1	E	112	145	9,350	13-1/8
	F	132	154	7,640	15-3/8
	B	178	146	9,330	13
	A	154	140	7,490	14
2	E	88	112	6,350	12-5/8
	F	115	115	6,090	13-5/8
	B	153	112	7,170	13-3/4
	A	168	119	7,200	13-1/2
3	E	135	98	N/A	N/A
	F	77	93	N/A	N/A
	B	104	115	N/A	N/A
	A	102	112	N/A	N/A

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1,000 psi = 6.89 MPa; and 1 kip/in. = 0.175 kN/mm.

In order to relate the results of the full-scale load tests to the expected strengths of the concrete pads at 407.2, full-depth cores were taken from pads A through H, and these cores were likewise trimmed and tested for compressive strength according to ASTM C 42. The thickness of the pads, as measured by the full-depth cores, agreed with the 12-in. (305-mm) dimension that was specified. The core strengths were highly variable ranging from 4,460–7,700 psi (30.8–53.1 MPa). The pads were cast at the job site from concrete remaining from various placements in the structure, and this may explain the high variability of the core strengths. The structural drawings did not specify a design compressive strength for the concrete pads.

The next component investigated was the sand jacks used to transfer load from the tower legs to the concrete pads. Relatively intact wooden sand jacks were recovered from the site and shipped to NBS along with a sample of fine slag similar to the material used as a filler in the boxes. Workers' statements revealed that problems had arisen with the wooden boxes during the early stages of construction. In addition, the structural drawing specified sand jacks fabricated from steel pipe. Thus, there were questions about the adequacy of the wooden sand jacks to transfer the leg loads to the pads. The tests revealed that the slag-filled wooden boxes could sustain tower leg loads up to 160 kips (712 kN) without distress. According to information in the structural drawings, the design load for the tower legs was calculated to be 81 kips (360 kN).

Data were not available on the tensile and compressive strengths of the bolt-on diagonal braces used to construct the towers from the prefabricated frames. Six braces similar to the diagonal braces used in the towers at 407.2 were obtained from the contractor's stockpile and shipped

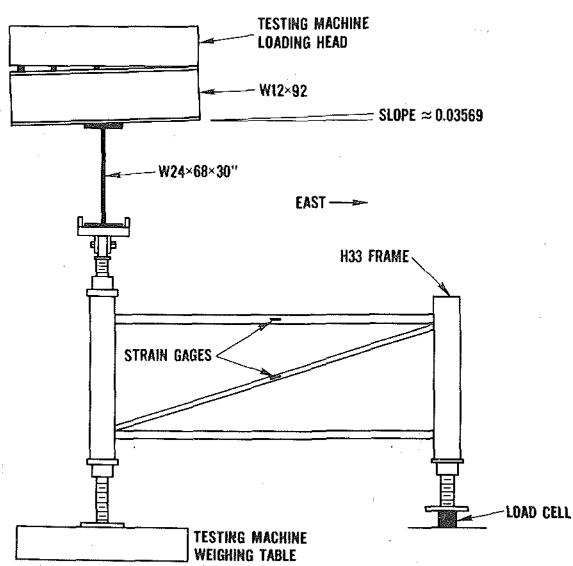

FIG. 13.—Arrangement of Falsework Assembly Tests

to NBS for testing. Testing fixtures were built so that the braces could be loaded with the same end conditions as in the towers.

The broken U-heads belonging to the towers at 407.2 were also shipped to NBS for examination in order to determine the quality of the welded joints that had fractured during the collapse (see Fig. 7). The critical portions of the U-heads were removed, sectioned, and prepared for metallographic examination. It was concluded that the weld quality in the U-heads recovered from the collapse site was poor. There was evidence of lack of fusion between the base metal and the weld metal, and some of the welds were less than 1/4-in. (6-mm) thick.

As previously mentioned, wedges were omitted from between crossbeams and stringers (Fig. 8). Tests were carried out to determine the consequences of this omission. The arrangement for these tests is shown in Fig. 13. A piece of a W12 × 92 beam was bolted to the head of a testing machine so that it had the approximate slope of the ramp. This beam was used to load the falsework assembly consisting of a crossbeam, a U-head, a welded frame, and two lower screw jacks. The threaded bar of the U-head was instrumented with strain gages from which the axial load and bending moment could be determined. The braces of the frame were also instrumented with strain gages so that these member loads could be determined. The results of these tests indicated that the absence of the wedges caused a horizontal force and a bending moment on the U-heads. It was determined that the horizontal force, acting toward the east, was about 2.4% of the vertical load. It must be realized that while the horizontal force seems to be small, when it is applied to the U-heads at the top of the 45-ft (14-m) towers, it creates a significant increase in the leg forces on the east side of the towers.

Structural Analysis

A structural analysis of the partially completed ramp was carried out in order to estimate the member loads likely to have been present just prior to the collapse. After reviewing workers' statements, construction records and local weather data, it was concluded that the only significant loads acting at the time of the accident were the weights of fresh concrete, hardened concrete and reinforcement, the weight of the formwork, and the self-weight of the falsework system. Using the structural drawings, test data on the unit weight of concrete, field-measured dimensions of structural members, and design handbooks, an estimate was made for each of these loads.

A structural model of the critical portion of the ramp (Fig. 10) was developed. Because of the two-stage method of constructing each unit, two models were needed to represent accurately the force distribution throughout the structure. During the casting of the bottom slab and the webs (compositely referred to as the "U-section") the weight of the fresh concrete within the forms was transferred to the stringers, which were simply supported by the crossbeams. Thus, for this case a tributary area approach was used to calculate the concentrated stringer reactions provided by the crossbeams. These reactions were applied to finite-element models of the shoring towers at 407.1, 407.2 and pier 408. The legs of the towers were assumed to be acting on elastic springs whose stiff-

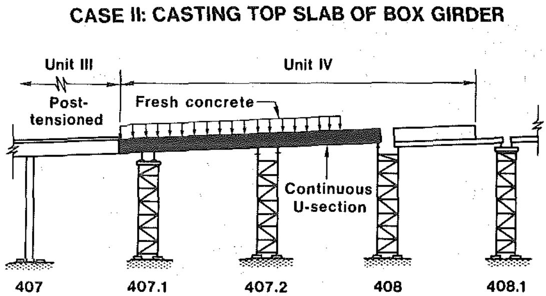

FIG. 14.—Structural Behavior of Partially Completed Ramp During Top-Slab Casting

nesses were derived from the full-scale load tests. The member forces resulting from the computer analysis were assumed to be present when the top-slab placement of unit IV was begun.

When the top slab was placed, an entirely different structural system was involved. Because the U-section had hardened, the load of the fresh top-slab concrete was distributed to the towers by the continuous-beam action of the U-section. This behavior is shown in Fig. 14. The flexural stiffness of the U-section was more than nine times that of the stringers, and for this case it was assumed that the stringers only served to transfer bearing reactions to the crossbeams. The U-section was modeled by a series of overlapping plate-bending and membrane finite elements, and was supported by the crossbeams. A series of elastic constraints were applied at the end of unit IV in contact with unit III to model the restraint provided by unit III. The portion of unit IV east of pier 408 was not included in this analysis because concrete was not being placed in this part. The loading for this second analysis included the weight of the fresh concrete placed to within 20 ft (6 m) of pier 408 (Fig. 10).

The structural model also included a provision for modeling the effects of no wedges by introducing to the top of the towers horizontal forces in proportion to the magnitude of the vertical leg loads. The results of the falsework assembly tests were used to develop the appropriate proportionality factor.

The final estimates of the loads acting in the various structural elements were obtained by superposing the results from the two analyses. The following forces (or stresses) were tabulated: the tensile bending stresses in the U-section, the end reactions of the stringers, the vertical leg loads at the top of the towers, the forces in the tower braces, and the vertical support reactions at the bottom of the towers. The analyses indicated that the falsework components at location 407.2 were about 50% more heavily loaded than at location 407.1 or pier 408.

Interpretation of Results

The results of the structural analyses were used to trace the load path from the top to the bottom of the falsework of unit IV. The resultant forces were compared with the capacities of the components in transmitting the forces. The capacities were derived from the NBS tests or from analytical expressions for the routine strength parameters, such as the bending and shear capacities of beams.

Based on comparisons of member loads to member resistances, the following conclusions were obtained:

1. The expected capacities of the stringers and crossbeams were sufficient to resist the applied loads.
2. While the quality of the welds in the U-heads was poor, it was unlikely that the loading condition existing prior to collapse would have resulted in fracture of the U-head welds.
3. The expected capacities of the shoring tower legs were sufficient to resist the applied axial loads.
4. The expected tensile strength of the diagonal braces in the shoring towers was sufficient to resist the computed diagonal member forces.
5. While buckling of some of the bolt-on diagonal braces could have occurred, additional structural analyses, with heavily loaded compression diagonals modelled as having failed, resulted in stable force redistributions within the towers.
6. The wooden sand jacks would have been able to resist the computed tower support reactions.

Thus, it was concluded that the collapse of unit IV was not triggered by the failure of any of these components: stringers, crossbeams, U-heads, shoring towers, and sand jacks. It was found, however, that some of the concrete pads were load critically.

The support reactions computed for the towers at 407.2 are given in the second column of Table 2. Using the results of the full-scale load tests and the compressive strengths and lengths of cores taken from the pads at 407.2, it was concluded that pad F had the lowest expected strength [94 kip (418 kN)]. Pad F was heavily loaded [90 kip (401 kN)], and it is reasonable to postulate that it had the highest likelihood of being the first to crack.

TABLE 2.—Computed Support Reactions at 407.2

Tower leg (1)	Load, in Thousands of Pounds: Prior to pad cracking (2)	Pad F cracked (3)	Pads E and F cracked (4)
A	67	75	91
B	70	85	91
C	70	76	77
D	67	68	65
E	86	101	55
F	90	6	16
G	90	97	98
H	86	87	85

Note: 1 kip = 4.45 kN.

The field investigation revealed that, at the time of the collapse, fresh concrete was not being added to the structure, and thus, additional load was not being transmitted to the concrete pads. In the full-scale load tests, many pads cracked during a constant-load holding period. It was hypothesized that during the holding period the soil pressure distribution was changing, thereby increasing the maximum flexural stresses in the pads. The experimental evidence thus supports the hypothesis that pads could have cracked while no additional load was being placed on the structure.

While it is reasonable to surmise that a pad probably cracked, this does not explain why unit IV collapsed. Cracking of a pad only provides a triggering mechanism, and it was necessary to determine how such an initial failure could have lead to an unstable condition leading to the total collapse of unit IV.

Failure Sequence

Additional structural analyses were performed to investigate the possible consequences of the cracking of pad F at tower location 407.2. The full-scale load tests indicated that pad cracking was followed by an instantaneous settlement of the screw jack resting in the sand jack. The structural model was subsequently analyzed with a 0.4-in. (10-mm) settlement applied to leg F. The 0.4-in. settlement was the average value observed in the load tests. The support reactions after cracking of pad F are given in Table 2; these values were obtained by adding the results of this analysis to the reactions present prior to pad cracking. Due to the failure of pad F, the support reactions at pads E and G increased to 101 kips (450 kN) and 97 kips (432 kN), respectively. Thus, pads E and

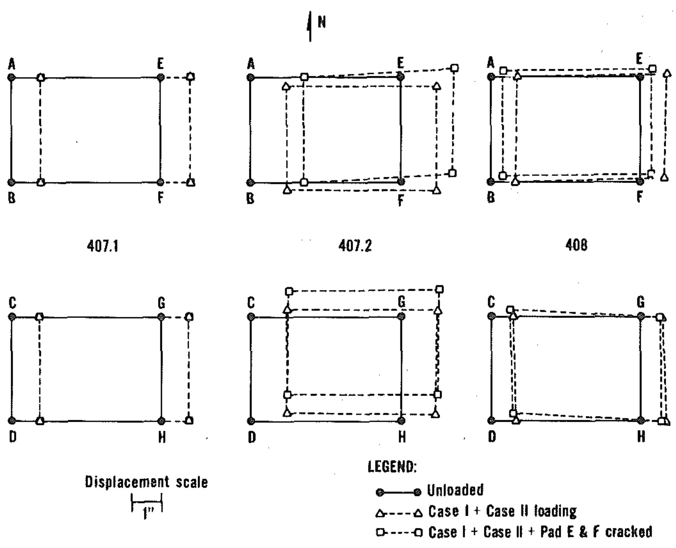

FIG. 15.—Longitudinal Displacements of Tops of Shoring Towers Under Various Loading Conditions

G were the next likely candidates for cracking. Based upon their expected strengths, it was concluded that there was a somewhat greater probability that pad G would have cracked before pad E, but the physical evidence indicated that the collapse of unit IV initiated on the north side of the ramp. It was, therefore assumed that pad E likely cracked subsequent to the failure of pad F.

A final analysis of the structural model was performed with a 0.4-in. settlement applied to leg E. The results were added to the support reactions with pad F cracked to obtain the values shown in the last column of Table 2. With pads E and F cracked, the reactions at pads A and B increased to 91 kips (405 kN), and based upon their expected strengths it is unlikely that these pads would have cracked.

A significant effect due to cracking of Pads E and F is shown in Fig. 15, which shows the longitudinal displacements of the tops of the shoring towers at 407.1, 407.2 and pier 408 under different loading conditions. Cracking of pads E and F resulted in a sudden increase of about 0.8 in. (20 mm) in the longitudinal displacement at the top of the north tower at 407.2.

As the tops of the tower translated longitudinally, the bottom flanges of the crossbeams would have translated also because they were held in the U-heads. The top flanges, however, would have remained fixed due to the friction between the stringers and the crossbeams. By means of finite-element analyses of the crossbeams at 407.2, it was shown that the increase in relative flange displacement due to cracking of pads E and F would have increased the bending moments in the U-heads (1). It was thus surmised that the next likely event in the failure sequence was fracture of the U-head welds.

While it was difficult to pinpoint which of the U-heads at 407.2 failed first, examination of the physical evidence, such as the positions of the fallen crossbeams and of the fractured U-heads, suggested that U-heads at legs A and B failed first and were followed by failure of those at legs C and D. When crossbeam CD7.2 (supported by U-heads A through D) lost its support, all loads at 407.2 were transferred to the other crossbeam (DU7.2 in Fig. 12). With the east side of the towers being loaded heavily, the towers collapsed in a jackknife mode and crossbeam CU7.2 was propelled toward pier 408. This failure mode is shown in Fig. 16.

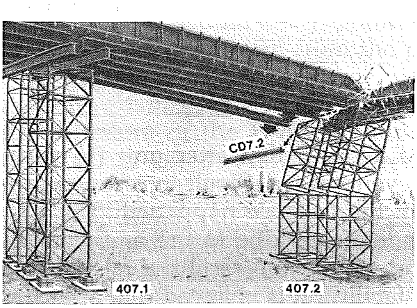

FIG. 16.—Rendering of Likely Failure Mode of Shoring Towers at 407.2 After Crossbeam on West Side Lost Support

At this point it is likely that the U-section of unit IV cracked and fell in the previously described V-shape.

Contributing Causes of Initial Collapse

The NBS investigation concluded that the collapse of unit IV was most likely initiated by the cracking of concrete pads supporting the falsework at tower location 407.2. Cracking of the pads increased the longitudinal displacement of the top of the north tower at 407.2, which in turn increased the relative flange displacement of the crossbeams. The relative flange displacement produced high bending moments in the U-heads supporting the crossbeams at 407.2. Due to their poor quality, the U-head welds fractured, then crossbeam CD7.2 lost its support, and an unstable condition was attained leading to the total collapse of unit IV.

Based on the preceding failure sequence, it was concluded that four deficiencies contributed directly to the collapse of unit IV:

1. Specified wedges were omitted between stringers and crossbeams.
2. The concrete pads had an inadequate factor of safety to resist the anticipated construction loads.
3. The tops of the shoring towers were not stabilized adequately in the longitudinal direction.
4. The quality of the U-head welds was poor.

Had the specified wedges been used, horizontal forces would not have been introduced at the U-heads. This would have resulted in less loads on concrete pads E through F, and the likelihood of pad cracking would have been reduced. Had the concrete pads been properly designed, the likelihood of their cracking would have been reduced greatly. Had positive longitudinal restraint been provided to the tops of the shoring towers, their longitudinal displacement would have been reduced and this, in turn, would have reduced the bending moments imparted to the U-heads. Finally, had the U-head welds been of good quality, they would likely have been able to resist the applied loads without fracturing, and crossbeam CD7.2 would not have fallen.

It is, therefore, reasonable to conclude that had any of the preceding deficiencies not existed, it is unlikely that the collapse of unit IV would have occurred.

Collapse of Unit V

Unit V collapsed about 5 min after unit IV collapsed. Although an exact sequence of events was difficult to reconstruct, it was not difficult to understand why the collapse happened. The falsework system was not tied together adequately due to the omission of specified bolts between stringers and crossbeams, and the falsework system lacked positive longitudinal stability. It is believed that after unit IV collapsed, the stringers that supported the west end of unit V (Fig. 10) also fell or were left dangling without support at one end. This left the west portion of unit V in an unstable condition, and eventually the towers at 408.1 collapsed. The failure of unit V then propagated in domino fashion from the west to the east as falling stringers struck the remaining towers.

Summary and Conclusions

At the request of the Occupational Safety and Health Administration, the National Bureau of Standards conducted an investigation to determine the technical reasons for the collapse of Ramp C of the Riley Road Interchange in East Chicago, Indiana. The investigative effort included an extensive field study to document the collapsed structure and to establish the conditions as they might have existed prior to the failure. In addition, tests were performed at NBS on critical components of the falsework system, and NBS participated in full-scale load tests of concrete pads similar to those used to support the shoring towers. Finally, structural analyses of the partially completed structure were performed in order to compute the forces that might have existed prior to the collapse. The structural models accounted for the two-stage construction technique and for soil-structure interaction.

Based on considerations of field observations, expected strengths of various components of the falsework system and the computed loads, the following conclusions were drawn by the investigative team (1):

1. The collapse of unit IV was initiated by a failure in the falsework system at shoring tower location 407.2. This conclusion is supported by the following observations: (a) The locations of key falsework components after the collapse indicated that the failure most likely initiated at location 407.2; (b) the accounts of eyewitnesses who were near location 407.2 at the time of the failure indicated that the failure started in this vicinity; and (c) the analytical investigation indicated that the falsework components at tower location 407.2 were about 50% more heavily loaded than at tower location 407.1 or at pier 408.
2. The collapse was most likely initiated by the cracking of a concrete pad supporting a shoring tower leg at location 407.2. This conclusion is supported by the following observations: (a) The concrete pads specified in the structural drawings did not have an adequate margin of safety for the expected loads; (b) any of the pads on the east side of location 407.2 could have been likely candidates for failure based on the estimated loads derived from the structural analysis and on their expected strengths derived from tests of cores taken from the pads; (c) although the U-head welds were variable in size and of poor quality, it is unlikely that the loading conditions existing prior to the initiation of the collapse would have produced a failure of the U-heads; and (d) all falsework components at location 407.2, except for the concrete pads, had adequate capacity to resist the applied loads.
3. Several deficiencies contributed to the collapse of unit IV. Had any of these deficiencies not existed, it is unlikely that the collapse would have occurred. They were as follows: (a) Specified wedges were not used between the stringers and crossbeams to accommodate for the slope of the ramp; (b) the concrete pads did not have an adequate margin of safety for the expected loads; (c) the tops of the shoring towers were not adequately stabilized against longitudinal translation; and (d) the welds

in the U-heads used at the towers at location 407.2 were of poor quality.

4. The collapse of unit V occurred about 5 min after that of unit IV. Unit IV collapsed suddenly and without warning, but unit V collapsed in a more gradual "domino-fashion." The following factors help to explain why unit V also collapsed: (a) The collapse of unit IV left unit V without any positive means to provide longitudinal and transverse stability to the falsework system; and (b) the lack of positive connections between stringers and crossbeams permitted falling stringers to strike shoring towers supporting unit V.

Appendix.—References

1. Carino, N. J., Lew, H. S., Stone, W. C., Chung, R. M., and Hoblitzell, J. R., "Investigation of Construction Failure of the Riley Road Interchange Ramp, East Chicago, Indiana," *NBSIR 82-2593*, National Bureau of Standards (U.S.), Oct., 1982.
2. "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," *ASTM C42-77*, 1982 Annual Book of ASTM Standards, Part 14, pp. 32–36.