5. Effect on traveling public,
6. Subjective reasons, and
7. Apprehension during design.

Case histories illustrate that sometimes large movements that cause some harm to a structure must be called tolerable because alternative choices would be excessively costly or otherwise objectionable and hence less desirable. Such situations should be made clear to all persons concerned, to avoid misunderstandings. Similarly, some cases occur where a small movement, which results in substantial cost to correct the resulting difficulty, should be labeled intolerable if the movement could have been prevented by an inexpensive provision in the design.

In the early stages of design at a structure site, the factors listed above should be considered. If necessary they should be studied carefully before making a decision on the location and type of structure. Incentives to avoid such studies by overdesigning are commonly known but should not be succumbed to without good cause.

In addition to the necessary structure analyses to determine the effects of foundation movements, the geotechnical work on which the predicted movements are based should in many cases be improved. Recommended ways of strengthening this work are given in Goughnour's report on the situation in the geotechnical operations in the state highway departments (6). In the matter of organization, he emphasizes that the geotechnical functions of a department should be combined in one unit, to provide for unified control of personnel and equipment that are involved with the same basic geotechnical problems.

In the geotechnical work, probably the greatest need in predicting structure foundation movements is in improving field observations during and after construction. These observations should preferably be made by the geotechnical engineering personnel. They are much more concerned with the results than are construction or maintenance personnel. They will use the data directly to learn how accurate (or inaccurate) their predictions were and to strengthen their knowledge of their work. Field observations could include readings on the structure units, settlement platforms, deeper settlement devices (such as Borros anchors), piezometers, and inclinometers, all based, of course, on proper bench marks and reference points.

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REFERENCES

1. J. Feld. Tolerance of Structures to Settlement. Specialty Conference on Design of Foundations for Control of Settlement, SM and F Div., ASCE, Evanston, IL, 1964, Vol. 90, SM5, p. 555.
2. O. J. Porter. Discussion of: The Use of Soil Mechanics in the Design and Construction of Bridge Foundations. Proc., Annual Convention, Association of Highway Officials of North Atlantic States, 1953.
3. P. Keene. Non-Engineering Elements in Factors of Safety in Soil and Foundation Engineering. HRB, Highway Research Record 269, 1969, pp. 1-6.
4. Science Cuts Bridge Costs. Hartford Times, Dec. 21, 1966.
5. Savings Cited on Bridges With New Techniques. Hartford Courant, Dec. 22, 1966.
6. R. D. Goughnour. Soil Management Studies in the State Highway Departments. Federal Highway Administration, Highway Focus, Vol. 8, No. 4, Dec. 1976, pp. 1-10.

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Survey of Bridge Movements in the Western United States

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The design of bridge superstructures is usually based on the assumption that less than 25 mm (1 in) of movement will occur within the substructure. Consequently, the foundation engineer must find soils at a proposed site of sufficient bearing capacity to limit movement to this small value. Often the movement criteria will dictate the size or depth of the foundation, thereby increasing its cost needlessly if the criteria established are too strict. The purpose of this survey is to document field performance of various structures that have moved and obtain an evaluation as to the acceptability of these movements by various state highway agencies. The 35 structures reported in this paper include 54 movements of structural elements. Sixty percent of these represent movements at the abutments and 40 percent represent movements at piers. Although some very large vertical movements were reported tolerable for some of the older structures, movements in excess of 63 mm (2.5 in) within the structure were usually considered objectionable from a rideability viewpoint. Horizontal movements in excess of 50 mm (2 in) usually caused structural distress that was considered harmful. Recommendations are made to improve reporting procedure and make use of available material. Emphasis is on more thorough involvement between designers and maintenence crews in solving movement problems.

This survey attempts to establish criteria that would define, from a practical viewpoint, tolerable movement of highway structures. Advances in our prediction techniques for soil movements based on new sampling, testing, and analysis techniques have often been undermined

by unrealistic design tolerances in the superstructures. The often heard, "We cannot tolerate more than one inch [25 mm]," from the structural engineer imposes an upper limit on movement that, on many occasions, leads to an overly conservative foundation design or undesirable superstructure characteristics, such as joints, which cause poor riding and difficult maintenance. Of interest for the survey are (a) horizontal and vertical movements of piers and abutments, (b) the type of structure, (c) the construction sequence, (d) the effects of the movements on the structural elements, and (e) an evaluation by the reporting agency of whether the observed movements are considered tolerable.

LITERATURE REVIEW

The design of structures based on static and stress analysis concepts has led to the development of standards or guidelines often used in the design of foundations. A review of the failure criteria in some of our most commonly used pile-load test procedures, for example, reveals usually low allowable movements of the pile head before the load test is stopped or declared a failure.

In the Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials (AASHTO) (1), a load test is considered to be a failure if after the continuous application of twice the design load for 48 h the net settlement of the pile head after unloading exceeds 6.35 mm (0.25 in). The standards of the American Society for Testing and Materials (ASTM) for static load tests for the years prior to 1969 gave little guidance as to failure criteria and generally only described a rather slow testing procedure to twice the proposed design load.

The drawbacks of some of these standard testing procedures prompted some agencies to research for quicker testing methods and different failure criteria in the early 1960s. The development of a quick test method by the Texas highway department is described by Fuller and Hay (2) and more recently promoted in a user's package form by Butler and Hay in a Federal Highway Administration (FHWA) implementation package (3). In this package, it is recommended that the piles or drilled shafts be loaded to plunging failure. Interpretation of the results of these tests is then based on the intersection of two tangents to the load-settlement curve and the safe design load found by applying an appropriate safety factor. With this information and a good knowledge of the subsurface conditions, economical and safe foundations can be designed.

Some of this pile-load test research work has led to the establishment of new standards and failure criteria as published in ASTM D1143-74 (4). Under article 4.5, which describes the constant rate of penetration test, it is recommended that the load required be held to achieve the specified penetration rate until the total penetration is at least 15 percent of the average pile diameter or diagonal dimension. Article 4.7 describes briefly the quickload test without any discussion of failure criteria, and article 4.8, which describes the settlement controlled method, states that the test is complete once the total pile-butt settlement equals 10 percent of the average pile diameter or diagonal dimension.

Note that none of the above failure criteria takes into account the length of the pile tested or its material composition. More work is needed in this area to provide a more engineered approach to interpretation of pile-load tests. Also of concern (as will be seen from the results of the survey) should be the lateral forces that are imposed on the foundations by the structure and surrounding soil. Again, structural analysis can give us good estimates of the lateral loads imposed by the superstructure, but many unknowns still exist as to the lateral loads generated by lateral movements of consolidating soils, for instance.

Some case histories in the literature show that excessive moments can be introduced into pile foundations due to consolidation of an adjacent embankment. Such a case is described in the proceedings of the 5th International Soil Mechanics and Foundation Engineering (ISMFE) conference (5) and followed up in the 6th ISMFE conference proceedings by Heyman (6). In the proceedings of the 8th ISMFE conference (7), another case history describes measurements made on piles subjected to both negative skin friction and lateral soil pressures. It is stated that the degree of consolidation of the soft layer had practically no effect on the magnitude of the maximum negative skin-friction load measured, which reached 54 Mg (60 tons) for some piles. On the other hand, the degree of consolidation was of great importance in reducing the maximum movement occurring in the foundation piles.

An example of another case history that supports the above conclusions was reported by Nicu, Antes, and Kessler (8) on a structure in Allamuchy, New Jersey. There one of the abutments was monitored during construction and backfilling operations and compared to the other abutment, which was backfilled at a slower rate. Considerably less lateral movement of the abutment occurred after some consolidation of the soft layer. This demonstrated clearly the value of this method in reducing horizontal movements.

In 1972, Marche and LaCroix (9) published their analysis of the stability of abutments founded on piles driven through soft soil layers for 15 different structures. These structures included some of the above case histories and some reported by Stermac, Devata, and Selby (10). Their results describe the influence of the type of abutment on the direction of movement relative to the superstructure, the relative stiffness of the pile-soil system, and the degree of loading imposed by the embankment.

Generally, movements became large if the embankment loading produced an increase of stress larger than three times the undrained shear strength of that soil at the surface of the consolidating layer. Documented failure occurred when the loading approached 5.14 times the undrained shear strength. Recommended solutions to this problem consist principally of either reduction of the load at the abutment by the use of lightweight fill or preconsolidation of the soft layer. Alternatively, from a structural standpoint, designing the structure to accommodate the anticipated movement may, in some cases, be more economical than to try and prevent it.

FIELD SURVEY

In September 1975 a survey questionnaire was sent to the highway departments of the 17 states shown in Figure 1. The information requested was to be completed on a prepared form from case histories documented in the agency's files.

As a guideline, the following definition of intolerable movement was given in the questionnaire: Movement is not tolerable if damage requires costly maintenance or repairs and a more expensive construction to avoid this would be preferable. Ten states gave information on a total of 35 structures; six states replied that this information was not documented in their files.

A list of the reported movements is given in Table 1. These represent movements of 54 structural elements, abutments, or piers in 35 highway bridges. These movements were divided into three classifications: (a) toler-

Figure 1. States surveyed.

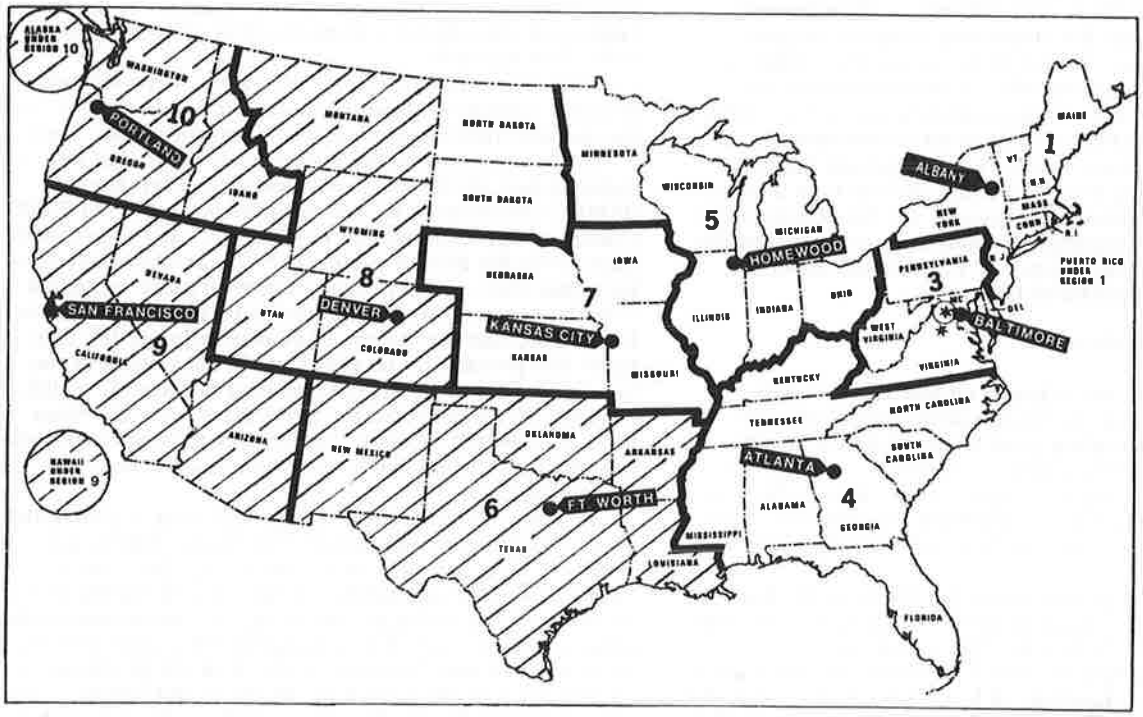

Table 1. Movements of structural elements reported.

	Simple Span				Continuous			
	Abutments (mm)		Piers (mm)		Abutments (mm)		Piers (mm)	
Movement	Vertical	Horizontal	Vertical	Horizontal	Vertical	Horizontal	Vertical	Horizontal
Tolerable								
1 Direction	125, 50, 25 450, 25	38	63,[a] 150 63	50	125, 50		75 to 88, 88, 25, 38, 38 13	25
2 Directions	25	50			13, 75	25, 25		
Intolerable, poor riding								
1 Direction	225, 225 300				200		63, 150	
2 Directions	125	25			450	87		
Intolerable, structural damage								
1 Direction		75	50,[a] 350, 75 to 125		50 to 125, 25 to 50, 150, 200 to 250	50, 50, 75 163	50 to 75[a]	
2 Directions	600, 50 400, 425	13, 75, 25, 63	600,[a] 100 225	150, 25, 25	63, 38	200, 50	13	25 to 88

Note: 1 mm = 0.04 in.

[a]Vertical heave.

able movements, (b) intolerable movements but only poor riding characteristics were cited, and (c) intolerable movements that resulted in structural damage. Each of these classifications was subdivided into one-direction and two-direction movements to indicate structural elements for which only vertical or horizontal movement occurred and others where both occurred. When both occurred, each structural element is represented by two numbers, one in the vertical column and one in the horizontal column, and listed in sequential order in each column. When differential movement was reported, the range is shown. Not all vertical movements were down, but significant heaves are also included in this table. Most of these unusual movements were due to frost or ice action on the foundations.

A review of the data shows a surprising range of values in each classification. Tolerable movements were reported for 11 abutments and 12 piers. Vertical displacements in this group varied from 13 to 450 mm (0.5 to 18 in) and horizontal movements were limited to a maximum of 50 mm (2 in). Much less movement was tolerable when both occurred together and the maximum reported was for an abutment that settled 75 mm (3 in) and moved horizontally 25 mm (1 in).

The second classification—intolerable movements, poor riding—was chosen in an attempt to define a consumer-related measure of tolerability. Eight elements are in this category; six were vertical settle-

Figure 2. Condition items of FHWA survey inventory and appraisal sheet.

CONDITION	MATERIAL	CONDITION ANALYSIS	RATING (9-0)
58 Deck			
59 Superstructure			
60 Substructure			
61 Channel & Channel Protection			
62 Culvert & Retaining Walls			
63 Estimated Remaining Life		65 Approach Roadway Alignment	
64 Operating Rating		66 Inventory Rating	

Figure 3. Structure condition rating system.

Recording and Coding Guide - July 1972		Commentary on July 1972 Coding Guide		BIT Course Coding	
		Unabridged	Abridged		
9	New condition	New condition	New condition	Good	The item is in new or good condition with no repairs necessary
8	Good condition - no repair necessary	No repair necessary. No sign of distress or deterioration	Good condition - no repair necessary		
7	Minor items in need of repair by maintenance forces	A defective or deteriorated secondary member, that will not progress to a serious defect if not repaired within a reasonable period of time. Most preventative maintenance is in this category.	Minor items in need of repair	Fair	The item is still performing the function for which it was intended. A minor or major item is in need of minor repair
6	Major items in need of repair by maintenance forces	A defective or deteriorated main supporting member or support system vital to the structural integrity of the bridge. Includes any progressive deterioration that can be arrested by maintenance repairs such as concrete cracking that can lead to rebar corrosion and steel cracks and corrosion that can lead to possible failure.	Major items in need of repair		
5	Major repair - contract needs to be let	Same as 6 except that the extent of deterioration is greater and repair could require complicated and/or extensive procedures.	Major rehabilitation needed	Poor	The item is still performing the function for which it was intended but at a minimum level. The item concerned is in need of major repair
4	Minimum adequate to tolerate present traffic, immediate rehabilitation necessary to keep open	The structure can marginally support loads from unrestricted legal load and posting should be considered. Continued observation indicates that failure is not progressive under restricted loading. This rating is relative to the class of loading using the bridge. This rating applies only to major components or elements.	Marginally adequate to tolerate unrestricted legal loads - consider restricting traffic and/or posting for less than maximum legal loads		
3	Inadequacy to tolerate present heavy load - warrants closing bridge to trucks	Major structural element deteriorated or damaged so as to reduce its capability of carrying trucks. Allow light loads only if stress check warrants and continued observation indicates failure is not progressive under these light loads.	Inadequate to tolerate legal loads. Post for light loads		
2	Inadequacy to tolerate any live load - warrants closing bridge to all traffic	Major structural element deteriorated or damaged so as to reduce its capability of carrying any loads. Stress check indicates structure cannot support any live load. Bridge should be closed.	Inadequate to tolerate any live load. Warrants closing bridge to all traffic	Critical	The item is not performing the function for which it was intended
1	Bridge repairable, if desirable to reopen to traffic	Bridge closed. Inspection indicates that bridge can be reopened with a complete rehabilitation.	Bridge closed. Complete rehabilitation needed to reopen		
0	Bridge conditions beyond repair - danger of immediate collapse	Bridge closed. Inspection indicates that bridge conditions are beyond repair and in danger of immediate collapse. Keep bridge closed.	Bridge closed. Bridge conditions beyond repair. Danger of immediate collapse.		

ments only, which varied from 63 to 300 mm (2.5 to 12 in). The other two are abutments where two-directional movements reached a maximum of 450 mm (18 in) vertically and 87 mm (3.5 in) horizontally. However, of more significance is the lowest value of vertical settlement reported intolerable from a rideability viewpoint. In this case it is 63 mm. This is a reasonable value because more than half of the tolerable vertical movements are at or below this displacement.

The third classification includes the remaining 23 structural elements reported. For each of these, some form of structural damage occurred as a result of the movements. In this group, vertical displacements varied from 13 to 600 mm (0.5 to 24 in), including some cases of vertical heave that equaled the maximum value reported. One-half of the displacements in this classification involved two directions. Horizontal movements were in many cases the cause of the damage to the structure. These ranged from 25 to 200 mm (1 to 8 in). Most of them were 50 mm (2 in) or more. This last value appears to be an upper value of tolerability, especially when movement is toward the deck and reduces the clearance for temperature expansion.

A review of the possible influence of the construction sequence on the movement of the abutments showed that 21 of the 27 abutments for which the construction sequence was known were built by first building the embankment, then the substructure, and then the superstructure. Of these movements, 10 were vertical only, 5 were horizontal only, and 6 were a combination of both.

The majority of the embankments were between a height of 3.6 and 7.1 m (12 to 30 ft) but otherwise varied from a low of 1.2 m (4 ft) to a high of 21.3 m (70 ft).

A review of the boring information made available for many of the structures showed that at nearly every site a layer of loose fine sand, sandy silt, or silty clay existed beneath the bridge approaches prior to construction. Only three sites described soft clays from which long-term consolidation problems could be anticipated.

It appears that the possibility of consolidation was recognized because 23 of the 32 abutments were on pile foundations. However, only eight movements were considered tolerable, so the piles were probably underdesigned for either the lateral loads or negative skin friction loads applied to them by the embankment. Unfortunately, it is not known how many construction projects included waiting periods between the bridge approach construction and bridge foundation construction. It seems that in most of these cases it was insufficient.

Following the receipt of these data and looking for a way to expand the number of case histories submitted, I became aware of the Structure Inventory and Appraisal of the Nation's Bridges program. This inventory was initiated by FHWA in 1972 in order that an accurate report might be made to Congress on the number and condition of the nation's bridges. Each state was requested to inventory all the bridges carrying and going over federal-aid highways. However, complete inspection was not mandatory. In July 1972 a recording and coding guide (11) was provided to the states as an example of the data base needed for the final report. Clarification of some items in the recording guide was made in July 1977 (12) as part of this continuing program. The data collected for each structure are summarized and reported to the FHWA in a structure inventory and appraisal (SI&A) sheet.

An extract of the SI&A sheet, which rates the condition of the structure, is shown in Figure 2. The rating system used for items 58 through 62, plus item 65, is shown in Figure 3. As can be seen from Table 1, this type of rating system does not allow the reviewer to determine if any down rating of the substructure (item 60) is due to movement or to other causes. This is unfortunate, because this type of information would be of interest to the foundation engineer for future designs.

RECOMMENDATIONS

Although this survey was limited in scope, it points out some important facts about the difficulties that exist in determining tolerable movements for various structures designed. Accurate movement data are difficult to obtain. Some agencies claim that this information is not kept in their files or are unwilling to part with this information. Some states, on the other hand, have quantities of data in their maintenance files but, because the bridge is complete from a construction point of view, design information must be obtained from one or more other offices within the agency.

Another problem is the reliance that one has to make on the memory of a few individuals to locate case histories of structures with movement. The ones that are remembered are usually only those structures that have had major movements, which caused some kind of structural distress. It would appear beneficial to develop a classification and retrieval method so that the designer can evaluate the performance of previous designs. In this age of rapidly rising construction costs it is important that geotechnical engineers and designers have all the information needed to design a safe and economical foundation without costly overdesign to take care of uncertainties.

From the values reported in the survey, horizontal movements were the most critical. Structural distress was usually reported if 50 mm (2 in) or more occurred. Vertical movements were reported tolerable up to some very high values but poor riding characteristics were mentioned once the settlement exceeded 63 mm (2.5 in) within the structure or approaches, for high-design roadways. Detrimental movements of smaller magnitude were reported when both horizontal and vertical displacements occurred simultaneously. These values appear to be reasonable limits for many structures as tolerable movements.

Since a large percentage of the abutment movements reported were founded on piles installed through in-place embankments, a closer look at probable embankment movements should be made during the investigation and design stage of the bridge approaches. Very little settlement of the embankment relative to the pile is needed to impose large negative friction loads and bending moments on the piles.

When the abutments are founded on piles installed through a thick layer of soft deposits, the geometry of the abutment will have an influence on the direction of the horizontal movements. Appropriate design features should be incorporated in the design to minimize damage to these structural elements.

Geotechnical engineers and designers should be involved early in monitoring of movements detected by bridge inspections. Field instrumentation installed at an early stage to detect direction and depth of movement provides invaluable information to the engineer for the design of correction schemes if necessary.

ACKNOWLEDGMENT

I wish to express appreciation to the agencies and their engineers who provided the data presented here. Their time and efforts made the preparation of this paper possible.

REFERENCES

1. Standard Specifications for Highway Bridges—with current revisions. AASHTO, 11th ed., Washington, 1973.
2. F. M. Fuller and H. E. Hay. Pile Load Tests Including Quick-Load Test Method, Conventional Methods, and Interpretations. HRB, Highway Research Record 333, 1970, pp. 74-86.
3. H. D. Butler and H. E. Hay. The Texas Quick-Load Method for Foundation Load Testing—User's Manual. Federal Highway Administration Implementation Package 77-8, March 1977.
4. Annual Book of ASTM Standards. ASTM, Philadelphia, 1975, pp. 178-189.
5. L. Heyman and L. Boersma. Bending Movements in Piles Due to Lateral Earth Pressure. Proc., 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, 1961, pp. 425-429.
6. L. Heyman. Measurement of the Influence of Lateral Earth Pressure on Pile Foundations. Proc., 6th International Conference on Soil Mechanics and Foundation Engineering, 1965, pp. 257-260.
7. W. T. Heijnen and P. Lubking. Lateral Soil Pressure and Negative Skin Friction on Piles. Proc., 8th International Conference on Soil Mechanics and Foundation Engineering, 1973, pp. 143-147.
8. N. D. Nicu, D. R. Antes, and R. S. Kessler. Field Measurements on Instrumented Piles Under an Overpass Abutment. HRB, Highway Research Record 354, 1971, pp. 90-99.
9. R. Marche and Y. LaCroix. Stabilité des Culées de Pontes Etablies Sur de Pieux Traversant une Couche Molle. Canadian Geotechnical Journal, Vol. 9, No. 1, 1972, pp. 1-24.
10. A. G. Stermac, M. Devata, and C. G. Selby. Unusual Movements of Abutments Supported on End-Bearing Piles. Canadian Geotechnical Journal, Vol. 5, No. 2, 1968, pp. 69-79.
11. Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges. Federal Highway Administration, Transmittal 123, Vol. 20, Appendix 54, July 1972.
12. Commentary on Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, July 1972. Federal Highway Administration, April 1977.

Discussion

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This paper, as well as the one by Grover in this Record, is an excellent presentation that provides an interesting account of the variability in the movement of piers, abutments, and approach slabs and in the relative effects of the magnitude of vertical and horizontal movements on ridability.

The effect of these movements on the bridge superstructure is of real concern to structural engineers. Differential vertical movements of piers and abutments may induce large support reactions and internal stresses. Differential horizontal movements require expansion supporting devices as well as deck expansion devices. The failure of these devices to function as intended in their design requires consideration of induced stresses. Many cases of abutment movement have resulted from the growth of approach slabs, the compaction or settlement of the approach fill, or the freezing of support or expansion devices.

The approaches of a number of bridges in California have been studied by Jones (13), who reported that more approach patching is required for closed-abutment bridges than for open-end structures. Examples of typical approach slab action and approach slab-pavement interaction are given by Strom (14).

A study of the annual movement of approximately 231 expansion joints (80 structures) in various climatic environments was made over a 3-year period by Stewart (15). His conclusions include (a) structures with regular expansion facilities at the abutments will have a higher average movement per unit length than will those that have no abutment expansion facilities; the larger the number of expansion joints in a structure, the less significant is abutment expansion; and (b) uniformly spaced expansion joints on long structures do not necessarily have the same movement.

In an effort to eliminate the settlement of approach slabs and the excessive movement of abutments (conditions that frequently result in broken abutment backwalls or the closing of expansion devices), many states have experimented with granular backfills, predrilling for piling, and stub abutments (16).

The general failure of expansion supporting devices to function as intended becomes apparent when they are observed in the field. During the summer of 1961, I conducted field observations of the behavior of bridge supporting and expansion devices on bridges located in Iowa, Kansas, and Nebraska (16). General limitations for observation were that the bridges should have three or more spans, either simply supported or continuous. The span lengths were usually 15.2 m (50 ft) or more. Of the 83 bridges tabulated, 39 had irregularities. Many of them had floor expansion devices that either had closed tight or had much less provision for further expansion than designed for at the observed temperature. Shifting of abutments, spalling and cracking of abutment backwalls, inconsistent rocker movement, cracked concrete bearing seats, and extrusion of asphaltic expansion joints were common observed irregularities. However, there appeared to be no direct relation between abutment movement and the age of the structure, type of approaches, or type of supporting or expansion devices. From the bridges observed, no definite trend or regularity of pattern could be isolated nor could a prediction be made as to which irregularity will occur or when it will occur. For example, both closed and open floor expansion devices were observed on bridges with concrete, asphalt, and gravel approaches. Similarly, evidence of abutment movement and irrational rocker movement was observed for all types of approaches and heights of approach fill. There were irregularities in both old and relatively new bridges.

To eliminate the problems of expansive supporting devices, one state highway department (circa 1950) began tying the superstructure to flexible piers and stub abutments. In 1934, a three-span, 57.9-m (190-ft), continuous steel beam bridge was constructed that utilized this type of construction. A single row of vertical piling with a concrete cap was used at the ends of the bridge, and a separate retaining wall was spaced 1.9 cm (0.75 in) at 27°C (80°F) from the end of the bridge. The concrete center piers rested on two vertical rows of piling. No roadway expansion device other than mastic was used. The bridge, which had had no maintenance except an occasional painting of structural members, had been subject to almost yearly flooding, and had supported a great

increase in traffic over the years, was in very good condition when observed in 1961 (16). Interest in and use of superstructures tied to flexible piers and stub abutments has continued to grow and is essentially limited only by the lack of suitable design criteria (17, 18, 19, 20).

Preboring for abutment piling is now commonly used. The paper reports that the possibility of consolidation apparently was recognized because 23 of the 32 (reported) abutments are on pile foundations. It is unfortunate that the respondents did not identify whether or not the pilings were prebored so that a comparison of abutment movement could be made. It also would be interesting to know whether the abutments are rigid high abutments or the spill-through type. It is believed that the influence of preboring, type of abutment, and superstructure-substructure interaction (connection), if known, would be reflected in modifications to Walkinshaw's recommendations.

In structural considerations, the effects of differential vertical movement and the restraint of horizontal movement should be compared and interrelated to thermally induced stresses [which may be significant (21, 22, 23)] in addition to the live-load stresses. Such analyses can be readily calculated by any highway bridge design group.

REFERENCES

13. C. W. Jones. Smoother Bridge Approaches. Civil Engineering, American Society of Civil Engineers, Vol. 29, No. 6, June 1959, pp. 407-409.
14. J. Strom. Maintenance of Bridge Expansion Joints. Univ. of Arkansas Engineering Experiment Station Bulletin 24, March 1957.
15. C. F. Stewart. Annual Movement Study of Bridge Deck Expansion Joints. California Division of Highways, R&D Rept. 2-69, June 1969.
16. J. H. Emanuel and C. E. Ekberg, Jr. Problems of Bridge Supporting and Expansion Devices and an Experimental Comparison of the Dynamic Behavior of Rigid and Elastomeric Bearings. Iowa Engineering Experiment Station, Iowa State Univ., Special Rept., Proj. 547-S, Sept. 1965.
17. C. E. Ekberg, Jr., and J. H. Emanuel. Current Design Practice for Bridge Bearings and Expansion Devices. Engineering Research Institute, Iowa State Univ., Proj. 547-S, 1967.
18. J. H. Emanuel and others. An Investigation of Design Criteria for Stresses Induced by Semi-Integral End Bends: Phase I—Feasibility Study. Missouri Cooperative Highway Research Program, Univ. of Missouri, Rolla, Final Rept. 72-9, 1973.
19. J. H. Emanuel and others. Current Design Practice for Bridge Superstructures Connected to Flexible Substructures. Department of Civil Engineering, Univ. of Missouri, Rolla, Civil Engineering Study 73-3, Structural Series, 1973.
20. J. D. Reynolds and J. H. Emanuel. Thermal Stresses and Movements in Bridges. Journal of the Structural Division, ASCE, Vol. 100, No. ST1, Proc. Paper 10275, Jan. 1974, pp. 63-78.
21. J. H. Emanuel and J. L. Hulsey. Thermal Stresses and Deformations in Nonprismatic Indeterminate Composite Bridges. TRB, Transportation Research Record 607, 1976, pp. 4-6.
22. J. H. Emanuel and D. J. Wisch. Thermal Stresses Induced in a Composite Model Bridge Structure. Missouri Cooperative Highway Research Program, Univ. of Missouri, Rolla, Study 75-2, Final Rept., 1977.
23. J. H. Emanuel and J. L. Hulsey. Temperature Distributions in Composite Bridges. Journal of the Structural Division, ASCE, Vol. 104, No. ST1, Proc. Paper 13474, Jan. 1978, pp. 65-78.

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Movements of Bridge Abutments and Settlements of Approach Pavements in Ohio

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The Ohio Department of Transportation experienced intolerable movements of bridge abutments and adjacent approach slabs within a short time after construction of a new highway facility. Two accurate field surveys measured the extent and magnitude of the settlements against the data on embankment heights, subsoil conditions, type of abutment design, and other conditions. The first study, in 1961, indicated that 90 percent of the surveyed bridge abutments had settlements of 10 cm (4 in) or less and only 20 percent of the settlements were 2.5 cm (1.0 in) or less. In most cases, the abutments were supported on spread footings, without piles, in the approach embankment. Major revisions were incorporated in the design and construction specifications. The two most important ones were the use of piles at the abutments and the increase of compaction requirements for the embankments. The second survey was conducted in 1975 to evaluate the revised policies. These data indicated that 70 percent of the surveyed bridge abutments had no measurable settlements and 20 percent had minor settlements even though supported by piling. Generally, the measured settlements were within tolerable limits. In 1961, the average approach slab settlement was 6.5 cm (2.5 in) and in 1975, the average approach slab settlement was 5.0 cm (2.0 in).

By the end of 1960, the Ohio Department of Transportation had completed a large portion of their Interstate highway system as well as the construction of other major highway facilities. This construction program included hundreds of kilometers of new highways and hundreds of new bridges. Over 90 percent of the bridges were steel beams or girders of multiple spans with continuity over the piers, as shown in Figure 1.

Within 1 to 3 years after construction intolerable movements (vertical settlements and horizontal displace-