Tolerable Movements of Bridge Foundations

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As many aspects of bridge engineering knowledge expand, it is desirable to examine the question of tolerable movements of bridge foundations. It is a subject that has had comparatively little study in the past; this lack has sometimes produced unrealistic results, especially in costly overdesigns. A nationwide investigation of the matter has found differences in definitions and practices. The definition of tolerable movement of highway structure foundations is often complicated. It involves amount of movement, type of structure, effect on each part of structure, cost of alternative choices, effect on traveling public, subjective reasons, and apprehension during design. Case histories illustrate that large movements that cause some harm should be called tolerable if alternative choices are excessively costly or more undersirable. Small movements that result in substantial costs for remedial work should be called intolerable if the movements could have been easily avoided. In addition to the structural analyses to determine the effects of prediction foundation movements, the geotechnical work on which the predicted movements are based should in many cases be improved. Recommendations are given to aid in improving the geotechnical work.

In engineering literature, there have been many publications on tolerable movements of buildings, such as those listed in the bibliography of Feld's paper (1), but few publications, if any, on tolerable movements of highway bridges and other structures. This lack of information on highway structures spurred a 3-year survey by a Transportation Research Board subcommittee of the Committee on Foundations of Bridges and Other Structures. Interest in this aspect of bridge behavior is in keeping with the progressive developments in many other aspects of bridges, such as prestressed concrete girders, orthotropic superstructures, integral abutments, weathering steel, and cantilevered end spans, as well as the use of computers in stress analysis and other calculations.

Movements of buildings are a more serious problem than movements of highway structures. Buildings are usually more sensitive because of glass panels, doors, windows, elevators, and utility configurations, whose performance could be affected by stresses caused by movements. The load on the foundation is added gradually during the orderly process of construction. By contrast, highway bridges are generally constructed more easily and, where necessary, construction can be done in stages. Approach embankments (often the chief cause of movements) and the substructures can be built and allowed to move for several months before starting the superstructure, the most delicate part of the bridge. In the case of box culverts, the structure is rugged and can have expansion joints at frequent intervals; differential movements and minor cracks sometimes are never known by anyone except occasional maintenance workers.

DEFINITION OF TOLERABLE MOVEMENT

The two chief considerations in bridge design and construction are safety and economy. Tolerable movements of highway structures, because the concern is small movements, are seldom concerned with safety. Safety involves large movements that lead to structural failure and collapse of the structure or a major part of it. Tolerable movements, then, are concerned mainly with economy; that is, can the movements be tolerated in order to save expense in money or time or should a more costly design and construction be used in order to reduce or eliminate the movements? Costs for the more economical choice may include temporary overload of approach embankments, proper sequence of operations, and observation of movements during construction. Also, later maintenance may require adjustments or repairs to bearing plates, expansion rockers, bridge railing, bridge deck, and approach pavements and traffic detours during the repair work. Costs for the more expensive choice may include additional spans, piles, or a more expensive site. The selection of the best choice depends on many factors, which may vary with each structure. To be avoided is a wasteful choice; to quote the late O. J. Porter, the internationally known soils and foundations engineer (2, p. 139), "While we have had many mistakes due to inadequate foundations, we have also had many buried treasures of money due to using an expensive pile foundation where spread footings could be safely used."

The incentives to overdesign are probably well known. They include: (a) easier analyses and hence cheaper design payroll costs; (b) a safer product that causes no worries; and (c) construction costs that are borne by another group, the taxpayers. Also, if a consulting firm is paid for the design on a percentage of the construction cost (no longer commonly done), there might be a tendency to overdesign to increase the fee. It can be added that another incentive for conservatism may be in lack of confidence in the design geotechnical work or in the supervision and inspection during construction.

An example of the savings realized when a wasteful choice is rejected is the twin bridges carrying I-91 over Silas Deane Highway in Connecticut (3), built in 1961 (Figure 1). The design consultant wanted piles under all footings. His soils engineer predicted large settlements without piles, based on superficial investigations and unrealistic analyses. The consultant had been caught in controversy on two previous projects in a neighboring state (one concerned an incinerator near a swimming pool and the other concerned a low-cost housing development built partly on new fill). On neither project was he in error, but his association with them was very unpleasant and it motivated his conservative solutions.

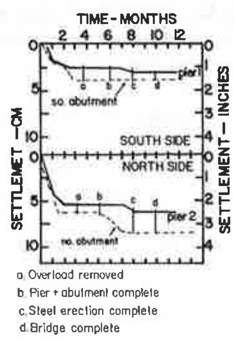
On the Connecticut Interstate project, after some detailed supplemental investigations at the site by the state's soils engineers, the state ordered the omission of all piles and the use of a 1.5-m (5-ft) overload on the approach embankments with a 3-month waiting period prior to the construction of any substructure units. The results for the northbound bridge are shown in Figure 2. Results for the southbound bridge are similar. Settlements of the superstructure were insignificant. The same procedure was ordered at two other pairs of bridges on this project, with results almost identical to those for the Silas Deane bridges. The net savings for the three pairs of bridges was $250 000, a large amount in 1961. The above are included in the savings of over $4 million realized on 33 Connecticut bridges, described at a bridge conference in 1966 (4, 5).

The definition of tolerable movements depends on several factors that vary in degree and importance with conditions at each structure. In special cases, movements might be called harmful but tolerable because the alternative choices are more undesirable. The chief factors in defining tolerable movements are

1. Amount of movement—Bozozuk suggests in a paper in this Record that 10 cm (4 in) of vertical or 5 cm (2 in) of horizontal movement is the limit of tolerability. Obviously these limits are too low for some cases and too high for others.
2. Type of structure—This has many varieties due to the many types of structures, span lengths, and skew angles. For example, a bridge with short continuous spans and deep girders will tolerate less differential settlement than one with long continuous spans and shallow girders. Connecticut made a table showing theoretical stress increase in girders for two-span continuous bridges of various span lengths, girder depths, and differential settlements (Table 1). This table was given to those design consultants who wanted zero settlement. Another feature to be remembered is that a bridge with a severe skew is more affected by horizontal movements than one that has no skew.
3. Effect on each part of structure—A common example is an abutment and wing walls built and backfilled as an early operation and allowed to settle. After a waiting period, the superstructure is erected. Here the substructure could tolerate large settlements, but the superstructure could not.
4. Cost of alternative choices—Frequently there is a question between various designs over the matter of movements of the structure. One choice of design will result in appreciable movements and consequent maintenance of the structure and another choice will reduce or eliminate the movements but be more costly. If the former choice will give only moderately undesirable results and the latter is very expensive, it is obviously a waste of money to adopt the latter. Movements involved in the former are tolerable and should be lived with, and the reasons made clear to all concerned.
5. Effect on traveling public—This should always be kept in mind, as the project is built to serve the public. Examples of adverse effects are ugly cracks in concrete at conspicuous locations; poor rideability, especially at high speed; and inconvenience to traffic during maintenance on high-capacity facilities.
6. Subjective reasons—Often the designers do not wish to make detailed studies and calculations where movements might occur. This may be due to inertia or to avoid trouble. Consultants may also wish to reduce their design payroll costs or increase their fee payments by using the easier, more expensive design.
7. Apprehensions during design—Where the design engineer does not trust the accuracy of the soils engineer's predictions of probable movements or fears poor construction operations, he or she may label the predicted movements intolerable. Actually, the fear is that the movements may be much greater and hence intolerable. It may be added that settlement predictions that have a 25 percent variance from the actual settlement are good and predictions having a 50 percent variance are fair if conditions are not complicated. Predictions of horizontal movements are more difficult.

Figure 1. I-91 over Silas Deane Highway.

Figure 2. Foundation settlement: I-91 northbound over Silas Deane Highway.



Note: 1 cm = 0.39 in.

Table 1. Two-span continuous bridges: increase in fiber stress due to 5-cm (2-in) differential settlement.

Differential Settlement	Increase in Fiber Stress Due to 5-cm Differential Settlement (kPa)			
	0.9 m Girders		1.2 m Girders	
	At Pier	At Mid-Span	At Pier	At Mid-Span
Two 38-m spans				
5 cm at pier	9 900	4800	13 100	6 600
5 cm at abut	5 000	2500	6 600	3 300
Two 27-m spans				
5 cm at pier	19 300	9700	25 600	12 700
5 cm at abut	9 700	4800	12 800	6 300

Notes: 1 m = 3.3 ft; 1 cm = 0.39 in.
Girders have constant moment of inertia.

SURVEYS AND CASE HISTORIES

Thirty-five states and Canadian provinces responded to the questionnaire issued by the TRB subcommittee. Ohio's contribution was its massive 1961 survey of 1525 bridges and its 1975 follow-up on 79 bridges; these are the subject of a paper in this Record by Grover of the Bureau of Bridges, Ohio Department of Transportation. The 19 reporting states west of the Mississippi River are covered by Walkinshaw's paper in this Record.

The remaining 14 reporting states east of the Mississippi and the Canadian provinces cover a total of 42 bridges. About one-half of these had intolerable movements. About two-thirds had simple spans and one-third had continuous spans. The embankment at one or both abutments was nearly always the prime cause of intolerable movements. Such movements were due to causes such as settlement of embankment, with drag down if piles were used, lateral pressure against abutment (and piles), and lateral movement of adjacent pier due to substantial lateral movement of the foundation soil.

However, remember that some bridges labeled with intolerable movements may have had harmful movements that actually were tolerable. As stated previously, these cases would exist where alternative choices of design and construction would be excessively costly in money

Figure 3. Profile of centerline of CT-15 Expressway over Silver Lake.

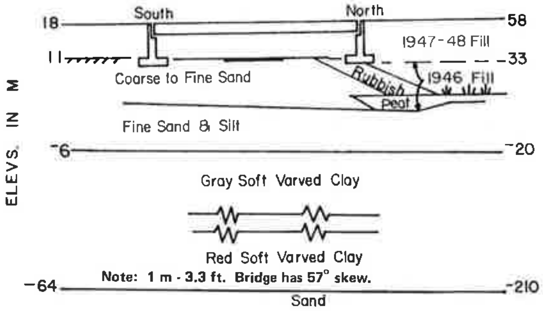


Figure 4. Shims under girder base plate.

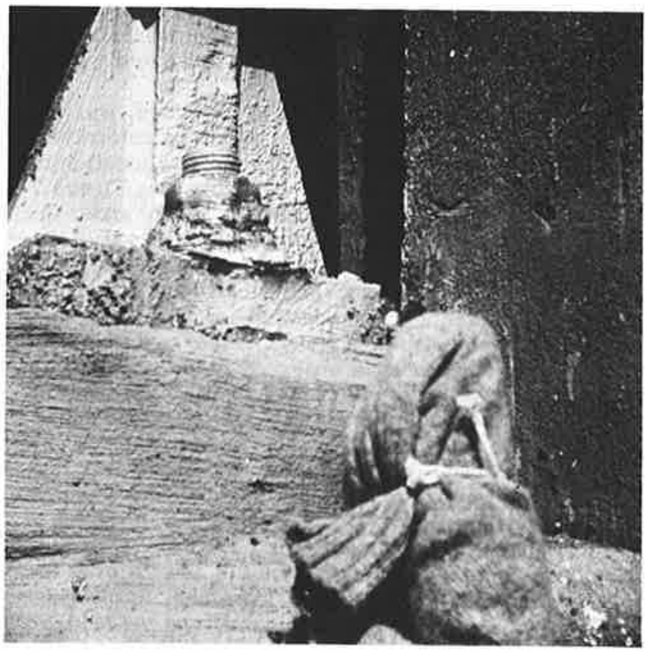


Figure 5. Tilt of rocker due to span shortening.

or time, very disruptive to neighboring property or local street systems, or highly displeasing aesthetically. In such cases the movements are harmful, but must be tolerated.

Of the 45 structures reported by Connecticut, 7 of the more significant ones will be described. The first 4 of these and the last 1 had rather harmful movements but are tolerable because of unusual factors. The other

Figure 6. I-84 over Hockanum River.

Figure 7. Conventional versus modern foundation treatment.

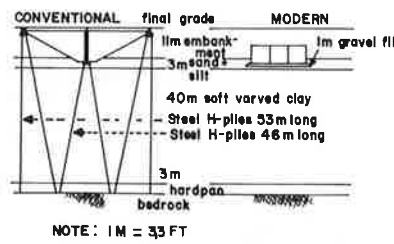


Figure 8. Profile of CT-2 Expressway over Willow Brook and Willow Street Extension.

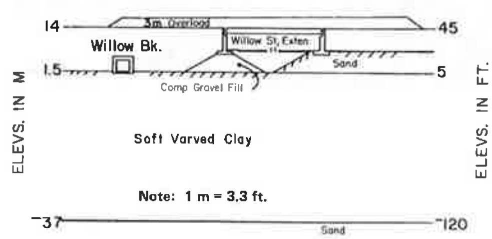


Figure 9. CT-15 Expressway over Folly Brook Boulevard.

2 should be considered not tolerable.

CT-15 Expressway over Silver Lane, East Hartford, was built in 1946 to 1948. The rubbish and peat shown in Figure 3 at the north abutment were removed and replaced by compacted gravel fill and the lower half of the embankment was placed. One year later the bridge was built and the final 7 m (24 ft) of embankment was placed. Settlement of the substructure before erecting the girders was 10 to 15 cm (4 to 6 in) in the north abutment and 2.5 cm (1 in) in the south abutment. Shims were placed under the north ends of the girders to bring them up to plan elevation. Figure 4 shows the largest shimming. A survey made 24 years later showed that the north ends of the girders had settled an additional 23 to 36 cm (9 to 14 in) and the south had settled 15 to 23 cm (6 to 9 in). Also, the span has lengthened 4 cm (1.5 in) at its east end and shortened 1 cm (0.5 in) at its west end (Figure 5). The abutments have a few hairline cracks. The greater vertical and horizontal movements were at the east end because of much more fill at the obtuse (skew) angle of the north abutment and northeast wing wall and the 16-m (54-ft) fill behind the north abutment. Short piles would have been ineffective to stop or reduce movements and long piles, over 76 m (250 ft) long, driven through the clay would have been extremely costly. Also, the omission of piles allowed the abutments to settle with the approach fill and avoid a bump at each end of the bridge deck.

I-84 over Hockanum River, East Hartford, was built in 1963 (Figure 6). As the river was to be relocated, the embankment was placed across the new location to grade and allowed to settle for 6 months (see Figure 7). At that time, settlements were 36 cm (14 in) under the expressway and 18 cm (7 in) under the midslope of the embankment. Then excavation was made for the triple-box culvert, and it was built and backfilled. The box settled 20 cm (8 in) under the pavement and 10 cm (4 in) at the ends. These settlements were 20 percent greater than predicted. No distress is observed in the structure. Piles would have had a net cost of $200 000 at that time.

CT-2 Expressway over Willow St. Extension and Willow Brook, East Hartford, was built in 1959 to 1960 (Figure 8). First, the Willow Brook box culvert, 192 m (630 ft) long, was built. Then the embankment was placed across the entire site to final grade plus 3 m (10 ft) overload and allowed to settle for 6 months. This produced settlements of 33 and 25 cm (13 and 10 in) at the north and south bridge abutment locations, respectively. Finally the bridge site was excavated, and the bridge was built and backfilled. The north and south abutments have settled 28 and 20 cm (11 and 8 in), respectively, with only a few hairline cracks in them. The box culvert settled 51 cm (20 in) under the expressway centerline and 20 cm (8 in) at 61 m (200 ft) left and right. It had no damage except faulting of 1 to 2 cm (0.5 to 1 in) at several expansion joints. The settlements were about 25 percent greater than predicted. The net saving by the omission of piles was $420 000 at that time.

CT-15 Expressway over Folly Brook Boulevard, Wethersfield, was built in 1941. This is a two-span continuous rigid-frame bridge with a 45° skew (Figure 9). The pier is on piles that straddle an old brick sewer. The abutments are on 2 m (7 ft) of desiccated clay overlying 7 m (23 ft) of soft varved clay. Piles for the abutments were deleted at the start of construction due to the steel shortage during World War II. Four months after the bridge was completed and approach embankments were placed, each abutment had settled 2.5 cm (1 in) at the end that had an acute angle with the wing wall and 4 cm (1.5 in) at the end that had an obtuse angle. Thrity-five years later, settlements were 4 and 5 to 8 cm (1.5 and 2 to 3 in), respectively. These are about 35 percent greater than predicted. These large differential settlements of the continuous frames would result in a substantial increase in steel fiber stress, but fortunately the settlements have occurred over a 35-year period. Consequently, plastic flow in the steel girders relieves much of the effect of the differential movement. The abutment walls have opened 1 cm (0.5 in) or less at expansion joints and each

Figure 10. Charter Oak Bridge over Connecticut River.

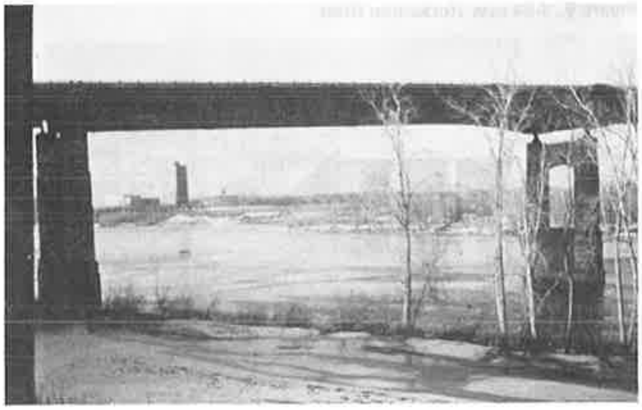


Figure 11. CT-185 bridge over Farmington River.

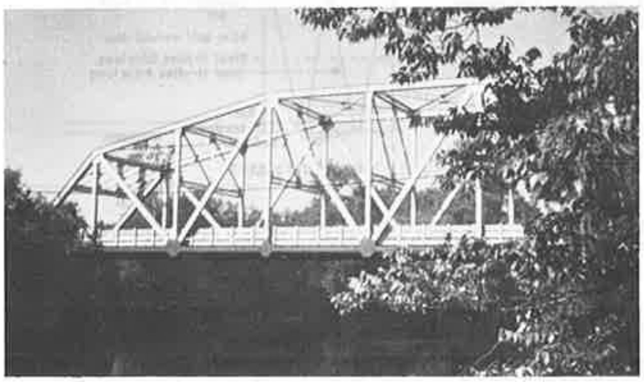
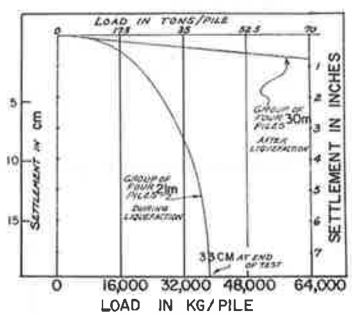
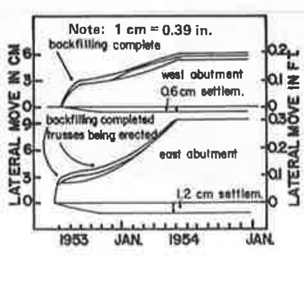


Figure 12. Pile-load test results at bridge over Farmington River.



Notes: 1 cm = 0.39 in, 1 kg = 0.001 ton.

Figure 13. Lateral and vertical movements of abutments.



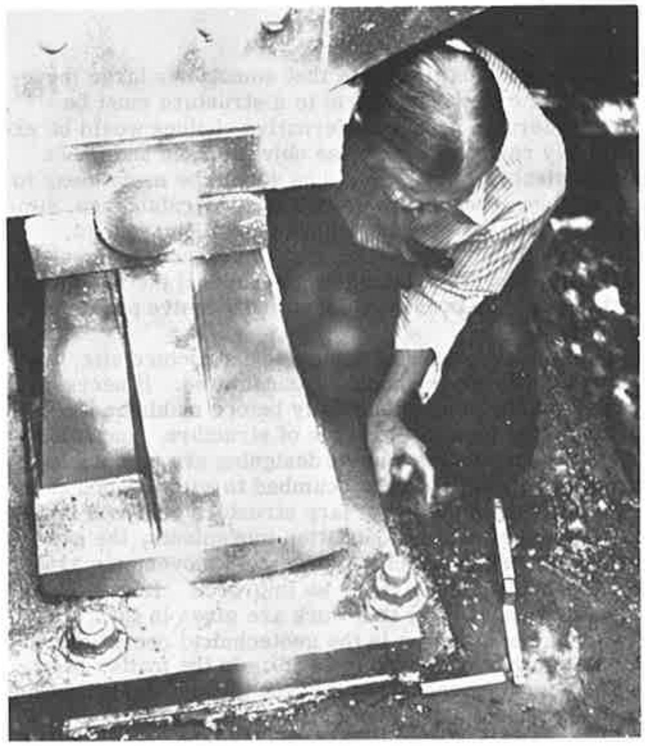


Figure 14. Shift of rocker base.

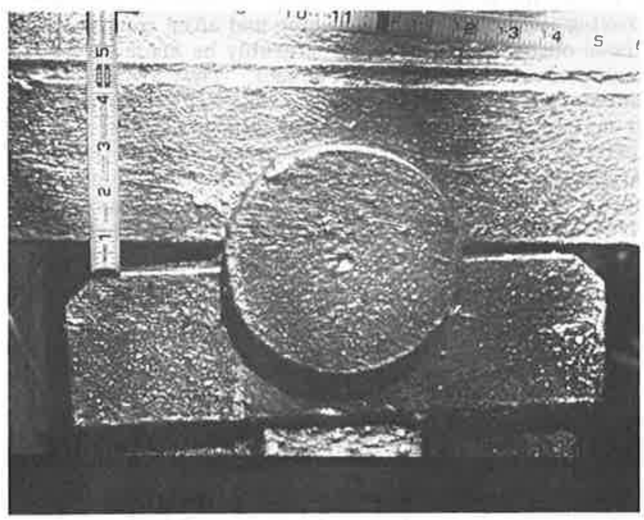


Figure 15. Tilt of rocker.

has moved forward about 2.5 cm. These movements are not very harmful and are tolerable. However, if deletion of the piles at the abutments had been foreseen during design, the embankment would have been placed first and allowed to settle to avoid a potentially intolerable situation.

It may be of interest to describe the compressibility of the soft varved clays under the foundations of these structures. These clays are part of glacial Lake Connecticut deposits. Their natural water content is approximately 50 percent and their resistance (N) in the standard penetration test is 2 to 4 blows/30 cm (12 in). However, they have been preconsolidated (overconsolidated) in their past history to about 1.5 to 2.5 kg/cm^2 (1.5 to 2.5 $tons/ft^2$) in excess of today's overburden load. This overconsolidation is indicated by laboratory consolidation tests on undisturbed samples and in part by geological studies. Consequently all or most of the compression in these varved clays, due to the project loads, is in the recompression phase and therefore the settlements are much less than if the clays had not been preconsolidated.

Turning to cases of movements that are largely or entirely horizontal, Charter Oak Bridge is over the Connecticut River, Hartford (Figure 10). Two cases occurred, one in 1941 and one in 1946, both due to lateral pressures against piers founded on long piles driven through soft varved clay. The first case involved piers 9 and 10, which moved away from each other 2.5 and 7.5 cm (1 and 3 in), respectively, due to a dike fill 8 m (25 ft) high placed between these piers. The resulting 10-cm (4-in) increase in span length required considerable adjustment of the girders when erected the next year; the girders for this span are barely seated on pier 10. The other case involved a roadway fill 5 m (15 ft) high placed some years later between piers 3 and 4. They moved about 2.5 cm each, causing the joint at the expansion pier 4 to open 5 cm (2 in). This became a hazard in cold weather—sufficiently so to be photographed for a front-page article in the evening newspaper. To resist any further movements, sloping steel raker piles were installed at both piers. Both of these cases can be considered not tolerable, as they involved harmful movements that could have been avoided by counterweight fills placed on the opposite side of each pier without difficulty.

The final case history concerns a Farmington River bridge in Simsbury built in 1953. This is a 69-m (224-ft) single-span through-truss bridge carrying CT-185 over the river (Figure 11). The soil below the footings is 6 m (20 ft) of brown silt and fine sand overlying 42 m (140 ft) of fairly firm brown silt; below this is sand. The structure is supported on 27-m (90-ft) cast-in-place concrete piles in Monotube shells. To simulate possible liquefaction of the soil, the four test piles, before they were loaded as a group, were surrounded by 27 other piles. Liquefaction occurred and was dissipated in 2 weeks after the 31 piles were driven. Figure 12 shows the results of load tests made during liquefaction and after it was gone. As seen in Figure 13, the abutments settled 0.5 to 1 cm (0.25 to 0.5 in) and moved toward the river 6 to 9 cm (2.5 to 3.5 in), chiefly by translation (not rotation). This shortening of the span by 15 cm (6 in) was anticipated in design. Figure 13 shows that 7.5 cm (3 in) of shortening occurred before the superstructure was erected and 7.5 cm after its erection. Some years later, at the expansion end of each truss, the rocker base was shifted 6 cm to make the rocker plumb at mean temperature. This was because the rocker system had not been designed for an excessive tilting of the rockers. Figures 14 and 15 show the results after these adjustments. It appears that the movements were slightly harmful but tolerable. The cost of shifting the two rocker supports was small and probably much less than that for adding a short span with stub (perched) abutment at each end of the bridge.

SUMMARY AND CONCLUSIONS

The definition of tolerable movement is often complicated by several factors, including

1. Amount of movement,
2. Type of structure,
3. Effect on each part of structure,
4. Cost of alternative choices,

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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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