Skew Effects on Backfill Pressures at Frame Bridge Abutments

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The abutments of frame bridges are integrally connected to the deck without expansion joints. Active soil pressures are normally considered in design despite the movement of the abutments into the soil from thermal expansion of the deck. Many abutments are located on a skew, but possible effects of this skew on the backfill soil pressures are not considered in design. To improve the knowledge of soil pressures behind a skewed integral abutment for use in designing this type of bridge, soil pressures were measured on an installed project for 33 months. The soil pressure measurements were taken using total pressure cells in the backfill on each side of the centerline for both abutments of a 20-degree skewed bridge in Maine. A total of 16 pressure cells plus temperature indicators have been monitored four times a day using a data acquisition system since October 1989. Expansion of the deck causes the pressure to increase well above the active conditions on the upper part of the abutment wall. Skew effects on the pressures that develop near the deck level behind the abutment wall of an integral abutment are substantial. When the greatest deck expansion occurs, the pressures at 3 m (10 ft) from centerline on the obtuse side reach almost three times the value at the corresponding distance on the acute side. The horizontal variation of pressure is greater than the vertical variation. A design envelope is proposed.

The typical bridge is designed with expansion joints to accommodate the deck movements related to temperature change. Abutments on shallow foundations are then able to rotate during construction and for a limited amount during operation under the action of the lateral soil pressure of the approach fill. Thus active pressure coefficients (K_A) as determined by the Rankine, Coulomb, or log-spiral method, as given by Naval Facilities Engineering Command (NAVFAC) (*1*) and elsewhere, are used in abutment design to determine soil pressures from the approach fill. In some abutments, there is restraint to movement due to wingwalls, pile foundations, or cantilevering. In these cases, higher lateral soil pressures may develop on the wall (*2–5*), and some designers (*1*) recommend the at-rest pressure coefficients (K_0) instead of the active coefficients. In all these designs, the lateral soil pressures vary with depth (see Figure 1), and a single profile is used across the abutment.

Bridges with the abutment rigidly connected to the deck slab are also being widely used. Dagher et al. (*6*) found in a survey that 11,500 bridges of this type with and without skew exist in 22 states. The elimination of the expansion joint allows thermal changes in the deck slab to move the abutment into or away from the fill. On the basis of field measurements, Broms and Ingleson (*7*) proposed a pressure envelope that combines a passive pressure envelope using Rankine passive pressure coefficients (K_P) in the upper third of the wall and a transition to the Rankine active case at the base of the wall (see Figure 1). This recognizes the abutment movement into the backfill at the top and the lack of movement at the base of the abutment. In these cases, the lateral pressure distribution varies with height, but the same distribution is used across the abutment. Most states that use pile-supported abutments ignore the effects of thermal expansion. Greimann et al. (*8*) indicate that "the survey responses show that most states ignore the thermally induced bending stress due to transverse thermal movement" and that "only a few states consider thermal, shrinking, and soil pressure forces when calculating pile loads." However, it is unclear that these soil forces can be ignored in all designs with pile foundations or any designs with shallow foundations.

For bridges with integral abutments on a skew, concerns are expressed about the effects of skew in the survey summarized by Greimann et al. (*8*). Some of the concerns included "rotational action caused by the active soil pressure on skewed bridges," as well as "rotational forces from the lateral earth pressure on the end walls [which] cause a failure of the pier anchor bolts on the exterior girders." However, the survey indicated that no special considerations are given to skew in determining pressures behind the abutment. For abutments on piles, Greimann et al. (8) state: "When the abutment is skewed, some twisting may be induced in the piles when the structure deflects, but this can be assumed to be of a minor nature and may be neglected. . . . No special treatments are usually given to backfill and pile cap on skewed bridges, and they might be constructed in the same way as on nonskewed bridges."

STUDY OBJECTIVE

The objective of the study was to determine if there are effects of skew of an integral bridge on the backfill pressures and how these relate to the deck movements arising from thermal changes and to develop a design guideline covering this effect. To accomplish this objective, the pressures occurring behind a skewed integral bridge abutment were continuously monitored.

INTEGRAL BRIDGE DESIGN

The Forks Bridge between the plantations of The Forks and West Forks in western Maine was opened to traffic in late

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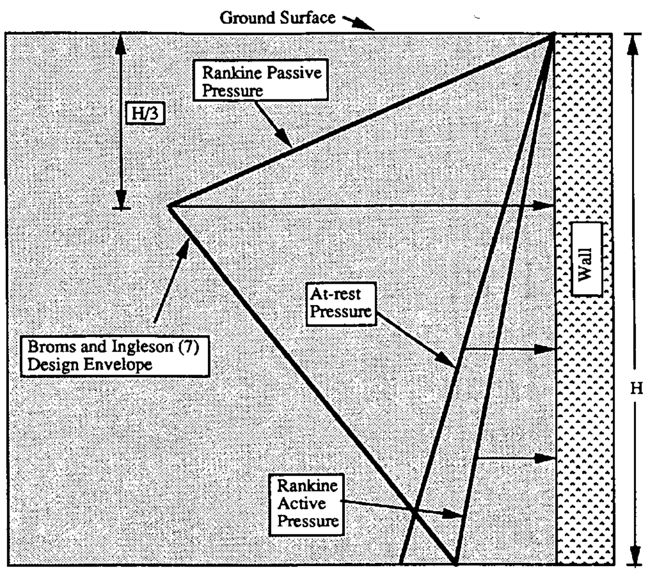

FIGURE 1 Design pressure distributions behind abutments.

1989. The stiff-legged, steel rigid frame bridge spans the Kennebec River. The bridge (see Figure 2) is a 20-degree skewed bridge with a 50.3-m (165-ft) span and an 11.5-m (37.67-ft) overall width. It consists of five steel frames resting on shallow foundations. The steel legs are encased in concrete to form the abutments (see Figure 3). The composite reinforced concrete deck is connected to the steel legs of the frames and the abutment encasement. The bridge design is described in more detail by Roberts (*9,10*).

The Maine Department of Transportation used a Rankine passive pressure in the upper third of the wall transitioning to an at-rest case at the base of the wall as described by Roberts (*10*) and shown in Figure 3. Thus the effect of expansion of the deck as found by Broms and Ingleson (*7*) was incorporated into the design. No effect of the skew on the soil pressures was considered. However, even this envelope raises questions. Because the Rankine passive coefficients neglect the effects of wall shear, could wall shear have a substantial effect on the passive resistance? For this length of bridge, will enough movement occur to develop full passive pressure? Does a skew of only 20 degrees affect the soil pres-

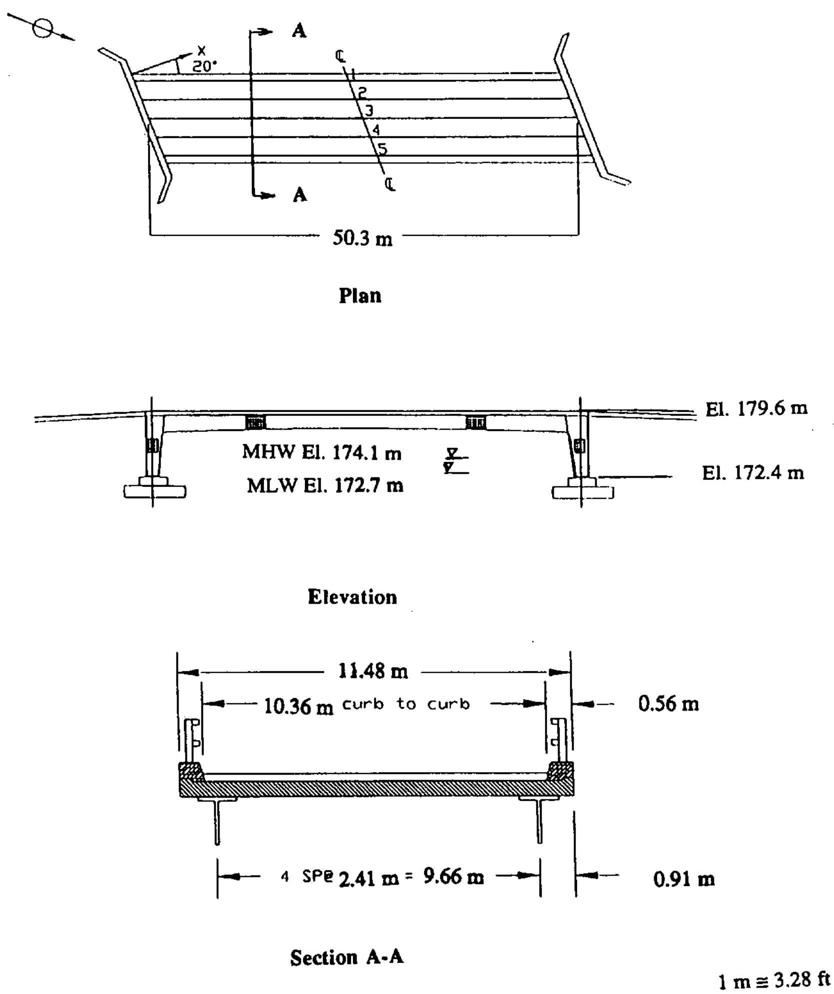

FIGURE 2 Plan, elevation, and section of The Forks Bridge.

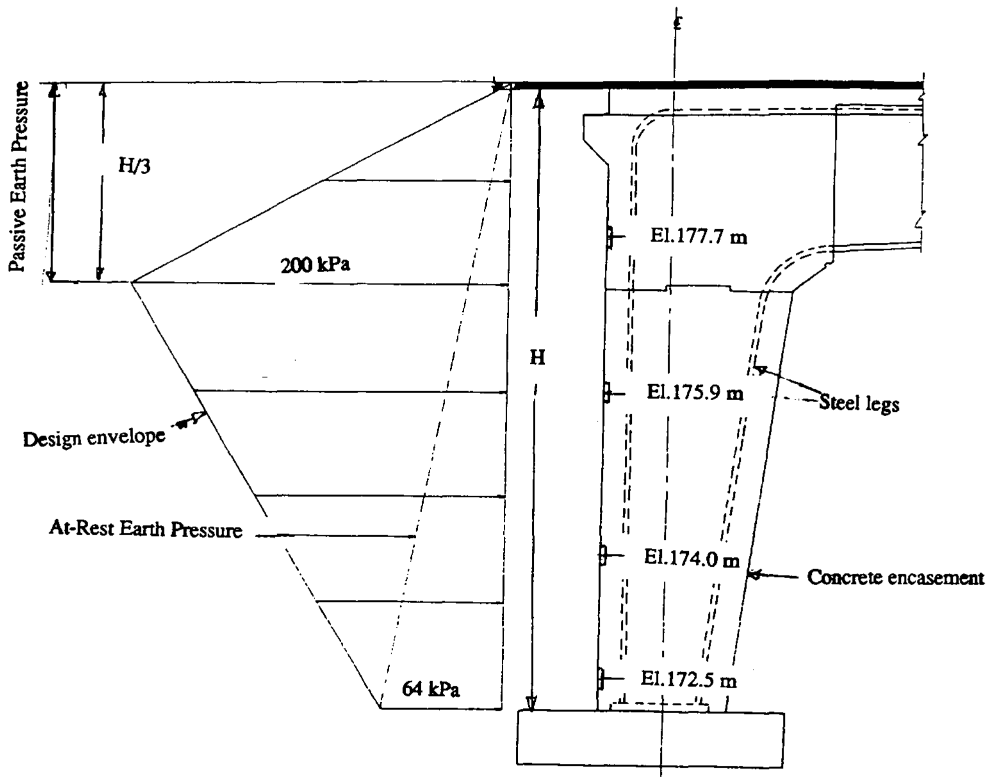

FIGURE 3 Design earth pressure for The Forks Bridge [after Roberts (*10*)].

sures? Will passive resistance increase with reloading in subsequent years? Because it had little experience with this size and type of bridge, the Maine Department of Transportation decided to monitor the soil pressures at the back of the abutment.

BACKFILL

The backfill was a coarse to fine sand and gravel with practically no fines as shown on a typical gradation in Figure 4. Triaxial tests using material compacted to 90 percent of AASHTO T99 gave a friction angle of 37 degrees for two different samples. The maximum dry density according to AASHTO T99 was 19.3 kN/m^3 (123 lbf/ft^3), and, after correction for oversize, the maximum dry density was 20.5 kN/m^3 (130.3 lbf/ft^3) with an optimum water content of 7.0 percent. The backfill was required to meet a minimum compaction dry density of 90 percent of the maximum, whereas the subbase was required to meet 95 percent of the maximum. The measured dry unit weights of the backfill averaged 94.9 percent of maximum, and the subbase averaged 96.4 percent.

INSTRUMENTATION

A total of 16 pressure cells were mounted at the back of the concrete abutments to monitor soil pressures for vertical variation and the effects of skew. Two temperature indicators monitored air temperature near the structure. The cells and other instrumentation were connected to a data acquisition system that could be remotely accessed by telephone to retrieve data or change the reading schedule on the instruments. The instruments are described in more detail by Elgaaly et al. (*11,12*).

The presence of a pressure cell alters soil pressure to some degree, and it is usually impossible to match the stiffness of the cell to that of the soil. Weiler and Kulhawy (*13*) and Dunnicliff (*14*) describe problems in obtaining reliable earth pressure readings. Thus a number of provisions were made to maximize the reliability of the readings. These included selecting a 228.6-mm (9-in.) diameter fluid-filled cell with a diameter-to-thickness ratio of 23. A vibrating wire transducer was used to measure pressures, and thermistors within the cells were used to correct the pressure signal for temperature effects. A bedding for each cell was prepared by casting a concrete block in the laboratory and setting the cell as nearly flush as possible into the wet concrete. After the concrete had set, the cell was removed, and the recess was ground as needed on the edges to remove concrete shrinkage pressures on the cell edges. Each cell was calibrated in this preformed bed in the concrete block, and then the cell and its concrete block were installed flush to the back of the abutment in the field as described in more detail by Elgaaly et al. (*11,12*). The backfill within 150 mm (6 in.) of each cell was restricted to a maximum size of 5 mm (0.2 in.) so that pressure concentrations would be reduced. The precast block containing the bedded cell protected the cell during concreting operations. The direct burial type cable was embedded in the abutment concrete and further protected with polyvinyl chloride tubing. Redundancy of readings was provided by installing instruments in both abutments as shown in Figure 5. All 16 cells

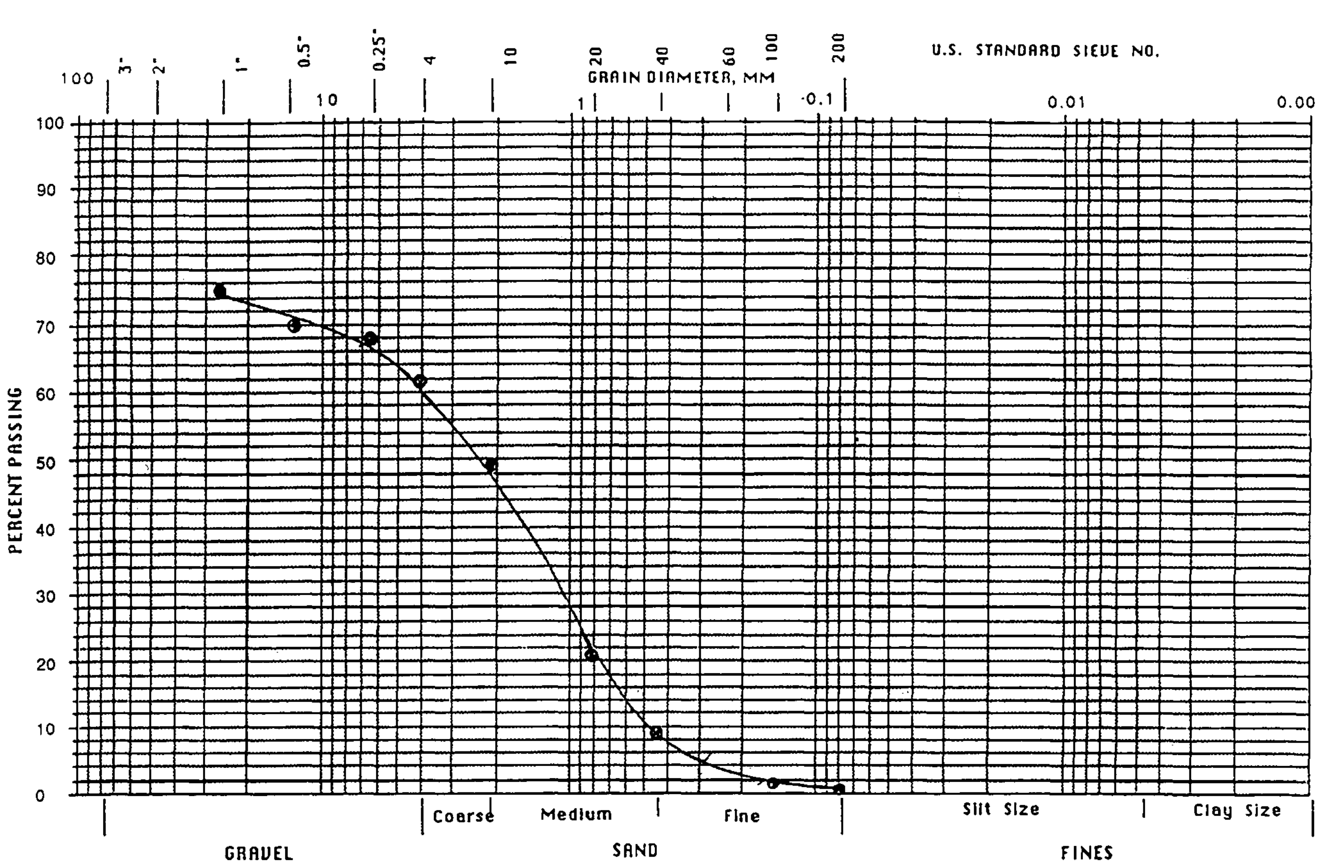

FIGURE 4 Grain size of approach fill.

were mounted 3 m (10 ft) from the centerline as shown in Figure 5. The elevations of the pressure cells with respect to the girders are shown in Figure 3.

Two resistance temperature indicators (RTDs) were located beneath the deck and measured the air temperature near the structure. The RTDs were calibrated in the field. The soil temperatures at the back of the abutment were available from thermistors located in each pressure cell. However, only six soil temperature locations at the back of the abutment were monitored because of channel availability on the datatakers. One of these locations was below river level. Because the in situ material was free draining, the temperature measured at this location was likely close to the river water temperature. The RTD air temperature indicators ceased operating on June 29, 1991 (Day 909), but the thermistors in the cells continued to monitor temperatures behind the abutment.

TEMPERATURES

The temperature of the structure determines how much expansion or contraction the structure will undergo. The structure temperature varies with seasons, weather changes, and diurnally as the sun heats the structure and with temperature changes in the air, adjacent soil, and water. Thus, a structure like The Forks Bridge continuously moves in response to these changes.

Two indicators of the superstructure temperature plus the water temperature in Maine over 33 months of monitoring are shown in Figure 6, in which the days are numbered beginning with 1 on January 1, 1989. The indicators of the superstructure temperature are the midday air temperature below the deck and the midday temperature at the back of the abutment near the girder level at Elevation 177.7 m (583 ft) in Cell 1 as shown in Figure 5. Midday water temperature as measured in Cell 7 shown in Figure 5 is also given in Figure 6. Despite three gaps where data were lost and the loss of the air temperature indicator on Day 909, the profile of temperatures at the bridge was obtained and is shown in Figure 6.

The seasonal changes in the air, soil, and water temperatures shown in Figure 6 are the largest temperature changes. This is important because seasonal temperature changes will result in a more or less uniform rise in the structure temperature. Therefore the air and soil temperatures should be good indicators for seasonal temperature change of the structure. Although data from every fifth day are plotted, the weather front changes are reflected in the variability of the air temperature. The magnitude of this variability is considerably less than the seasonal changes. Diurnal air temperature changes are not shown because only midday temperatures are plotted, but the effects are generally less than the weather front changes. The short-term changes will result in nonuniform temperatures in the structure. The temperatures in the structure are not uniform because of the rate of heat transfer, the heat capacity, heat sources and sinks at various locations on the structure, and a complicated geometry. For example, during the day, the top of the deck with an asphalt cover will reach a higher temperature than most of the steel, which is covered by the concrete deck. The most westerly girder will likely be

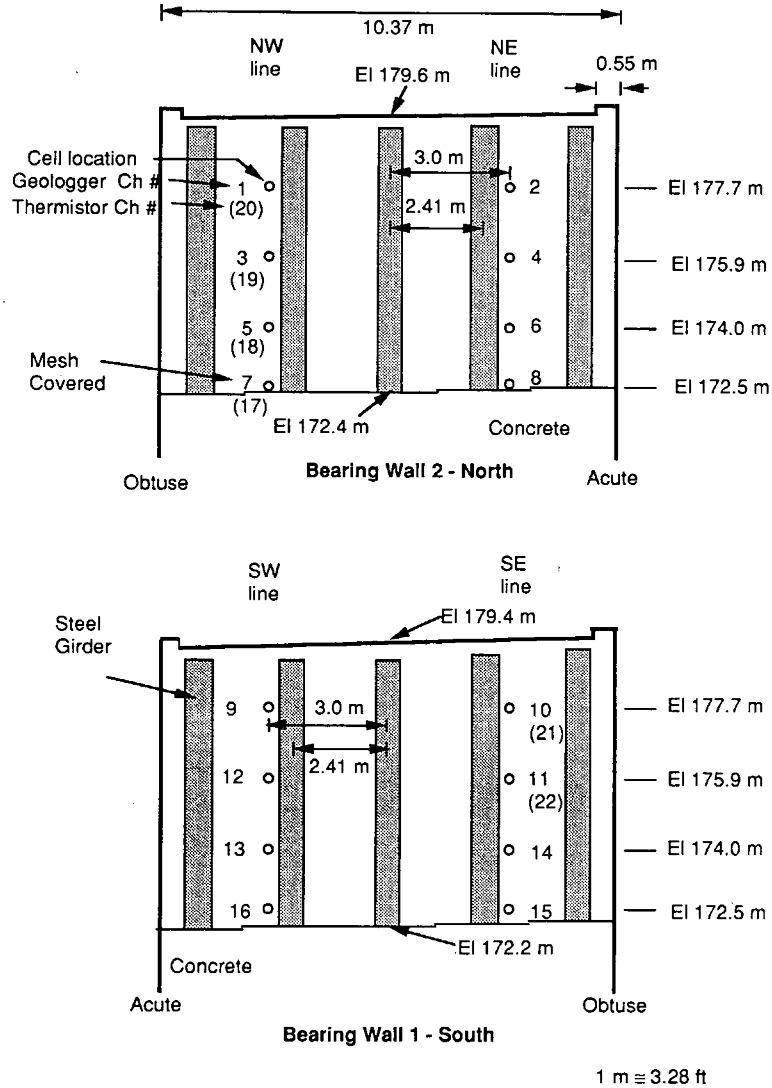

FIGURE 5 **Pressure cell locations.**

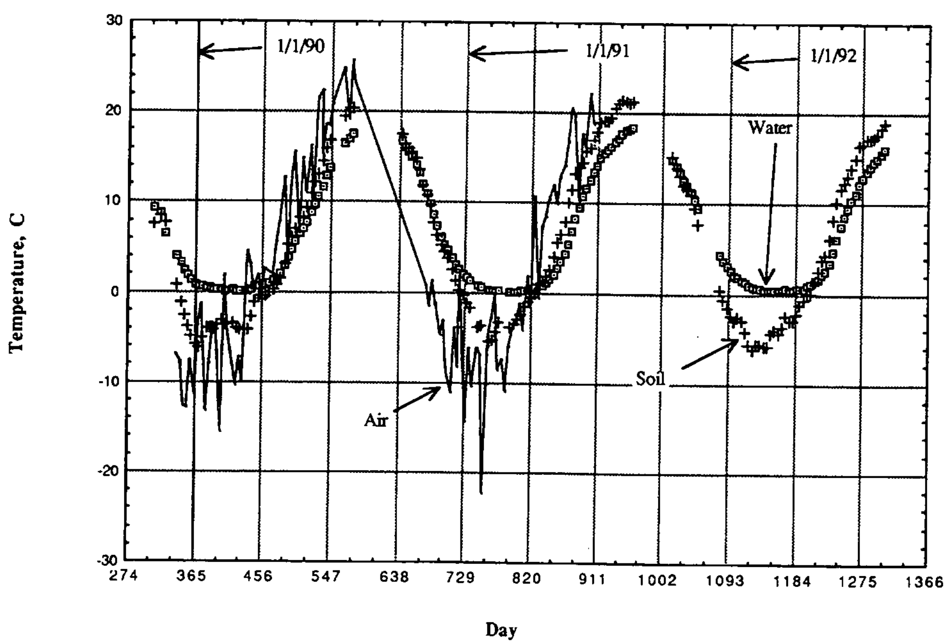

FIGURE 6 **Temperatures at The Forks Bridge.**

warmer in the afternoon than the other four girders. Air temperatures below the deck and soil temperatures behind the abutment may have shortcomings for predicting distortions from short-term temperature fluctuations. Although the plots of midday temperatures do not show the diurnal variation of temperature, they do include the higher diurnal temperatures occurring at the bridge.

The seasonal air, soil, and water temperatures and patterns of change that were monitored do not appear unreasonable for this location in Maine. The summer of 1992 was cooler than the two previous summers, as indicated by lower soil temperatures. The river current is too swift at the bridge for freezing to occur even though the air temperature in the winter is often well below freezing. Because the measured water temperatures go right to freezing but not below freezing, this increases the confidence in the water temperature readings. Freezing was indicated in the soil at a depth of 1.9 m (6.23 ft). This is ordinarily below the maximum depth of frost penetration. However, the cell is located at the back of the concrete wall. Although the concrete is about 2.5 m (8.20 ft) thick at this point, the freezing front penetrates the concrete rapidly.

LATERAL SOIL PRESSURE DISTRIBUTION IN VERTICAL PLANE

Because the bridge has no expansion joints, thermal expansion of the deck due to temperature rise during the summer will push the abutments into the approach fills; winter contraction will then relieve the pressure. Soil pressure results for approximately 7 months were given by Elgaaly et al. (*11*). Additional results for a period of 33 months of the average pressures at each elevation behind the backfill are shown in Figure 7. At each level, with the exception of Elevation 172.5 m (566 ft), the plotted pressure is the average of four pressure cells. At Elevation 172.5 m (566 ft), one cell has been covered with a porous screen and measures only water pressure. This water pressure is used to find an effective pressure at that level and at Elevation 174.0 m (571 ft). The effective pressure at Elevation 172.5 m (566 ft) is thus the average of only three cells. These average midday pressures thus combine results from both the acute and obtuse sides of the centerline.

Despite the precautions taken during their installation, the pressure cells may give misleading results (*13,14*), but the results shown in Figure 7 do not appear unreasonable. Soil pressures vary vertically close to at-rest pressure or slightly lower when the bridge contracts during colder weather as shown in Figure 8. Near the footing at Elevation 172.5 m (566 ft), the pressures stay relatively constant throughout the year, reflecting little movement at that depth. Because these are effective pressures, there is some change in the results due to higher water levels in the spring. During warmer weather, the superstructure pushes into the soil at higher levels, and then the pressures at the higher elevations increase. For Elevation 174.0 m (571 ft), which is located in a zone of fluctuating water level of 1.4 m (4.7 ft) caused by upstream dam releases, there are spikes in pressures from about January 1 to mid-March of each year. It is believed that this is frost buildup due to the fluctuating water level at this location combined with an air temperature below freezing. This is a separate phenomenon from pressure buildup due to thermal changes. Considering the movement of the abutment from thermal expansion or contraction of the deck, the level of pressures in all seasons is not unreasonable. However, recent values at Elevation 172.5 m (566 ft) do show some erratic behavior in one cell.

Clearly, the active or even the at-rest pressures are inadequate for predicting the pressures that are generated in the upper levels of the abutment during expansion. On average, the results were similar to those found by Broms and Ingleson (*7*). The Rankine passive pressure down to one-third of the

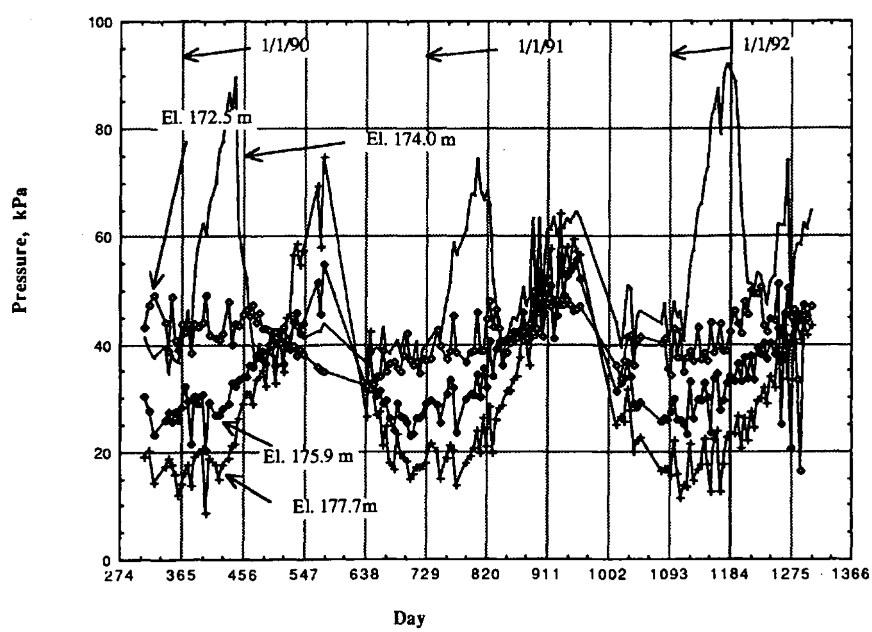

FIGURE 7 Pressure averages behind abutment.

wall height, which transitions to the at-rest condition at the base, forms an envelope for the results of the average pressures in this bridge, as shown in Figure 8. For locations other than Maine, the temperature changes will be different. Elgaaly et al. (*12*) found for this 50.3-m (165-ft) bridge that at Elevation 177.7 (583 ft), the change of pressure per degree change in temperature was 1.46 kPa/°C (17 lbf/ft²/°F) for seasonal, 0.51 kPa/°C (5.9 lbf/ft²/°F) for weather fronts, and 0.28 kPa/°C (3.3 lbf/ft²/°F) for diurnal temperature changes. There was concern that the soil reaction during the seasons following the first season would show stiffening and thus give higher pressures for the same expansion. The pressure cells do not indicate that this is happening.

SKEW EFFECTS ON PRESSURE

If pressures are affected by skew, then laterally nonuniform pressures will be exerted on abutment walls that may have implications for structural design of the abutment walls. To measure the effects of the skew on the soil pressures, pressure cells were installed on both abutments on both sides of the centerline as shown in Figure 5. Under a uniform temperature change with no constraint of the soil, it was anticipated that the skewed superstructure with a span of 50.3 m (165 ft) would expand or contract uniformly parallel to the roadway centerline as the temperature changed.

As shown in Figure 5, at Elevation 177.7 m (583 ft) two total pressure cells were installed on the acute side (Cell 2 on the northeast line and Cell 9 on the southwest line) at 3 m (10 ft) from the centerline and two pressure cells on the obtuse side (Cell 1 on the northwest line and Cell 10 on the southeast line) at the same distance from centerline.

The average pressures of the two obtuse cells are compared with those of the two acute cells in Figure 9. The pressures are about the same when the bridge contracts from November through February of each year. The pressures are close to the anticipated at-rest pressure of 13.3 kPa (280 lbf/ft²) and appear to sometimes reach the anticipated active pressure of 8.3 kPa (175 lbf/ft²). This is not unexpected and gives confidence that the pressure cells are reading at an appropriate level. During March of each of the three years, the seasonal temperature increase begins to increase the pressure at all cells. However, there is a distinct difference at this elevation of the superstructure between the pressures on the acute side and the pressures on the obtuse side. By the end of July, which is the peak temperature in Maine (as shown in Figure 6), the pres-

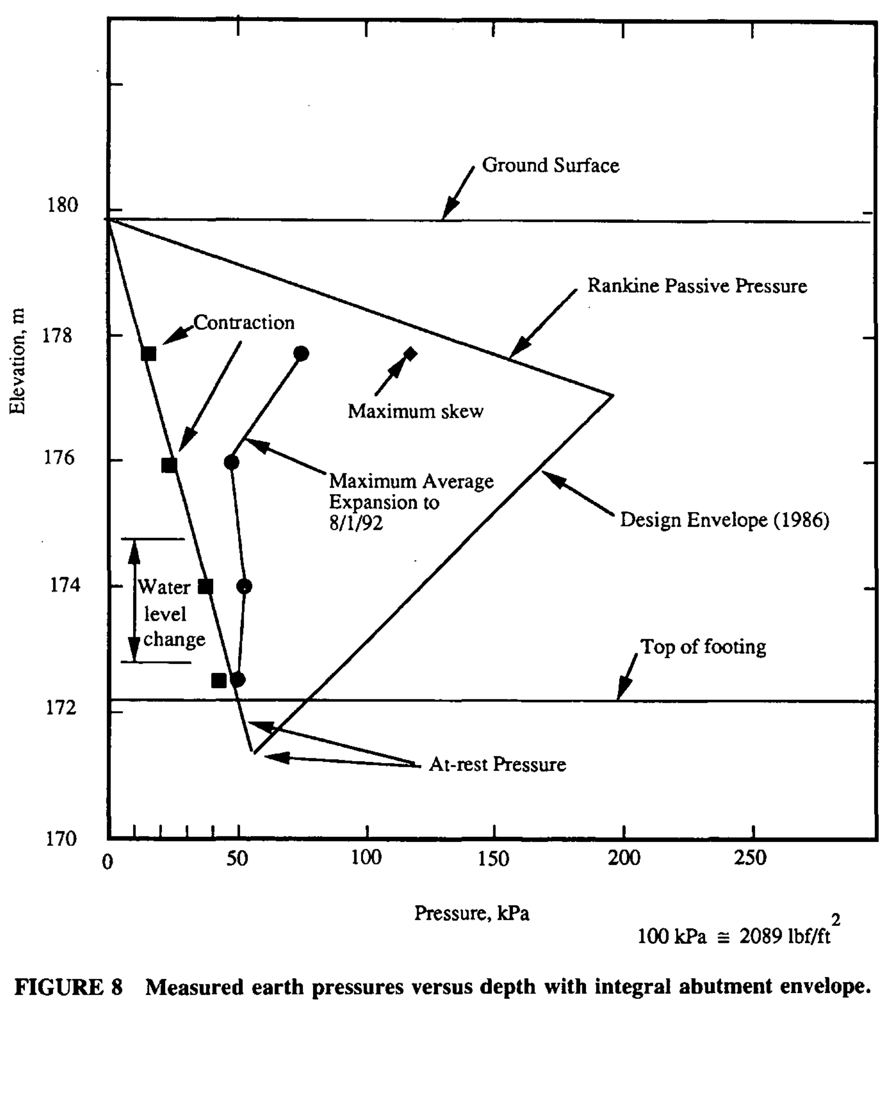

FIGURE 8 Measured earth pressures versus depth with integral abutment envelope.

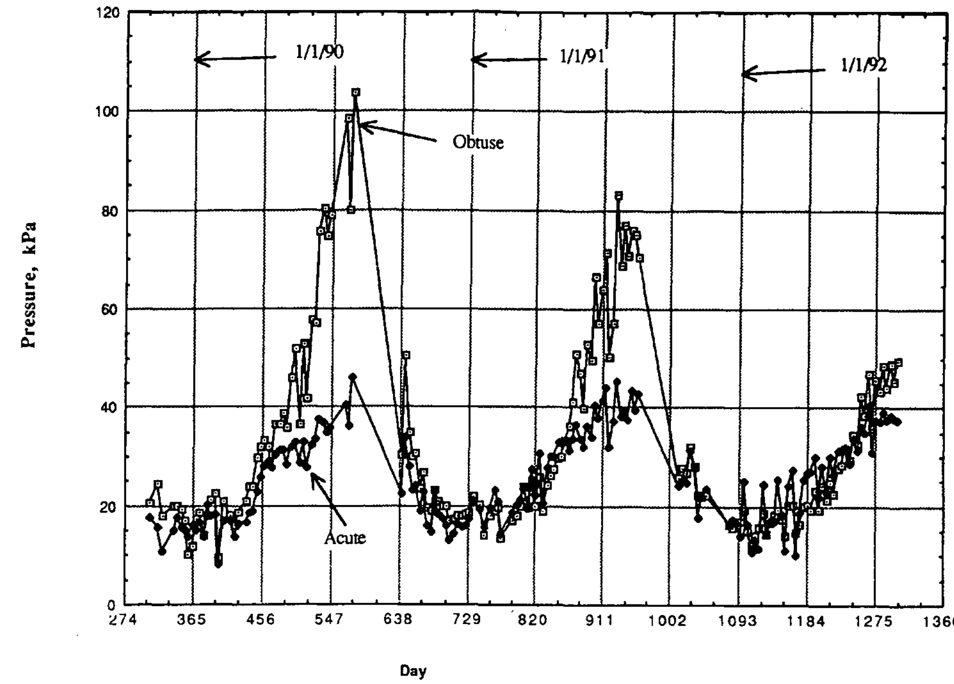

FIGURE 9 Skew effects on pressures, elevation 177.7 m.

sures on the obtuse side have increased by four to six times the cold weather value, whereas the pressures on the acute side have increased only two to three times.

The difference in pressure between the cells on the obtuse side and those on the acute side is shown in Figure 10. For 4 to 5 months during the warmest weather, there are consistent differences in pressure between the obtuse side and the acute side of this skewed bridge. The difference occurs seasonally and thus indicates that the skew effect is not a reflection of distortions caused by nonuniform temperatures in the structure from short-term changes in temperature. The effect of the skew appears to be diminishing with time, and thus there

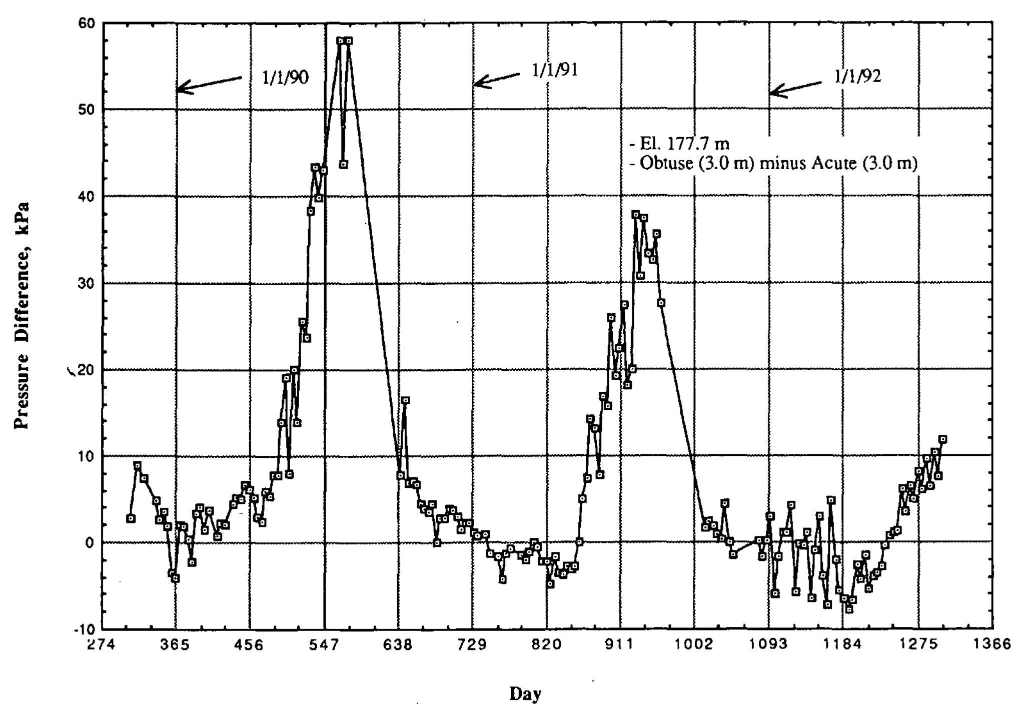

FIGURE 10 Lateral pressure difference at girder elevation.

is no indication that the effects of the cyclic stiffening of the soil are increasing skew differences.

The pressure difference across the centerline indicates that a greater movement is possibly occurring on the obtuse side than on the acute side; that is, a rotation of the abutment at the level of the superstructure is occurring. When the air temperature increases, the bridge deck experiences thermal expansion, which is restrained by the massive abutments. This restrained thermal expansion will result in thermal stresses and strains, which have been found to be larger at the obtuse corner than at the acute corner (*6*). Hence, the bridge has the potential to push more on the soil at the obtuse corners and cause higher pressures at the obtuse corner. The diminishing of the skew effects with time indicates that the expansions may have caused more permanent deformation of the backfill on the obtuse side than on the acute side.

For a linear rotation of the abutment, the pressures near the ends at 4.88 m (16 ft) from the centerline are estimated and given in Table 1. The estimated pressure difference across the skewed abutment at Elevation 177.7 m (583 ft) is almost 96 kPa (2000 lbf/ft^2). This is a larger difference than occurs from top to bottom during contraction or expansion.

It is possible that some of the skew effects arise from differences in stiffness of the soil. However, the magnitude of difference that was measured was more than could be expected from soil property variations. The average unit weight of the backfill on the acute side was 19.4 kN/m^3 (123.6 lbf/ft^3) on the basis of 14 tests. On the obtuse side, the average unit weight was 19.5 kN/m^3 (124.2 lbf/ft^3) on the basis of 21 tests. The behavior was replicated on each abutment, which makes it less likely to be caused by soil property variations. There is more apparent confinement of the soil on the acute side than on the obtuse side, which would give the opposite effect from that measured.

At the next lower level of cells at Elevation 175.9 m (577 ft), which is below the girder level, there is substantial buildup of pressure as expansion occurs in the warm months, as shown in Figure 7. However, when the obtuse and acute cells are distinguished, there are some differences, as seen in Figure 11, but it is not the same pattern as for the girder elevation in Figure 10. The magnitude of pressure differences is small at Elevation 175.9 m (577 ft) and can almost be considered as having no skew effect. This indicates that the thermal expansion restraining effects are diminishing below the deck-abutment junction, as would be expected.

CONCLUSIONS

For integral abutment bridges, thermal expansion will be resisted by abutment soil pressures, which will rise above the active or at-rest conditions according to the magnitude of movement depending on the length of the bridge, the temperature change, and the sharing of the resistance with the foundation piling. Measurements taken on a 50.3 m (165 ft) span bridge in Maine showed that average soil pressures at the girder level reached five times the at-rest condition (Figure 8).

For this 50.3-m (165-ft) span bridge, the Maine Department of Transportation proposed a modification to the Broms and Ingleson (*7*) design envelope incorporating passive pressures (Figure 3). Monitoring indicated that these design pressures were an envelope for the actual soil pressures. Most designers of these abutments now use active pressures (*8*), which can be significantly less than the actual pressures.

With a 20-degree skew, measurements showed a variation in soil pressure laterally at a height near the base of the girder. The pressure at 3.0 m (10 ft) from the centerline on the obtuse side at this level was measured to be more than two times the pressure at the same distance on the acute side (Table 1 and Figure 12). It is anticipated that the pressure ratio of the obtuse to the acute at the ends of the abutment 4.88 m (16 ft) from the centerline could reach a ratio of 4. The lateral variation of pressure for the deck expansion is higher than the vertical variation of pressure due to the expansion. This lateral variation has apparently not been considered by designers of these abutments (*8*).

RECOMMENDATIONS

As a result of this study, the authors recommend the following:

- For design of integral abutments, a vertical pressure envelope similar to that of Broms and Ingleson (*7*) or that of the Maine Department of Transportation (Figure 3) should be considered.
- For skewed abutments, a horizontal soil envelope that has the Rankine passive pressure at the obtuse end of the wall and the Rankine active pressure at the acute end should be considered (Figure 12). This should be analyzed to the

TABLE 1 Pressures at Skewed Abutment, The Forks, Maine

Distance from Centerline, m (ft)	Maximum Pressure, kPa (lbf/ft^2)	Pressure[d] Difference, kPa (lbf/ft^2)
4.88 (16), Obtuse	121 (2530)[c]	104 (2170)
3.0 (10), Obtuse	103 (2160)[a]	86 (1800)
0.0	75 (1560)[b]	57 (1200)
3.0 (10), Acute	46 (960)[a]	29 (600)
4.88 (16), Acute	28 (590)[c]	11 (230)

(a) Measured value in late July 1990
(b) Interpolated value
(c) Extrapolated value (linear extrapolation)
(d) Maximum pressure minus measured pressure for cold weather of 17 kPa (360 lbf/ft^2).

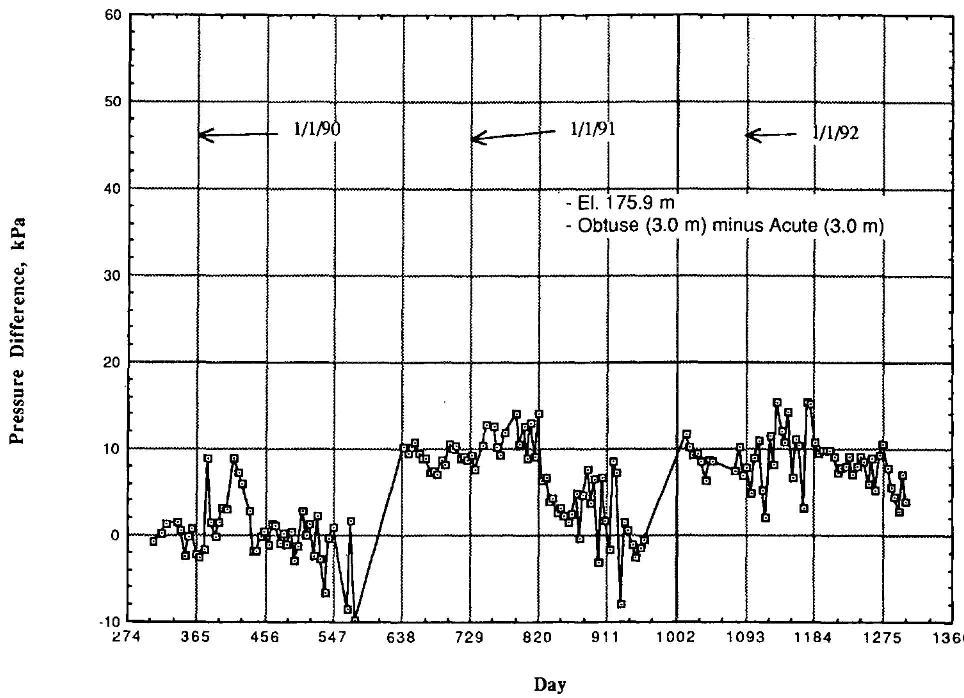

FIGURE 11 Lateral pressure difference below girder level.

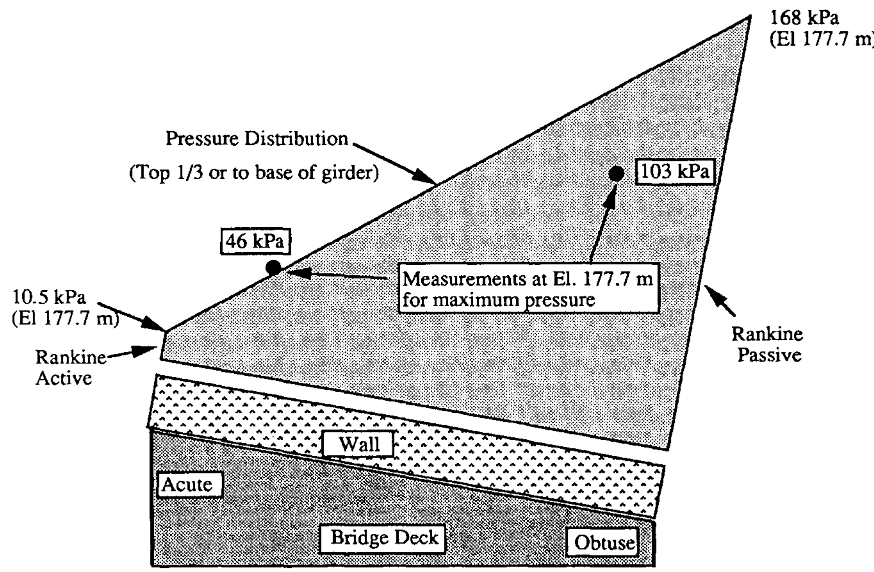

FIGURE 12 Design lateral pressure distribution for skewed abutments.

maximum depth of either the upper third of the height of the wall or the distance to the base of the girder.

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