REINFORCED EARTH® BRIDGE ABUTMENTS

Groupe TAI
Early on in the use of Reinforced Earth® to construct very high or very heavily loaded retaining walls, it was found that this technology could be easily adapted to the construction of abutments for the direct support of bridge superstructures. However, although the general principles involved are the same, the concentrated loads created by bridge superstructures significantly affect the distribution of stress and strain within the reinforcements. As with research on Reinforced Earth retaining walls, studies on the effect of such loadings and the development of increasingly precise design methods included measurements of actual projects, reduced-scale models, and finite element studies.

In the design of load-bearing abutments, detailed analysis of the abutment geometry, of the bridge supporting structures, and of the provisions for handling water are required. Construction demands strict adherence to the specifications governing the selection and compaction of backfills.

Reinforced Earth's inherent flexibility makes it possible to construct bridge abutments on soft soils. Special foundations are not required, although in some cases simple soil improvement techniques are recommended.

The design and construction scheduling of each project must take into consideration the characteristics of the superstructure, phasing and waiting-period requirements, geotechnical data, and the inherent properties of Reinforced Earth construction.

In special cases, it may be necessary to separate the retaining and bearing functions of a structure by constructing a mixed abutment. In such cases, if the foundation soils are good, the use of a "pier abutment" – a type of mixed abutment with interior supports – may be viable.

The worldwide experience gained in developing various designs and configurations of hundreds of abutments enables Reinforced Earth engineers to determine the optimum solution for each application.
In 1969, the successful construction and performance of several very high retaining walls demonstrated that Reinforced Earth technology could also be used for the construction of bridge abutments.

1969 - Strasbourg
The engineering department of Electricité de France provided an opportunity to confirm this thinking on a bridge project near Strasbourg. La Terre Armée, Paris, designed and supervised the construction of two abutments for a bridge to carry very heavy truck loads on a service road leading to hydroelectric dams on the Rhine River. These first abutments proved to be both technically and economically successful.

1972 - Thionville
Following construction of the Strasbourg prototype structures, the first highway abutment was built at Thionville, France, in 1972. The structure is 15 meters high and supports the end span of a 76 meter-long prestressed concrete viaduct that crosses the Moselle River on the Nancy-Luxembourg Highway. (A second bridge has since been added. It is supported by a portion of the Reinforced Earth abutment built for this purpose 15 years before).

Dunkirk
The ambitious Thionville project was made possible by the successful construction of a large coal and ore loading facility at the Port of Dunkirk, France. Reinforced Earth's inherent earth retaining capacity and exceptional load handling characteristics were utilized in the design of parallel walls, up to 15 meters high and 550 meters long, to create a storage area and to support traveling gantry cranes with wheel loads in excess of 1,000 tons (Fig. 1).

This heavily surcharged structure, like Thionville, was extensively instrumented. Analysis of the results served as the basis for the rational design procedures utilized for subsequent Reinforced Earth abutments. Fifteen years after Dunkirk, more than 1,700 bridge abutments were in service worldwide.

Conceptual Design
A Reinforced Earth abutment essentially consists of a conventional Reinforced Earth retaining wall designed to support the earth pressures behind it, as well as the heavy, concentrated vertical and horizontal surcharge loads imposed on it by the bridge superstructure and traffic loads (Fig. 2a). Superstructure loads are transmitted by a reinforced concrete beam seat which distributes the stresses to the top of the Reinforced Earth structure (Fig. 2b).

Research on Abutment Behavior
Research studies have been conducted in a variety of ways, including theoretical research using computer modelling, as well as the instrumentation of scale models, prototypes, and full-scale in-service structures. These studies conclusively demonstrated the mechanisms by which the loads imposed by the beam seat affect the behavior of the Reinforced Earth mass, and how these loads are subsequently distributed through the mass to the foundation soils. They have provided the means to accurately predict tension in the reinforcements and deformation of the Reinforced Earth structure as a whole.
Monitorying of Actual Structures

Measurement Principles

A number of in-service structures have been instrumented with strain gauges and pressure cells to measure the effects of concentrated loads from bridge superstructures. These effects are derived by measuring and comparing the relative tensile stress levels in the reinforcements prior to and then following placement of the superstructure.

Dunkirk

The Port of Dunkirk structures do not carry the relatively static loads of a bridge superstructure. Rather, loads are imposed by two traveling gantry cranes (Fig. 3). Consequently, the interpretation of data is somewhat complicated—both by the presence of residual stresses which diminish slowly after a crane has passed, and by the uncertainties associated with the manner in which the concrete supports distribute the localized load of the wheels. Nevertheless, measurements made along several cross-sections clearly indicate a variation in additional tensile stresses in the reinforcements from one level to another, as well as the influence of lateral distribution of the load from one section to another (Fig. 4a).

Abutments in France

The abutments that were instrumented at Thionville, Angers, and Lille have made it possible to assess the effects of each construction phase on the development of maximum tension levels in the reinforcements for abutments of differing sizes and proportions subjected to varying loading conditions (Fig. 4b).

Amersfoort

The most accurate and complete set of measurements on an in-service abutment are those obtained at Amersfoort, the Netherlands, in 1984 (Fig. 5). In this structure built for the Ministry of Public Affairs/Department of Bridges, forty-two measurement points were distributed over eight levels of reinforcements. Readings were taken at six successive stages of construction. The results of the measurements were in close agreement with computations made according to state-of-the-art design procedures, taking into account that the stiffness of the foundation soil at the base of this structure greatly reduces the tension in the lower levels of reinforcements strips. Deformations of the facing were also monitored using rods anchored in the fill well behind the Reinforced Earth mass. Measured movements of the facing were less than 0.1 mm.

Figure 3: Measurement profiles, Dunkirk structure.

Figure 4a, 4b: Tensile stress measurement in reinforcements at Dunkirk and Angers.

Figure 5: Tensile stress measurement at Amersfoort.
Other measurements have been taken on full scale experimental walls supporting heavy, concentrated loads in the form of concrete slabs or steel bars placed directly behind the facing panels.

**Triel**

In the experimental wall at Triel in France, built by La Terre Armée in 1975, surcharge loading was successfully increased to 90 kPa over a two-meter width behind the facing. The measurements, which involve three levels of reinforcements, are very reliable and conform well to the theory, especially with respect to load distribution.

**Millville**

At Millville in the United States, the Reinforced Earth Company built an experimental wall using short reinforcements. This structure was gradually subjected to a load of 40 kPa over a width of 1.5 meters (Fig. 6). Due to the wall’s very narrow profile (L/H = 0.45), the structure was more sensitive to the effects of moments developed by overturning, particularly those produced by the eccentricity of the surcharge. The vertical stresses measured by pressure cells placed under the entire width of the structure clearly demonstrated the need for designers to consider these moments in the computation of load distributions.

**Fremersdorf**

The Fremersdorf Wall constructed in Germany in 1980 was built for normal earth retention service, not for experimental purposes. However, the owner took the opportunity to experiment before the wall was placed in service. A localized, temporary load of 650 kN was placed on the reinforced volume at a point slightly further behind the wall facing than usual for a typical Reinforced Earth abutment load. The additional stresses measured in the reinforcements were in remarkable conformity with theoretical predictions based on abutment design methods (Fig. 7).

**Fontainebleau**

In 1988, a large, narrow, experimental wall built at Fontainebleau, France, by the Reinforced Earth Group was prepared for subjugation to concentrated surcharges analogous to those experienced by an abutment. Its upper portion was modified to form a layer of unreinforced fill overlaying the uppermost reinforcements. Loads were designed for distribution through vertical tension rods anchored in the sub-soil (Fig. 8).
Studies Conducted by Terre Armée Internationale

The performance of several series of bi- and tri-dimensional reduced-scale models has been extensively studied in the laboratory. These models were subjected to surcharge loading similar to that exerted on bridge abutments.

One of the most interesting experiments was conducted in 1982 in the Paris laboratories of the Center for Research and Study in Soil Mechanics (CERMES) at the request and sponsorship of Terre Armée Internationale. The CERMES study involved three-dimensional sand models, 60cm high reinforced with thin aluminium strips. In order to eliminate side wall effects, the models were separated into three vertical sections.

Study of the Failure Mode

In the initial series, 15 models were loaded to failure under increasing loads placed at various distances from the face of the structure (Fig. 9). Propagation of the failure surface was observed by using lights mounted in series with the reinforcements. Upon disassembly, all failure locations were carefully measured. By incorporating correction factors to compensate for the effect of the rigid base of the model, satisfactory agreement was obtained between the measured experimental failure loads and those predicted by theoretical computations (Fig. 10). In addition, it was consistently noted that the failure surface is influenced by the location of the surcharge. (The analysis of all results clearly indicated that the location of the failure surface is relative to the location of the surcharge load).

Stress Measurements

In two other series, models of equal height (80 cm) were subjected to 19 different loading conditions, with stresses measured using 30 strain gauges attached to the 0.2mm thick reinforcements (Fig. 11). Although it is difficult to accurately and reliably measure strains at the low levels of stress inherent in reduced-scale models, it is possible to observe a rather close agreement between the experimental and theoretical values, and the magnitude and location of maximum tensile stress in the reinforcements. Near the top of the structure, the location of the maximum stress in the reinforcements was observed to shift towards the midpoint of the load, away from its normal location in retaining walls.

Figure 9: CERMES models loaded to failure.

Figure 10: CERMES models. Recordings of loads and failure lines.

Figure 11: Models with wire strain gauges. Maximum tensile stress readings.
Principles

Mathematical models using the finite element method have also been used by Terre Armée Internationale to study the behavior of Reinforced Earth abutments and to analyze the influence of the principal design parameters. These models have been designed using the same elastoplastic modeling principles as those used in the mathematical study of retaining walls. Computations were performed using a specially developed computer program, Rosalie.

Design Parameters Studied

In tests conducted in 1984 and 1985 by Terre Armée Internationale, more than 50 different mathematical models were studied in which the following parameters were simultaneously varied:

- The height of the Reinforced Earth structure (6.0 meters and 10.5 meters).
- The length of the reinforcements (7.0 meters and 10.0 meters).
- The distribution of the reinforcements, using three typical configurations that differed primarily at the top of the structure.
- The dimensions and load at the beam seat (Fig. 12) corresponding to a bridge with a ten meter span or a bridge with a 30 meter span.
- The loading conditions (Fig. 13), including the following four stages:
  1. The structure without surcharge loading.
  2. The structure completed to the top of the beam seat, with permanent bridge load in place.
  3. The finished structure with total vertical loads and surcharges.
  4. The same as #3 above, with horizontal reactions added.

Results

As in the study of retaining walls, graphic superimposition of the results in bridge abutments allowed direct observation of the development of tension in the various reinforcement levels as loadings were increased. Results were also obtained on the effects of the different design parameters (Fig. 14). Further, the vertical stresses at every level of reinforcement (particularly at the foundation) and the deformations of the structure were recorded.
The Principle of Superimposition

Interpretation of experimental data and the results of finite element analyses have confirmed the reliability of analysis of an abutment structure by superimposing the stresses from two separate load conditions: stresses from the retaining wall conditions, and stresses caused by the bridge loading.

Stresses from Bridge Loading

Distribution:
The study of stresses arising from bridge loading involves the distribution of a vertical load within the reinforced volume. The results show that Boussinesq’s formula (Fig. 15a) is perfectly satisfactory in defining this distribution, whether toward the rear of the beam seat (using an assumption of an equivalent symmetrical surcharge) or laterally.

As vertical stresses from the various surcharge loadings diffuse with depth, the center of gravity of the Boussinesq stress distribution moves away from the wall facing; that is, the resultant of the overall vertical stresses moves rearward. This movement of the resultant creates an overturning moment that increases with depth. This overturning moment must be considered in the overall stability of the structure as an increase in the stress component with depth (Fig. 15b).

However, the surcharge load is effectively distributed only insofar as it loads, in combination with this moment, to a dispersion of stress. This defines the depth limit y, for load distribution.

Horizontal forces:
The horizontal forces applied to the beam seat from the superstructure and horizontal earth pressure behind the beam seat also create an increasing overturning moment. This moment affects the overall stability of the structure, even when these loads are transmitted first and directly to the uppermost layers of reinforcements, as indicated by finite element analysis (Fig. 16a).

Stresses from Retaining Wall Conditions

The study of stresses arising from earth retention follows standard Reinforced Earth retaining wall criteria. Thus, the effect of a structure’s weight and the overturning moment generated by the active earth pressure behind it are considered. They are then taken in combination with horizontal loads and the overturning moments generated by the shift of vertical load distributions (Fig. 16b).

Figures 15a, 15b: Principle of the distribution of vertical loads according to Boussinesq.

Figure 16a: Effects of concentrated horizontal stress according to the finite element models.

Figure 16b: Stresses assigned to the support function.
The tensile stress measured along a reinforcement is the resultant of the combined effects of the two loading conditions previously discussed.

Due to earth retention conditions, a maximum tensile stress develops (which under certain circumstances may be a secondary maximum) together with a potential failure surface similar to that observed in normal Reinforced Earth retaining walls (Fig. 17a). However, under the influence of wide beam seats, this line of maximum tension moves away from the face of the structure towards the heel of the beam seat. This potential failure line is always contained within the typical Coulomb failure wedge (Fig. 17b).

Due to bridge loading conditions, another potential failure surface develops. This surface originates at the top of the structure near the center of the beam seat and intersects the facing at the point of the critical wedge defined by the geometry of the beam seat (Figs. 17a, 17b). As a result, when relatively narrow beam seats are used, maximum tensile stresses are generally encountered near the facing, beyond this wedge.

Value of Maximum Tension in the Reinforcement

Experimental data confirms that the maximum tensile stresses \( T_{m} \) on both potential failure lines are again related to the total vertical stress exerted at the same points by the relation \( T_m = K_v N \), where \( N \) is the number of reinforcing strips per square meter of facing panel and \( K \) varies from \( K_1 \) to \( K_6 \) in the upper six meters of the structure.

Resistant Zones

The potential for the existence of two possible failure lines where the reinforcements are subjected to secondary maximum tensile stresses requires verification of adherence conditions over two different resistant lengths \( L_{st} \) and \( L_{ad} \) (Fig. 18).
The practical methods of computation apply to abutments in which the beam seat is of limited width and is situated slightly behind the facing.

**Applied Loads**

1. For all loading conditions to be considered, the forces exerted by the beam seat are expressed as a horizontal force, $F_h$, and an equivalent uniform vertical load, $q$, distributed over a width slightly smaller than the beam seat.

2. This vertical load, together with the loads imposed by the fill behind the beam seat, is replaced by an overall uniform surcharge, $q_s$ (as used in the retaining wall analysis), together with additional surcharge loads adjacent to the facing that simulate greater or lesser strip loads (equivalent, once superimposed, to the bridge seat load). Taken together, these surcharge loads are considered in the computation of distribution (Fig. 19).

**Stress Distribution**

1. Distribution toward the rear of each loaded strip is computed using Boussinesq's equation:

$$
\sigma(x, y) = \frac{q}{\pi} \left( \frac{1}{x^2 + y^2} \right) \left\{ \begin{array}{ll}
\frac{x + a}{y} & \text{if } x + a > 0 \\
\frac{x - a}{y} & \text{if } x - a > 0
\end{array} \right.
$$

At each reinforcing strip level the value of $\sigma$ as a function of the distance $x$ from the face, which corresponds to each of the strip loads, is summed up:

$$
\Sigma \sigma_x = \sigma_{x_1} (x, y)
$$

2. Lateral distribution is estimated in simplified fashion by a truncated pyramid, possibly curtailed by the wing walls. From this distribution a reduction factor $\lambda$ of $\sigma_x$ may be deduced (Fig. 20).

3. The loaded strips are distributed only to depth $y_s$ where $\frac{d \sigma_s}{dy} = 0$ for the maximum total stress. In practice, $y_s$ is given by the equation:

$$
y^2_s - \left( \frac{L}{2} - 2e \right) y_s + 0.83 \ell^2 = 0
$$

where $\ell$ is the width of the beam seat measured from the face and $e$ is the eccentricity calculated from the earth retention computation.

**Earth Retention Stresses**

To the moment produced by the earth pressure and the horizontal force at the top of the structure are added the moments arising from the shifting of the diffused bridge loads as follows:

$$
M = \lambda \frac{\gamma A^2}{\pi} \left\{ p(1+p) \left( \frac{E}{2} - \arctan p \right) \right\}
$$

where $p = y/a$, and $y \leq y_s$.

Under the effect of these moments, the resultant of all of the vertical loads (except those which are distributed) is characterized by an eccentricity $e$. This corresponds to a uniform vertical stress:

$$
\sigma_{ii} = R_x / (L - 2e)
$$
Superimposition

An envelope of the total vertical stress (Fig. 21) is defined by:

\[ \sigma_1 = \sigma_{11} + \sigma_{12}(x) \]

Potential Failure Lines

In current analyses, the first potential failure surface drops vertically from the center of the beam seat and intersects the facing at a point located at depth 2\(a\). The second potential failure line is analogous to that observed in a retaining wall, or it passes by the heel of the beam seat.

Stresses in the Reinforcements

On the first or second potential failure lines, the tensile stress in the reinforcements, distributed as N per m², is given by:

\[ T = K (\sigma_{11} + \sigma_{12}(x))/N \]  

(Fig. 22a). On the first line, \( \alpha \) equals 1.0 at the center line of the beam seat.

At the facing, \( \alpha = T_f / T_{fs} \); i.e., the ratio used at the same strip level in retaining walls. Over the remaining portion of the line, \( \alpha \) is interpolated. On the second line, \( \alpha \) is always equal to 1.0.

At the facing, \( T_f = K (\sigma_{11} + \beta \sigma_{12}(x))/N \).  

The coefficient \( \beta \) equals 0.85 under the beam seat and increases to 1.0 below a depth of \( 2a \).

\( K \) varies between \( K_s \) at the surface and \( K_n \) at a depth of six meters (Fig. 22b).

For reinforcements located at a depth \( y \) below the beam seat, where \( y \) is less than \( L \), the tensile stresses are increased by:

\[ \Delta T = 2F \left( 1 - \frac{y}{L} \right)/N \]  

(Fig. 23).

Sections of reinforcement are checked at both the gross section and at the net section with due consideration to service life design.

Adherence

Adherence is checked for each of the two potential failure lines, one and two, and for the corresponding level of tensile stress, \( T_{fs} \), and \( T_{sl} \). The computation consists of verifying by an integration over the length of adherence \( L_{ad} \) (or \( L_{es} \)) that:

\[ T_s \leq T_{fs} = \frac{1}{2b} \int^{y_0}_{y_0} T(x) \left( y + q_s + \sigma_{12}(x) \right) dx \]

For high-adherence reinforcements, \( T \) varies along the reinforcement and in relationship with \( \sigma_{12} = \gamma y \sigma_s + \sigma_{12}(x) \), between \( T_{fs} = 1.5 \) for \( \sigma_{12} = 0 \), and \( \tan \phi \), for \( \sigma_{12} \geq 1.20 \) kPa.

It should be noted that one design loading condition that may have a decisive effect on adherence conditions is that in which the backfill placement has only reached the level of the top of the beam seat while the total permanent load of the bridge is already in place.
Essentially, there are two types of Reinforced Earth bridge abutments: closed abutments with return walls, and open abutments with wing walls. The choice between these two types depends primarily on site conditions and constraints.

Closed Abutments

Return walls are required when the access ramp to the bridge is confined by long retaining walls. If this type of abutment is also chosen for shorter return walls, constraints will be imposed on the installation, including several successive foundation levels for the return walls and a delayed completion of the top portion of the return walls until the beam seat and its cheek walls are completed.

Open Abutments

Wing walls may be collinear with the abutment itself, they may curve or angle slightly inward (Fig. 26), or they may be oblique (Fig. 27). The entire structure, including wing walls, is generally founded at the same level, and is built in a single phase prior to construction of the beam seat. No special equipment or coping is required at the top of the walls, which are topped out using special panels cast with sloping top edges. On the other hand, the extremities of the beam seat must be well protected against erosion. An additional benefit of open abutments with collinear wingwalls is that their configuration easily allows for bridge widening should it become necessary in the future.

Skewed Bridges

Skewed bridges involve several design peculiarities. Rather than excessively skewing the reinforcements from a position perpendicular to the wall face and placing the backfill within a highly restricted sharp angle, it is preferred, where possible, to offset the return wall somewhat and truncate the point of the angle. When site conditions allow construction of a wing wall, the optimum solution is the projection of the slope of the embankment, with the reinforcements being angled only slightly from perpendicular to the wall face (Fig. 28).
Width of the Beam Seat

Generally, reinforced concrete beam seats are dimensioned so that the contact pressure imparted to the Reinforced Earth mass will be as uniform as possible and will be less than 150 kPa under permanent loading conditions. Furthermore, the centerline of bearing should be located at least one meter from the facing. These rules are good engineering practices that result in a negligible amount of settlement under the beam seat. In finite element analyses, the amount of settlement under these conditions, with the normal density of metal reinforcements, is on the order of 7.5 mm (Fig. 29).

Backwall

For large bridges, either in terms of span or traffic volume, a backwall that may include the stationary part of the bridge deck expansion joint is incorporated into the beam seat (Fig. 30a). For others, where this joint is not necessary, the beam seat consists of a simple thick concrete slab, and the bridge deck is isolated from the embankment by a cast-in-place end section at the end of the bridge beams (Fig. 30b).

Approach Slab

Reinforced Earth abutments do not require approach slabs. In fact, there is no differential settlement between the approach embankment and the bridge deck, since the latter is supported by the embankment itself. At the most, a short approach or transition slab is sometimes provided on large girder bridges to accommodate any small settlement that may occur in the fill behind the backwall itself (Fig. 31).

Jacking Recesses

As with any other type of abutment, spaces are normally provided between the beam seat and the bridge roadway for lift jacks in anticipation of maintenance work on the bearing pads.

For abutments built on highly compressible soils, the jacks also make it possible, if necessary, to compensate for secondary settlement due to consolidation of the foundation soils.

It should be noted that if the jacks are located in front of the normal centerline of bearing of the bridge, jacking will constitute a special loading condition that must be taken into account in the design calculations of the structure.
Before examining how Reinforced Earth abutments make it possible to accommodate settlement of the foundation soils, precaution must be taken to minimize deformations in the structure itself following placement of the superstructure and its resulting movements within the beam seat.

Differential settlement of the beam seat, accompanied by rotation, could result in damaging distortion of the bearing pads and the expansion joint. However, there is no danger of this condition occurring if the fill has been properly compacted.

As previously discussed, when the Reinforced Earth volume has been properly compacted, the amount of settlement under the beam seat usually will not exceed a few millimeters under normal design situations.

Compaction and Selection of Backfill

In order to limit the amount of settlement in an abutment to negligible levels, it is necessary to follow precisely the standard specifications for the selection, placement, and compaction of backfill materials used in the construction of embankments built beneath roadbeds.

Without going into unnecessary detail regarding these specifications, it should be noted that water-sensitive backfill materials must not be placed during rainy weather, or used if they are excessively wet when delivered. Conversely, provided they are not rejected in the first place, if these materials are too dry they must be moistened as needed and vigorously compacted.

In the area immediately behind the facing and directly beneath the beam seat, where only lighter compaction equipment can be used, it is recommended that smaller backfill lift thickness be specified. In addition, the use of subgrade-type backfill, which provides the additional advantage of good drainage, is advisable as a distribution layer beneath the beam seat (Fig. 32).

Drainage

It is imperative that special care be taken in the collection and removal of water that can penetrate the expansion joint or filters at the point of contact between the embankment and the backwall or roadway. Water must be prevented from seeping under the beam seat, where it could cause settlement or subsidence after saturation, or leach and wash away fines.

Therefore, the beam seat design must include the necessary slopes and gutters. In some cases drains must be installed (Fig. 33). The design and proper maintenance of drainage outlets must always be provided. Ideally, outlets should be located outside the structure, easily accessible for maintenance purposes.

Protection against water is also very important during each phase of construction, particularly during the phase when the bridge deck has been placed on the beam seat, but neither the approach embankment nor the surface drainage system have been constructed.

Figure 32: Layers of materials selected and compacted under the beam seat.

Figure 33: Typical drainage details.
In constructing Reinforced Earth bridge abutments, the compressibility of the foundation soil is first assessed relative to the bridge itself. Pertinent factors include the weight of the superstructure relative to the size of the abutments, the degree of rigidity of the structure, and the criticality of the scope of the project. Another important factor is the rate at which the soil will consolidate, considered in terms of the overall project schedule.

Under good conditions, abutments and the superstructure can be constructed together without phasing the work or taking special measures to improve soil properties. However, an identical site with a different type of bridge may require special considerations and procedures. In effect, every project is unique.

Direct Construction

Light Bridges: Reinforced Earth abutments distribute loads exerted by the bridge. Therefore, when an abutment is high relative to the length of the span, as at Val d’Esnons in France, pressure on the foundation is derived primarily from the weight of the backfill. If a sufficient period of time is allowed for the settlement and consolidation of the foundation due to the weight of the Reinforced Earth mass prior to placement of the beam seat and superstructure, only the Reinforced Earth structure and its facing will be affected. Both the structure and its facing have a relatively high tolerance for such deformation.

At Vallon des Acacias in Nice, abutments 17 meters high were built askew to an old river channel filled with compressible alluvial deposits. These abutments settled 40 to 70cm and experienced 1.5 percent differential settlement without damage. The 9.5 meter roadway spans were installed several months later and exhibited no significant movement.

Ordinary bridges: Many other structures of more common dimensions—where the weight of the superstructure is relatively greater—are built in the same way. For single-span structures, residual settlement levels of several centimeters are generally allowable.

The structure built at Antoing, Belgium, is a typical example. The foundation soil, made up of five meters of loose, clayey sand, quickly settled 65mm under the weight of the Reinforced Earth structures (180 kPa) (Fig. 34). In the second phase, construction of the superstructure increased the loading on the foundation by a little more than a third, and caused additional, very homogeneous settlement of 25mm. This was the only settlement to affect the superstructure. The record of settlement that occurred prior to installation of the superstructure made it possible to refine the estimate of final settlement levels, and to use this estimate in designing the supports. Such refined estimates are usually one of the major advantages of this method of phased construction.

Bridges built in this manner on Reinforced Earth abutments, taken together with structures built on good soils, make up about 90 percent of all projects.
Special Procedures

Where the bearing capacity of the soil does not allow for support of the total load imposed by the abutment, special procedures are required. In such cases, the danger exists for a deep slippage or a bearing capacity failure. Special measures are also called for when the anticipated amount of residual settlement under the abutments is incompatible with the longitudinal structure of the bridge, as is generally the case with multiple, continuous-span bridges. Nevertheless, certain flexible slab-bridges, like those forming the overpass bridges of the Dijon-Geneva Highway in France, easily tolerate settlement levels of several centimeters at their end supports.

Special steps are also needed when the soil quality is so inconsistent that it creates a risk of causing unacceptable levels of residual differential settlement of the abutment perpendicular to the roadway, thereby throwing the supports out of level.

Certain measures are also required when the level of delayed settlement is exceptionally high and at the same time rather uncertain, or too slow to enable control of the project or its longitudinal profile.

Most often, the methods used where Reinforced Earth abutments are built on highly compressible soils involve ground improvement techniques to reduce foundation soil compressibility or predrying of the structures prior to superstructure installation in order to anticipate the deformations that will occur.

Soil Improvement Techniques

Substitution is by far the most common method used to improve foundation soil. This involves replacing the existing surface soil to the depth of a few meters—where the most compressible soil often lies—with a good, compacted fill.

An overpass section of the Ring de Kortrijk in Belgium was built on Reinforced Earth abutments at a site where the very thick Flanders clay had been covered with four meters of clayey hydraulic soil (Fig. 35). This layer, in which the peak resistance was as low as six bars, was replaced, resulting in a settlement level of only 50mm during construction of the abutments and 35mm after placement of the superstructure.

Under more exceptional circumstances, the foundation soil is reinforced ahead of time using a system of ballasted “stone columns” set in place by a large vibrator. For example, design of the Grossbliederstroff Bridge crossing the Saar River at the French-German border called for a 29-meter span supported on six-meter high Reinforced Earth abutments (Fig. 36). In this case, the superstructure is heavy relative to the weight of the abutments, so that most of the foundation soil settlement would ramify through the superstructure. Soil at the site consisted of about ten meters of clayey alluvial deposits of mixed quality, which could be expected to settle 15cm or more. In addition, the site did not offer adequate security against a major slide failure. By incorporating stone columns into the design ~80cm in diameter, ten meters in depth and spaced every 1.5 meters—stability of the bridge structure was guaranteed, and settlement was limited to 5cm.
Preloading

Preloading is another classic method of improving foundation soil. Three types of preloading can be used in the construction of Reinforced Earth abutments on compressible soil.

Preloading Exerted by Reinforced Earth

This variant is used in cases such as those described above in which structures are built directly and without prior treatment. The site is preloaded by the Reinforced Earth embankment itself before the superstructure is set in place.

Preloading Using Ordinary Fill

The second, more traditional method involves preloading the site with an ordinary fill intended to remain in place only temporarily. This method is used when the surcharge must be put in place quickly, or when the levels of consolidation and settlement during this phase would exceed the permissible facing deformation level. This technique was used near Champlain in Canada for an overpass of highway A 40 (Fig. 37a). The 76 meter bridge, consisting of three independent spans, is supported by two Reinforced Earth abutments built on 24 meters of compressible clay. Because the anticipated level of settlement was 3.0 to 4.5 m, a temporary surcharge 6.4 meters thick was put in place two years before construction of the abutments. When the settlement level had reached 25 cm the temporary surcharge was partially cleared away and replaced with Reinforced Earth structures. Five years after construction of the bridge, the abutments had dropped about three centimeters, causing no problems.

Combined Preloading

A third method consists of precharging the site simultaneously with the final Reinforced Earth structure and a temporary topping of fill or concrete blocks. The topping is designed to stand in for part or all of the future superstructure's weight. Consolidation is thereby accelerated and, if the construction schedule permits, final levels of settlement and deformation of the Reinforced Earth abutment can be reached before the roadway is built. This method was used for a bridge in Rocquencourt, France, with concrete blocks providing the surcharge. The site consisted of eight to ten meters of uncompacted fill which consolidated quickly. On the Route

One bridge over the Boston and Maine Railroad in the United States, built on 40 meters of relatively loose sand and wet clay of average consistency, 3.3 meters of additional fill were enough to produce one-third (22 cm) of the anticipated long-term settlement (Fig. 37b). The flexible nature of this independent metal span structure is expected to adapt well to the remaining settlement.

Whichever method is used for building Reinforced Earth abutments on compressible soils, the purpose should always be to optimize the use of Reinforced Earth's inherent flexibility. Additional measures are employed only when necessary.

Every Reinforced Earth structure built on compressible soil constitutes a new and unique project. In designing each project, company engineers take into account the nature of the superstructure, scheduling requirements and deadlines, geotechnical data, and the flexible possibilities of Reinforced Earth.

Figures 37a, 37b: Preloading for the Champlain and Boston and Maine railroad bridges.
There are certain cases in which a foundation soil that is too compressible for the proposed structure (e.g., a large continuous viaduct) cannot be sufficiently improved to result in acceptable limits of consolidation. In such cases, a mixed solution can be adopted in which the beam seat and bridge deck are supported by piling or other special foundations, and the embankment is retained by Reinforced Earth. This solution makes the support structures nearly independent of the embankments. Also, by building the Reinforced Earth structure in advance, the amount of negative skin friction on the piling can be reduced. In the case of a mixed abutment where the bridge is no longer supported by the Reinforced Earth structure, a conventional approach slab is used as a transition.

**Mixed Abutments with Exterior Supports**

In these cases, piers are usually placed in front of the Reinforced Earth structure, either in the form of a row of columns capped by a cap beam, or as a full pier that runs along the face of the approach structure. This arrangement reduces the span of the bridge to a minimum and, in effect, separates the construction of the two parts of the project.

The distance to be maintained between the pilings (or pier) and the Reinforced Earth structure depends primarily on either the width of the foundation, or on the clearance required by the construction equipment. When the distance is small, the Reinforced Earth facing is connected to the cap beam or to the end wall of the beam seat (Fig. 38a). Clearances and overlaps are provided as needed, so that any possible subsequent movement caused by consolidation of the foundation soil will not result in contact or loss of fill. When the distance is large, the transition slab assumes the form of a small connecting span over this area (Fig. 38b). All aspects of the connection require detailed study, particularly at the shoulders and edges of the embankment.

**Mixed Abutments with Interior Supports**

For aesthetic reasons, pilings are sometimes embedded within the Reinforced Earth structure. There are two instances where this is a customary practice. In the first, pilings are driven prior to construction of the Reinforced Earth embankment, requiring that backfill be carefully placed and compacted around each column. In the second instance, pilings are driven through the Reinforced Earth structure after the latter has been completed. In both cases, sleeves are used to separate the supports from the fill, leaving sufficient clearance to prevent any transmission of lateral load. The sleeves also make it possible to drive the pilings without cutting or damaging the reinforcing strips.

**Pier Abutments**

A disadvantage of mixed abutments with interior support is that the pilings must be placed relatively far behind the facing, thereby lengthening the bridge span. When the foundation soil is good, a special type of mixed abutment with interior supports, called

**Combined abutment of the Epron Bridge, National Highway RN 57 Beaugé-Épinal, France.**

**Figures 38a, 38b:** Typical details of the connection between an exterior pier and the Reinforced Earth structure.
a "pier abutment", makes it possible to set the supports closer to the facing (Fig. 39).

In these cases, concrete piles are poured directly into blockouts cast into specially made facing panels. At present, these special facing panels are prefabricated in two pieces and assembled at the site. These piles transmit the bridge loads through the reinforced volume to a footing below the structure. The piles are poured only after the Reinforced Earth structure has been completed and the embankment has been backfilled. The reinforcements will then be in tension and any deformations will have occurred.

After the concrete has been poured, the special panels are made integral with the concrete piles. That is why this type of structure is suitable only for sites with good foundation soils, where the flexibility of the facing is no longer indispensable once construction is complete. Subsequent settlement of the foundation would create the possibility of secondary stresses in the concrete piles through interaction with the reinforcements and the facing.

To accommodate horizontal stresses from the roadway without transmitting these stresses to the piling, additional reinforcements are either placed above the piles or attached directly to the cap beam. Since pier abutments are used only on good foundation soils, approach slabs are usually not required.
Reinforced Earth technology has made possible new and economical designs for bridge abutments adaptable to the widest variety of superstructures and foundation soils. Complementing the performance of engineering techniques that have undergone more than 15 years of research and development are all the advantages of quick and scaffold-free construction. The appearance can take many forms. Each year, over 120 bridges are built on Reinforced Earth abutments.

A WORLDWIDE ORGANIZATION

Licensed under the patents issued to Henri Vidal throughout the world, the Reinforced Earth Group of companies operates in 33 countries on six continents. Although part of the same Group, each company is independently managed by nationals of that country who are professional engineers that understand local conditions, codes of practice, and construction capabilities and techniques. Research and other technological activities among the different companies are coordinated from Paris, France, by Terre Armée Internationale, parent company of the Group. For new applications and special or unusual projects, Terre Armée Internationale can pool the resources of several companies to create optimum project designs and material specifications. It also acts as the central technical service organization, and maintains the primary information database collected from new applications, special projects and research.

Terre Armée Internationale takes the lead in organizing and developing research projects, both under its own direction and through coordination among the other companies. Analysis and synthesis of research and technical data by Terre Armée Internationale results in technical recommendations and design improvements published in routine reports disseminated to all the Reinforced Earth Group companies.

The dynamics of this organization allow each Reinforced Earth company to offer government agencies, owners, consultants and contractors the understanding and flexibility of a local business, combined with the vast resources and technological advantages of a global concern.