The influence of unstable submerged river bank slopes on large caisson foundations in the Jamuna River

Victor A. Sowa, PhD., PEng
Jacques Whitford and Associates Limited, Vancouver, B.C.

Abstract: Much of the country of Bangladesh in the Indian Sub-Continent is a huge delta formed by the Jamuna-Ganges-Meghna River system. A 230 kv electrical transmission line was required to cross the Jamuna River which is a large migrating braided river that is up to 20 km wide during the flood season. The transmission line crossing of the Jamuna River at a location where the river is about 10 km wide consisted of 11 transmission towers, 90 m high and 1,220 m apart. Each tower is supported on a single large 10.7 m diameter caisson with the caisson depths ranging from 91 to 112 m. The river delta soils in the vicinity of the transmission line crossing are erodible and essentially the entire height of the eroding river banks are submerged during the flood season. The submerged river banks that are comprised primarily of clayey soil extend to greater depths than the river banks comprised of sandy soil. A significant problem occurs when a river bank is being eroded adjacent to a caisson and the unstable submerged river bank slope imposes a large lateral load on the caisson. The large lateral river bank load was a major design issue, and the solution proposed to minimize this problem is presented.

Introduction

The Brahmaputra-Jamuna River is one of the largest rivers in the world. It rises in Tibet on the northern slopes of the Himalayas and flows eastward through Tibet and turns southward to cross the Himalayas into Assam (India), then flows westward in Assam before turning to flow southward to enter Bangladesh. The Brahmaputra-Jamuna River then flows generally in a southerly direction discharging into the Bay of Bengal.

Along the way the Brahmaputra-Jamuna River is joined by the Teesta River, the Ganges River, and the Meghna River. The river system in Bangladesh is known by various names in different parts of the country and is called the Brahmaputra-Jamuna-Ganges-Padma-Meghna River. For the purposes of this paper the area of interest is in the vicinity of the confluence of the Jamuna River and the Ganges River, and the river will be referenced simply as the Jamuna River.

The Jamuna-Ganges river system, in terms of the volume of flow during the flood season, is the second largest river system in the world. The maximum recorded flow for the Jamuna River is 92,300 m^3/s (3.3×10^6 cfs) and the maximum flow for the Ganges River is 73,000m^3/s (2.6×10^6 cfs).

The Jamuna River system essentially divides the country of Bangladesh into two; the western and the eastern regions. The dividing river is a major obstacle that separates the two parts of the country and inhibits the development of the country in many ways. There has been a strong need to construct an electrical transmission line to cross the Jamuna River to connect the western and eastern portions of the country. As a result the 230 kv East-West Interconnector Project came into being, connecting Ishurdi in the west to Ghorasal in the east as shown on Fig. 1.

The Interconnector Project has many benefits. The main benefits included the transmission of considerably cheaper electrical power developed in the eastern part of the country to the western part, and the desire to connect the west and the east electrical grids to improve and to provide a more stable electrical grid system. In view of the benefits, the Bangladesh Power Development Board determined that engineering studies for the Interconnector Project should be undertaken.

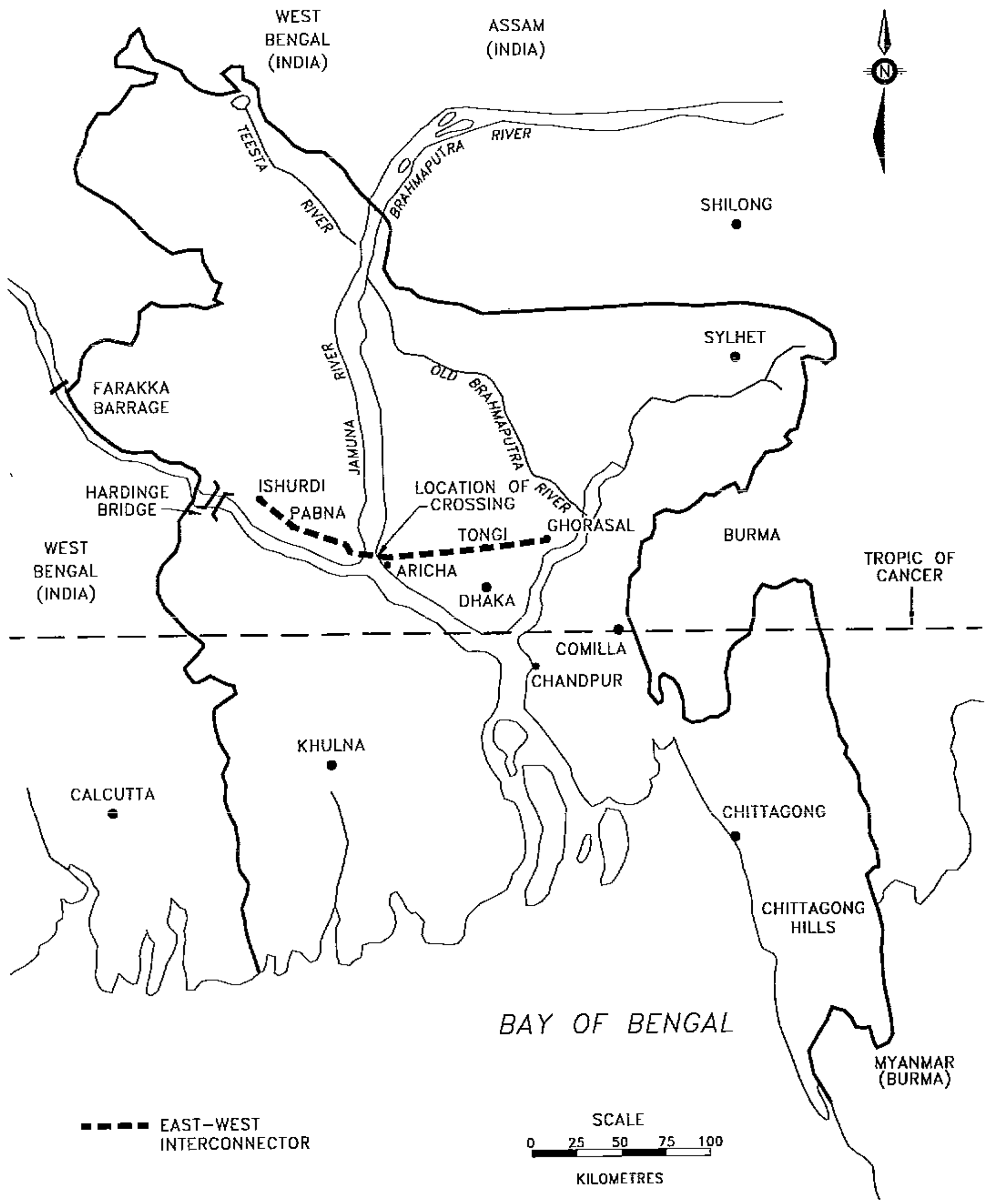
The main physical obstacle to constructing the East-West Interconnector was the crossing of the Jamuna River. At the time that this project was designed and constructed there was no bridge crossing of the Jamuna River in Bangladesh. Ferries were the only means of crossing the river, and even the railroads crossed the river using the ferries.

Engineering studies of the East-West Interconnector Project were undertaken at various times, but it was not until 1970 that a complete engineering study was undertaken by a consortium of Acres International Limited and Consulting Engineers (Pak) Limited on behalf of the Bangladesh Power Development Board. The results of this study are summarized in the 1970 Acres report listed in the references, Acres (1970).

The engineering study commenced in 1967 and was concluded in 1970 with the submission of a final engineering report and construction documents. The engineering study included the results of site investigations, primarily geotechnical, geomorphological, and hydrological, that were undertaken during the 1967, 1968 and 1969 seasons.

Shortly after the completion of the East-West Interconnector Project engineering study, Acres International Limited headed a consortium of engineering

Fig. 1. Map of Bangladesh and the location of the East-West Interconnector transmission line.



and technical advisors that were successful in their submission to serve as General Consultants to the then East Pakistan Water and Power Development Authority (EPWAPDA). EPWAPDA was the forerunner to the Bangladesh Power Development Board that was formed later to look after the power development in Bangladesh. EPWAPDA was responsible for all the water and power development in Bangladesh (then known as East Pakistan).

Acres International Limited, as the new General Consultants to EPWAPDA, could not continue their involvement with the East-West Interconnector Project during the construction phase, since as advisors to EPWAPDA, this would be a conflict of interest. As a result, the project would need to be taken over by other consultants.

During the process of selecting alternate consultants and obtaining funds for construction of the East-West Interconnector Project, political events created further delay to the project. West Pakistan and East Pakistan were one country that was formed during the partition of India after World War II. The western and eastern portions of Pakistan were separated by the entire width of India (about 1,600 km). The physical separation and the considerable differences in culture eventually proved too much, and East and West Pakistan separated into two separate countries, with East Pakistan becoming a new country in 1971 named Bangladesh.

The political events that accompanied the creation of Bangladesh, not surprisingly, delayed much of the development of the country as Bangladesh struggled in its new role. Eventually, the abnormally high price of fuel oil in the late 1970's that supplied much of the power for electrical generation in the western portion of Bangladesh proved to be too a large burden. The economics of proceeding with the East-West Interconnector Project became increasingly attractive. In 1979 the Bangladesh Power Development Board appointed consultants from Great Britain, namely Merz and McLellan; and Rendel, Palmer and Tritton to continue with the East-West Interconnector Project. Their terms of reference were to review and update the design to current standards, and to carry the project through to construction.

After a review of the project to ensure that the project was compatible with the current standards and the current site conditions, the new consultants made only relatively minor changes to the main physical elements of the Jamuna River crossing. The original Acres International Limited design called for a river crossing of the transmission line supported on 11 transmission towers spaced at 1,220 m. The 90 m high transmission line towers were to be founded on deep cantilever caisson foundations 10.7 m in diameter. The design depth of the caisson foundations ranged from 91 to 96 m, with one of the caissons extending to a depth of 112 m. The design after review by the new consultants called for a river crossing of the transmission line supported on 11 transmission line towers spaced at 1,220 m. The 111 m high transmission towers were founded on deep cantilever caisson foundations 12.2 m in diameter. The design depth of the caissons ranged from 91 to 103 m.

Construction of the East-West Interconnector Project based on the Rendel, Palmer and Tritton design was completed in 1982. Four technical papers describing the design and the construction of the East-West Interconnector Project appeared together in the same issue of the *Proceedings of the Institute of Civil Engineers* in 1984. The geotechnical aspects are covered in one of the four papers, namely the paper by Hinch et al. (1984).

The author of this paper was part of the original Acres International Limited design team and was responsible for the geotechnical design of the project. The design team undertook the field investigations and preliminary design in Bangladesh, and the final design and preparation of the contract documents were completed in Canada.

This article focuses mainly on two of the loads applied to the caisson foundations. The electrical transmission line towers are subjected to a variety of conditions and loads, including wind, river flow hydraulic forces, river scour, and lateral soil loads. The two loads considered are the lateral loads imposed by the river bank and the river bed sand waves. These loads were not covered in detail by Hinch et al. (1984). The geomorphology of the river delta development is also described to provide a background for understanding the river migration and the stability of the river banks. The following presentation is based on the results of the original engineering work undertaken by the author for the East-West Interconnector Project, and is based on the information available and analysis undertaken at that time. The presentation is based on the 1970 Acres design with the 10.7 m diameter caissons.

Location of the transmission line crossing of the Jamuna River

General

A suitable location for the transmission line crossing of the Jamuna River was recognized at the outset as one of the most important design issues to be resolved for the East-West Interconnector Project. The intersection of the shortest distance between the electrical substations at Pabna and Tongi with the Jamuna River would normally be the river crossing location of choice. However, it was recognized that there could be considerable departure from such a location if a more stable river crossing was identified.

The Jamuna River is a very wide braided river during flood. The main river channel between the "valley walls" can be as wide as 20 km. The rate of erosion of the river banks and the banks of the sand islands within the valley walls of this braided river can range up to 1,200 m in a single year, with most of that occurring during the flood season. Unless the crossing location is chosen carefully, the designers faced the daunting prospect that the river crossing could be by-passed a few years after construction.

In order to select the most suitable river crossing it was essential that a thorough understanding of the characteristics and the behaviour of the river be developed. This, in essence, meant understanding the geomorphology of the river and the formation of the river delta in Bangladesh. Most of the country of Bangladesh is comprised of the delta of the rivers. A description of the general geology and the geomorphology of the Jamuna-Ganges-Meghna delta is given in the following. Much of this description is based on the work undertaken by J. M. Coleman on behalf of the East-West Interconnector Project, and is summarized in Coleman (1969).

Migratory behaviour of rivers

For the most part, Bangladesh is comprised of the delta formed by the Ganges, Jamuna and the Meghna River systems. The Bengal Basin geological unit is located mainly in Bangladesh. Apart from isolated, higher-lying remnants of the old Pleistocene surface, the terrain is comprised of flat-lying recent alluvial sediments dissected by numerous tributaries and distributaries of the major river systems. The recent alluvial sediments are found at the Jamuna River crossing and along most of the transmission line route.

The rivers in Bangladesh are of the meandering or braided channel type and a characteristic feature of most of the rivers in Bangladesh is the tendency for rapid migration and meandering. The migration of rivers continues largely unabated since there is very little flood control and construction of river training works. At the outset, it was concluded that river training works to control the Jamuna River in the vicinity of the crossing would be uneconomical. The design of the transmission line crossing of the Jamuna River was based on the assumption that river migration would continue during the service life of the Interconnector Project.

The sediments transported by the river systems of Bangladesh consist primarily of sand, silt and some clay. These soils are also the constituents of the recent alluvial sediments. Being recently deposited, the sediments are of loose density. The great quantity of easily eroded sandy sediments which are deposited annually, combined with the large river flows, cause the rivers to constantly adjust the river bed configuration to accommodate differing flow regimes. This has resulted in the major river systems occupying and abandoning numerous river courses.

The rate of river bank erosion and therefore the rate of river migration is dependent on a number of factors, including volume of flow, river type, the direction of the river flow at any particular location, and the composition of the river bank sediments.

Assuming that the other factors are equal in importance, the erosion of the river bank proceeds more slowly when the banks are comprised of clayey soil rather than sandy soils. Furthermore, the greater the thickness of clayey strata in the river bank, the slower the rate of erosion. Since the rate of river bank erosion is significantly governed by the presence of the clayey soils, it is necessary to understand the basic processes underlying the deposition of these soils in the Bengal Basin delta.

Deposition of erosion resistant clayey soils

The rivers in Bangladesh overtop their banks annually during each flood season, and flood approximately 40 percent of the total land area. The capacity of the rivers to transport sediment is the greatest during the flood season. When the river banks are overtopped, the flood water carries the sediment in suspension. The finer the sediments, the greater the distance that the sediments are transported over the flooded land area. The fine to medium size sandy soils are deposited in the river channels, and the finer sandy and silty soils are deposited on the river banks forming the natural river levees. The finest silts are deposited further from the banks, and the cohesive clayey soils are deposited the greatest distance from the rivers. A fresh deposit of floodwater alluvium is deposited annually following each flood. Eventually, if other factors remained static, the elevation of the ground surface would build up to a level corresponding to the highest flood level.

The total depth of the alluvial sediments in Bangladesh in the project area is greater than 3,000 m. Consolidation of these sediments is occurring on a continuing basis and could result in the accumulation of thick deposits of cohesive clayey soils. The migrating rivers, however, usually erode the cohesive clayey sediments before they become very thick, and replace these sediments by coarser-grained sandy soils transported in their channels.

The thickness of the cohesive clayey soil strata along the overland portion of the transmission line route for this project generally did not exceed 6 m, indicating that river migration interrupts the sedimentation process before an additional thickness of clayey soils can accumulate. As frequently occurs, there are exceptions to the idealized concept, and considerably thicker strata of cohesive clayey soil were found. The thickness of cohesive clayey soil at one location along the east bank of the Jamuna River near the crossing at Aricha is approximately 28 m.

Soil stratigraphy - Jamuna River crossing area

Fig. 2 shows the location of the transmission line crossing of the Jamuna River, the tower and caisson foundation locations, and the river channels for the Ganges and the Jamuna Rivers. Fig. 2 illustrates the location of the "river valley" and "river valley walls" for the braided Jamuna River. The western and eastern river valley walls at the crossing are shown on Fig. 2, and the river valley spans between the valley walls. The river valley is considered to be that portion of the delta within which the river has flowed since occupying its present course.

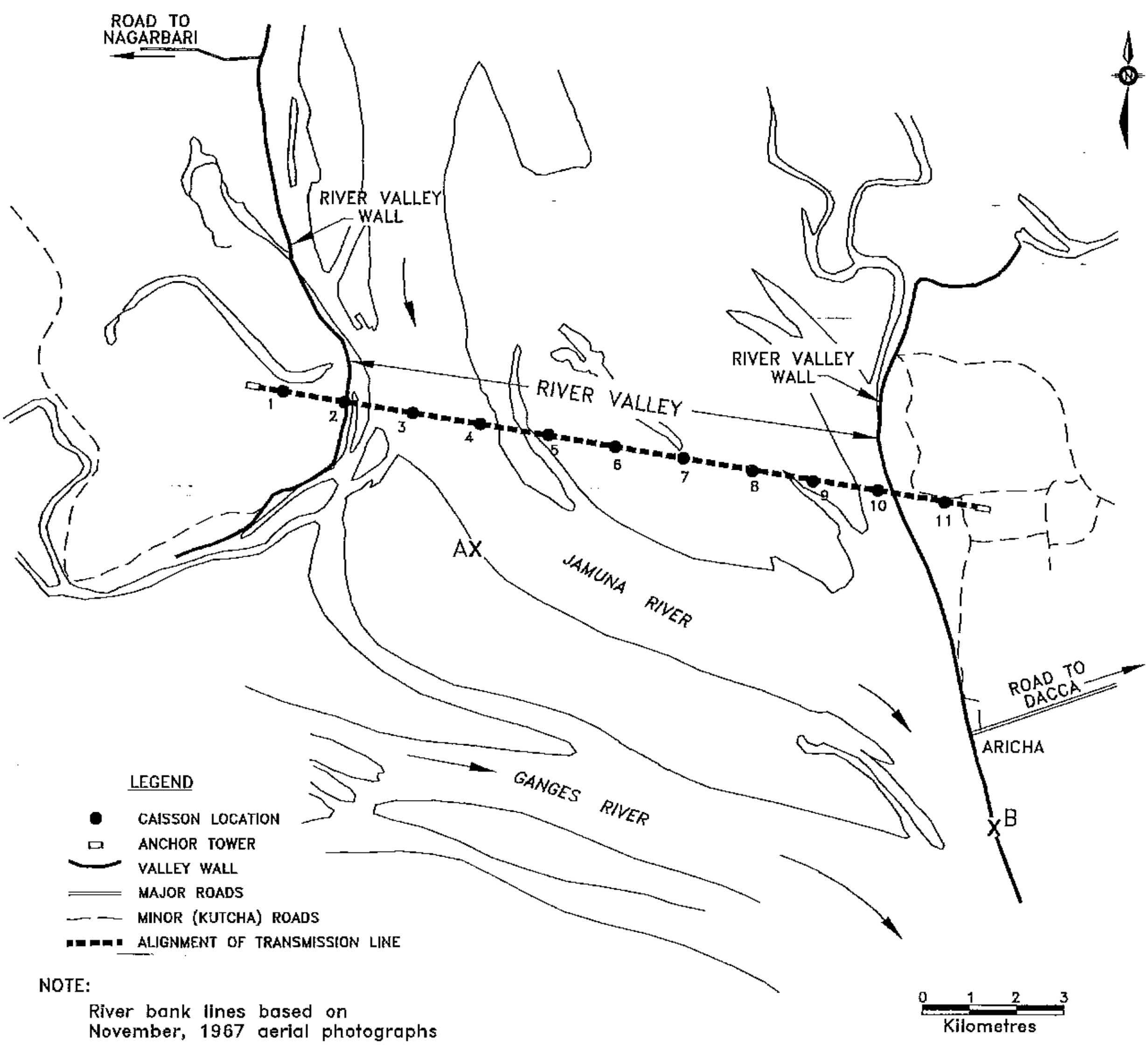
The basic soil stratigraphy at the river crossing consists of clayey soil at the valley walls, and sandy soil in the valley within the valley walls. The thickness of the upper clay in the valley walls is variable, typically 9 to 18 m, and the clay is underlain by sand. The sandy soil condition is applicable to the sand islands in the valley between the valley walls. The upper sands in the sand islands may be siltier, but they are underlain by cleaner sand.

The term "clay bank" or "clay bank condition" refers to a river bank with the upper portion comprised of clayey soil. The term "sand bank" or "sand bank condition" refers to a river bank comprised almost entirely of sandy soil.

The sediments in the river valley to within the depth of the river scour have been reworked and redeposited very recently. Since these sediments are of loose density and sandy, they are readily erodible. The sediments comprising the river valley walls, although still considered as recent sediments in geological terms, are several thousand years older than the very recent sediments within the river valley. The results of radio-carbon dating undertaken on two samples of organic matter obtained in the Aricha area at depths of 18 and 90 m indicated the matter to be 2,100 and 29,000 years old respectively.

The presence of the clayey soil in the river valley walls increases the erosion resistance considerably and since the

Fig. 2. Jamuna River Transmission Line Crossing Location.



river is restrained in moving laterally, the energy of the river is directed downwards in eroding a deeper channel.

A feature of a river valley of a huge braided river such as the Jamuna River, is that the river valley includes large sand chars (sand islands or sandbars) which can be up to several kilometers in size. The sand chars, although having the appearance of being relatively permanent land because of intensive cultivation and the presence of large villages, are actually of a temporary nature. In many cases, a sand char does not remain at the same location more than a few years before being eroded.

Selection of the Jamuna River crossing location

Once the geomorphology of the Jamuna River was better understood, the identification of the river valley walls was undertaken. The river valley walls were defined for a significant reach of the river, extending from Aricha, located as shown on Fig. 2, northward for about 50 km. A significant reach of the river was considered because of the importance of selecting the shortest river crossing in the most stable location to minimize costs.

The river valley walls were identified primarily from a series of aerial photographs taken at various years over a period of time. The data obtained from the aerial photographs was supplemented with surveyed river bank line maps, and river navigation maps, some dating as far back as 1830.

Evaluation of the data revealed that the distance between the river valley walls ranged from about 10 km at locations near Aricha, and up to 20 km at other locations north of Aricha. The rate of sand bank erosion within the valley walls ranged up to 1,200 m annually. The very large erosion rate is due to the low erosion resistance of the sandy soil comprising the sand banks in very actively eroding channels. The annual rate of erosion of the more clayey valley walls subjected to direct river attack ranged from about 15 m to about 60 m. The lowest rates of annual valley wall erosion were found within the Aricha area.

The preceding evaluation revealed that the shortest river crossing between valley walls with the lowest rate of annual river bank erosion was located in the Aricha area. The final Jamuna River crossing location for the transmission line is that shown on Fig. 2. The estimated average annual rates of erosion of the west and the east river valley walls were 56 and 28 m respectively, but the actual annual rates may be less. River erosion is not expected to affect the western and the eastern anchor transmission line towers for approximately 40 years or more.

Configuration of the Jamuna River crossing

The Jamuna River is a large river with deep natural river scour and the river valley is about 10 km wide at the crossing location. It was recognized that the foundations for the transmission line towers would need to be deep and spaced relatively far apart for reasons of economy and to minimize the total number of tower foundations. Various foundations for the towers were considered, including deep piles, and large and small diameter gravity sunk caissons. The loading conditions on the towers and the foundations, and the river scour conditions influenced the final selection of the tower foundations.

The natural river scour is very deep for a river of the size of the Jamuna, but in addition, local scour occurs around a foundation that adds to the total scour depth. The larger the foundation width, the greater the local scour, and the larger the corresponding total hydraulic drag force on the foundation from the flowing river. A large diameter caisson usually depends primarily on end bearing for stability. The large diameter caisson was proving to be uneconomic so the approach was reversed. A smaller diameter deeply embedded cantilever caisson foundation that creates smaller local scour was considered. The smaller diameter caisson depends primarily on deep embedment and lateral support for stability. The deep smaller size diameter caisson proved to be the most suitable foundation solution.

The final crossing configuration for the 1970 Acres design consisted of 11 transmission towers spaced at 1,220 m apart. The towers were 90 m high and were supported on 10.7 m diameter deep caisson foundations. The maximum combined design river scour and local scour around the caisson foundations within the river valley was 50 m. The required depth of caisson embedment within the sand foundation soil was 41 m, which resulted in caissons 91 m deep. The caissons located on the valley walls would be subject to greater natural river scour because the clay banks drive the river scour deeper. The caissons located within the valley walls were designed for a total maximum scour, including local scour, of 54 m, which required caissons 96 m in depth.

The eastern most caisson foundation on the east valley wall was located within the deepest clay bank which resulted in even deeper river scour. For the most likely severe angle of river flow attack on the river bank of the east valley wall, the estimated total scour, including local scour, was 68 m which required a caisson 112 m in depth. This caisson, which was significantly deeper and extrapolated the previous construction experience even further, presented a construction risk since it might not be feasible to attain the required depth. An alternative approach was presented which permitted construction to a shallower depth of 96 m. The alternative of using a 96 m caisson was a future operational risk since it required dumping of riprap into the scour hole around the caisson during flood.

The significance and the magnitude of the dimensions given in the preceding text is perhaps difficult to comprehend without making reference to familiar physical features and landmarks. The following references are for those who are familiar with the city of Vancouver. The width of the selected Jamuna River crossing for the transmission line between the valley walls is about 10 km. This is comparable to a river width that is equivalent to the straight line distance from the north end of the Oak Street Bridge to the West Georgia Street entrance to Stanley Park in downtown Vancouver.

The Alex Fraser River Bridge (Annacis Island Bridge) is a cable-stayed bridge in Greater Vancouver which spans the main channel of the Fraser River. The main towers of the bridge support a main span of 465 m. The distance between the Jamuna River caisson supported towers is 1,220 m, which is more that 2.5 times the length of the main span of the Alex Fraser Bridge. The design depth of the caisson foundations ranged from 91 to 112 m, and for comparison, the height of the main Alex Fraser Bridge towers above the bridge deck is about 95 m. The maximum total scour depth around the caisson foundation is 68 m, and for comparison, this exceeds the maximum height of the Alex Fraser Bridge deck above the river which is about 60 m.

Construction of the caissons

The caissons were constructed by sinking under their own weight as the foundation soil was excavated from inside the caissons. The construction procedure consisted of constructing the lower portion of the caisson foundation, excavating inside the caisson to allow the caisson to sink under its own weight, and then construct another section of the caisson. The process of constructing sections of the caisson, alternating with excavation inside the caisson to sink the caisson, continues until the required depth of the caisson was obtained. Bentonite mud fluid was circulated along the exterior wall of the caisson to reduce the skin friction to aid the sinking of the caissons.

The overall plan was to construct as many of the caissons as possible, using a land-based operation to construct the caissons on the sand chars (sand islands). For the few caissons located in the water between the sand chars, artificial islands were constructed during the low water season to serve as construction platforms.

Loads on the caissons

The caisson foundations are subjected to a variety of loads. One of the main loads on the transmission line towers and foundations is that due to wind, and Bangladesh experiences tropical hurricanes. Other loads on the tower foundations include the hydraulic drag load caused by the river during flood, storm waves, lateral river bank loads, underwater river bed load sand waves, and earthquake loading. Various load combinations were evaluated to determine the most critical combination relative to the caisson stability and structural capacity. Not all loads were considered to be applicable simultaneously.

Only two of the loads on the caissons are considered in this article. The loads are the lateral load imposed on the caisson from the river bank as the river bank is eroded by the river, and also the load imposed on the caisson as the migrating river bed load sand wave moves past the caisson.

River bank failure loads on the caissons

General

The caisson foundations are subjected to a variety of loads. One of the loads is the lateral load imposed by the unstable submerged river bank slope on the caisson as the eroding slope fails and this load is considered in the following.

Erosion and undercutting of the submerged river bank slopes causes the banks to fail and slide into the river. Observations during flooding indicated that the banks either fail as numerous small blocks of soil sliding into the river, or as a large mass of soil sliding as a single unit. The sliding soil mass can range from soil blocks a few cubic metres in size, usually characteristic of the banks comprised of sandy soil, and up to soil blocks 3 to 15 m wide and 15 to 60 m in length, for banks comprised of clayey soil.

The presence and the benefit of the clayey soil at the river crossing is, to some extent, a contradictory benefit. On one hand the greater erosion resistance of the clayey soil is a beneficial feature that was the main reason for selecting the Aricha crossing. On the other hand, this feature is also a liability because the presence of the erosion resistant clayey soil also allows the development of much steeper and deeper submerged river bank slopes than would develop in the case of sandy soils. The important benefit of the more stable river banks and the shorter crossing at the Aricha location by far exceeds the problem created by the lateral river bank load on the caisson. As a result, it was essential that the issue of the lateral river bank load on the caisson be resolved in some practical manner.

Evaluation of the lateral river bank load on the caisson foundation during erosion requires information on the shear strength parameters, the boundary conditions, and selection of a method of analysis.

The shear strength information was obtained from the results of the site investigation and laboratory tests. Additional information on the shear strength and the boundary conditions was obtained from stability back analysis of the river bank slopes that are on the verge of failure with a factor of safety of 1.0 (unity). Both the shear strength parameters and the stability back analysis of the failing slopes are considered in the following.

Selecting a suitable method of analysis proved to be problematic, partly because this problem does not fit into one of the more conventional analyses, and partly because the actual mode of failure is not known. The process of analyzing the stability of slopes is well established. However, a slope stability analysis approach is probably not valid when the size of the caisson embedded in the slope is similar to the height of the river bank slope. Similarly, using an earth pressure method to estimate the lateral load on the caisson as for a retaining wall is also not appropriate because of the size of the caisson compared to the slope. The basic problem in analyzing the lateral load on the caisson foundations is that actual mode of failure of the river bank adjacent to the caisson is not known for conditions applicable for the size and type of river such as the Jamuna River.

In view of the difficulty in defining a suitable failure mode and in selecting a suitable method of analysis, the procedure finally used and described later was an adaptation of a method proposed by Hansen (1961) for determining the lateral bearing capacity of piles.

Stability back analysis of unstable river bank slopes to establish soil parameters

The banks of the Jamuna River erode and fail on a continuing basis during each flood season. Since the factor of safety of a river bank on the verge of failure must be marginally less than unity, it is possible to select an appropriate combination of soil parameters and boundary conditions for this condition. The same soil parameters can then be used for analyzing the river bank loading on the caisson foundation.

Stability back analysis of river bank slope failures was undertaken for two locations on the Jamuna River, Locations A and B, as shown on Fig. 2. These two locations are representative of the range of bank conditions likely to be encountered at the caisson foundations. Location A is representative of a "sand bank" slope where the river bank slope is comprised primarily of sandy soil. Location B is representative of a "clay bank" slope where a significant portion of the upper river bank is comprised of clayey soil.

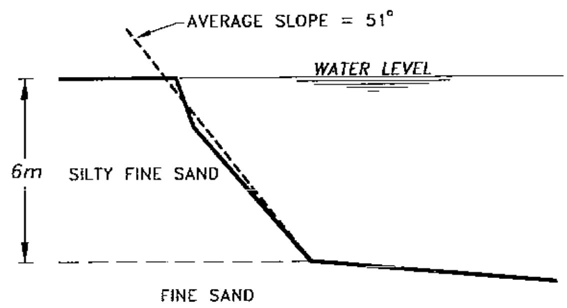
The profiles of the river bank slopes at the two locations were established by means of surveying and depth soundings obtained during the flood season when the active river bank failure was occurring. The river bank profiles at Locations A and B are shown on Figs. 3 and 4 respectively. The river level during the flood season corresponds approximately to the top of the river bank when essentially the entire bank becomes submerged.

The shear strength parameters used for the stability back analysis of the failing river bank slopes were based on the results of the site investigation and laboratory testing. These results are given in Table 1.

Table 1. Soil strength parameters.

Sandy Soil	
Angle of shearing resistance	= 30 degrees
Saturated unit weight	= 18.1 kN/m^3
Buoyant unit weight	= 8.3 kN/m^3
Clay Soil	
Undrained shear strength	= 28.7 kPa
Saturated unit weight	= 18.1 kN/m^3
Buoyant unit weight	= 8.3 kN/m^3

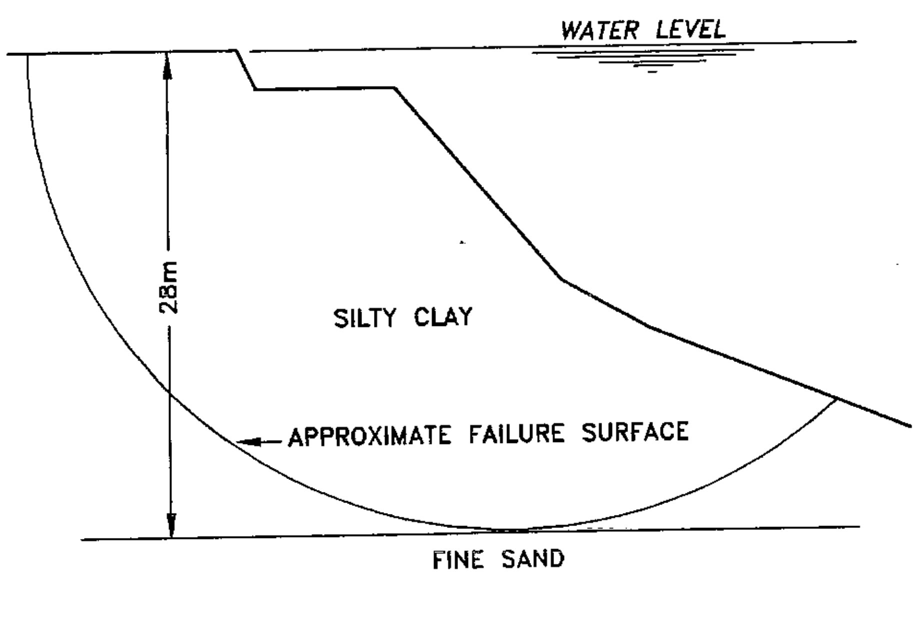
Fig. 3. Profile of submerged sand bank river slope.



The angle of shearing resistance for the sandy soil was obtained from drained direct shear tests and triaxial compression tests for a range of densities. A value of 30 degrees was considered to be representative of the looser siltier sand comprising the upper sand bank. The maximum and minimum values of saturated unit weight of the sandy soil determined from laboratory tests ranged between 17.3 kN/m^3 and 21.5 kN/m^3, and a value of 18.1 kN/m^3 was considered to be representative of the looser sand.

The undrained shear strength of the clay soil ranged from 9.6 to 96 kPa. Most of the shear strength values were within a smaller range and a value of 28.7 kPa was adopted for general design. The saturated unit weight determined from the undisturbed soil samples usually ranged from 15.7 to 19.2 kN/m^3, and a value of 18.1 kN/m^3 was considered representative of the clay soil in the upper portion of the river banks.

Fig. 4. Profile of submerged clay bank river slope.



Sand bank slope

The sand bank condition at Location A, Fig. 2, is representative of the river bank on the sand chars. The sand bank condition was considered typical of the foundation conditions likely to exist for the eight caissons located within the river valley. The river bank profile of the sand bank during river bank erosion and failure is shown on Fig. 3 and consists of approximately 6 m of silty fine sand overlying fine sand. The average slope of the river bank section on Fig. 3 is approximately 51 degrees above horizontal. The angle of repose of the silty fine sand is expected to be about 30 degrees. Since the bank slope is considerably steeper, some other factor must be responsible for maintaining the steeper slope. Possible factors include:

(1) The presence of a small amount of cohesion in the silty fine sand soil.
(2) The water level in the river is rising faster then the groundwater level in the river bank. Water seeping into the sand river bank will increase the effective stress and consequently the bank stability.
(3) The rate of bank erosion is so rapid at some locations that as bank slumping is occurring, negative pore water pressure develops owing to soil dilatancy. The negative pore water pressure causes a small increase in effective stress, thereby temporarily maintaining a steeper bank.

Any of the foregoing factors, or a combination of these factors could account for a slope steeper than about 30 degrees. For the purpose of stability analysis, the effect of any of these factors can be considered in terms of an equivalent amount of cohesion.

The stability of the sand bank shown on Fig. 3 was analyzed on an effective stress basis, using an angle of shearing resistance of 30 degrees, and assuming a rotational type of bank slump. One of the purposes of stability analysis was to establish the value of cohesion which would yield a factor of safety of 1.0 (unity).

The analysis illustrated that a small value of cohesion of about 2.4 kPa is sufficient to increase the steepness of the slope so that a factor of safety of 1.0 could be obtained for the geometry and the soil conditions assumed. This amount of cohesion is very small and can easily be accounted for by a trace of actual cohesion, or due to an apparent cohesion resulting from a temporary increase in effective stress.

Clay bank slope

The clay bank condition at Location B, Fig. 2, is representative of the east river valley wall. The river bank profile of the clay bank during river bank erosion and failure is shown on Fig. 4. The layer of clay soil was 28 m thick at this location and the clayey soil overlies fine sand.

The stability of the clay bank was analyzed on a total stress basis. It is a reasonable assumption that the rate of erosion during the flood season is so rapid that the undrained condition is applicable. It was assumed that the failure surface is circular and tangential to the fine sand strata. The average value of the undrained shear strength obtained from the stability analysis corresponding to a factor of safety of 1.0, was 31.1 kPa. This undrained shear strength value was similar to the representative value of 28.7 kPa obtained from laboratory tests as given in Table 1.

Estimate of river bank loading on the caissons

When the river bank has eroded to the caisson, the presence of the caisson restrains sliding of the bank, and accordingly, a load is imposed on the caisson. As erosion continues, the river bank load on the caisson increases until a maximum river bank load occurs. Eventually the river bank on both sides of the caisson fails and slides into the river, relieving the river bank lateral load.

In order to estimate the lateral load on the caisson, it was assumed that the mode of failure was similar to forcing the caisson into the river bank until the soil fails laterally in bearing capacity. The method of analysis selected was an adaptation of the lateral bearing capacity analysis proposed by Hansen, (1961) for evaluating the ultimate resistance of rigid piles subjected to an applied horizontal load.

The analysis of river bank loading was undertaken for three cases. Case I corresponds to a river bank of silty fine sand soil found within the river valley. The soil conditions for Case I are assumed to be identical to the soil conditions for the sand bank failure profile at Location A, as shown on Fig. 2. Case I is considered representative of the conditions applicable to Caissons No. 3 to 10 inclusive. Case II is assumed to correspond to the clay bank condition most likely found along the west valley wall in the vicinity of Caissons No. 1 and 2. Case III is assumed to correspond to the clay bank condition along the east valley wall in the vicinity of Caisson No. 11.

The effective stress parameters are assumed to be applicable for the sandy soil in Case I, while for Cases II and III, the undrained shear strength of the clayey soil was used in the analysis. The shear strength parameters determined in the previous section were used for the analysis. The groundwater level was considered to correspond to the top of the bank.

The thickness of the silty sand for Case I was taken as 6 m. The thickness of the clayey soil for the Cases II and III was 9 and 18 m respectively.

The calculated lateral river banks loads on the caissons are given in Table 2.

Table 2. Lateral river bank loads on the caissons.

Case	Calculated Lateral Load - kN	Factor of Safety	Design Lateral Load - kN
I	8,900	1.1	9,800
II	11,100	1.5	16,600
III	26,200	1.5	39,300

As can be seen the calculated lateral load at bearing capacity failure ranges from 8,900 kN to 26,200 kN for Cases I, II, and III. For the purpose of caisson design, a factor of safety must be included to allow for variability in the soil conditions, depth of bank and other conditions.

A factor of safety of 1.1 was taken for Case I because it was considered that the soil conditions for this case are reasonably well known, including the depth of the silty fine sand. A factor of safety of 1.5 was adopted for Cases II and III since the shear strength of the soil and the depth of soil strata were more variable. The caisson should be designed to accommodate lateral river bank design loads ranging from 9,800 to 39,300 kN.

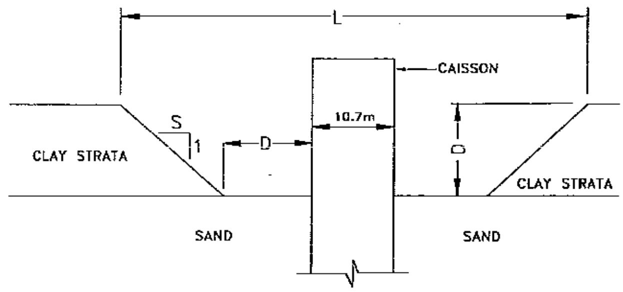
It is believed that the adoption of Hansen's (1961) analysis was a reasonable assumption but, nevertheless, a simplifying one. As the issue of the lateral river bank load on the caisson evolved and was being resolved, it became apparent that an approximate value would be adequate so that the appropriate solution would be undertaken.

The solution for accommodating the design lateral river bank loads on the caissons given in Table 2 must satisfy two requirements. The caissons must have adequate overall lateral stability to sustain the combination of loads applied to the caissons, and the lateral river bank load is one of these loads. The other requirement is that the caissons must have adequate structural strength to withstand the applied loads.

An evaluation of both of the above requirements for the condition when the maximum lateral river bank load is being applied showed that it was structural caisson capacity requirement that governed, and not the overall caisson lateral stability. This arises because at the time that the maximum lateral river bank load is applicable, the total scour around the caisson is relatively shallow compared to the maximum scour that can occur. Since the scour is shallow, the depth of caisson embedment is large, and as a result, the lateral caisson stability is substantial.

In summary, the lateral river bank loads have the greatest effect on the caisson structural strength required for the upper section of the caissons, particularly for Cases II and III as considered below.

Fig. 5. Excavation around caisson to reduce river bank load.



Reduction of the river bank loading on the caissons

The structural strength of the caisson required for the Case I lateral river bank load can be accommodated reasonably well within the structural capacity of the caisson. The river bank loading conditions for Cases II and III are too large for practical purposes to be resisted by providing the caisson with sufficient structural strength. Consequently, reduction of the river bank loading was therefore required.

Reduction of the river bank loads can be accomplished by excavating the clayey soil strata around the caisson and backfilling with sand dredged from the river. This procedure reduces the lateral river bank loading to a level similar to the Case I loading condition for all caissons. When the river bank erodes to the sand backfilled excavation, the sand backfill around the caisson will erode easily. This should prevent the development of a high steep river bank with the corresponding large lateral earth load transmitted to the caisson.

Excavation of the clayey soil strata should proceed to the bottom of the clayey soil. The excavation should be horizontal at the bottom, as shown on Fig. 5, and be circular in plan. The side slopes of the excavation should be sufficiently stable to permit excavation and subsequently backfilling with sand. Backfilling with sand is recommended since this will allow using the land for agricultural purposes for many years until river erosion has progressed to the caisson.

The shear strength parameters adopted in the analysis to evaluate the required steepness of the excavation slopes were the same as used for the river bank stability analysis applicable during erosion failure. The only difference was that since the excavation would be undertaken during the low river conditions in the dry season, the groundwater level was assumed to be 6 m below the top of the banks.

The dimensions of the excavation required for various depths of clay strata are illustrated on Fig. 5, with some values of "S" and "D" given below. A factor of safety of 1.1 was adopted for construction purposes. A minor amount of river bank slumping or failure may occur during excavation and may require some additional excavation during construction.

The sand used for the backfill should be clean sand dredged from the bottom of the river. The sand can be placed hydraulically but placement should be undertaken in layers to allow consolidation and dissipation of pore water pressure.

The volume of excavation and sand backfill can be quite substantial. If the thickness of the clay, dimension "D" on Fig. 5 is about 9 m, the value of "S" on the excavation slope will be 1.0, the volume of excavation will about 10,000 cubic metres, and the dimension "L" from crest to crest of the excavation will be about 47 m. If the thickness of the clay, "D" is about 18 m, the value of "S" on the excavation slope, Fig. 5, will be 2.0, the volume of excavation will about 105,000 cubic metres, and the dimension "L" from crest to crest of the excavation will be about 120 m.

The alternative to excavating the upper clay strata around the caissons after construction is to do nothing until necessary. This is a reasonable approach since the rate of erosion of the river valley wall is relatively slow and reasonably predictable. The excavation around the Case II and III caissons can deferred until about two years before it is expected that river erosion will reach the caissons. The land can be used for agricultural purposes until that time. This was, in fact, the decision and the process that was adopted by the Bangladesh Power Development Board.

Sand wave loading on the caissons

The hydrographic surveys undertaken on the Jamuna River have shown that large submerged bedform sand waves (dunes) migrate along the bottom of the river bed during the flood season. These waves, ranging in height from a few metres up to 8 m, will impose a load on the caisson as they progress past the caisson and an evaluation of the magnitude of this load was required. The sand wave loading was used in combination with other loads, but it was not combined with the river bank loading.

It was assumed that when the sand wave is moving past the caisson, the front slope of the wave will be at an angle of repose of 30 degrees. The load on the caisson is that due to the difference in sand height on the upstream and downstream sides of the caisson. The angle of shearing resistance was taken as 30 degrees and the buoyant soil weight as 8.3 kN/m^3. The sand wave load was calculated as 1,200 kN. This load was increased by a factor of safety of 1.5 to 1,800 kN to account for variations in steepness of the slope, height of the wave, and the simplified analytical approach adopted.

The submerged sand waves develop during the river flood reaching a maximum height during the rising stage of the river. When the depth of the natural river scour reaches its maximum, the sand waves tend to be smaller. Also, as the sand wave passes a caisson local scour will occur that will neutralize, to some extent, the local effect of the sand wave on the caisson and consequently the sand wave loading. Because of the foregoing considerations, it was recognized that the calculation of the sand wave load was approximate and somewhat arbitrary. However, the calculated value provided an allowance for the sand wave load as one of the design loads that were applied to the caisson foundations.

Conclusions

The failing Jamuna River bank slopes adjacent to the electric transmission line foundation caissons during erosion of the river banks in the flood season will impose a significant lateral load on the caisson foundations. Three river bank cases were considered; a sand bank case with a bank about 6 m in height, and two clay bank conditions with the clay being 9 or 18 m in height. The analysis of the above cases showed that the lateral river bank loads have limited impact on the caisson stability since the depth of scour around the caissons during the lateral bank load condition is relatively shallow. The lateral load, however, does have a very significant effect on the structural strength required for the upper portion of the caisson. The structural strength of the caisson was adequate for the shallower sand bank condition. The structural strength required for the two clay bank conditions was, however, too large to be practical.

As a result, it was necessary to develop a plan which would minimize that lateral river bank load. The solution adopted was to excavate the upper clay soil and replace with sand. This procedure reduces the maximum lateral load on the caisson during river bank erosion to the corresponding level of the sand bank condition.

Large submerged sand waves (dunes) migrate along the bottom of the river bed during the flood season. These submerged sand waves create a lateral load on the caisson, and values of the load were estimated for design purposes.

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