CONSTRUCTION PROBLEMS IN DELTAIC AREAS: ROADS, AIRPORT RUNWAYS AND SHALLOW FOUNDATIONS

M. J. TOMLINSON FICE FIStructE[1]

ABSTRACT

Tomlinson, M. J. 1979 Construction problems in deltaic areas: roads, airport runways and shallow foundations. In: E. Oele (ed.): Proceedings of the symposium on engineering-geological aspects of deltaic areas (Renesse, 1979) - Geol. Mijnbouw 58: 397-404.

The wide variety of soil types present in deltaic deposits, the occurrence of weak compressible soils extending to considerable depths, and the unstable topog?aphy of the deltaic terrain can give rise to difficult problems in the design and construction of roadworks, airports, and the shallow foundations of buildings and engineering structures.

This paper describes some of the topographic features and geotechnical characteristics of deltas, as they affect the siting, design, and construction of highways, airports and buildings. Examples are given of construction projects in each category.

TOPOGRAPHIC CONDITIONS IN DELTAS

The principal zones of a deltaic region which will be referred to in this paper are the *meander belt* in which the estuarial or flood-plain channels of the river follow a meandering course; the *coastal belt,* which is an area of instability affected by periodic incursions of the sea; and the *pro-delta* where the river-borne sediments are deposited on the sea bed on an advancing front.

Within the meander belt erosion of unprotected river banks takes place on the outsides of bends, and deposition on the inner sides of the bends. In wide sharply curving meanders a major flood flow can cut through the banks. The river then follows a new course and leaves a stagnant lake (oxbow lake) on its former location. The lake is gradually filled by soft silt, clay and swamp vegetation forming the backswamp areas behind the natural levees bordering the present river channel. Typical conditions in the meander belt of the Mississippi River at Greenville are shown in figure 1. Land clearance, reclamation and cultivation can obliterate the ancient buried river channels, but aerial photography can be an effective means of locating them when planning new highway routes.

In the coastal belt deposition of the coarser sediments at the mouth of the estuary forms a horseshoe-shaped sand bar which is exposed at low tide. In severe storm conditions the storm waves build up the bank above highwater mark which then deflects the course of the tidal river channel. With time the combined action of sediment-bearing river currents and littoral drift along the shoreline produces an ever-lengthening sand-spit and the river channel follows a course parallel to the coastline (Fig. 2). Sand dunes build up on the spit, followed by vegetation to form stable terrain. On the landward side of the river channel incursions of the tide form complex intersecting swampy tidal creeks. In the Niger Delta the creeks are bordered by mangrove trees, and the islands covered by mangrove scrub are submerged at half-tide level (Fig. 3).

Catastrophic floods and changes in the elevation of the land surface can deflect the main river channel to a new course when the tidal creeks are abandoned and gradually become filled with silt and peaty swamp vegetation. In the ancient delta of the Niger River at Lagos 20 m deep channels are filled with soft organic clay (HENKEL, 1965).

GEOTECHNICAL CHARACTERISTICS OF DELTAIC SOILS

KOLB & SHOCKLEY (1975) classified the soils of the flood plain and delta of the Mississippi River in relation to their location in the meander belt and the more seaward areas of the delta

[1] Consulting engineer, 23 High Street, BURTON-ON-STATHER, South Humberside DN15 9DG, United Kingdom.

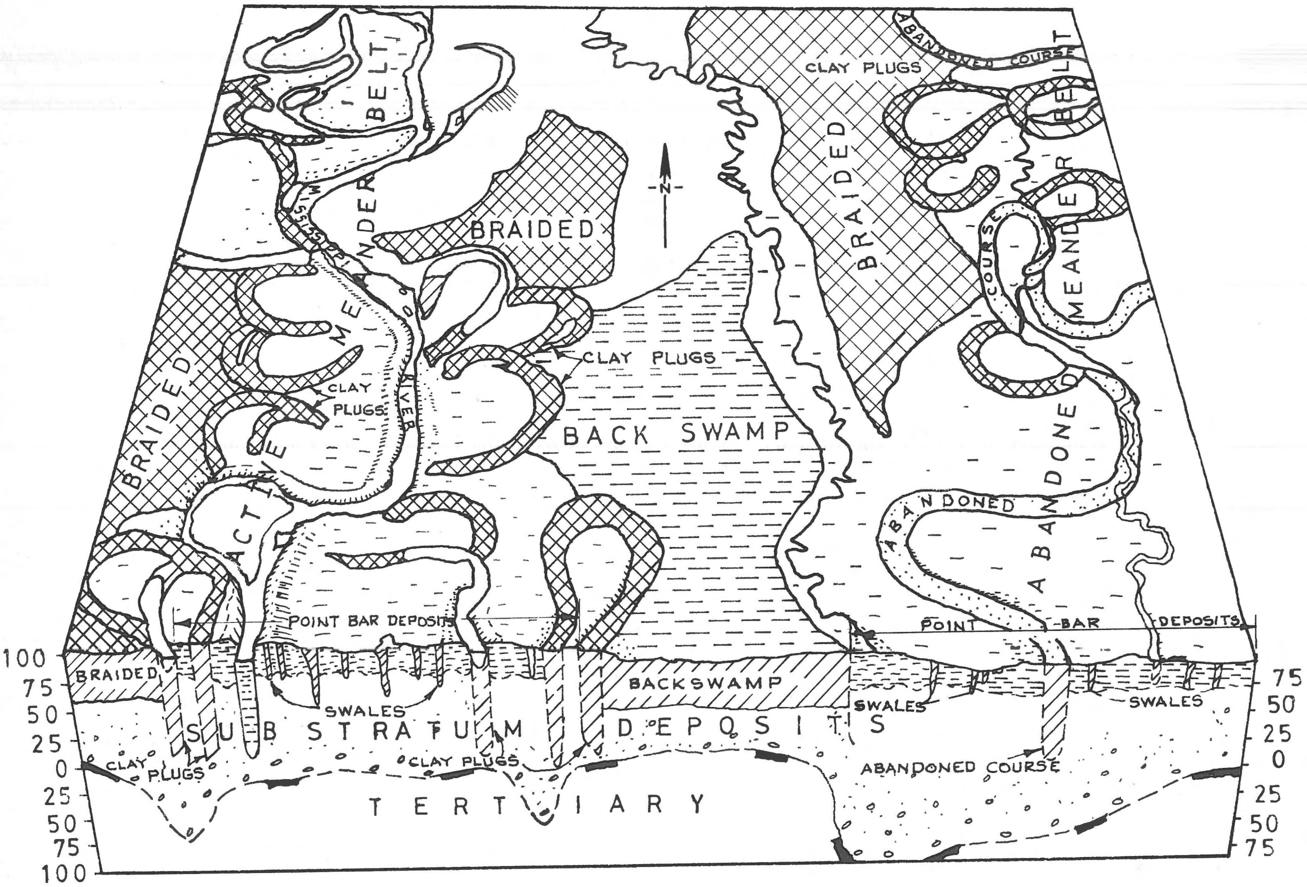

Fig. 1
Topographic features and associated soil types in the Mississippi Delta (after Kolb and Shockley, 1957).
Heigth in feet above mean sea level.

as follows:
Braided stream
Natural levee
Point bar ridges (on insides of river bends)
Abandoned channel
Backswamp
Swamp
Marsh
Pro-delta
Lacustrine
Beach
Bay-sound
Sub-stratum
Pleistocene

The geotechnical characteristics of the soil in each of the above classes are shown in table I.

Curves showing the relationship between the undrained shear strength (c_u) and depth below ground level of deltaic clays and silts at a number of locations are shown in figure 4. Also shown are the undrained shear strength-effective overburden pressure (c_u/p) relationships for normally-consolidated clays. BISHOP & HENKEL 1957) showed that this relationship varied between 0.22 and 0.33 for clays with plasticity indices of 30 and 60 respectively.

It will be seen from figure 4 that at a number of locations the clays are over-consolidated over depths of 5-10 m below the ground surface. This is due to desiccation by atmospheric drying and the growth of vegetation. The desiccation effects are most marked in arid regions as will be seen for the soils of the Shatt al Arab delta at Khorramshar, Iran, and Fao, Iraq.

Over-consolidation can also result from the overburden pressure caused by deposits of wind-blown sand, sandy alluvium, or reclamation fill. In contrast to this the clays of the pro-delta are likely to be under-consolidated as a result of slumping and disturbance of storm waves, as shown by the curve for the Mississippi pro-delta in figure 4.

Normally or lightly over-consolidated deltaic silts and clays generally exhibit static cone resistance values in the range of 2 to 15 kg/cm². Some typical cone resistance diagrams for clayey and sandy deltaic soils are shown in figure 5.

HIGHWAY LOCATION AND CONSTRUCTION

The earliest means of communication in deltaic regions was by boat. The townships which were established along the deeper navigable channels were later linked by roads constructed along the sandy natural levees. Tributary rivers and streams were crossed at fords or by primitive ferries. These communication routes followed lines at right angles to the delta front; transverse communication routes were possible only at some distance behind the delta front where the river followed a stable channel capable of being bridged.

Improvement of highway routes following natural levees involves embanking them above highest flood levels and bridging or culverting tributary rivers and streams. Improvement of the alignment is likely to involve crossing backswamp areas and the abandoned meander channels (EIDE, 1965). In the Mississippi delta embanked highway construction over the swampy areas resulted in long-term settlement of up to 1.5 m with problems in maintaining satisfactory vertical alignment at approaches to bridges and culverts on piled foundations. To avoid these problems in the re-alignment and improvement of US Highway 51, organic deposits 8 m thick were removed before constructing the new embankments (KOLB & SHOCKLEY, 1957).

The soft organic clays were not removed from beneath the route of the Ebute Metta Causeway at Lagos, Nigeria (HENKEL, 1965). In order to achieve an elevation of the causeway some 2-2.5 m above swamp level it was necessary to place a total thickness of about 7.5 m of hydraulic fill (pumped sand). Fill placement took place over a period of about $1^1/_2$ years and a further period of 4 years was required before consolidation settlement of the embankment was virtually complete.

For secondary roads where some irregular settlement of the road surface can be permitted, the road embankments can be constructed on a brushwood mattress. In the rainforest areas of the Niger Delta the mattresses are formed by felling trees and piling uprooted scrub on the right-of-

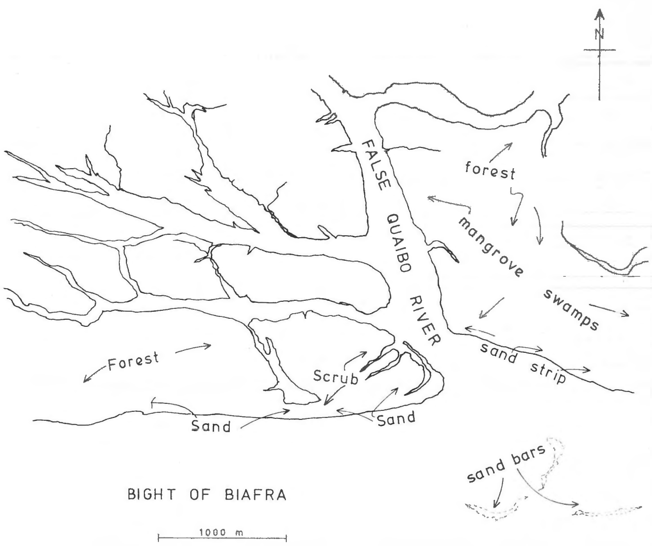

Fig. 2
Sand-spit development on seaward side of tidal creeks, False Quaibo River Estuary, Nigeria.

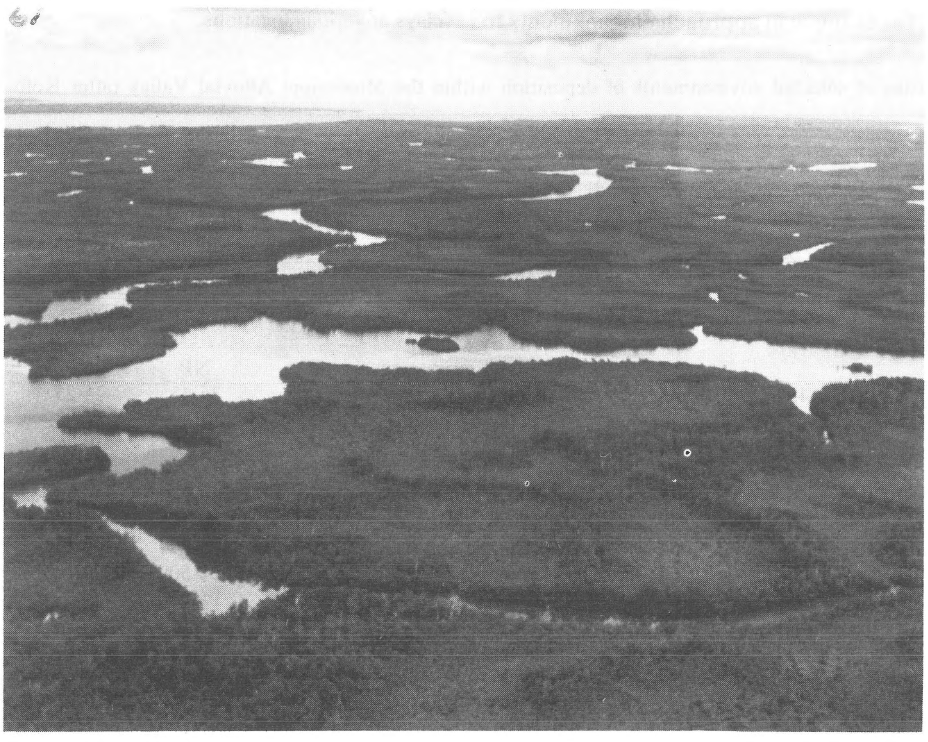

Fig. 3
Tidal creeks bordered by mangrove swamps in the Niger Delta.

way. Where suitable vegetation is not available on the highway location, imported materials must be used. CODDE (1957) described the use of bundles of 100 mm mangrove branches placed at 0.9 m centres transversely and longitudinally with leafy brushwood between them. The 60×30×0.5 m mattresses were floated into position, then sunk by the addition of stone. The road embankment was then formed from hydraulically-placed sand fill.

Where suitable timber or brushwood is not available mattresses can be formed from porous plastic sheeting.

Mattressing is not always necessary on highway routes across the intertidal zone in mangrove swamps, where the root zone of the mangrove trees is in the form of a relatively stable natural mattress 1-1.5 m thick. This mattress is composed of strong fine fibrous roots capable of supporting a low embankment.

Comparing the stability of the terrain in the intertidal 'islands' of the mangrove swamps and the intervening mud-filled tidal creeks, ROSEVEAR (1953) wrote:

'The newly deposited mud is of a blue-black colour and so soft that a man sinks immediately to his knees; as the first (Rhizophora) crop becomes mature this soil becomes filled with minute rootlets until the whole is transformed into a thick and hard carpet . . . there being, to all intents and purposes, no roots of a larger size than a piece of twine in the ground. On this it is possible to walk with the utmost ease at low tide getting not more than the sole of ones shoe damp.'

Construction of high embankments on the soft compressible deltaic clays, for example in approach embankments to

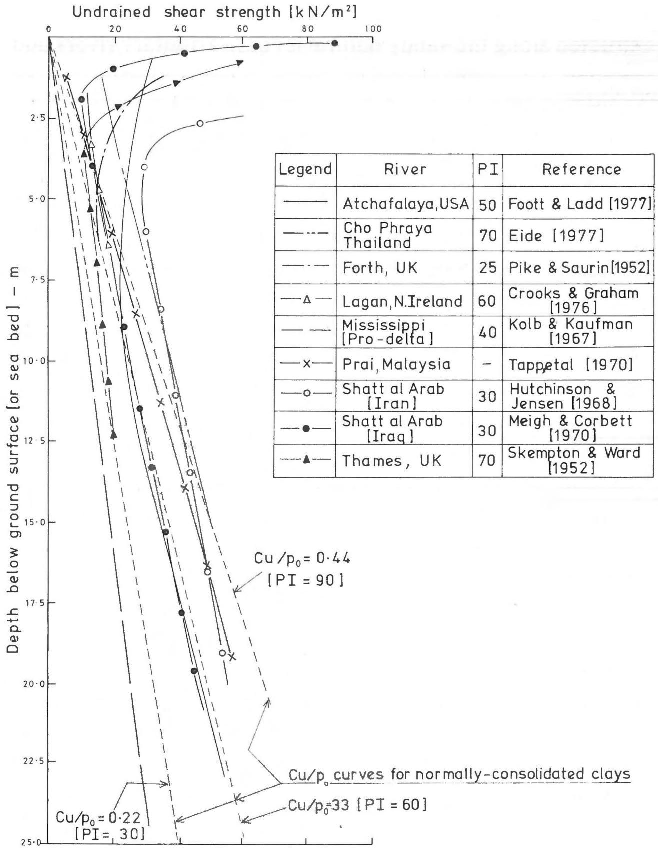

Fig. 4
Undrained shear strength versus depth relationships for deltaic soft clays at various locations.

Table I
Geotechnical properties of selected environments of deposition within the Mississippi Alluvial Valley (after Kolb & Shockley, 1957).

Environment	Particle-size distribution and organic content (%)					Predominant soil texture	Natural water content (%)	Liquid limit (%)	Plasticity Index	Unconfined shear strength	
	Gravel	Sand	Silt	Clay	Organic					Cohesion (kN/m²)	Angle of shearing resistance
Braided stream	–	25	30	45	–	Clay sands to silty sands	25-40	30-75	10-55	10-50	30
						Sands	–	NP	NP	0	30-40
Natural levee	–	15	40	45	–	Clays	25-35	35-45	15-25	17-57	0
						Silts	15-35	NP-35	NP-5	9-33	10-35
Point bar ridges	–	47	45	10	3	Silts and silty sands	25-45	30-55	10-25	0-40	25-35
Abandoned channel	–	5	20	72	3	Clays	30-95	30-100	10-65	14-50	0
Backswamp	–	5	5	80	10	Clays	25-70	40-115	25-100	19-120	0
Swamp	–	5	10	50	35	Organic clays	110-265	135-200	100-165	very low	
Marsh	–	0	–	20	80	Peat	160-465	250-500	150-400	very low	
Pro-delta	–	15	12	63	10	Clay	20-120	25-95	10-80	8-14	0
Lacustrine	–	–	5	90	5	Clay	45-165	85-115	65-95	36-72	0
Beach	–	85	10	5	–	Sand	saturated	NP	NP	0	30
Bay sound	–	25	30	40	5	Clay silts and silty clays	20-70	40-80	25-65	12-33	15-20
Substratum	25	70	5	–	–	Sand	saturated	NP	NP	0	30-38
Pleistocene	–	15	25	60	–	Clays and silty clays	15-30	25-80	20-75	26-240	0

bridges over river or minor roads, requires careful control of construction to ensure that over-stressing of the underlying soil followed by slumping of the embankment does not occur. This control can take the form of observations of pore-pressure increase and dissipation and of distortion of the embankment by means of an elaborate system of piezometers linked to a control chamber together with remote-controlled inclinometer and settlement gauges. However, simpler and very much cheaper control measures can be just as effective. HOLT (1978) described the use of settlement gauges in the form of a flat plate set on the ground with a vertical pole extending up through fill. Horizontal movement was monitored by a line of stakes near the toe of the embankment. This method was used to control the rate of placement of 7 m of fill forming a bridge approach sited over soft clays in the tidal estuary of the River Neath in South Wales.

The design of road pavements on embankments over soft deltaic soils is governed by the characteristics of the embankment fill. Flexible pavements are preferred since they can be more readily re-shaped and re-surfaced after major settlement has taken place, whereas rigid concrete pavements are likely to suffer widely spaced major cracks and stepping at joints. Consideration should be given to the possibility of longitudinal cracking of highway pavements consequent on the horizontal displacement of the soil beneath the embankments which must necessarily accompany vertical settlements.

KAUFMAN & WEAVER (1967) described the experiences in constructing flood protection levees along the banks of the Atchafalaya River in the Mississippi Delta. The highly plastic backswamp clays of the river basin are characterized by very low undrained shear strengths. A minimum shear strength of about 15 kN/m² was measured at a depth of 10 m, making it impracticable to construct the embankments to their full height in a single stage. Experimental embankments were constructed with shoulders at the toe and with wide berms for a first stage of placement to provide a calculated safety factor of 1.1. It was then planned to raise the embankments to their final height of 5.5 m above original crest level by further fill placed in lifts (Fig. 6).

Measurements of pore pressure made beneath the first stage berms and shoulders showed that very little dissipation had occurred during and shortly after construction. The large lateral deformations which occurred during this period represented the undrained response of the clay to the applied loading. The lateral deformations continued over a long period after completing the first stage embankment causing excessive sinking along the centre line of the earthworks. Measurements of the deformations within the clay below the embankment showed that the greater part of the lateral movement occurred at depths of 6-12 m below the base of the levee. A study of the shear strength profile indicated the presence of zones of somewhat stiffer clay at depths of 12-18 and 25-30 m. These zones represented periods of overconsolidation in the earlier stress history of the deposits.

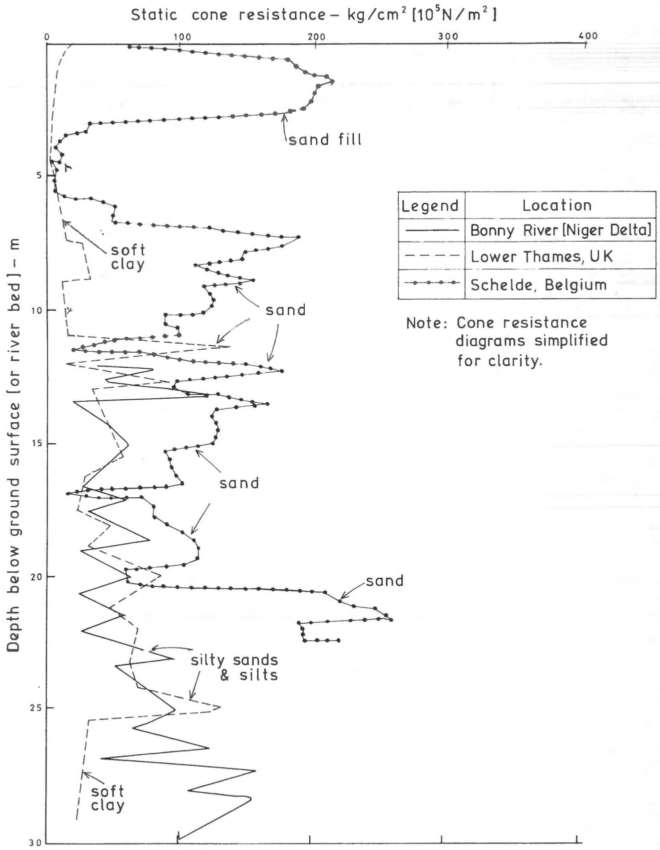

Fig. 5
Static cone penetration values versus depth for deltaic soft clays, silts and sands at various locations.

Concentrations of stress within the soft clays between these stiffer zones caused yielding of the clays and the accompanying high lateral strains.

Analyses of the behaviour of the trial embankments using finite element techniques and conventional slip circle analyses have been described by FOOTT & LADD (1977).

AIRPORT CONSTRUCTION

The problems of airport construction in deltaic regions are mainly those of selecting a site which will minimize the high cost of earthworks and drainage in reclamation and flood protection. It may be possible to site low cost airstrips for short feeder routes on the sand-spit areas of the coastal belt where these are aligned to allow construction of the runway in a direction favourable to the prevailing wind.

Major airports, or those planned for long-term stage development, are best sited in the more stable regions inland of the intertidal and fresh-water swamp zones. Examples of these are the airports at Benin City and Port Harcourt in the Niger delta. However, with the present shortage of land suitable for housing and industrial development and the need to

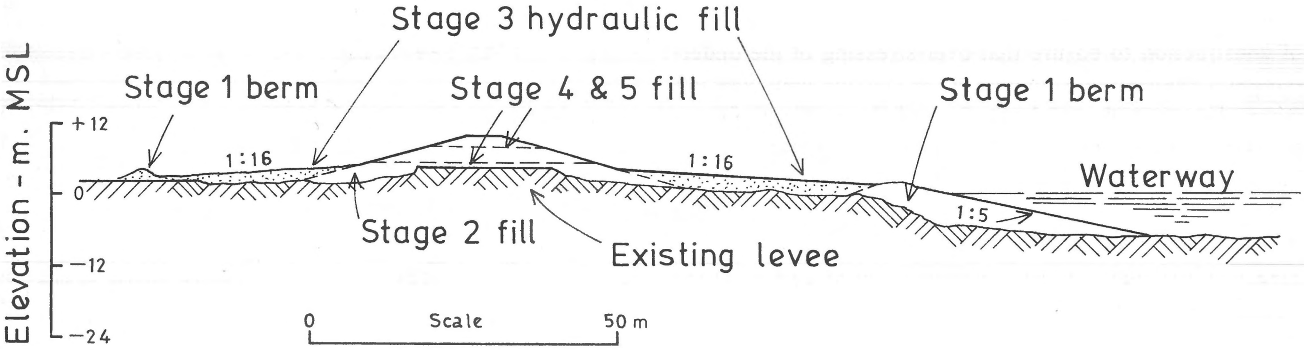

Fig. 6
Cross-section of experimental embankment for raising levees along the Atchafalaya River, Mississippi Delta.

protect developed areas against noise pollution, it may be necessary to seek sites for airports in swamps or tidal lands. An example of this was the proposal to site the Third London Airport on land reclaimed from the tidal creeks of the lower Thames estuary.

On estuarial sites of this type a major consideration is the need to protect the airport against flooding from astronomical high tides or storm surges. It is possible to construct the airport runways and terminal area at an elevation a little above natural ground surface with a raised embankment around the whole site as a protection against flooding. This method has the advantages of minimizing the quantities of imported fill, and because the airport pavements are constructed at or only a little above the original ground surface the vertical deformations caused by the consolidation of any soft soil beneath the pavements are quite small. Earthworks beneath the pavements are usually limited to removal of peat and any similar highly compressible swampy soil which could cause waviness to develop in the pavements. However the surrounding embankment located within the runway approach and take-off funnels can be a severe flying hazard. Also vehicular access tunnels beneath runways to the central terminal areas are likely to be constructed in difficult conditions below the groundwater table. A further requirement is that all surface water discharged from paved surfaces and buildings must be removed by pumping. Schiphol Airport is constructed at 4.2 m below mean sea level and is protected against flooding by the dikes of the Haarlemmermeer Polder. A pumped drainage system maintains the groundwater level at 2.1 m below the ground surface (CLERX & WEINBERG, 1948).

The problems of flying hazards caused by the flood protection embankment and the need for an internal pumped drainage system can be avoided by raising the whole airport area by means of hydraulically-placed sand fill. This method was proposed for the Third London Airport at Maplin in the Thames estuary. It may be necessary to raise the surface levels by some 5-7 m giving rise to problems of long-term settlement of fill placed on soft or peaty soils. Where these soils contain natural drainage channels in the form of layers or laminations of sand and silt the settlement of the fill can be rapid with little residual movement after compacting the fill to its final level (ROWE, 1972). However where these natural features are not present, either a long period of consolidation must be allowed or artificial subsoil drainage provided in the form of vertical sand drains or porous 'wick' drains. Alternatively, surcharging can be used as a means of accelerating settlement. This method was used for the international airport at Brunei where the runway was sited over swamps (STERLING ET AL., 1979). A few metres of peat were removed but the underlying soft clay extended to depths of up to 30 m. The surcharge was in the form of a sand fill embankment with a trapezoidal shape in cross-section increasing in height from 2 m on one side of the runway where the soft clay was 5 m thick to 4 m on the opposite side over the deepest clay (Fig. 7).

Pavement design for airport runways and taxiways is governed by the characteristics of the sub-base or embankment fill. In the case of the Brunei Airport designed for operations by Concorde aircraft (177 tonne AUW) the pavement consisted of:
127 mm marshall asphalt wearing course;
356 mm cement-bound crushed gravel base;
229 mm cement-stabilised beach sand;
508 mm beach sand sub-base.

At Schiphol Airport sited on the soft clays and silty sands of the Rhine delta the pavement design in the post-war years consisted of a sandwich construction comprising (KELLERSMAN, 1973):
70 mm asphalt wearing and base course;
180 mm asphalt base;
150 mm cement-stabilised soil;
130 mm sand;
400 mm lean-concrete sub-base.

The purpose of the concrete sub-base at Schiphol was to spread the aircraft wheel load on the low bearing capacity clays, but by placing the slab below the asphalt layers the concrete was insulated from the wide temperature variations which can occur at the pavement surface. Hence the problem of costly joint construction and joint maintenance which is a characteristic of airports with conventional unreinforced concrete pavements was avoided. In more recent years a variation of the sandwich construction has been a pavement consisting of:

180 mm prestressed concrete;
2 mm asphalt-sand friction reducing course;
150 mm soil cement;
180 mm sand;
400 mm lean-concrete sub-base.

The purpose of adopting a prestressed concrete pavement was again to avoid the need for transverse joints thereby minimizing interruption to flying operations by maintenance work. KELLERSMAN (1973) reported that the runway design had been successful in this respect.

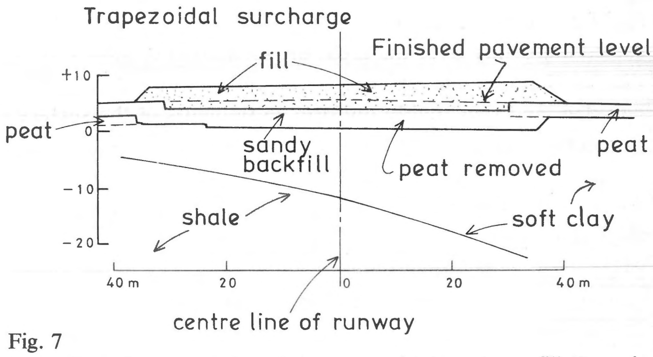

Fig. 7
Preloading of runway subgrade by trapezoidal surcharge fill, Brunei International Airport (after Sterling et al, 1977).
Height in m above mean sea level.

SHALLOW FOUNDATIONS

In the landward areas of estuaries where atmospheric drying and the growth of vegetation has produced a firm or stiff crust of clay overlying the soft normally consolidated deposits of clays and silts, it is possible to construct single and two-storey structures on strip or raft foundations bearing at as high a level as practicable within the crust. However the crust is usually of limited thickness and the foundations of heavier buildings can transmit stresses to the underlying soft compressible soils. The resulting consolidation of these soils can cause damaging settlements of the buildings.

GREEN ET AL. (1976) described the behaviour of two buildings of load-bearing wall construction founded on the post-glacial mudflats of the Forth estuary in Scotland. A 3-storey block of flats founded on a semi-buoyant shallow cellular raft foundation settled by about 10 mm during the first two years after construction with no significant cracking of the superstructure. A 4-storey block of similar construction founded on a flat slab raft settled by varying amounts up to 30 mm, with 20 mm of differential movement from front to rear over a period of four years. Quite severe cracking took place in the superstructure.

A buoyant raft is a design expedient which can be adopted for the foundations of heavy structures on soft deltaic clays. The method was used successfully for a power station sited on the soft clay deposits of the Forth estuary at a location where these highly compressible soils extended to a depth of about 30 m (PIKE & SAURIN, 1953). The power station was designed to be supported by a group of four cellular rafts of reinforced concrete construction covering an area of 52 m^2. The mass of soil displaced by the rafts at a founding depth of 4.7 m was 23,500 tonne compared with the mass of the cellular structure of 8200 tonne. Thus an uplift capacity of 15,300 tonne was available to limit the net bearing pressure imposed by the power station superstructure and generating plant to about 25 kN/m^2. The rafts were sunk by grabbing the soil from within the cells. To control verticality the soil was removed only from the outer cells during the early stages of sinking. Then on approaching founding level the corner cells were plugged with mass concrete to arrest further sinking. This was followed by plugging and sealing the remaining outer cells, and then the interior cells were excavated and plugged. About 30 mm of settlement were observed over a two-year period after commencing the construction of the superstructure. This included about 10 mm of settlement in the 9-month period after commissioning the generation equipment with movement still continuing at a slow rate.

Although the buoyancy raft principle is at first sight an attractive alternative to long piles to support heavily loaded foundations in areas of deep compressible deltaic soils, there are considerable practical difficulties in the construction of foundations of this type to the depth required to give adequate buoyancy. Deep excavation in soft clays involves a risk of massive heave of the soil at the base of the excavation. Open excavations must be cut back to flat stable slopes, or the alternative of vertical sides requires heavy shoring and strutting. Even if stable conditions are achieved, removal of overburden pressure is accompanied by upward elastic deformation of the soil at the base of the excavation. The subsequent re-compression of the heaved soil during early stages of the application of foundation loading then causes additional settlement. However, it should be noted that the alternative of long piles will not reduce settlements to a value less than those obtainable with a buoyancy raft, unless the piles can be driven down to a relatively incompressible stratum such as stiff clay or dense sand. Such soils may not be present at a depth at which piling is economical or practicable in deltaic regions.

A further point to note regarding construction on shallow raft foundations on compressible soils is the risk of serious settlement where construction schemes involve lowering the groundwater level, for example in the construction of a vehicular underpass. This can cause settlement of the neighbouring buildings on shallow foundations as a result of an increase in the vertical effective stress within the compressible soil layers.

It is possible to construct buildings above flood level in areas of low-lying swampy estuarial soils by founding them on hydraulically-placed sand fill. If the surface of the fill is raised uniformly over the area, the formation of mud-waves due to shear displacement of the underlying soft clays is avoided, and reasonably uniform settlements of the surface of the filled ground can be obtained. The sites of individual buildings can be surcharged by pre-loading with mounds of fill imposing ground pressures equal to or somewhat greater than that of the building. The fill is allowed to remain in place until level observations have shown that the time-settlement curve has approached the horizontal. The mound of fill is then moved to the next building location (TOMLINSON & WILSON, 1973)

An occasional problem of construction on swampy soils is the generation of marsh gas caused by the decay of organic matter. The gas can seep through shallow foundations of buildings giving rise to an explosion hazard. RUTLEDGE (1970) described the use of a sand blanket beneath buildings to bleed off the gas and dissipate excess pressure.

Major structures have been built on deep soft clay deposits after pre-loading with surcharge fill. SANGLERAT ET AL. (1977) described the construction of reinforced concrete cooling towers for an electricity generating station in France, where 5 m of recent alluvium were overlying 28 m of soft clay. A 10 m high annular embankment was dumped over the location of the 100 m diameter ring beam support to the tower structure. This embankment produced a surcharge pressure of 180 kN/m^2 corresponding to that imposed by the dead load of the tower structure. The fill was allowed to remain in place for 9 months during which time the alluvium consolidated by 80-110 mm and the underlying soft clay by 150-200 mm. Predictions of the subsequent settlement of the tower structure were made using finite element techniques and quasi-elastic analyses based on soil compressibility parameters derived from empirical correlations with static cone penetration tests.

Petroleum storage tanks are notable examples of the feasibility of constructing heavily loaded structures on soft deltaic soils using a slow controlled rate of loading. The problems of settlement and distortion of oil tanks have been reviewed comprehensively by PENMAN (1977). Although stability can be achieved by controlled rates of loading the consolidation of the soft compressible soils is accompanied by quite severe distortion of the tank structure which induce bending and tensile stresses in the steel plates. For this reason pre-loading is undertaken by filling with water. Slow consolidation of the soils below foundation level can cause continuing distortion of the tank plates and it may be necessary to re-level the tanks by jacking from time to time.

ACKNOWLEDGEMENTS

The author is grateful to Wimpey Laboratories Limited and to Fugro Limited for the supply of information on the geotechnical characteristics of deltaic soils.

REFERENCES

Bishop, A. W. & D. J. Henkel 1957 The measurement of soil properties in the triaxial test – Edward Arnold (London).

Clerx, J. W. & L. J. H. Weinberg 1948 Soil investigations and runway construction at Schiphol Airport – Proc. 2nd Int. Conf. Soil Mech. and Found. Eng. (Rotterdam) 8: 197-211.

Coode, D. C. 1957 Ebute Metta Causeway, Lagos, Nigeria – Chartered Civ. Eng. May 1957: 11.

Crooks, J. H. A. & J. Graham 1976 Geotechnical properties of the Beleast estuarine deposits – Geotechn. 26: 293-315.

Eide, O. 1977 Exploration, sampling and testing of soft clay in the Bangkok area – Proc. Conf. geotechn. aspects of soft clay (Asian Inst. Techn., Bangkok): 359-374.

Foott, R. & C. C. Ladd 1977 Behaviour of Atchafalaya levees during construction – Geotechn. 27: 137-160.

Green, D. R., I. A. Macleod & W. G. Stark 1976 Observation and analysis of brick structures on soft clay. In: Proc. Conf. Performance of Building Structures – Pentech Press: 321-336.

Henkel, D. J. 1965 Problems associated with the construction of the Ebute Metta Causeway over soft clays in Lagos, Nigeria – Proc. 6th Int. Conf. Soil Mech. and Found. Eng. (Montreal) 2: 74-77.

Holt, J. B. 1978 Discussion on D. H. New et al.: Some features of the Aberdulais to Llandarcy section of the Neath-Abergavenny trunk road – Proc. Inst. Civ. Eng. 64: 154-155.

Hutchinson, J. N. & E. V. Jensen 1968 Loading tests on piles driven into estuarine clays at Port of Khorramshahr – NGI Publ. 78: 1-12.

Kaufman, R. I. & F. J. Weaver 1967 Stability of Atchafalaya levees – Proc. Amer. Soc. Civ. Eng. 93 (SM4): 157-176.

Kolb, C. R. & R. I. Kaufman 1967 Prodelta clays of south-east Louisiana – Marine Geotechnique (Univ. Illinois Press): 3-13.

Kolb, C. R. & W. G. Shockley 1957 Mississippi Valley geology - its engineering significance – Proc. Amer. Soc. Civ. Eng. 83 (SM3): 1289-1/14.

Meigh, A. C. & B. O. Corbett 1970 A comparison of in-situ measurements in soft clay with laboratory tests and the settlement of oil tanks – Proc. Conf. in-situ investigations in soils and rocks – British Geotechn. Soc. (London): 173-179.

Penman, A. D. M. 1977 Soil-structure interaction and deformation problems with large oil tanks – Proc. Int. Symp. Soil-Structure Interaction (Univ. Roorkee, India) 1: 521-536.

Pike, C. W. & B. F. Saurin 1952 Buoyant foundations in soft clay for oil refinery structures – Proc. Inst. Civ. Eng. 1: 301-334.

Rosevear, D. R. 1953 The Nigeria handbook, Chapter 15 (Vegetation) – Crown Agents (London).

Rowe, P. W. 1972 The relevance of soil fabric to site investigation practice – Geotechn. 22: 195-300.

Rutledge, P. C. 1970 Utilization of marginal lands for urban development – Proc. Amer. Soc. Civ. Eng. 96 (SM1): 3-72.

Sanglerat, G. et al. 1977 Surcharge fill settlements on soft clay at the location of two air cooling towers – Proc. Conf. geotechn. aspects of soft clays (Asian Inst. Techn. Bangkok): 285-300.

Skempton, A. W. & W. H. Ward 1952 Investigations concerning a deep cofferdam in the Thames Estuary at Shell Haven – Geotechn. 3: 119-139.

Sterling, W. A. D., C. B. Paget, J. M. Eddison & M. C. Earwaker 1977 Construction of new international airport, Brunei – Proc. Inst. Civ. Eng. 62: 605-622.

Tapp, J. W. T., A. J. B. Dixon & A. C. Carpenter 1970 The design and construction of Butterworth Wharves, Malaysia – Proc. Inst. Civ. Eng. Paper 7323 S: 223-266.

Tomlinson, M. J. & D. M. Wilson 1973 Preloading of foundations by surcharge on filled ground – Geotechn. 23: 117-120.