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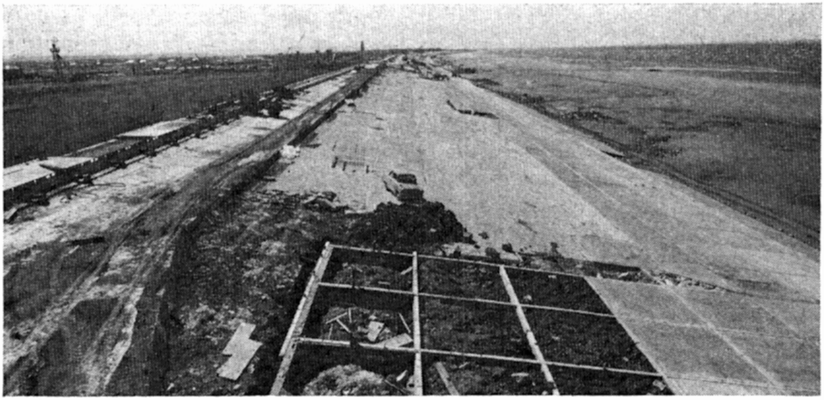

Fig. 1. General view of embankment

7833 The Wallasey Embankment

M. N. BELL, BSc, FICE*
P. C. BARBER, MICE, MIMunE, MIPHE†
D. G. E. SMITH, MSc, DIC, MICE*

The Paper traces the concept and history of the Wallasey Embankment from its origin in the early 19th century when coast erosion on the Wirral and the development of the port of Liverpool made its construction necessary. Subsequent maintenance methods and improvements to the structure, which were developed in the face of increasingly rigorous exposure conditions experienced since the original construction, are described. An account is given of emergency procedures adopted when the integrity of the embankment has been put in jeopardy by severe storm damage. Emphasis is placed upon the desirability in a structure of this type, of exposed elements being designed so as to control the spread of storm damage and to permit permanent repairs and replacements to be executed rapidly using readily available resources. The current reconstruction programme and methods which have been developed to this end are described in detail, including modifications to accommodate difficult ground conditions and improved construction techniques. Finally the principles of the design and construction are considered in the general context of protective structures in the marine environment. Authorship of the Paper is shared between representatives of the Local Authority engineering staff and the Consulting Engineers' staff, in harmony with the procedure of shared responsibility and effort which has been successfully adopted for the design and contract supervision of the reconstruction.

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* Scott Wilson Kirkpatrick & Partners.
† Chief Resident Engineer, Metropolitan Borough of Wirral.

Introduction

1. The Wallasey Embankment (Fig. 1) is a sea defence and coast protection structure, 3·6 km long and rising approximately 10 m above beach level, which was constructed during the first half of the 19th century under an Act of Parliament 'for the purpose of preventing further encroachment of the sea and the injury to arise therefrom to the contiguous lowlands and to the Port of Liverpool'. The embankment lies along the low-lying middle third of the Wirral Coast between Wallasey and Hoylake in the Metropolitan Borough of Wirral. It faces squarely into NW gales which are the principal cause of coast erosion in the Liverpool Bay sector of the Irish Sea (Fig. 2). It is the sole protection against sea-flood afforded to some 18 km^2 of low-lying land which has been extensively developed for industrial and residential use (Fig. 3).

General history

Concept

2. There is evidence of a general lowering of the land relative to mean sea level at the head of Liverpool Bay during historic times, estimated as proceeding at an average rate of nearly 1 m/100 years.[1] Today some 18 km^2 of the northern end of the Wirral peninsula is below the level of the highest tides. This land was originally protected from inundation by a continuous system of coastal dunes between Hoylake and Wallasey.

3. In addition to vertical movement of the land mass, there has been net marine erosion of the exposed coastline SW of the River Mersey, due to a deficiency in the supply of sediment in the predominant NE littoral drift along the North Wales and Wirral coasts. By the latter part of the 18th century encroachment by the sea had reached the stage at which it was feared that a breach of the dunes was imminent. A breach would have caused flooding of the adjacent low-lying land and perhaps the creation of a new outlet from the Mersey between Birkenhead and Wallasey through Wallasey Pool. In 1794, in the face of this danger, an attempt was made to arrest erosion by forming 'a slope wall or pavement' along the seaward face of the dunes. This was successful only in the short term and erosion soon resumed. In 1813, at the request of the proprietors of lands below or near high water level, William Chapman reported on the adverse effect that a permanent breach would have upon the port of Liverpool due to higher tide levels in the river and reduced channel scour which would result from a bifurcation of the ebb stream. In 1829, following further representations, the Wallasey Embankment Bill was presented to Parliament by Liverpool Corporation, at that time the dock undertakers. The purpose of the bill was to set up a permanent commission to be responsible for the construction and maintenance of a sea defence structure along the most vulnerable stretch of shore. This structure, approximately 2·65 km long, is now known as the Old Embankment and is the principal subject of this Paper.

4. Following serious erosion around the flanks of the Old Embankment the works were extended at both ends under an Act of Parliament in 1894, increasing the total length of the structure to about 3·6 km. These two extensions are known as the New Embankment (Fig. 4).

5. The enclosure of Wallasey Pool by a dock system between 1840 and 1860 reduced the risk to the flow in the River Mersey. Industrial and residential

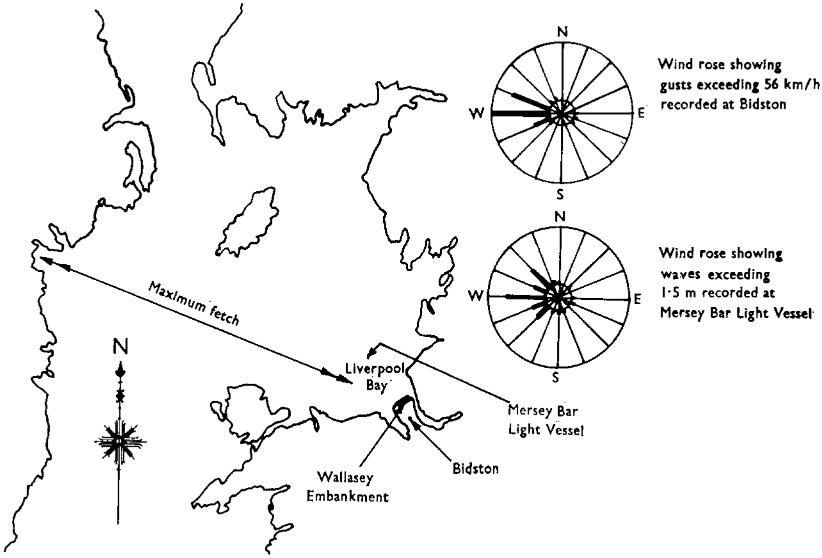

Fig. 2. Map with wind and wave roses indicating degree of exposure

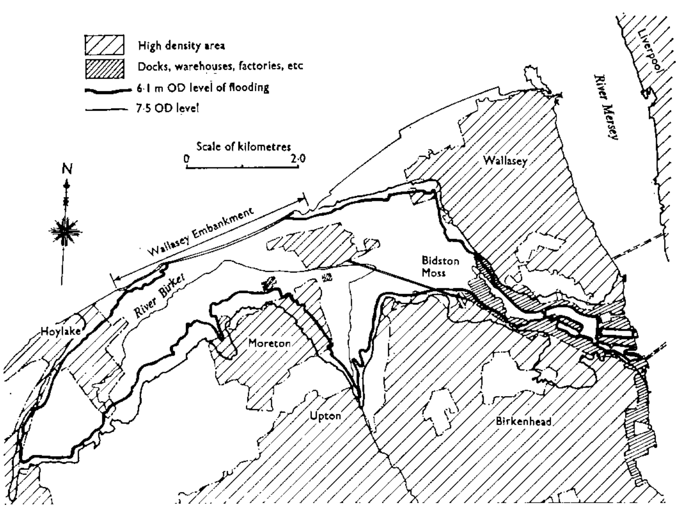

Fig. 3. Location of embankment showing areas liable to flood in the event of a breach

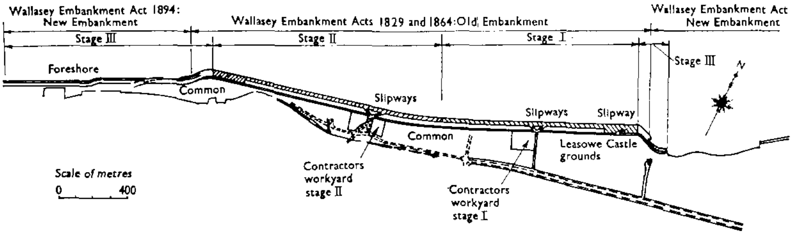

Fig. 4. Plan of embankment showing extent of each stage of the current reconstruction programme

development subsequently took place in the protected area, especially since 1920. The functional emphasis of the embankment thus progressively shifted from navigation protection for the port to direct protection of life and property. This change of emphasis eliminated any acceptability of flood conditions. By 1972, due mainly to industrial and residential development behind the embankment, the direct replacement value alone of property protected from flooding was estimated to be approximately £30 million (Table 1).

Table 1. Properties below high tidal level

Details of property	Value, £ × 1000*
1800 houses (£8 000 each)	£14 400
180 shops (£20 000 each)	3 600
8 schools (£200 000 each)	1 600
35 public houses (£30 000 each)	1 050
1 hospital	2 000
1 convalescent home	250
Cadburys factory	2 500
Squibbs factory	2 000
2 brickworks (£15 000 each)	30
1 caravan site	10
12 small industrial establishments (£20 000 each)	240
6 railway stations (£50 000 each)	300
North Wirral main drainage pumping station	1 000
1100 acres farmland (£250/acre)	275
Total	£29 255

* No account is taken in this estimate of the cost of emergency services, temporary rehousing, disruption of industry and transport, etc.

Initial construction

6. The basic form of the Old Embankment, which has stood well the test of time, is a bank of indigenous material, having a shallow seaward slope provided with an impermeable facing. This profile induces the dissipation of energy by causing the waves to break and by absorbing the remaining energy of the swash by frictional and gravitational resistance.

7. From boreholes and from observations during stage I of the recent reconstruction work along the eastern half of the Old Embankment it has been found that the heart is mainly of sand, clay, silt and peat. This confirms that the formation was for the most part simply from the natural material trimmed to a regular profile, any deficiency being made up from the beach.

8. The seaward slope of the embankment was profiled at gradients between 1 in 4 and 1 in 8 and was initially faced with clay, of local origin, approximately 0·6 m thick. This facing was in due course protected from erosion by an armour layer of sandstone blocks, varying in size from 0·20 m to 0·45 m equivalent cubes, up to highest tide level, and ultimately by cobbles in the swash zone above. The blocks within the tidal zone were dressed to a rectangular plan with a slight taper into the embankment and tooled on the butting faces so as to interlock. They were laid dry in bonded courses along the embankment face. At that time the facing simply continued into a trench dug along the beach and no cut-off into the underlying boulder clay was provided. A tribute to the engineers and craftsmen of the time is that some sections of the original masonry facing are still functioning today, up to 140 years after construction, having withstood the impact of perhaps 150 million waves.

9. The initial cost of constructing the Old Embankment is recorded as about £32 000 and that of constructing the New Embankment as about £16 370.

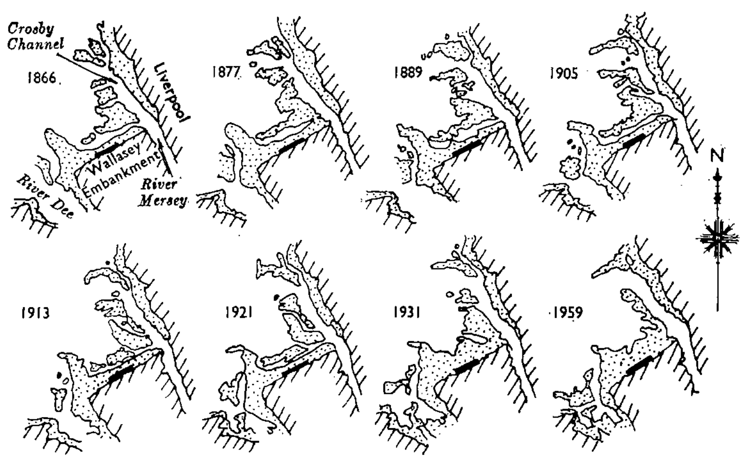

Fig. 5. Changes in foreshore configuration between 1866 and 1959

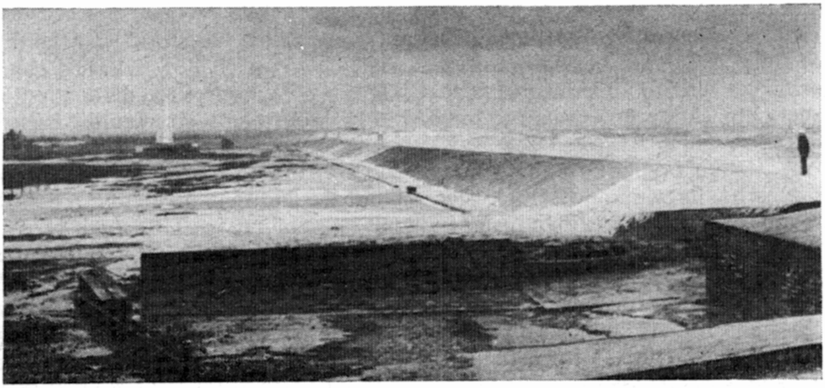

Fig. 6. Overtopping of embankment during NW gale, 2 April, 1973

Regime changes

10. The trends of long-term erosion and subsidence have already been mentioned. Apart from the effects of these, the beach level at the embankment toe has usually been relatively stable in the month-to-month time-scale, subject to local fluctuations due to channel movements, storms and the effect of artificial obstructions. However, during the life of the embankment major changes in the configuration of the sandbanks between the Dee and the Mersey have taken place (Fig. 5). The principal change affecting the embankment has been the migration of the Rock Channel entrance eastwards due to beach movements, influenced by the construction of the Crosby Channel training banks between 1911 and 1957. This has had the effect of transferring the incidence of maximum wave energy from the western end of the embankment to the eastern end, an effect confirmed by records of storm damage repairs.

11. As the beach eroded and progressive failure of the toe and facing was threatened, a sloping apron of stone blocks, and later concrete, was added in front of the toe to reduce scour; however, further beach erosion and tidal action in turn threatened even this construction. Beach erosion was combatted by the use of various types of groyne, including the extensive use of a fascine type constructed of cherry tree branches.

12. The increased depth of water over the lowered foreshore has allowed the incidence of larger waves (Fig. 6). To reduce overtopping, therefore, a raised crest was added to the original embankment and more recently, to effect virtual elimination of overtopping, a wave-return wall has become necessary. With regard to the profile prior to the present reconstruction, model tests conducted at the Hydraulics Research Station in 1969[3] at the request of the Mersey and Weaver River Authority suggested that if the 1 in 100 year tidal level were associated with the highest yearly wave, with a 9 s period, the rate of overtopping would be of the order of 65 m^3/h per m run of embankment. If this overtopping rate were combined with a fresh water flood discharge, the capacity of the River Birket, which drains the adjacent low ground, could be exceeded. The model tests further showed that a 0·9 m high curved wave wall at a fixed level constructed along the crest of the embankment could almost eliminate overtopping at the 100 year tidal level.

Administration

13. The administration and finance of the construction and maintenance of the embankment under successive Acts of Parliament are summarized in Table 2. Consulting Engineers to Wallasey and Wirral Borough Councils since 1944 have been successively Sir Cyril Kirkpatrick & Partners and Scott Wilson Kirkpatrick & Partners.

History of maintenance

Storm damage

14. The initial absence of a cut-off at the toe of the clay sealing layer allowed heart material to be washed out, with consequent subsidence of the facing. It is thus reasonable to assume that most storm damage to the sandstone facing would have started at or near the toe in areas weakened by this process and by

Table 2. Administration under successive Acts of Parliament

Act of Parliament	Responsible body	Finance	Remarks
Wallasey Embankment Act, 1829	Wallasey Embankment Commission	Annual rate levy on landowners protected, plus substantial contribution from Liverpool Corporation	Authorized construction of Old Embankment
Wallasey Embankment Act, 1864	Wallasey Embankment Commission, reconstituted	As before but Mersey Docks and Harbour Board replacing Liverpool Corporation	Maintenance and improvements subsequently carried out by Mersey Docks and Harbour Board staff
Wallasey Embankment Act, 1894	As before	As before; borrowing powers extended	Authorized construction of New Embankment
Wallasey Embankment Act, 1923	As before	Increased borrowing powers	
Wallasey Corporation Act, 1945, Part III	As before	As before	Authorized transfer of Commission's duties to Wallasey Corporation, but agreement on terms of transfer not reached until 1962
Wallasey Embankment Act, 1962	County Borough of Wallasey	General rate	Structure eligible for grant aid under Coast Protection Act of 1949
Local Government Act, 1974	Metropolitan Borough of Wirral	General rate	Structure eligible for grant aid under Coast Protection Act, 1949

beach lowering. To stabilize the repaired toe and prevent recurrence of washout, a toe wall was constructed in front of each break, which was then sealed with clay, pitched sandstone blocks and concrete. The toe wall was up to 3 m deep, being carried down 0·5 m into the boulder clay where found, and was eventually constructed along most of the toe of the embankment. An apparent random alignment of toe wall sections is somewhat perplexing; one possible explanation is that the fixing of the toe wall shutter in a damaged area was dependent on the location of the landward face of the break. There appears not to have been any major modification to this approach up to 1941, by which time some 85% of the structure had been protected with a toe wall to secure the original sandstone paving.

15. Records of severe storms in 1883 and 1919 have been found, including photographs which show that the structure suffered severe damage. Certainly in the latter instance the embankment was threatened with a breach (Fig. 7). The affected area was eventually contained and stabilized by constructing an irregular network of massive concrete walls, within which it was possible to replace the heart and reconstruct the facing. Between 1955 and 1970, as a consequence of a reduced level of maintenance work, there were several severe breaks, the largest of which involved partial dislocation of the eastern corner, although a breach of the embankment was averted. Repairs during this period took the form of hardcore infill and mass concrete slabbing.

Maintenance of facing

16. Throughout the period between 1829 and 1962 there was a gang of men continuously employed in the construction and maintenance of the embankment.

17. The original facing performed well, being effective, durable (except in certain areas where soft sandstone was used) and sufficiently flexible to accommodate settlement of the poor hearting material. However, its great structural weakness was that, once breached, the disintegration spread quickly and was difficult to arrest in the event of prolonged adverse weather. Records show that from 1870 concrete was used increasingly to supplement the sandstone blocks as a material for paving the structure.

18. It is clear that throughout the later history of the embankment there has been recognition of the need to restructure the original facing so as to facilitate maintenance and allow quick effective emergency repairs, with disturbance of the heart being kept to a minimum. In recent reconstruction work it has become established policy to achieve an element of 'fail-safe' construction, designed to eliminate as far as possible the need for temporary repairs to storm-damaged or eroded facing and to enable permanent repairs to be effected by simple workmanship in a single tide. Design philosophy has been evolutionary, in that previous work has been successively evaluated and modified to suit changing conditions and available techniques.

19. A landmark in the history of the embankment was provided by the change in facing reconstruction pattern adopted by the Mersey Docks and Harbour Board in 1941. The main outcome of this change was the adoption of cellular construction, resembling that used before mainly for major emergency repairs but using modern materials and arranged in a regular grid pattern. This development for the more exposed parts of the face took the form of a network of cells approximately 4 m square. The cell boundary walls

Fig. 7. Storm damage incurred during December 1919 showing cut-offs walls under construction

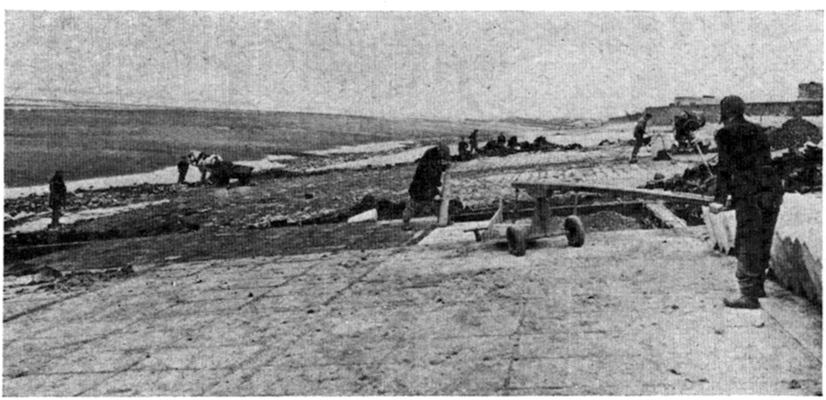

Fig. 8. Cellular paving under construction by manual method, 1949

and infill paving panels were formed in concrete, sometimes in situ and sometimes precast, reinforced and unreinforced (Fig. 8). The clay seal was usually maintained beneath the infill paving. The purpose of the cellular construction was to limit the extent of washout, upon eventual failure of the infill concrete skin, to a single bay of a size which could be made good in one tide by the maintenance gang. In general this design proved successful in containing

Table 3. Anticipated annual renewal of concrete elements following reconstruction

Zone	Type	Area, m^2	Date constructed	Estimated Life, years	First renewal period	First renewal rate, m^2/year
Apron	New 0·30 m concrete	21 000	1973	60–120	2033–2093	350
Toe beam	New 0·7 m × 1·0 m concrete	(3 000 m)	1973	60–150	2033–2123	(33 m)
Facing below MHWST	New 0·30 m concrete Old 0·15 m concrete	33 000 14 000	1973 1941–1957	40–80 20–40	2013–2053 1973–1997	830 580
First panel above MHWST	New 0·225 m concrete Old 0·15 m concrete	4 000 6 000	1973 1941–1957	40–80 30–60	2013–2053 1973–2017	100 140
Second panel above MHWST	New 0·225 m concrete Old 0·10 m concrete Old 0·15 m concrete	2 000 4 000 4 000	1973 1952–1957 1941–1951	50–100 30–60 40–80	2023–2073 1982–2017 1981–2031	40 120 80
Upper facing	New 0·15 m concrete Old 0·10 m concrete Old concrete on stones	2 000 30 000 12 000	1973 1941–1957 1957–1972	50–100 40–80 20–30	2013–2073 1981–2037 1977–2002	40 530 480
Access road	New concrete	13 000	1973	60–120	2033–2093	220
Wave wall	New concrete wave wall	(2 700 m)	1973	40–80	2013–2053	(70 m)
Rear face	Old concrete (part)	15 000	1941–1957	40–80	1981–2037	270
Total paving		160 000 m^2				

Notes: (*a*) For simplicity, surfacing renewed within present reconstruction is all shown as constructed in 1973.
(*b*) Cut-off walls are not indicated as it is assumed that their repair will be concurrent with the replacement of the facing.

Fig. 9. Deteriorated facing prior to reconstruction, 1974: new precast toe beam in position

damage. However, the facing panels laid at this time in the tidal zone were only about 0·15 m thick, and a concrete of relatively low durability was used (Fig. 9). Also, due to the haunched cross-section of the cell cut-off walls providing peripheral bearing, the infill slabs were subject to undetected undermining, leading eventually to sudden collapse. A large amount of reconstruction was subsequently undertaken and all 'breaks' sustained up to 1957 were reinstated using this method. Only a small length of new toe wall was constructed during this period, although the existing wall was maintained.

20. From 1955 up to the eventual transference of responsibility for the structure in 1962, negotiations were in progress between the Dock Board and Wallasey Council. During this period little work on the structure was undertaken and in the period immediately after 1962 only strictly essential works were carried out using mass concrete paving: few cut-off walls were renewed or repaired during either of these periods.

21. Until the present reconstruction, adequate access for maintenance by hand methods was provided by a slightly raised barrow-way, about 1 m wide, along the crest of the embankment.

22. When all stages of the present partial reconstruction are complete it is planned to introduce a definite maintenance programme (estimated cost £40 000/year at 1975 prices) to include the repair of damaged areas to reconstruction standard and to allow an annual extension to the reconstructed area on a damage prevention basis (Table 3).

New works

23. Following the assumption of responsibility for the embankment by the County Borough of Wallasey in 1962, escalation of the routine maintenance

costs prompted a detailed engineering inspection of the structure. This inspection revealed an acceleration of facing failure previously identified in reports from the Consultants. As a consequence a major reconstruction was considered necessary and a programme for this work, in three stages, was initiated in 1972. Details are given in Fig. 4 and Table 4.

Philosophy of design

24. The design, construction and maintenance of all marine and coastal structures are very closely governed by ambient conditions at the site. At the Wallasey coast there is a combination of great tidal range at high spring tides and severe exposure to waves generated during NW gales. The principal physical parameters governing the design are given in Table 5.

25. Soil conditions at the embankment toe are generally beach sand, at present approximately 2 m thick on average, overlying boulder clay at an average level of −1·6 m OD. There is evidence that in earlier days the clay was exposed after layers of silt and peat had been eroded and that prior to this the whole had been covered with dune sand.

26. The solution adopted in reconstructing the toe has been to provide a reinforced concrete 'beam' along the approximate line or to seaward of the

Table 4. Construction programme

	Stage I	Stage II	Stage III
Construction period	January 1973– October 1974	May 1975– August 1977	1978
Contractor	Mears Construction Ltd	Dredging and Construction Co. Ltd	—
Cost	£875 000 (estimated final, June 1975)	£1 603 745 (tender)	—
Financial assistance	Dept of the Environment (Coast Protection Act, 1949)	Ministry of Agriculture, Fisheries and Food (Land Drainage Act, 1930) through North West Water Authority, for whom Metropolitan Borough of Wirral acts as Agent	

Table 5. Physical parameters

Extreme annual tidal range (predicted)[2]	10·0 m
100 year maximum level (predicted)[3]	5·89 m OD
Storm surge set-up assumed	0·97 m
Maximum fetch	180 km from WNW
Wave height at Mersey Bar producing highest wave at embankment[3]	3·7 m
Design wave period[3]	7 s
Percentage of time embankment not exposed to sea action	54%
Approximate flood current (from SW)[3]	0·60 m/s
Approximate ebb current (from NE)[3]	0·40 m/s
Net bed velocity (from SW)[3]	1200 m/tidal cycle

Fig. 10. New toe sheet piling being driven to seaward of old toe construction

Fig. 11. New toe construction completed before start of facing reconstruction, showing sand accretion

original toe wall to form a substantial footing for the new facing construction, but not necessarily deep enough to form a cut-off into the clay. Between 5 and 6 m to seaward of this toe beam a continuous curtain of steel sheet piling has been provided, penetrating the boulder clay a minimum of 0·6 m (Fig. 10). The design level for the top of the sheet piling has been fixed at 0·6 m below the average existing foreshore level, derived from available foreshore surveys covering the last 50 years. The level of the top of the toe beam was fixed to allow adequate formation drainage and an extended construction period for the more intricate cellular slabbing on the main sloping face of the embankment. The toe beam and sheet piling are connected by a reinforced concrete apron slab, laid at an average slope of 1 in 4. By maintaining this slope, scour of the foreshore is minimized provided that the sheet piles remain below beach level. The sheet piling, apron and toe beam function as a composite unit of relatively rigid construction to retain the foot of the embankment (Fig. 11).

27. A modified form of the cellular facing developed previously was specified for stage I, using precast concrete cut-off walls and in situ mass concrete infill panels 0·38 m thick (including blinding) in the tidal zone, and a reduced slab thickness above mean high water (Fig. 12). The infill panels were designed on a gravity basis, the weight of the panels being such that the mass of the fragments to be expected after failure at a prescribed erosion state would be great enough to inhibit movement by the tide.

28. The life of the exposed members is dependent upon concrete quality. In the tidal zone durability is the critical long-term factor, coupled with the workability necessary for bay preparation, placement, finishing and achievement of initial set, all during a single exposure period of about 6–8 h. The concrete mixes for the reconstruction were designed to achieve various levels of workability in the wet condition, a rapid initial set and maximum durability in the finished work. The maximum slump allowed to avoid the concrete flowing in the steeper apron slabs, which have a mean thickness of 350 mm and a gradient of 1 in 4, was 50 mm. Durability under marine conditions depends upon the free water/cement ratio and the cement content. Under coastal conditions, where abrasion from sand and gravel is likely, the aggregate type is of comparable importance. The most severe physical conditions to which concrete is normally exposed are those involving wetting and drying. In these conditions the minimum cement content, suggested by FIP,[4] is 400 kg/m^3 and the corresponding free water/cement ratio is 0·40–0·45. Abrasion is considered to be the principal cause of deterioration on concrete structures along the North Wales and Wirral coastlines. Observations on these structures indicate that amongst the many forms of construction used over the years those incorporating an igneous coarse aggregate have greater durability than those incorporating limestone, despite the tendency to achieve greater compressive strength with the latter (§ 41). Another factor which affects the durability of concrete in sea water is the tricalcium aluminate content of the cement. It is still uncertain how significant this is in temperate climates and unfortunately on stage I records of the C_3A contents were not immediately available.

29. A trial section of mesh reinforced guniting was applied to a section of old facing, but the thickness in places was in excess of 150 mm on account of the variable nature of the embankment surface, and the method was not pursued because of the inherent slow production rate and the relatively high cost of the process in these conditions.

30. Fibre contents were examined, but the only one not likely to increase material costs by more than 20% was a polypropylene fibre used in the highest concentration practicable. Theory did not suggest any appreciable increase in strength but nevertheless trial fibre-reinforced slabs have been placed on the embankment for observation.

31. The life of a structure exposed to wave action is also closely governed

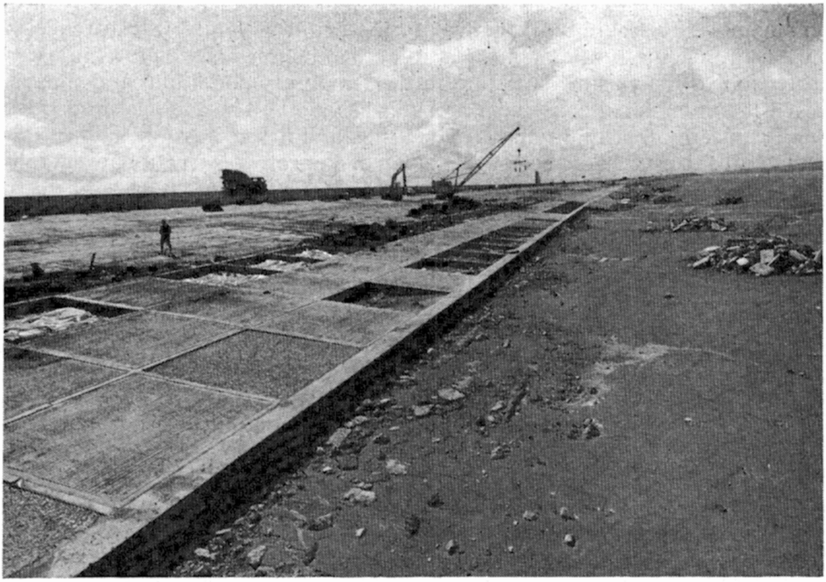

Fig. 12. Reconstruction stage I facing under construction showing some unfilled cells in temporarily stable condition (the infill concrete in one of the bottom-row cells has been damaged by sea action and was replaced in due course)

by detailing, particularly of exposed edges and joints where shock compression of entrapped air can cause spalling and lead to early disruption of the concrete. To reduce this vulnerability in the reconstruction all significant exposed edges and joints are radiused. Away from the slab edges a ripple finish has been applied by hand tamping, partly to provide a safer walking surface and partly, at the apron, to lessen the boundary layer run-off velocity and hence to reduce scour of the beach at the toe of the embankment.

32. The provision of the wave wall mentioned in § 12 has been included in the reconstruction and the current design is completed by the establishment of an access road for maintenance vehicles and plant along the crest, superseding the old barrow-way, with slipway access to the facing. The road has become necessary following the development of construction plant, and it provides the subsidiary benefit of serving as a pleasant promenade for the general public.

Reconstruction: stage I

Methods

33. In stage I (Fig. 13) the Contractor elected to precast the toe beam sections, and this method proved successful, although the weight of each unit (over 8 t) complicated handling. It was found quite practicable to store the precast sections along the foreshore before placing. When placed in position the toe beam sections, if slightly haunched with concrete to prevent base lubrication, were stable in all the sea conditions experienced and were installed prior to tidal concreting in any area. Once the toe beam was in place and

temporarily sealed to the old facing with a skin of concrete the embankment was temporarily secure until the new facing could be laid. It also rendered the existing facing in the area for apron construction redundant as integrity protection to the existing structure in the temporary condition, and thus allowed the unhindered construction of this element. The average production rate for placing the toe beam was 20 m/tide, and it was possible with certain precautions to lay in advance of the facing and apron construction, eventually allowing additional working faces for both elements.

34. The line of sheet piling was severely obstructed by old facing blocks and other debris, but once these were cleared it was found possible to drive up to 20 m of piling per tide. Alignment was generally good, with a maximum deviation of about 50 mm.

35. The apron is the lowest in situ concrete element in the reconstruction and its construction was affected by fluctuations in disturbed foreshore levels on a tide-to-tide basis. A maximum period of 6 h is available from the time the tide leaves the toe until its return, and during this period the apron area had to be excavated to formation, the edges trimmed (especially around the sheet piling), a stop-end shutter erected and secured and threaded anchor bolts fixed to the sheet piles. Individual apron pours were on average approximately 11 m^3 each. After the initial working-up period an average production of 2 pours/tide per gang was achieved.

36. It was observed that precast units weighing 6 t, standing near crest level on the embankment, could be moved by the swash from heavy seas. It was found essential to backfill excavations and to reinstate the facing as soon as possible, even along the crest, up to over 4 m above high water spring tides, to avoid disruption of new work.

37. During the early period of the contract, installation of the cut-off units in the facing was found to be difficult, the difficulty being accentuated by bad ground conditions. The walls were therefore redesigned as a series of self-stable cruciform units (Fig. 14). An additional beneficial effect, due to the reduced haunching necessary, was a further reduction of possible settlement restraint upon the infill panels, thus achieving a more flexible pavement. In the swash zone above the sixth panel, the precast cruciform units were replaced by a grid of standard concrete road kerbs, bedded and haunched, and set flush.

38. The life of the facing depends on a high standard of workmanship. Tight tolerances were laid down and observed for precasting the units, but due to the poor ground conditions it was frequently found difficult to achieve accurate placing. Old facing blocks, $0{\cdot}60 \times 0{\cdot}90 \times 0{\cdot}15$ m, were used as founding pads under each leg, and hardwood wedges were used for fine adjustments to level. After placing, simple purpose-made clamps were used to secure the units in the temporary condition until the unit and infill panel were backfilled and concreted. Average production per gang during good weather in a single tide was four cruciform units placed or six infill bays concreted.

39. When work began in January 1973 the structure was far from uniform in facing texture, being aptly described as a 'patchwork quilt' of sandstone blocks, in situ concrete, concrete block work of many shapes and sizes, and cellular construction. The full extent of variation in depth of parts of the existing concrete facing was not appreciated at that time, nor was the variable character of much of the underlying formation in respect of its resistance to

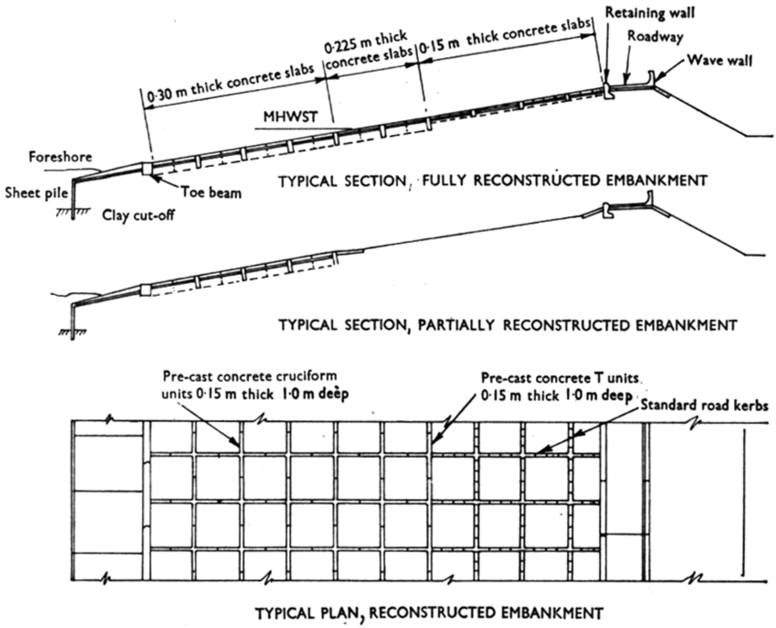

Fig. 13. Typical plan and cross-section for stage I of the reconstruction

Fig. 14. Placing precast cruciform units in stage I facing

tidal action when exposed. The unexpectedly varying conditions meant that significant areas of the existing surface took longer to remove, and that the formation of the structure was exposed longer. For the major part of the work these conditions were overcome by the design and construction modifications described above.

40. The concrete employed within the tidal zone on stage I of the reconstruction had a free water/cement ratio of 0·44 and a cement content of 435 kg/m^3; with this mix a 40 mm slump was obtained without the use of a plasticizer. In situations clear of the high tide swash level the cement content was reduced to 360 kg/m^3, as suggested by FIP,[4] and the free water/cement ratio was increased to 0·52.

41. With regard to materials, the coarse aggregates available in the area are a rounded irregular quartzite from the Staffordshire Bunter beds, a crushed angular irregular limestone from Lancashire or North Wales and a crushed igneous rock, locally known as Penmaenmawr granite, also from North Wales. The quartzite was used for some time for work above the tidal zone but was eventually discarded on account of lignite contamination and replaced by the limestone, an aggregate which is attractive as it contains cementitious fines which increase the 28 day cube strengths by between 1·0 and 2·0 N/mm^2.

42. Mersey grits, which feature prominently among the fine aggregates used in the area, were used. They are found in a wide range of gradings, varying from zone 3 to zone 6. The degree of fineness depends upon the location from which the material is dredged; the very finest grading is landed at Liverpool and was regarded as not available for use on the embankment. An unfortunate disadvantage of Mersey grit is its occasional contamination with lignite, the origin of which is not known. The degree of contamination is, however, relatively small and has been tolerated, mainly due to the high cost and/or equal contamination of the fine aggregates from alternative sources.

43. It is well known that the durability of concrete roads which are normally subject to both wetting and drying and freeze/thaw conditions is markedly improved by the use of an air-entraining agent. Such admixtures were not permitted as it was considered that the suspected reduction in abrasion resistance would probably outweigh the advantage of reduced moisture movements. In this, account was taken of the fact that the embankment experiences freeze/thaw conditions far less frequently than inland sites, due to the effect of the sea. Other admixtures used at various times were a calcium chloride accelerator in solution, which was necessary to hasten the set of the unreinforced tidal slabbing in cold weather, and a carboxylic plasticizer, also in solution, which was found very effective in the retaining wall and wave wall construction to eliminate surface voids. In addition during the summer months the retarding properties of the plasticizer were advantageous.

Emergency operations during a storm

44. On one occasion both adverse conditions described in § 39 occurred at their most extreme, and their combined effect led to the development of the most dangerous threat to the integrity of the structure since the major break of 1919 (Fig. 15).

45. The Contractor for the stage I contract did not work through weekends, but in severe conditions weekend inspections were undertaken after each tide.

Fig. 15. Part of the deep break in November 1973 showing old 'non-tidal' facing concrete undercut by wave action

At an inspection at 1600 h on a Sunday afternoon in November 1973, after three successive tides of 8·8 m above chart datum (Liverpool Bay datum), with a following NW wind of force 7–9, it was discovered that an area of the existing structure approximately 90 m² adjacent to incomplete new works, had begun to break up. A labour force was recruited, but only backfilling of the area with clay was possible at this time.

46. The affected area was known to be protected by a continuous mass concrete skin 150–200 mm thick extending from the working area to the crest of the structure, with no apparent cut-off walls. What had not been realised until the tide exposed the edge of the area was that here the underlying clay was absent and the formation consisted of brick hardcore and demolition rubble.

47. Inspection after the night tide showed that the clay fill had been removed and the break up of the area had extended. The Contractor mounted a major repair operation for the Monday afternoon. Meanwhile the morning tide accelerated the deterioration. The area had now extended to 600 m² with a depth on the landward face of 3 m below adjacent paving level, constituting a major break with definite breach potential. The Contractor's repair effort was plagued with almost impossible working conditions (driving hail and sleet, in force 10 winds), and due to the poor formation and the extent of its exposure his access was severely hampered. The technique adopted was a general backfilling with available material and protection with an in situ concrete cover. The extreme ambient conditions, together with the increased

area for repair, resulted in little confidence being sustained by the Contractor and the Chief Resident Engineer in the survival of the repair.

48. Inspection early on Tuesday morning showed the repair work to have been obliterated and the break to have worsened to such an extent that the danger of losing an entire section of the structure was imminent. The area of the break was now 800 m^2. Temporary backfilling was undertaken before the Tuesday morning tide and 'urgent repairs' were put in hand for the Tuesday afternoon. These repairs were based on a 'terracing principle', taking advantage of the fact that a tide attempts to reduce all upstands to a horizontal condition. Thus it was hoped that if a tide-resistant bund could be established across the lower section of the break, connecting the existing structure remaining on each side, the landward side could be backfilled to the level of the bund with natural fill only and would still be resistant to tidal action. The procedure could then be repeated at successively higher levels until the break was secured with final tide-resistant sealing between the fill and the existing structure along the sides of the break. Facing the area with an in situ concrete skin could then be carried out at comparative leisure. This method had already been tried in a makeshift manner on the Monday evening with the facilities then available. On that occasion it had proved ineffective, but it was felt that the principle was sound.

49. The material chosen for the lower bund wall was standard cement-filled bags. This material was chosen for its immediate availability as well as its resistant properties. The upper bund wall comprised rejected toe beam units and the natural fill was crushed rocksand from nearby rock tunnelling contracts. Over 100 t of bagged cement were placed by hand and machine in the lower bund, which was approximately 40·0 m long and 1·5 m high, pumped sea water being used to cool the bund during all stages of construction. Over 1000 m^3 of rocksand were placed behind the bund wall and three 5·0 m long toe beam units were established as the upper bund wall.

50. All this work was completed on the Tuesday evening (7 h low water period) in weather conditions as severe as those of the previous evening. Added to that, the evening tide was again over 8·8 m above chart datum and backed by a NW gale of force 9 with gusts up to force 10. Inspection on the Wednesday morning showed the scheme to have been completely successful. Some minor erosion had occurred along the edges of the break which had not been sealed the previous evening. These edges were sealed, and held during the next tide. The break was then finally protected with in situ concrete over the next two low water periods.

51. The weather conditions were extreme and the tide levels were in excess of 8 m above chart datum for a period of 8 days commencing 3 days before the discovery of the break. As a consequence of this experience intensive operations within the winter period were abandoned, although this measure served only to reduce the degree of risk inherent in the work.

Stage II

52. For stage II the modified facing design, which proved successful during stage I, has been changed by using precast T units in place of the cruciform units to form the sides of the square bays (Fig. 16). This should provide an improved facility for correcting alignment errors, and virtually eliminates the need for 'specials' at the top and bottom of the replacement areas. Initially

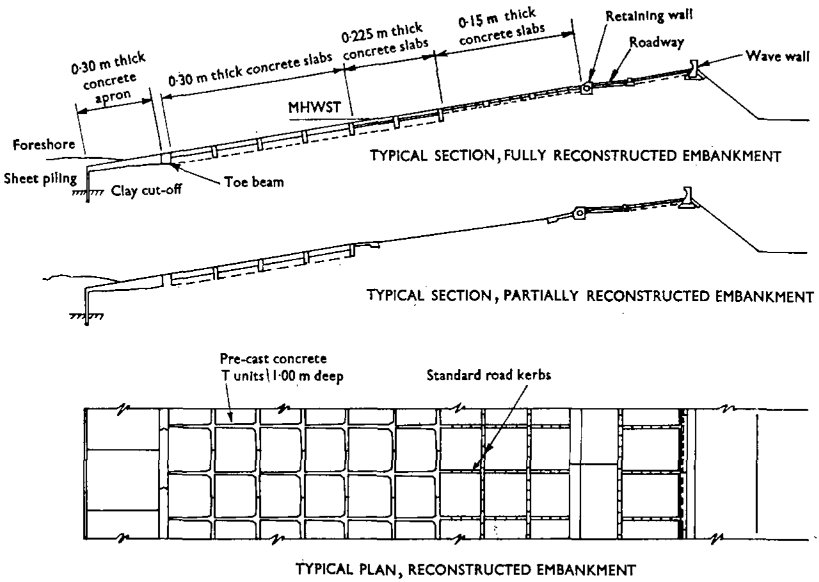

Fig. 16. Typical plan and cross-section for stage II of reconstruction

some handling difficulties were experienced with the cruciform units, both in the precast yard and on the embankment. A number of units cracked at the roots of the arms as a result of minor mishandling. To overcome this tendency it has been decided to introduce tapered arms and radiused roots for the T-units. Some lack of control of the desired surface texture of the slabbing was experienced in stage I where the ripple finish was produced by hand tamping. This aspect is under review for stage II.

53. In stage I the wave wall was cast in situ, monolithically with the access road slab. In stage II the access road is aligned some distance below the crest, allowing the wave wall, which is retained along the crest, to be an independent element precast in sections and set in a mass concrete footing.

54. On account of the variability of the supply, the mix design was examined to determine the effect should the finer sands mentioned in § 49 be substituted. It was interesting to learn that local firms preferred to work with the finer sand owing to the achievement of a mix which was cohesive and therefore avoided the problems of segregation produced with the coarser sands. Despite the increase in cement required to produce workable concrete the material was still economic owing to a cost saving on the finer sand. In view of the rather narrow precast concrete T units, the available flowing concretes were examined at the end of stage I and experiments were undertaken with admixtures of two different types. Neither was successful but the possible cause of the failure of one was shown to be 34% fines content of the aggregate used in the stage I mix, as opposed to the 39% considered necessary for a satisfactory flowing concrete with that particular admixture.

55. In the Liverpool area it has been standard practice to base concrete mix designs upon total water/cement ratios and this precedent was followed

for stage I. It is considered that a more consistent concrete is achieved if the free water/cement ratio is used in conjunction with the 20 min aggregate absorption, and this has been specified for stage II.

Conclusions

56. In the Authors' opinion the Wallasey Embankment may fairly be described as a successful structure in relation to the following aspects:

(*a*) its function has been fulfilled under conditions of increasing severity and it has never been breached, having withstood the onslaught of the sea for some 140 years;

(*b*) readily available materials, some of them by no means ideal, have been successfully employed and currently remain as integral parts of the embankment;

(*c*) as far as may be judged, the embankment, which has a profile somewhat similar to a natural storm beach, has caused negligible disturbance to the foreshore regime and adjoining coasts;

(*d*) recognizing that there is no permanence in any sea defence and that work must always be carried out where it is desired to resist dynamic forces of such magnitude and persistence, the protective facing is designed to confine ultimate breakdown to individual cells which then remain stable in the short term and allow permanent reinstatement to be effected simply and rapidly;

(*e*) aesthetically the long curved sweep of the embankment, with its gently sloping profile merging into the wide expanse of sand which lies uncovered at low water, avoids an unpleasant intrusion into the landscape often experienced with structures of this magnitude; this aspect, together with its function as a promenade, gives the structure further important status as an amenity for the people of Wirral and Merseyside.

57. The Authors consider that the principles of the design and construction ultimately developed and proved on this project provide a viable alternative for examination when approaching the problem of a new sea defence, especially where land reclamation forms an intrinsic part of the work.

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