

The reconstruction of King's Wharf, Suva, Fiji

by

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Dr T. M. Prus-Chacinski (Partner, C. H. Dobbie & Partners) said that the Paper was of very great practical value and contained a lot of useful information, including cost figures. The difficulties during construction were mostly due to soil conditions. These were overcome with great speed and determination and the result was certainly satisfactory because the total cost of the job was less than estimated, and it was finished five months ahead of the programme. This spoke very well for the general planning, the good supervision, and the manner in which difficulties were promptly dealt with.

41. The speaker said that he had experienced difficulties with piezometers. Even in ground which seemed modestly impermeable the water level had followed tides. He knew of at least two instances in Britain, although in one case there had been a considerable time-lag, measured in several hours, so he did not know whether the piezometers were showing wrong readings. There seemed to be insufficient information on this problem.

42. The acceleration due to earthquake had been taken as 0.1 *g*. This was the figure normally assumed. He had been concerned in the design of some structures in New Zealand using the same figure, but the Japanese now used 0.2 *g*. Of course, Japanese conditions were more dangerous from the point of view of earthquakes, and in any case this type of construction would seem to be quite safe because the wharf would not be particularly vulnerable.

43. The Author had mentioned in the Paper that even if the ship wall failed the wharf should be safe, and recent Japanese information appeared to confirm this.

44. In Appendix II the Author had described the pile testing, and he completely agreed with his comments on the formula for this sort of ground. The speaker had known more than one case where the soil mechanics specialist had advised that the best thing to do was to go somewhere else or to do something which would cost about ten times more than the budget allowed. But the test pile in these cases had demonstrated quite a large bearing capacity, due to what was commonly known as 'suction'. Unfortunately, very little was known about this, and there was a considerable field for useful research here. The difficulty was that some silts produced this suction and some did not, and it was not possible to predict with any certainty whether a given seal was of one type or the other.

45. The Author had also mentioned in the Paper that it was necessary to use 90 tons force to pull the pile out. The speaker suggested that if this silt were analysed by an expert in soil mechanics it would be found to have no bearing capacity at all.

46. This effect, he thought, must be due to particles of silt plus water. He had read a few papers about it. Test piles could be quite difficult, particularly for small jobs, because sometimes the cost of test piles was quite substantial, but he recommended using them in all cases where soil mechanics would not recommend piles and they were obviously the cheapest solution.

Mr D. H. Little (Engineer, Walter C. Andrew & Partners) fully agreed with the Author that pile driving dynamic formulae did not work in this sort of ground. Indeed, he would suggest they did not really work in any sort of ground. His experience differed from the Author's in that he usually found pile driving formulae gave too high an assessment, whereas in this job they gave too low an assessment. A classic example occurred during the war in Belfast Sleaford, where the ground had a shear strength of no more than 150 lb/sq. ft and the dynamic formula on certain piles gave a result of 40 ton/pile. The four piles were capped with $15 \times 15 \times 4$ ft thick slab to make a dolphin. This resulted in a dead load 16 tons per pile, and each settled nearly 3 ft, so that the 40 tons was a gross over-assessment.

48. A common standard pile driving specification is 1 in. for 10 blows, repeated three times, and that would not be regarded as refusal. But on this job $\frac{1}{2}$ in. per blow was regarded as refusal. This was nearly 3 in. for 10 blows. The generally accepted figure on this job was 3 in. per blow, which was 2 ft 6 in. for 10 blows. This was not pile driving but pile pushing. As the piles were 140 ft long and weighed 26 tons, and the hammer only weighed 12 tons this was inevitable. But it did not seem to matter, in this sort of ground, how piles were put in. The resistance was invariably proportional to the shearing strength of the soil times the embedded area of the pile.

49. He had thought he was clear about the design until he saw the slides. From the drawings he had thought the bridging portion was continuous but from the slides it seemed that it was only intermittent. If it had been continuous it did not seem to him that the structure as a whole would be very safe against earthquakes. He would have thought an earthquake would bring the old sheet piling down and that the bridge would fall down with it, so that it would not be possible to get on the wharf. In the design adapted piles with a carrying capacity of 56 tons were spaced over areas of only 100 sq. ft. This would mean about 20 tons of live load per pile, plus 10 tons of decking, and the pile itself weighed 26 tons. When there was a 12 ton hammer just resting on the pile that was almost a working load. He realized that a technique in existence on the job had been used, and certainly the piles, at £500, were not all that expensive, but he thought a neater design would have been circular steel piles, 30 in. in diameter, and $\frac{1}{2}$ in. thick. These would weigh 8 tons. The ground was poor but nothing like as poor as much in the north of Scotland and in Belfast. It had a mean shear strength of 800 lb/sq. ft and a pile 100 ft long with an 8 ft girth would have taken a 75 ton load so that pile spacing could have been doubled. The bridging section could have been incorporated in with the wharf and there would have been a complete wharf 75 ft wide in which there was an embankment dipping down under it. If that had then slipped in an earthquake the jetty as a whole could have been repaired a bit quicker than this one could be in the same circumstances.

50. With regard to the deterioration of the timbers, it appeared from the slides as though the topmost timber bearers were being eaten away. Did they suffer from Teredo in that port, and how would greenheart have behaved? Any jetty in this sort of ground was really floating in the ground and possibly a better approach, if the timber could stand up to it, would be to have a lighter weight timber, something lighter than a pile weighing in itself virtually as much as the load coming on to it.

Mr G. B. Lewis (Heap and Digby) said that his contribution would be concerned with the last two words in the Paper, which were 'cathodic protection'. Before dealing with this he wished to say how sympathetic and understanding were the Author and his team in providing accommodation for cables, etc, invariably in those parts of the structures where steel was thickest.

52. The difficulties of the site, to which the Author had referred, were reflected in the problem of affording adequate protection against corrosion to the steel pile surfaces. The protection of the surfaces exposed to sea-water was a comparatively

simple matter but there was a problem in making sure that adequate protection was given to the landward side of the main wall and to both sides of the anchor wall.

53. It was decided that an impressed current system was essential but that the way it should be applied could best be left to the specialist manufacturers to whom the problem would be referred in its broadest terms and to give them a clear instruction as regards the standard of surface protection to be afforded and of electrical installation, insulation, cable fittings, etc. Four bids were received, one at £25 000 proposed an array of anodes buried between the walls and the other three were of £13 000, £11 000 and £10 000 for systems employing a series of anodes to seaward of the main wall.

54. The lowest was investigated, selected and in due course installed.

55. Records received from test points on the site over the first year and a half of service have shown that adequate protection is being afforded to all surfaces. The installation comprises a series of non-deteriorating anodes suspended beneath the main decking and these effectively protect the seaward surfaces. For the 'tween wall surfaces a smaller number of anodes were suspended beneath the decking as far to seaward as practicable. The theory of the firm concerned was that the current path taken from the farthestmost anodes would be directly downwards into the sea-bed and that from there it would distribute via the ground mass to the 'tween wall surfaces. The test results taken indicate that this is in fact being achieved. Along the Prince's Wharf to the east, where there was no detached wharf, the anodes were installed close alongside the piling.

56. The principal difficulty that was met was the use of hard plastic conduit and fittings which were totally unsuitable. They have been proved perfectly satisfactory in similar installations in this country but in the high ambient temperature of Fiji they wilted noticeably and had to be replaced.

57. The protection system as a whole consisted of eight transformer rectifiers each having a rating of around 8 kW. Practically all that output was used before the surfaces were polarized, but as polarization of the landward surfaces came about, the demand by each transformer rectifier fell to around 4 kW and the records indicate that it is tapering off at about this level.

Mr A. F. Daldy (Senior Engineer, Building Research Station) said that he had particularly enjoyed reading the Paper, having visited Fiji some months ago.

59. Referring to Fig. 5, he asked for a little more detail as to the articulation joints between the various sections to allow for relative earthquake movement. What was the size of the opening? Was it something like an expansion joint on the bridge over the top of the piles?

60. In § 28 there was a reference to the fill in earthquake conditions. This had been done slowly and carefully, to give plenty of time for consolidation, but, even so, there was a bit of outward and downward movement of the sheet pile curtain. In the next earthquake, if there was the sudden shaking and compaction normally experienced, might there not be a severe movement of the curtain piling? Would the Author give some explanation of what he expected here, perhaps with reference to not having some additional fill on the outside of the curtain, which might, when it also compacted, assist in preventing the movement of the pile curtain?

Dr A. J. Dutt (Senior Structural Engineer, Greater London Council) said that Suva was located in an active seismic zone. Earthquake and hurricane effects appeared to be predominant factors in the design considerations for the King's Wharf, and as such he wished to raise a few points on these and other related aspects.

62. It had been stated in the Paper that an acceleration of 0.1 g had been assumed to act horizontally through the centre of gravity of the vertical loads. An earthquake would produce very high sea waves, as distinct from an earthquake in an inland area, in addition to the normal sea wave. High sea waves could result in an uplift combined with the horizontal forces of the earthquake and the gale. Had any

consideration been given to this effect caused by the high sea waves produced by earthquakes?

63. Another point worth considering was the estimation of the period of vibration of the wharf structure which was so much more vulnerable to earthquake damage. With this was also associated the yielding of the ground under the foundations. This consideration was more important when the soil conditions were unfavourable.

64. Had any special protective measures been provided for the 30 in. diameter prestressed concrete piles subject to sea action? Normally a concrete cover of 2½ in. was provided to the reinforcements in marine structures. The thickness of the walls of the piles being 4½ in., it would not be possible to comply with this requirement for the prestressing wires.

65. Would the Author like to say something about the structural framing of the transit sheds? It was necessary to make a strong and rigid connexion of the members in the structural framing to avoid excessive deformation and to maintain initial shape under high seismic stresses.

66. Could the Author give an idea of the cost that would have been incurred if such a project were to be built in this country, taking into account the high cost of materials and labour?

Mr G. W. Rooke (Consulting Engineer, Lewis & Duvivier) said that a number of the questions he had prepared had been adequately answered by the Author's excellent photographs. It was difficult to comment constructively after reading what was only a brief résumé of quite a large project, and without knowing all the background to the hard thinking which must have gone into the preparation of the design.

68. What was the size of the original timber piles in the King's Wharf? They must have been very long and he presumed they were round piles. Within what limits did their diameter vary from head to tip?

69. The Author had stated in § 12 that he experienced no trouble when driving the sheet piles in the apron wall due to obstruction. He certainly had good cause for congratulation that this proved to be the case.

70. The Author had also stated in § 12 that the wharf itself was designed to be earthquake resistant. Had it any earthquake resisting features other than the fact that it was not rigidly connected to the apron at the back?

71. The design of the deck was of beam and slab construction. He had been designing heavy slab decking for some 20 years and still preferred that system from the point of view of ease of construction and also maintenance and the spreading of impact loads from the fendering. There might be some good reason why it was not adopted here and it would be interesting to have the Author's views on whether he considered it would have been suitable. The spacing of the piles was 15 ft and 7 ft 6 in. and this would have avoided the necessity for any very heavy soffit shuttering or supports.

72. In § 14 of the Paper was a description of three articulated joints in the 1500 ft length of the structure. Were there any other intermediate contraction joints of any sort? He asked this with particular interest, since during his early days at Southampton he had found that the more contraction joints were put in the more the concrete contracted, so that when designing wharf widening some 1600 ft long he had only put in two contraction joints.

73. Was the fender thrust regarded as taken by the structure itself? If it received the heaviest blow for which it was designed, would the piled structure have closed the 6 in. gap and thrust against the bridges at the back? Possibly the Author was satisfied, with the great length of the structure, that all those piles would be sufficient to take the thrust without the 6 in. movement, which was fairly substantial. On the other hand, the piles were very 'leggy', and he supposed they were quite flexible, in spite of their large section.

74. He had noticed from one of the photographs that one end, at any rate, of the

piles (presumably the lower end) appeared to be plugged. Was this the case or was any light protective shoe provided round the rim? What was the design of the toe? If, on the other hand, the lower ends were open, was there any record of how far the material into which they were driven plugged the toe and rose up into the pile?

75. The piles were pitched by lifting at three points. Had it been necessary to take any great care in order to ensure that any of the supporting ropes did not get too long or too short and over-stress the piles?

76. With regard to the impact, and the ineffectiveness of dynamic formulae for calculating the bearing capacity of piles in cohesive ground, everyone would be entirely in agreement with the Author. He had been under the impression that these impact formulae gave too high a figure, but all the soil mechanics experts had assured him that they gave too low a value, and he was prepared to accept this.

77. Referring to page 538, Fig. 6, what was the progressive settlement of the test pile during the 88 hours of the second cycle for which the 100 ton load was held? It had seemed to settle quite a bit and he wondered whether it had started off at a rapid rate and then faded out to virtually no settlement over the last quarter or so of the 88 hour period.

78. He noted that the shear strength of the clayey, sandy silts was given as 800 lb/sq. ft combined with a ϕ of 30°–35°. This seemed to be rather unusual. Was the Author surprised to get such a high ϕ in a cohesive material? A ϕ of 15° or 20° would be normal for cohesive materials, but 30° to 35° was approaching the ϕ of a granular material, and he found it hard to reconcile such a figure with a cohesion of 800 lb/sq. ft.

79. Having been responsible for three contracts for sheet piled walls in almost the worst ground conditions imaginable, he agreed that he might be inclined to be a bit jittery about them, but the Author appeared to him to have sailed fairly close to the wind in his design of the apron wall. He may of course have had perfectly good reasons for doing so. He was obviously keeping an eye on the anchorage, which was a vital part of the structure. Why had he not carried his sheet piling a little bit below the two critical slip circles and got a little more toe support for the sheet piling, just in case?

80. Was the Author surprised that the curtain wall had settled as much as 6 in. and moved forward as much as 5·7 in.? These seemed to be rather substantial movements for a sheet piled wall and were, he felt, somewhat surprising. Could the Author enlarge on what he had written in the second half of § 29 on page 541: 'Looking back now the Author tends to the view that they were, in fact, due more to the location and method of construction of the anchor wall, which resulted in appreciable "take-up" of the material in front of the anchor wall before stability was reached, than to any deep-seated ground instability.' Would the Author amplify this statement to some extent in the light of his detailed knowledge of the design and construction?

81. With regard to the pile bearing formula, he felt that soil mechanics experts would agree with that given by the Author, who had taken the support provided by the toe and shaft of the pile in the cohesive material. He had, however, not allowed anything for the friction of the sandy element for which a ϕ of 35° had been given. There was no doubt good reason for this but it seemed to call for a little explanation as to what had happened to the ϕ of 30° to 35° in the formula. If it had been taken into account it would have substantially increased the bearing capacity above that arrived at by the Author.

82. The figure for the extraction of the timber pile, 90 tons, was most interesting, but had the Author any records which would give the average value of the extracting force these timber piles had required? It was always valuable to know what a pile would take in tension but no one ever seemed to have any accurate ideas on the subject for any given set of ground conditions.

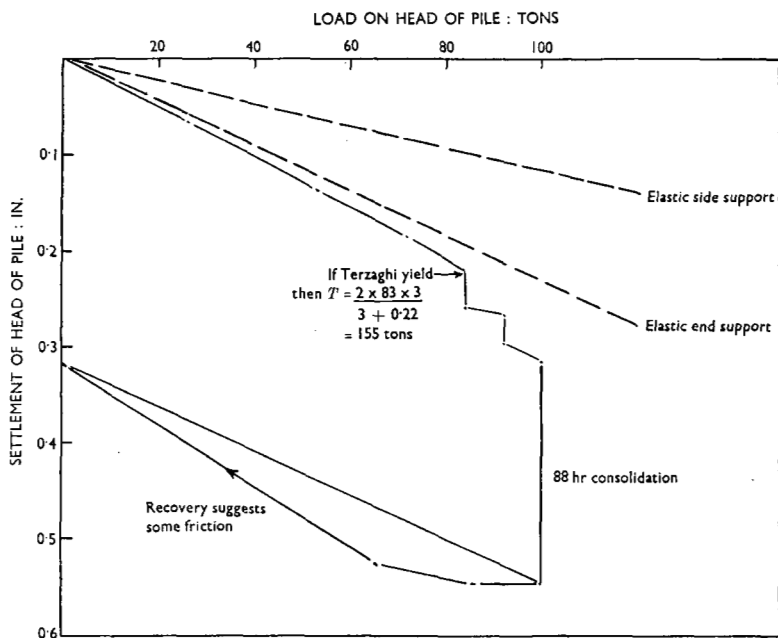


FIG. 15: MR BULLEN'S REDRAWING OF FIG. 6

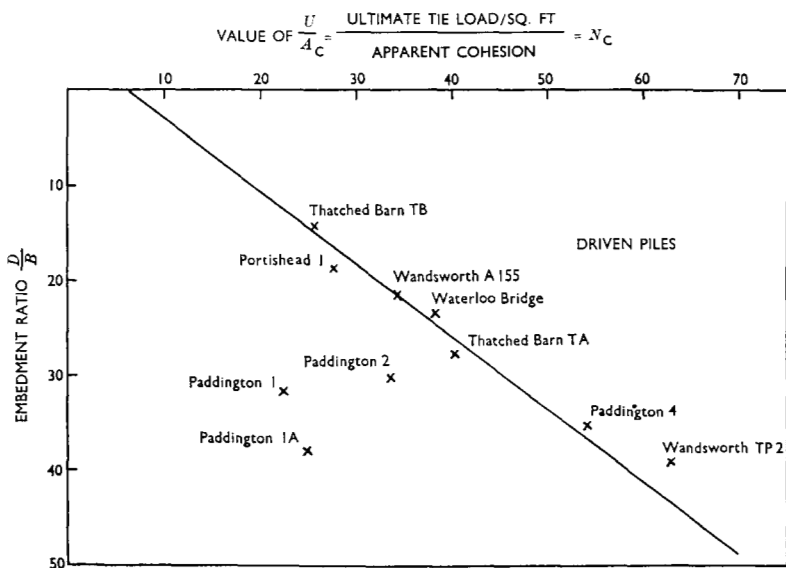


FIG. 16

Mr F. R. Bullen (Consulting Engineer) said that he had noticed the spacing of the piles was 7 ft 6 in. for a 2 ft 6 in. diameter pile. Some had thought that for friction piles the spacing should be a little larger than that, and possibly the circumference of the pile. On the other hand, if they were end bearing piles, twice the diameter of the pile was thought to be sufficient; clearly it was assumed that the load was carried partly in friction and partly in bearing!

84. With regard to the embedment of these piles, was he to understand that the 80 ft pile would be embedded about 30 ft? If so, according to the Author's figures, the ultimate friction would be about 42 tons. He took it that a 140 ft pile would be embedded about 90 ft, in which case the friction would be of the order of 126 tons.

85. He questioned a term which was new to him. The Author stated that K1 and K2 were 'factors of restitution'. There were certainly factors which tended to relate adhesion to shear strength but he had not thought of these as 'factors of restitution'.

86. Considering the load on the toe, and assuming a cohesion of 1000 lb/sq. ft, it seemed to him that the toe value would be only about 22 tons, so that it would follow that the 140 ft piles were predominantly friction piles, and there was little point, therefore, in driving them into the soapstone.

87. He had ventured to re-draw part of Fig. 6 (Fig. 15) in a way which he found easier to follow. This diagram seemed quite normal for a partially tested pile. If the pile were considered as having been entirely supported at the toe, with no friction around it, and the load were applied at the head, the movement of the head would be in the following form added to the diagram. If, on the other hand, the pile was entirely supported by friction, then the movement of the head would be as shown in the diagram.

88. The recovery would be seen to have taken place along the line indicated in the diagram, so that he concluded that the pile was behaving as if supported almost entirely at the toe and only partly by friction. This conflicted with the calculation made previously.

89. He would venture to comment upon the Author's suggestion that a static calculation was necessarily right. The speaker would not be prepared to say unreservedly that a dynamic one was right, but he would not agree that a static one was right either. Possibly the dynamic one might be right if the formula used were correct. How, then, were the piles supported? With driven piles in cohesive materials a plot relating N_c to embedment ratio could be a straight line, meeting the horizontal axis at a value of about 6, which it should do.

90. On the piles that were being discussed (Fig. 16), taking the embedment length over the diameter, he found that N_c was about 50 (he was sure this was wrong by all the soil mechanics arguments, but this was what one found by experience). Taking this value the toe value would be about 120 tons, and if one could then offer the suggestion that the average friction or adhesion on the piles would be about 0.1 ton, or 2 cwt/sq. ft (the sort of friction one would think of in sinking a caisson or something like that), one obtained 70 tons or so in friction, so that there was a total value for the pile of 190 tons at ultimate.

91. The Author had suggested that one of the kinks in the settlement diagram should be regarded as a 'Terzaghi kink'. The speaker's own term was 'Terzaghi yield', as mentioned by the Author (page 539, § 23). If this were applied in the way which had been described, the ultimate value would be of the order of 160 tons, which was not far away from the previous figures. But much of this was speculation because one had not enough information; he wanted therefore to ask whether it would be possible to have more details of the helmets which were used, and the packings? This was information which was regularly omitted from papers, though he did not think the Institution should be blamed for it.

92. He would like to know a little more about the hammers which were used, their make, if possible, and whether the Author had at any time applied varying drops

of the hammer to a pile and measured the varying sets, so that some idea could be gained about refusal drops?

93. He had made some assumptions which might well be wrong due to the lack of information, but if they were anywhere near right his calculation suggested that these piles, assuming the toes were solid, had a bearing capacity at the toe of about 90 tons ultimate, which was not far out from the figure he had given earlier, and if one added friction at 70 tons, making a total of 160 tons, which he believed to be the value of these piles in the ultimate, one could only say what a pity it was that the tests had not been carried nearer to the ultimate load.

Mr T. N. W. Akroyd (Consulting Engineer) said that his questions were particularly concerned with the clayey silt and the problem of the angle of friction ϕ . It had already been mentioned that a value of ϕ of 30° – 35° was quite high with such cohesive materials. They were described as clayey, sandy silts. Could it be that these angles of ϕ were measured in the normal undrained triaxial test? If so, were they not merely a function of the saturation of the sample, and, since the sample was more than likely to be saturated *in situ*, were they not a function of the sampling of the silt? This could quite well be, because it seemed unlikely that one would develop an angle of ϕ of 30° – 35° in a material which was silt unless one had a negative pore water pressure. If in an undrained triaxial test on silt the cell pressure were raised high enough, then one would obtain an angle of ϕ equal to zero. If this kind of condition could be produced during the behaviour of the soil *in situ*, then the angle of ϕ must be discounted. As soon as one talked about clayey, sandy silts and earthquakes, one could immediately connect the two, silt could become liquefied due to the generation of positive pore water pressures during earthquake movements, and hence an angle of ϕ equal to zero appeared to be the right solution. To make an assumption that the angle of ϕ might be 30° or 35° *in situ* would be the wrong approach. Ultimately, during the design the ϕ was assumed to be zero. It would appear that this was right.

95. In his experience Mr Akroyd considered piezometers became unreliable because of the difficulty in so installing them that they reflected the pore water pressure of the ground in which they were embedded and not the ground above; this difficulty was due to connecting drainage paths existing after the piezometers had been installed. All the troubles he had ever had with piezometers had been due to the method of installation—failing to ensure that the piezometer itself was cut off from the ground above. If there were a drainage path due to the method of installation, the piezometer could be expected to reflect the normal tidal movement, with a suitable time lag, which depended on the length of the drainage path along the piezometer conduit tube.

Mr E. Loewy (Consulting Engineer, Sir William Halcrow & Partners) said that many years ago he and the Author had been colleagues and had often discussed the conservativeness at that time of Sir William Halcrow & Partners towards the use of prestressed concrete in marine works, and chaffed at what seemed to them unreasonable restrictions on the use of prestressed piles, which could only be used after a lot of argument. Now that the Author was running his own organization he had clearly made amends for that, for on a rough calculation he had 100 000 ft run and 20 000 tons worth of prestressed piles on this job, so presumably he was not worried about the matter any more. He was sure that the Author was right in this, but there certainly were a lot of piles, weighing a great deal and very close together, as Mr Little had said.

97. Could further information be given about the relation of King's Wharf to the over-all master plan of development of the Port of Suva?

98. He found the references to a comprehensive scheme difficult to understand—a few dotted lines on a tiny diagram which did not seem to mean very much.

99. He thought it was right that every port development which had any promise of being only a stage in a long exercise should be preceded by some kind of overall development plan, however fanciful, however imaginary, and however much only in the engineer's mind. Clients were not always very anxious to pay for this kind of thing; all they wanted was some particular piece of reconstruction. But the photograph and the drawing seemed to show that this wharf did not lend itself as easily as it might to further development. It also seemed unnaturally and unhappily close to the town, and he feared that the time would come when this would be regretted and, unbeknown to the Author, somebody would ask, 'Who was the silly engineer who put all this money out on this form of development? Why did not they go somewhere else?'

100. This was a particularly troublesome problem for consulting engineers in developing countries, particularly ex-colonial ones. Their origins and history were much as the Author had described in this case. Wharves were built as close as possible to towns, or wharves were built somewhere and towns gradually grew up alongside them. But everywhere the time came when, for a really significant long-term development, the decision had to be made to abandon existing investment *in toto* and go to a completely different place where there was more space, easier land communications, and the possibility of really modern development, for which ample space was always the first requirement. Could the Author say something about this aspect?

101. The concept of the structure with intermittent bridges between the apron at the face and the apron behind by the sheds seemed to him rather curious. He would have thought it was a fatal hindrance to efficient port operation to have only about 50% solid working area as soon as one passed the first 38 ft, and also to have a portion of the deck surface (for this was what the bridges were) with a much lower permissible deck loading. One of his colleagues had said that the work had been done very cheaply at £8 10s. 0d./sq. ft, so what was there to criticize, however many piles there were and whatever their weight? If one regarded the intermittent bridges as an extemporized solution to a problem partly of money and partly of linkage, and realized that the proper job must fill those in, then there was something of the order of a third of a million pounds to add to the total costs, so perhaps the costs per sq. ft would not look quite so attractive then.

102. He could not quite accept the statements made rejecting simple sheet piling construction and filling in, because it seemed to him that the first reason given, namely, that the subsoil was not considered very suitable to put the filling on, was an even greater criticism of founding friction piles in that material. These did not seem to him to be consistent arguing points.

103. He was not quite sure what was meant by the point that a sheet pile wall of the elemental form had an unpredictable behaviour in earthquakes. He would have thought it was perfectly capable of being designed for, and that the only reason for rejecting such a proposal was that when all the allowances had been made the cost came out too high, though in that respect the Author stated that it was attractive. Perhaps some more information could be given on that. Of course, settlement of the surface would take place on such a construction, but that was nothing unusual and nothing which could not be easily catered for by various means of temporary surfacing or surfacing with precast slabs which could be lifted and packed. That in itself was not an unusual problem.

104. Would the Author not have considered a light form of construction eliminating the point loads that piles must impose, namely, some form of precast caisson Mulberry-type units, for convenience of explanation, which could be left empty at the front and loaded at the back to equalize the foundation pressures?

105. The fact that this material was so bad for friction piles, leading to this enormous length and number of large diameter piles was a reason, he would have thought, for reducing the weight of the structure, to come away from point loads and to have spread loads and the caisson form of construction, or even the circular smaller

sheet pile construction would have considerable merit. Could the Author give some opinions about that, for usually all manner of solutions were tried in an office and they could not all be described in a paper.

The following contributions were received in writing:

K. H. Sørensen (Vice President, Christiani & Nielsen A/S, Copenhagen) wrote to make a few comments to the Author's remarks on the ingress of water into the hollow prestressed piles. Even if it is a fact that quite a number of the hollow piles seem to remain dry it can, generally speaking, hardly be permissible to assume that piles of this type remain so for the reason that one cannot assume absolute water tightness of a 4½ in. thick concrete shell under a hydraulic head of more than 100 ft, particularly so when this concrete has been exposed to driving. In the writer's opinion it would be safest after driving to fill piles of this type with water of the same type as the water which surrounds the piles in order to ensure that there is no tendency for movement of water or salts through the concrete.

107. The reason for the slow ingress of the water can probably be divided into two principal causes: (1) flaws in the concrete, and (2) fine cracks.

108. As regards the first cause the weak points will probably be round precast blocks which are often essential to keep the inner forms in position.

109. As regards the cracks in the piles there are two categories:

- (a) fine cracks which tend to develop at the time of casting the piles because prestress only can be applied after a number of days. In tropical countries the temperature of the concrete may rise rather quickly after the pouring to something like 130°F and after a few days it will drop back to the normal environment temperature of say 80°F. This drop in temperature tends to develop very fine cracks as the concrete is prevented from shrinking by the reinforcement. Most of the cracks will, however, close up when the wires are released but small specks of sand in the cracks may prevent this;
- (b) cracks developed during the handling and driving process, in this respect particularly a problem caused by the tensile stress waves experienced in driving under very soft conditions which can create difficulties in a long prestressed pile. It is a fact that the tendency towards development of tensile cracks is reduced by using a heavier hammer, naturally with correspondingly smaller drops, and this is the reason why a 12 ton hammer was later used in Suva in preference to the 8 ton hammer originally used—although the latter was quite capable of driving the piles to the required resistance.

110. It may be added that experience of reinforced concrete has indicated that cracks of this nature in piled structures are benevolent, provided the quality of the concrete is good and the cover adequate, and there seems no reason why this should be different in prestressed piles where both these conditions are fulfilled generally to a higher degree than in reinforcement concrete piles.

Mr D. V. Buck (late Senior Partner, Messrs Wilton & Bell, Consulting Engineers to the Government of Fiji) wrote that it might be of interest to mention some of the major defects which were disclosed by the earthquake of 1953 which resulted in the Consultants advising the Government of Fiji that the original King's Wharf shown in Fig. 2 was beyond reasonable repair and that reconstruction must be seriously considered at an early date.

112. The damage attributed to the earthquake was not a direct consequence of the shock, but rather of the resulting tidal wave, reported to be a 10 ft surge wave.

113. The hydrostatic pressure resulting from this wave becoming trapped under the decking resulted in:

- (a) collapse of some lengths of the precast concrete retaining wall allowing water to scour out the retained filling behind the wall and thus to release the anchors of the land ties;
- (b) areas of the timber decking were lifted off the supporting piles, thus leaving the heads of the piles free to move;
- (c) the suction of the escaping water following the recession of the surge dislodged parts of the rubble bank, broke off a number of the piles and moved others to an extent which did not allow the timber deck to return to its proper seatings.

114. A survey of the damage also disclosed that joints in the concrete casings of the timber piles had, over the years, become impaired by flexing of the structure under its normal usage, thus allowing marine borers to gain access to the timber cores, which were in many cases almost completely honeycombed; that the heavy timber cross heads connecting the pile heads and also many of the timber deck beams, all above water level, had been seriously attacked by termites to the extent that the timber could be crushed by hand pressure.

115. The timber piles, cross heads and deck beams were all of Australian Iron-barks, Eucalyptus, Turpentine, etc, but all species had suffered equal attack either from marine borers or termites; it must, however, be remembered that the structure was by now some 35 years old.

116. Marine borers are common in all tropical waters, but efforts to trace the origin of the termites were unsuccessful. The particular species was unknown in Fiji and it was, therefore, assumed that it had been imported from Australia in the original timber; this assumption was, however, denied in Australia, where the species was again unknown; no satisfactory conclusion could therefore be reached.

117. Temporary repairs were carried out intended to maintain the wharf in use pending early reconstruction. A number of new R.C. piles up to 80 ft long were dropped into the holes left in the rubble bank by the extraction of defective piles and were driven to a set consistent with their loading. Timber lattice girders were prefabricated and erected on either side of the pile heads across the width of the deck to replace the original timber cross heads where these were found to be beyond repair and a number of the land ties were reinstated. These repairs enabled the wharf to provide berthing facilities for large ocean going vessels for some 8-9 years until the new wharf was constructed in 1961/63.

118. The repairs were carried out by the Public Works Department of the Government of Fiji by direct labour and it is worth noting that very little skilled labour was available in Fiji in those days.

119. He referred to the original design for the reconstruction of the wharf which was produced in 1952/53 immediately before the earthquake and is as shown on Fig. 3. The original design for the wharf reconstruction provided for R.C. piles to be sunk into holes through the rubble bank left by the extraction of the original concrete cased piles. These new piles were capped by transverse R.C. cross head beams which extend seawards to provide the required depth of water at the berth. The seaward ends were to be supported on two rows of 5 ft diameter concrete cylinders sunk into the soft clayey sandy silt by kentledge and excavation of the core material until a sufficient frictional plus bearing resistance was reached. The open bottoms of the cylinders were to be sealed with a concrete plug, an in situ R.C. beam and slab deck providing a monolithic structure.

120. A curtain wall of R.C. sheet piles retained the reclaimed land behind the wharf, the whole structure being designed to absorb the berthing forces and those resulting from hurricane force winds.

121. At this stage, whereas good aggregate in the form of basalt rock was known to exist on the island, it was only known in places quite inaccessible to the machinery for its recovery. The only reasonably accessible aggregates were of relatively poor quality and design stresses were, therefore, kept low.

122. As the Author had already mentioned in his Paper, the original design was later adapted to suit modern construction equipment and techniques and especially the highly trained labour forces then available in the island, as a result of the Contractor's (Christiani Nielsen and Gammon) initial contract for the construction of the wharf at Lautoka and the Slipways at Suva, carried out during the years 1959/60.

The Author, in reply, thanked all those who had contributed to the Discussion; he was particularly grateful to Mr Buck and Mr Sørensen for their written comments. Both had been closely involved in the project.

124. Mr Buck had provided some interesting facts regarding earthquake damage and, in company with Mr Little and Mr Rooke, about the behaviour of timber in tropical waters. Some of the marine borers found in the Fiji piles had been identified as *Terrido*; the Author could not comment on the possible behaviour of greenheart which had not, to his knowledge, been used in Fiji. However, as noted in Appendix 1, Turpentine was proving reasonably resistant and appreciably better than Ironbark, this confirming earlier experience on the original King's Wharf. Referring to timber piles, these were round, being approximately 15/18 in. dia at the head tapering to perhaps 12 in. at the toe. Lengths varied from 50 ft to 90 ft, including one or more splices.

125. Regarding earthquakes, which had been mentioned by several of the contributors, he would point out that continuous bridging would not have had the effect of reducing earthquake resistance of the wharf because of the clear gap between wharf and bridge decks. It was true that collapse of the curtain wall might affect the bridges themselves (although these had deeper, box-pile abutments), but provided the wharf proper remained serviceable, the rapid provision of temporary bridging should be a comparatively easy matter. The articulation joints were simply breaks in the deck, leaving clear gaps of a few inches, which were bridged by sliding plates. Between articulation joints the deck was concreted by the alternate bay method without any permanent joints. This had been entirely successful. The Author believed (despite Mr Loewy's comment!) that it was not realistic to deduce precisely from calculations, the behaviour of a sheet pile curtain, supporting fill, under earthquake conditions. In designing King's Wharf he had envisaged that local damage, due to some quite unpredictable ground disturbance, might be severe. Subsequent actual experience of the behaviour of a sheet pile wharf in an earthquake suggested that his original fears had been exaggerated, but nevertheless he still believed there to be an element of uncertainty. High sea waves had been considered, but the 'draw down' following a high 'tidal' wave could be much more catastrophic than the wave itself in the case of a sheet-piled quay. Mr Buck's comments also answered this point. The transit shed framing was of the normal portal-frame design with rigid knees and apex and hinged feet. The loadings imposed by hurricanes were more severe than earthquake loadings on the sheds; hence the former governed the design. In the case of the wharf itself, in answer to Mr Rooke, apart from its isolation, the structure was designed for the moments and shears developed by the horizontal loading derived from an acceleration of 0.1 g.

126. Three contributors had discussed ground conditions in relation to sheet piling. The Author recognized the difficulty in reconciling the apparent inconsistency of 'poor ground' with a ϕ value as high as 35°. The inconsistency nevertheless remained the fact and the soil mechanics advisers had summed it up by saying, in effect, 'This is a most unusual and interesting material.' Certainly the Author had occasionally used adjectives other than 'interesting'. It was confirmed that the triaxial tests were undrained. Mr Rooke may have been misled by Fig. 4 into thinking that sheet piling had deliberately been terminated at the depth of the critical slip circles. In fact, these circles were the critical circles for piling of the design length, not vice versa. Regarding anticipated settlements and movements, the Author had expected ground settlement immediately behind the sheet piling of perhaps 12 in. (hence the generous surface drainage falls provided); he had not expected the piling itself to be carried

down as much as this and agreed that the maximum recorded piling movements, of about 6 in., exceeded his expectations. The example of 6 in. was an isolated maximum and check point 78, in Fig. 7, gave more typical values. The Author thought that these movements might have been reduced by siting the anchorage further back and by a somewhat more elaborate backfilling procedure. Both of these would, however, have involved increased direct cost and considerably greater time and disruption to the Client during construction.

127. Dr Prus-Chacinski and Mr Akroyd both sounded a rather pessimistic note concerning piezometers which confirmed the unsuccessful efforts with these instruments at King's Wharf.

128. As expected, the main piling aroused considerable comment and was referred to by four of the contributors. Mr Little's description of 'pile pushing', rather than 'pile driving' was most apt. This really confirmed the Author's growing belief that the means of getting the pile into place should not be used as the means of determining its carrying capacity; what was needed by practising engineers was a simple, reliable, method of calculating safe load from soil properties. It was confirmed that a 140 ft pile was embedded to between 90 ft and 100 ft. The Author did take issue with Mr Bullen on the need to drive the piles to soapstone. Even accepting Mr Bullen's calculation of an appreciably higher safe load with the toe above soapstone level, the extra few feet involved to reach soapstone was surely the correct practical solution, especially in view of the history of the old wharf, which had continued to settle throughout its life. Regarding pile driving details, the helmet packing of the pile driver was softwood and the hammer a single-acting steam hammer; enquiry had not, unfortunately, produced the manufacturer's name. In general the Author felt intuitively that the piles might well have a higher safe load limit than indicated in Appendix 2 (i.e. in his view the constants used in calculation might have been too conservative), but he doubted whether an ultimate load of 160 ton would be achieved. This led directly to the question of pile testing. It was unfortunate that tests to ultimate load could not be carried out; only certain limited equipment was available in Fiji and it would have been costly and time consuming to have extended the test load range. This was unavoidable; but controlled tests with varying hammer drops had not been carried out, and this was by no means unavoidable. Both ends of the piles were solid, the toe having a slight 'point' formed in the shutters, but without any metal shoe. Unfortunately there was no record of the rate of settlement over the 88-h period queried by Mr Rooke, nor of the average pull exerted to extract the timber pile referred to in Appendix 2.

129. Dealing now with some general topics, and in reply to Dr Dutt, high-quality concrete was used for the 4½-in. pile walls. Dr Dutt had asked about costs on a U.K. basis. It was not an easy comparison to make because shipping costs, in addition to comparative labour and material costs, affected the matter. The Author thought that total costs in the two places would, very approximately, be equal.

130. Mr Lewis had kindly contributed some details of the cathodic protection installation. It might be of interest to add that the annual running cost of the installation is about 2500 pounds sterling p.a.

131. Mr Rooke had discussed fendering; the thrusts were taken into the wharf structure, but the maximum design blow was calculated to produce a deflexion of less than 1 in. On the occasion of the 'Monteray' incident, when a 15 000-ton ship hit the wharf bow-on, no tremor at all was felt by those at the point of impact.

132. Three contributors had made suggestions for alternative designs. These were interesting and under other circumstances the Author would be in agreement with a number of the ideas put forward; indeed 'Alternative Designs' forms one section of the Paper. In the event, the availability of equipment in Lautoka and experience in Fiji were clearly of such enormous advantage as to rule out other considerations for King's Wharf.

133. Mr Sørensen advocated filling the hollow piles with water (taken from the

surrounding water); having been initially opposed to this, the Author now tended to the view that it was, in fact, a logical and realistic thing to do. It would be borne in mind next time!

134. The question of an overall master plan had been referred to by Mr Loewy. This was indeed a matter of vital importance and yet for some reason it seemed that Consultants had a continuous battle to impress this fact on Clients. When the Suva Docks scheme (the 'few dotted lines' on Fig. 1) had to be abandoned it had proved impossible to get any form of revised overall plan accepted. Obviously this was most disappointing to the Engineers. It was however some compensation to know that, fortuitously, King's Wharf could be extended and still fit reasonably into the general development pattern of Suva.

135. Finally, referring to wharf operation in relation to the intermittent bridges, the Author was aware that any defence of these would savour of making a virtue out of necessity. The plain fact was, however, that in Suva, without railways or dock-side cranes, these bridges constituted a system of traffic control for the trains of mechanical runabouts operating in and out of the sheds. There had been no criticism from the operators and as a result the Author would have no qualms about introducing the same system in another project. The proof of the pudding was, after all, in the eating. It was not inconsistent to design the bridges for a reduced live load because cargo was stacked for short periods on the wharf proper, but never on the bridges.