Determining Ultimate Bearing Capacity of Precast Reinforced Concrete Piles from Deep Sounding Tests in Alsancak Harbour

Détermination de la portance des pieux battus en béton armé à partir des indications de pénétromètres au port d'Alsancak

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Summary

The author suggests an alternative method of estimating the bearing capacity of reinforced concrete driven piles from the results of deep sounding, borings and load tests.

The main feature of this procedure is neglect of the skin friction of clay, when the point of pile is supported by a gravel and sand stratum, or when the point resistance is greater than a definite value.

Sommaire

Le but de cet article est de proposer une méthode de détermination de la portance des pieux battus en béton armé, basée sur les résultats d'essais au pénétromètre, d'essais de chargements et de sondages.

La caractéristique principale de ce mode de calcul consiste à négliger le frottement latéral dans les couches argileuses lorsque la pointe des pieux s'appuie sur une couche de gravier et sable, ou quand la pression à la pointe du pénétromètre dépasse une valeur définie à l'avance.

Introduction

Exploratory borings—At the construction site of Alsancak Harbour, twenty-two exploratory borings had been carried out. The soil profiles and some geotechnical properties near the site of the loading tests were determined in the Technical University of Istanbul, and they are shown on Fig. 1 ; in order to measure the density of sand strata, S.P. tests and œdometer tests were carried out :

Description of loading tests—Eight loading tests were carried out with hydraulic jacks deriving their reaction from a dead load of 200 tons. The settlement readings, the loading and unloading operations were performed as explained in reference [1].

The settlement for eight piles under continuous load ranged from 1 to 2 mm over the initial period of from 6 to 8 days.

Each reinforced concrete test pile had an octogonal cross section of 25 cm apothem, a weight of 12 tons and a length of 30 metres.

The result of loading test No. 6 and the sections of pile are shown in Fig. 2. The ultimate bearing capacities after eight weeks from driving of the piles are given in the table at the end of this paper.

Penetrometer tool—The Dutch deep sounding tool of the improved 1936 pattern was used in penetrometer tests. The longitudinal section of this tool is shown in Fig. 3.

The interpretation of deep sounding charts—The deep sounding results obtained in the vicinity of each load test are given in Fig. 3. The point (p) and total (P) manometrical pressures are denoted as (cone resistance) and (skin resistance) there.

By interpreting the charts, the manometrical skin resistances K_{ci} (for cohesive soils) and K_{si} (for cohesionless soils) for each layer are obtained ; and then, the coefficients ξ and η in equations $K_c = \Sigma \pm \xi_i K_{ci}$ and $K_s = \Sigma \pm \eta_i K_{si}$ are estimated. (These coefficients have been recommended by ord. prof. H. Peynircioglu.) Nevertheless, in this case, it is assumed $\xi = 1$, $\eta = 1$ except the upper layer, where ξ is assumed to be zero *.

In cases where cone resistances are found in excess of their real values because of gravel pieces, the point bearing capacities (q_f) have been estimated as given in references [2, p. 220] and [3, p. 127].

Calculation of bearing capacities

This problem consists determining the ratio between the skin resistance of a concrete pile and that of a penetrometer pipe. In this case, these ratios of skin resistance have been obtained by means of trial and error calculation, so that the calculated ultimate bearing capacities had to be equal to that found from load tests.

In these calculations, the ultimate bearing capacity of pile Q_f has been separated in two parts as skin resistance $Q_f s$ and point resistance Q_{fp} ; also skin resistance is in two parts as skin resistance in sand and gravel $Q_f s_s$ and that in clay and silt $Q_f s_c$.

In cases where the pile point was supported by a sand or gravel stratum, the skin resistances of clay or silt layers were not taken into consideration. This was in accordance with usual consideration about soil and pile deformations (Ref. 2, p. 219).

(a) *The determination of point resistance :*

After the formulae of point ultimate bearing capacity (Ref. 2, p. 220), the relationship between the point resistance of pile and the point resistance of penetrometer easy can be expressed in Kg as follows :

* Negative friction due to upper layer of sand was considered for piles behind the steel sheet piling. It was taken into account when designing the foundation of the quay.

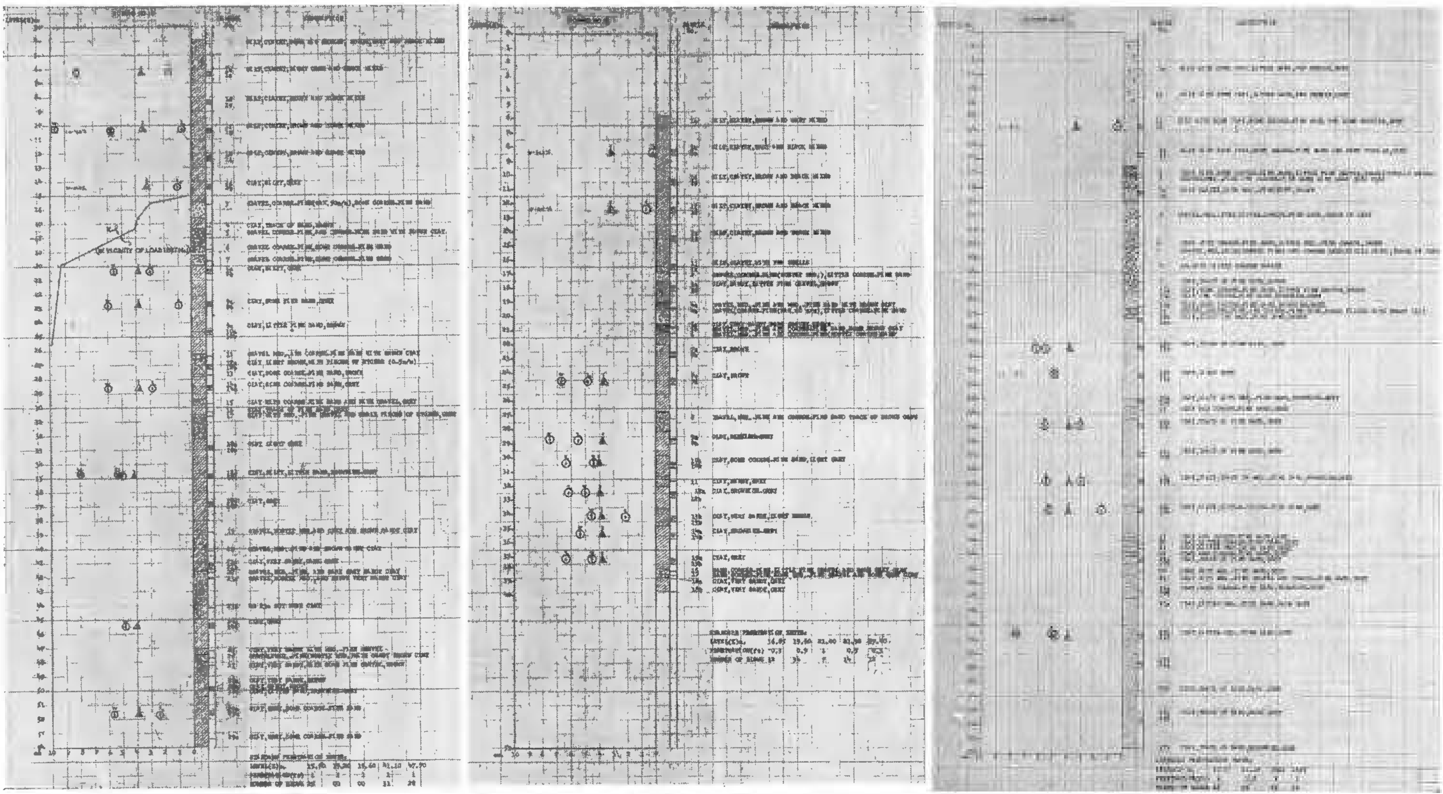

Fig. 1 Soil profiles
Profils de sol.

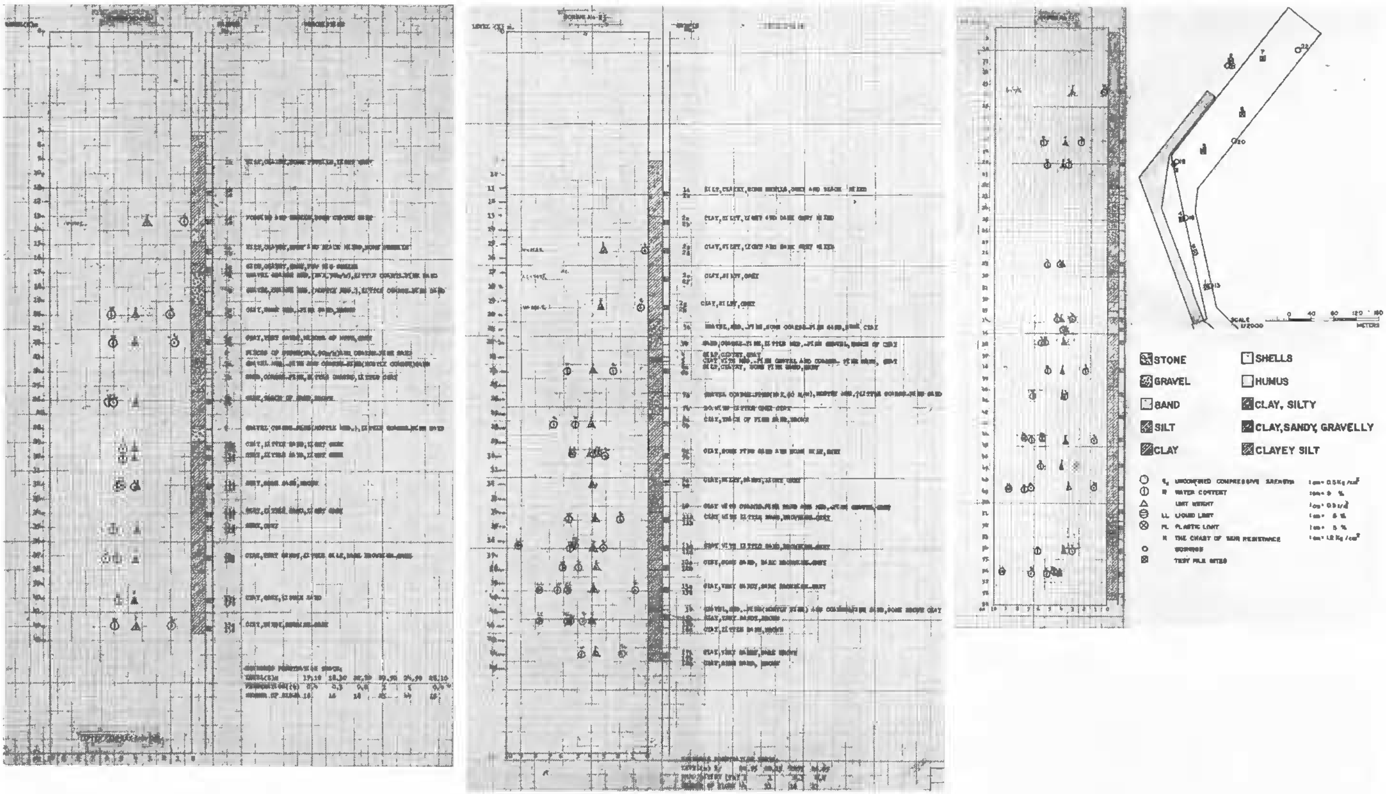
STONE
GRAVEL
SAND
SILT
CLAY
SHELLS
HUMUS
CLAY, SILTY
CLAY, SANDY, GRAVELLY
CLAYEY SILT
SCALE
METERS

Fig. 1 (suite)

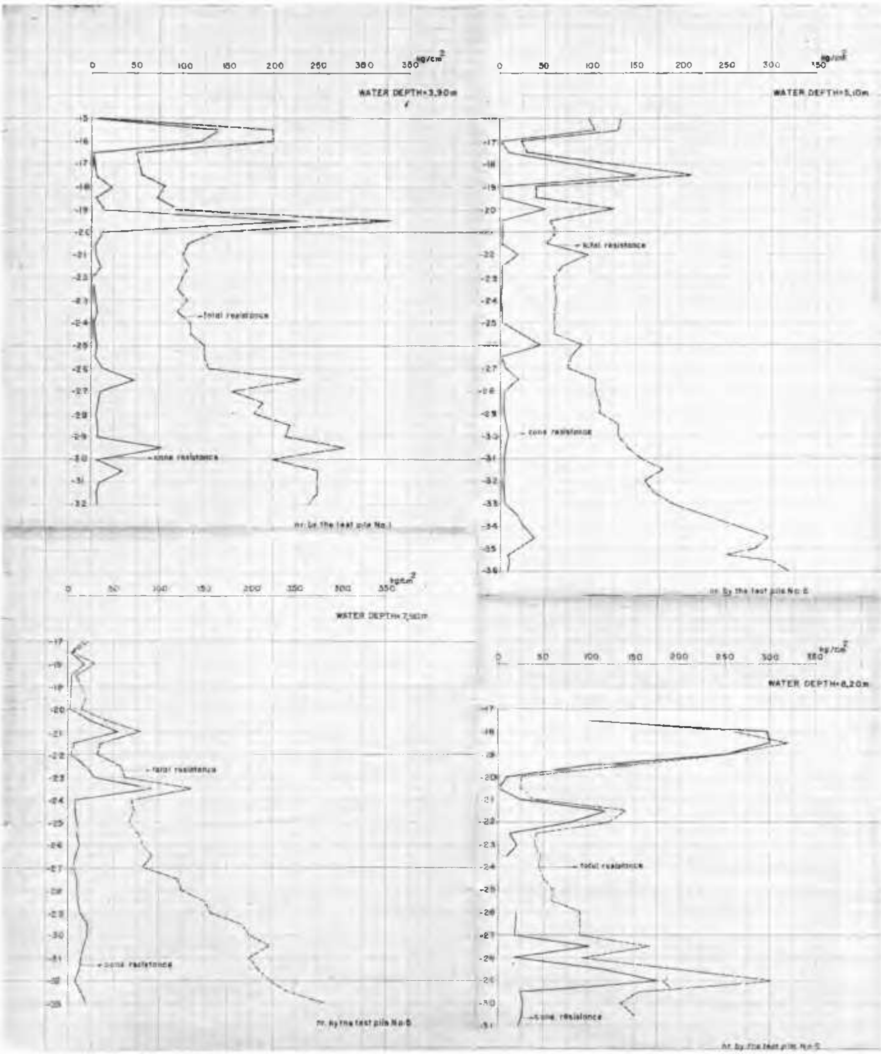
WATER DEPTH=3,90m
total resistance
cone resistance
WATER DEPTH=5,10m
total resistance
cone resistance
total resistance
cone resistance
WATER DEPTH=8,20m
total resistance
cone resistance

Fig. 3

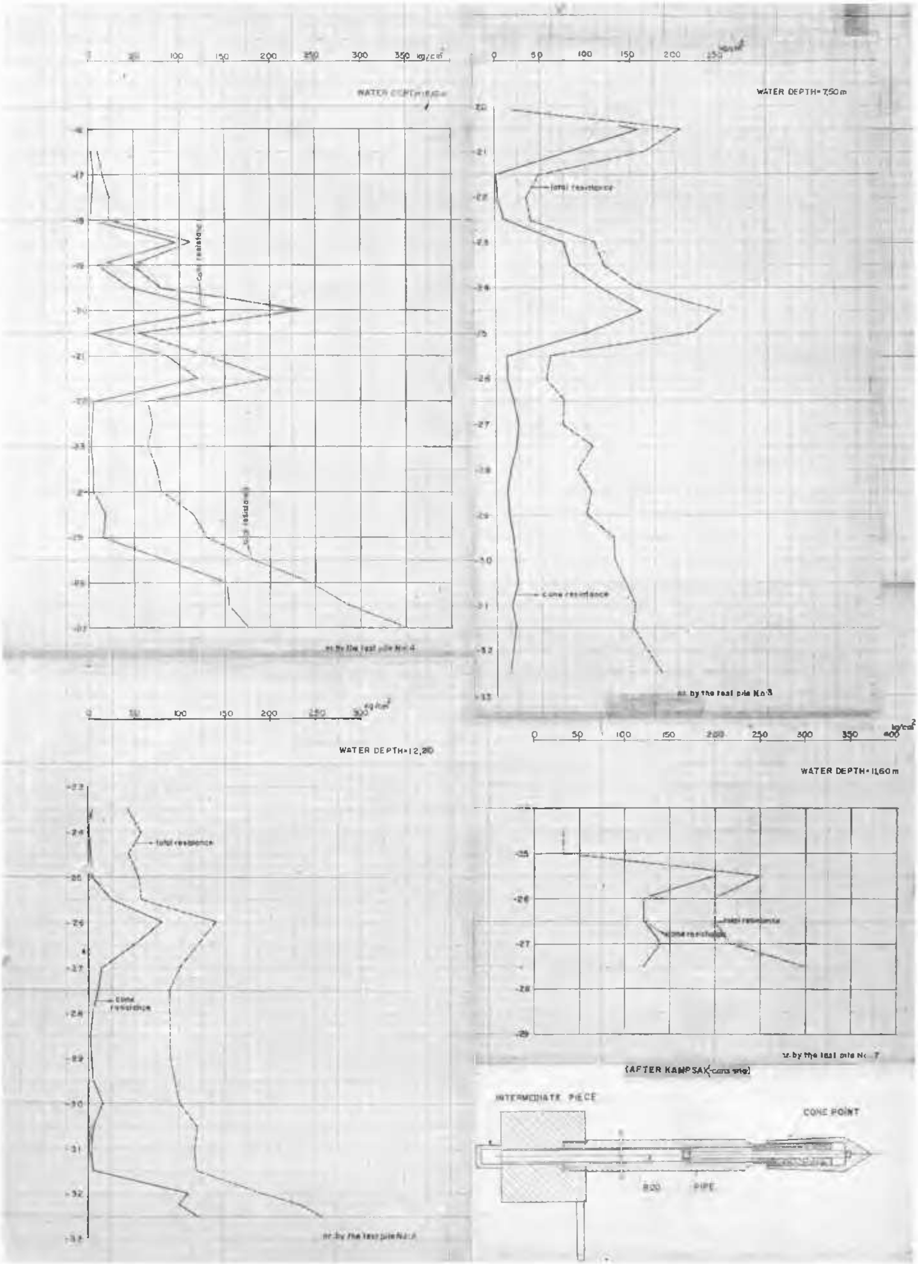

Fig. 3 (Suite) Deep sounding charts in the vicinty of test piles, and profile of deep sounding tool.
Diagrammes de « Deep Sounding » au voisinage des pieux d'essai, et schéma de l'outil de « D. S. ».

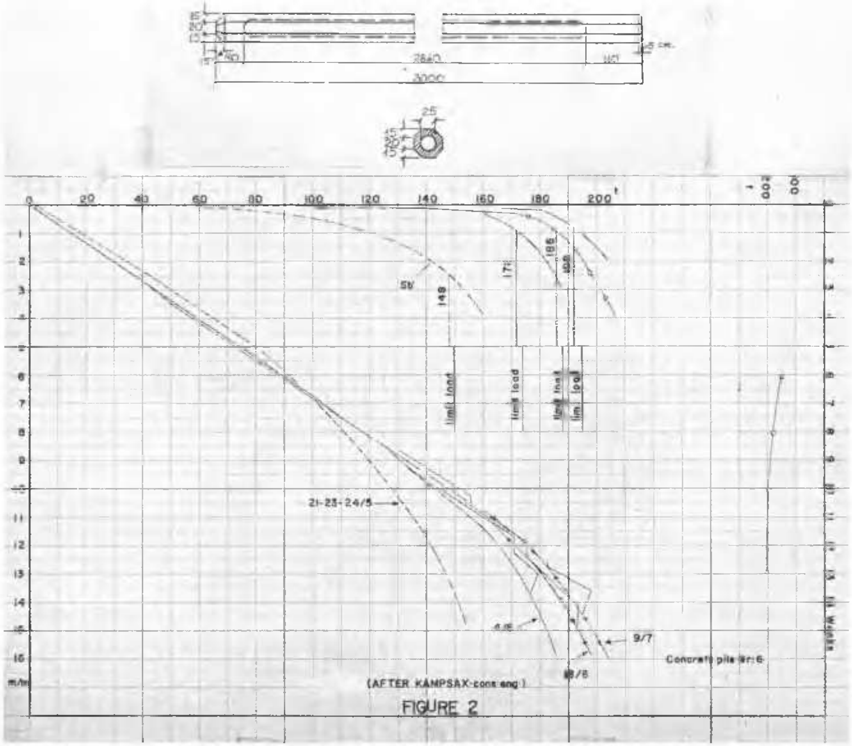

Fig. 2 Curves of load versus settlement for test No. 6, and Sections of test pile.
Courbes de chargement pour le pieu n° 6, et sections du pieu d'essai.

$$Q_{fp} = 2Ap + 0{\cdot}12AZ \qquad (1)$$

Z = depth of point in metre;
A = base area of pile (2 074 sq. cm);
p = manometrical point (cone) resistance of penetrometer in Kg per sq. cm.

For the piles Nos. 3, 4, 7, the point resistance of pile is calculated as explained above :

$$Q_{fp} = Aq_f \qquad (2)$$

(b) *The determination of skin friction :*

Skin Friction in Sand and Gravel ($Q_f s_s$)—As seen in the formulae of skin friction (Ref. 2, p. 211), in order to determine the ratio of skin frictions, the ratio α_s of the friction coefficients α_r should be found. Here α_r depends on roughness.

In this case, this ratio had been found $\alpha_s = 2$.
Hence,

$$Q_f s_s = 0{\cdot}59 K_s + \frac{0{\cdot}179 Z K_s}{K} \qquad (4)$$

Skin Friction in Clay and Silt ($Q_f s_c$)—For cohesive soils, the essential part of this problem is to determine the ratio α_c of the friction coefficients α_{rt}. Here α_{rt} depends on roughness and time.

In this case, this ratio had been found $\alpha_c = 4{\cdot}50$.
Hence,

$$Q_f s_c = 1{\cdot}325 K_c + \frac{0{\cdot}391 Z K_c}{K} \quad \ldots . \quad (5)$$

The ultimate bearing capacities calculated with values for α_s and α_c determined above, and the results of load tests are given in the following table.

The design of pile adopted is based on the collective efforts of Professor Hansen and Kampsax, who acted as Consulting Engineers. This first stage of the Alsancak Harbour construction required some 6 000 piles for various types of foundation, for which the contractors were the Société Construction de Batignolles. Supervision was undertaken by the Department of Harbour Works and Kampsax. The author was the Resident Engineer and he proposed the use of deep sounding tests for determining the ultimate bearing capacities of the piles.

The author is grateful to the president of Research Society on Soil Mechanics, Ord. Prof. Dr. Ing. H. Peynircioglu for sincerely examining this paper and sacrificing to this communication a part of the space allotment assigned to his

The number of test pile		1	2	3	4	5	6	7	8
The Point Depth (Z)	metres	26	26·8	23·7	26·3	27·8	27·5	25·9	25·9
The Soil Type at Point		Clay	Gravel, little Sand and Clay	Gravel Sand	Clay probably little Sand and Gravel	Clay	Clay	Sandy Gravel	Clay
Cone resistance (p)	kg/cm^2	12·5	28	95	153·5	10	17·5	135	19
q_f	—	—	—	72·1	48	—	—	65·4	—
K	—	117·6	76·3	45	115	110	97·5	77	65
K_s	—	74·2	76·3	40	70·25	51	37·25	57	60
K_r	—	43·4	0	5	44·75	59	60·25	20	5
Q_{fp}	ton	58·4	123	149·75	99·6	48·5	79·5	135·65	85·4
Q_{fS_s}	—	46·7	49·8	27·4	44·2	32·4	23·9	37·2	39·7
Q_{fS_r}	—	61·3	0	disrega.	63·4	84	86·4	disrega.	7·4
Q_f	—	166·4	172·8	177·15	207·2	164·9	189·8	172·85	132·5
The Result of load test + 12 t (The Pile Dead weight).	—	167	173	177	210	150	204	182	130

paper, and to the member of Research Society, assistant V. Kumbasar for sincerely examining, to Kampsax Consulting Engineers, Chief Engineer E. Nielsen, and the Technical Head of Harbour Works Department H. Okçu for documents

References

[1] SCHENCK, W. (1951). Der Rammpfahl, Wilh. ernst und Sohn, Berlin.

[2] PEYNIRCIOGLU, H. (1957). Zemin Mekaniği, Vol. 1, part 2, second edit. Istanbul Technical University.

[3] TERZAGHI, K. (1944). Theoritical Soil Mechanics, John Wiley and Sons.

Fig. 4 Partial view of Izmir-Alsancak harbour works showing the construction work nearly completed.
Vue partielle des installations du port de Izmir-Alsancak vers la fin de la construction.