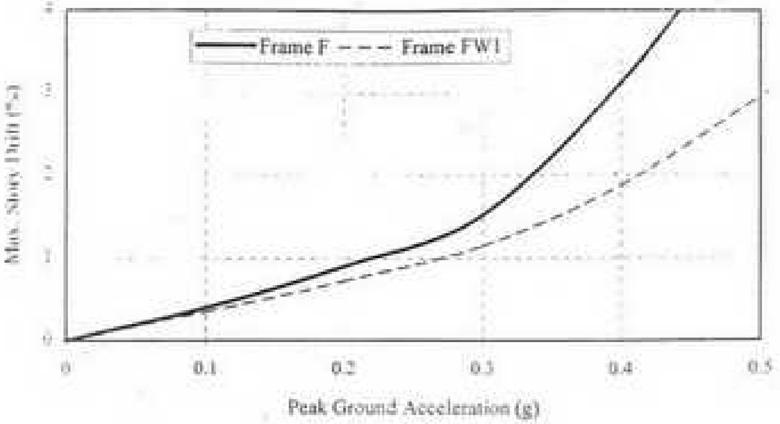

Fig. 6: Relation between Mean Value of Maximum Story Drift and Peak Ground

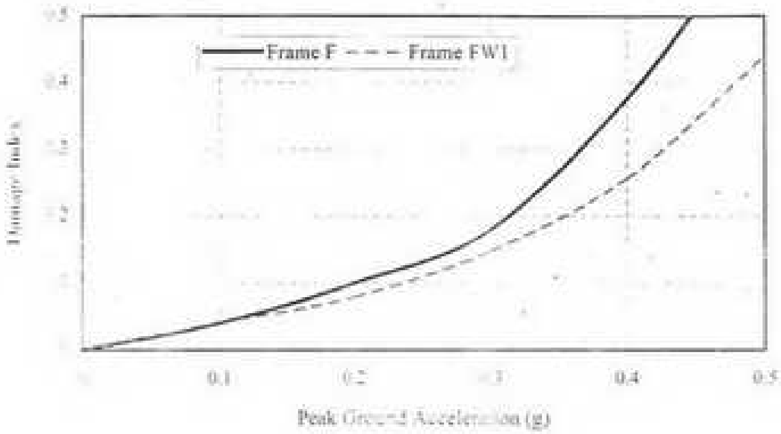

Fig. 7: Relation between Damage Index and Peak Ground Acceleration

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تقييم هبوط مسجد البوميس في عدن – دراسة حالة

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ملخص : تناقش الورقة دراسة حالة مسجد البوميس الذي يحتل مساحة ٦٢٢ مترا مربعا ويتكون من دور واحد . خلال فترة استخدام المسجد لمدة ١٨ سنة بدأت الشروخ تظهر بشكل شعيري وتطورت إلى تصدع في مكونات المبنى مصحوبة بهبوط ، وفي عام ١٩٩٨م هدم المسجد وبدأ بناء مسجد جديد مكانه . تم دراسة وتحليل هذه الحالة حيث اعتمدت الدراسة على : ١- ملاحظة الشروخ / التصدعات وتطورها . ٢- دراسة التربة وذلك من خلال حفر ٣ نقاط (مجسات). ٣- دراسة تصميم المسجد المهدم ، وقد لوحظت شروخ وتصدعات أفقية وشروخ مائلة بزاوية وهي شروخ خطيرة . وكان الموقع ذا ميول ولذا فقد تمت دراسة التربة بواسطة حفر ٣ مجسات إلى عمق ٣,٢٥ أمتار في موقع المسجد المهدم لمعرفة نوع وطبقات التربة ومكوناتها . وأسفرت الدراسة عن أن أساس مبنى المسجد غير ملائم لأن الأرضية ترتكز على جدار ساند من أصل الأساس وذي ميل يتطلب تسويته إلى عمق ٢,٤ متر وبتحليل الجدار الساند أظهرت الدراسة أن عامل الأمان للانقلاب والانزلاق أقل من واحد.

الكلمات الدالة: هبوط ، شروخ ، تقييم التصميم ، إنهيار الأساسات ، إستقرار .

SETTLEMENT EVALUATION OF POMISE MOSQUE AT ADEN - A CASE STUDY

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Abstract: The present paper discusses the case study of a mosque that occupied an area of $622m^2$ and consisted of a single story. During the use of the mosque for 18 years, cracks took place as minor then widened and increased, excessive settlement took place. In 1998 the mosque was demolished and reconstruction carried out. An attempt has been made to study and analyze this case in depth. The study is based on 1. Observation of the cracks, 2. Site investigation and 3. Study the design of the demolished mosque. The cracks appeared as horizontal, diagonal and of dangerous types. For soil investigation, three- bore holes (BH) were made up to a depth of 3.25m in the site of the demolished mosque. The site was on a slope. The evaluation and study of the case found that the design of the foundation was inadequate bearing on wall acting as a retaining wall up to a depth of 2.4m to be level with other footings. The analysis of the retaining wall shows that the factor of safety against overturning and sliding is less than one.

Keywords: Settlement, Cracks, Design Evaluation, Foundation Failure, Stability.

INTRODUCTION

For foundations on medium – dense to dense granular soil, the immediate and consolidation settlements are of relatively small order. A high proportion of the total settlement is almost completed by the time full loading comes on the foundations. Similarly, a high proportion of settlement of foundations on loose granular soil takes place as the load is applied. Settlement of foundation is not necessarily confined to very large and heavy structures. Settlement and cracking occurred in two – story houses founded on soft silty clay in Scotland (Tomlinson, 1986). Settlement and cracking occurred in single story building (Hanan et. al. 1998) in hill side slope which is liable to long – term movement which usually takes the form of mass of soil on a relatively shallow surface sliding or slip down hill. In the present study the evaluation of the settlement and cracks of the mosque was based on:

1. Observation and study of the different types of cracks.
2. Site investigation by conducting 3BH in the demolished mosque site.
3. Evaluation of the design of the demolished mosque.

LOCATION AND SITE INVESTIGATION

The general locality of the mosque is in the southern part of Al –Taweela region in Aden City known Al-Zariba; this region is an extension of mountain series of Shamsan Mountain with a valley as shown in Fig.1, this place is a pumice volcanic environment. The mosque is located in the Al-Taweela yard far around 150m from the valley. The site plan of the mosque is given in Fig.2. The pomise mosque was built in 1972, but due to shortage of fund the construction was stopped, then the construction has been started and completed in 1979. The mosque occupied an area of 622m^2 and consisted of single story with attached tow story building (school and Imam house). In 1998 the mosque was demolished due to series of dangerous cracks that appeared in the walls and floors, in general in all the structure. Before starting the construction of the new mosque, three trial pit bore holes (BH) up to a depth of 3.25m have been conducted. The location of these BH is shown in Fig 2. From each BH disturbed samples have been collected and tested in the Soil Mechanics Laboratory of Civil Engg. Dept., Faculty of Engineering. The test results and soil profiles of the 3BH of the site are illustrated in Fig.3. It can be found that there are four layers. The upper layer of thickness 1.0–1.4m of gravelly sand, second layer of 0.15– 0.25m as a pocket of sandy gravel, the thickness of the third layer is 0.75–1.10m of gravelly sand and the last layer ranges from 0.95–1.12m of gravelly sand. It can be said that the soil in general is gravelly sand except the pocket of sandy gravel. The results of the grain size distribution given in Fig.4, show that the soil is gravelly sand. The values of coefficients of curvature Cc and uniformity Cu are varying from 0.25–1.1 and 3.25–42.86 respectively. According to the unified soil classification system (Wagner, 1957); the soil can be classified GP, a poorly – graded gravelly sand. In the present investigation, calculation of void ratio has been made based on the values of field density obtained in the site (14kN/m^3) and results of specific gravity obtained in the laboratory (2.65). The computed value of the voids ratio is found 0.893 which seems to be very high when compared with typical values given by Das, 1985.

OBSERVATION AND STUDY OF CRACKS

In the present case study, the cracks are concentrated in the corner of the mosque of diagonal types as shown in Plate 1. Cracks were also observed near the openings (windows/doors) at the top and at the bottom as illustrated in Plate 2. These cracks started as minor then widened at the end. Horizontal cracks were observed in the load - bearing walls at the top of the walls (Plate 3). Random cracks were also observed in different places of the partition walls. The study of the types of cracks reveals the following:

1- Diagonal cracks (Plate 1), observed in load bearing walls, might be due to differential settlement and shear failure (Shamsher, 1998).
2- Corner and angular cracks (Plate 2) developed due to mistake in design (Al – Issa, 1998). The cracks observed in windows/doors (Plate 2) might be due to poor materials / construction (Mustafa and Shonoda, 1996).
3- Horizontal cracks as shown in the corner of the mosque (Plate 1 and 3) may be due to poor materials used, and discontinuity of the construction.

EVALUATION OF DESIGN AND SETTLEMENT

An attempt has been made to study and recalculate the design of the mosque to know the causes of deformation and cracks thoroughly as follows:

1. **Nature of the Structure:** the mosque occupied an area of 622m^2 and, consisted of single story attached with big store, Imam house and Quran boys school in ground floor. In the first floor, there was a mosque terrace with library and Quran girls school as shown in Fig.5 a and b. The terrace was also used by prayers specially on Fridays. It was made of timber joists and timber boarding covered by plain concrete of thickness 10cm. It should be noted here that in the design drawing (Fig.6) it is mentioned that R.C.C slab is used but in the actual site visit it was found made of timber joists. This may be due to shortage of fund. The load was transmitted from terrace to the load bearing walls of masonry stones 40cm thick. The foundation type is strip footing with a width of 1.2m and average depth of 0.9m. The base of footing is of plain concrete slab with depth of 40cm for single and also for two story footing as illustrated in Fig.7. The nature of the site is slope with an angle of 6 . So that the mosque footings are based on a wall foundation as a retaining wall of 0.9m thick masonry stone wall up to a depth of 2.4m to be leveled with attached two story footing as shown in Fig.7.

2. **Bearing Capacity:** the site is of slope nature with an angle w = 6 which has an influence in the computation of the bearing capacity. Referring to the design of footing (Fig.7), the width of the strip footing is B = 1.2m with an average depth of D = 0.9m. Based on the results of soil testing and site investigation of upper layer, the soil is poorly - graded gravelly sand (GP), having internal friction angle Ø = 30 [illegible] 14.0kN/m^3 [illegible]. To compute the ultimate bearing capacity (q_{ult}) of strip footing founded on the face of slope, Meyerhof's (1957) equation is adopted: $q_u = 0.5$ [illegible] where N [illegible] is bearing capacity factor depending on D/B ratio, Ø and w, obtained from chart given by Meyerhof. The ultimate bearing capacity is found equal to 420kN/m^2. The back calculation of the design load, including the dead load of the masonry wall transmitted to the strip footing, gives P = 500kN/m. Then the actual pressure on the soil

due to the weight of structure q = 500/1.2 * 1 = 417kN/m^2. To check the factor of safety (F.S) with respect to shear failure, in terms of the net ultimate bearing capacity = (q_{ult} - γD) and net foundation pressure = (q - γD), the following equation was used to check the factor of safety: F.S = (q_{ult} - γD) / (q - γD), the computation of the factor of safety is found FS = 1. If the factor of safety is considered 3 as it is common in practice then the allowable bearing capacity $q_{all} = q_{ult}$ / F.S = 140 kN/m^2. Therefore the allowable load P_{all} = (140 + 12.6) * 1.2 * 1 = 183 kN only, where the required allowable load is 500kN. This indicated that shear failure has occurred to the base of the footing which confirms the cracks observed in the corners.

3. **Retaining wall:** as it is mentioned before the site is on a slope with angle β = 6°, the foundation supports a retaining wall of 0.9m thick masonry stone up to a depth of 2.4m to be level with other footings Fig 7. The length of the wall is found 28m, carrying a load of 500 kN/m transmitted to the width of the retaining wall 0.9m, the surcharge q = 500 / 0.9 * 28 = 20 kN/m^2 is considered. The final distribution of loads on the retaining wall is illustrated in Fig 8. No shear stresses act on this vertical wall, therefore the Rankine theory is used to calculate the active and passive pressures. For passive resistance calculation Tomlinson (1986) procedure was adopted. The pressure distribution is shown in Fig 9. The unit weight of masonry stone and concrete base are taken to be 20kN/m^3 and 23kN/m^3 respectively. The active and passive pressures are calculated on the vertical through the toe of the wall given in Table 1. The total active and passive forces are calculated from the numbered triangles and rectangles as cited in Fig 9 and presented in Table 2. To determine the position of the base reaction, the moments of all forces about the toe of the wall (x) are calculated and given in Table 3.

Lever arm of base resultant:

$$\frac{M_x}{R_v} = \frac{\Sigma M_o + \Sigma M_R}{\Sigma P_v + \Sigma W} = \frac{47.36 + 52.77}{6.31 + 85.32} = 1.1m$$

Table 1 - Calculation of Active and Passive Pressures

Soil	Depth	Ø	Ka	Active pressure (kN/m^2)
Surcharge	0	-	0.339	$K_{a1}q$ = 0.339 * 20 = 6.78
Soil (1)	1.25	30°	0.339	$K_{a1}q + K_{a1}\gamma H_1$ = (0.339 * 20) + (0.339 * 14 * 1.25) = 6.78 + 5.93 = 12.71
Soil (2)	1.40	15°	0.612	$K_{a2}(q + \gamma h_1) + k_{a2}\gamma H_2$ = 0.612 (20 + 14 * 1.25) + (0.612 * 14 * 0.15) = 22.95 + 1.29 = 24.24
Soil (3)	2.8	24°	0.431	$K_{a3}(q + \gamma H_1 + \gamma H_2) + K_{a3}\gamma H_3$ = 0.431(20 + 14 * 1.25 + 14 * 0.15) + (0.431 * 14 * 1.4) = 17.07 + 8.4 = 25.52
Soil	Depth	Ø	K_p	Passive pressure (kN/m^2)
Surcharge	0	-	-	0
Soil (1)	1.25	30°	2.95	$K_{p1}\gamma H$ = 2.95 * 14 * 1.25 = 51.63
Soil (2)	1.40	15°	1.63	$K_{p2}\gamma H_1 + K_{p2}\gamma H_2$ = 1.63(14 * 1.25) + 1.63(14 * 0.15) = 28.53 + 3.42 = 31.95
Soil (3)	2.8	24°	2.32	$K_{p3}(\gamma H_1 + \gamma H_2) + K_{p3}\gamma H_3$ = 2.32(14 * 1.25 + 14 * 0.15) + (2.32 * 14 * 1.4) = 45.47 + 45.47 = 90.94

Table 2 Active and Passive Forces

Active			Passive		
Elem.	Pressure (kN/m^2)	Force (kN/m)	Elem.	Pressure (kN/m^2)	Force (kN/m)
(1)	6.78	6.78 * 2.8 = 18.98	(1)	51.63*0.8= 41.3	½ * 41.3 * 1.25 = 25.82
(2)	5.93	½ * 5.93 * 1.25 = 3.71	(2)	28.53*0.8=22.8	22.8 * 0.15 = 3.42
(3)	16.17	16.17 * 0.15 = 2.43	(3)	3.42 *0.8=2.7	½ * 2.7 * 0.15 = 0.21
(4)	1.29	½ * 1.29 * 0.15 = 0.097	(4)	45.47*0.8=36.4	36.4 * 1.4 = 50.93
(5)	10.29	10.29 * 1.4 = 14.41	(5)	45.47*0.8=36.4	½ * 36.4 * 1.4 = 25.46
(6)	8.45	½ * 8.45 * 1.4 = 5.92			
		Total 45.55			Total 105.84

Table 3. Moments Acting on the Wall

Force type	Elem.	Force (kN/m)		Arm (m)	Moment (kN.m/ m)
Active Horizontal Forces	(1)	18.98 Cos 6° =	- 18.88	1.4	- 26.43
	(20	3.71 Cos 6° =	- 3.69	1.97	- 7.27
	(3)	2.43 Cos 6° =	- 2.42	1.475	- 3.56
	(4)	0.097 Cos 6° =	- 0.096	1.45	- 0.14
	(5)	14.41 Cos 6° =	- 14.33	0.7	- 10.03
	(6)	5.92 Cos 6° =	- 5.89	0.47	- 2.77
Passive Horizontal Forces	(1)	25.82 Cos 6° =	25.68	1.97	50.59
	(2)	3.42 Cos 6° =	3.4	1.475	5.02
	(3)	0.21 Cos 6° =	0.209	1.45	0.30
	(4)	50.93 Cos 6° =	50.65	0.7	35.46
	(5)	25.46 Cos 6° =	25.32	0.47	11.9
		Total ΣP_h = 59.9			
Active Vertical Forces	(1)	18.98 sin 6° =	- 1.98	1.2	- 2.38
	(2)	3.71 sin 6° =	- 0.388	1.2	- 0.47
	(3)	2.43 sin 6° =	- 0.25	1.2	- 0.30
	(4)	0.097 sin 6° =	- 0.01	1.2	- 0.012
	(5)	14.41 sin 6° =	- 1.51	1.2	- 1.81
	(6)	5.92 sin 6° =	- 0.62	1.2	- 0.74
Passive Vertical Forces	(1)	25.82 sin 6° =	2.69	0	0
	(2)	3.42 sin 6° =	0.38	0	0
	(3)	0.21 sin 6° =	0.022	0	0
	(4)	50.93 sin 6° =	5.32	0	0
	(5)	25.46 sin 6° =	2.66	0	0
		Total ΣP_v = 6.3[illegible]			ΣM_o =47.36
Surcharge	Q	20 * 1.05 =	[illegible]	0.675	14.18
Wall	W_1	2.4 * 0.9 * 20 =	43.2	0.600	25.92
Base	W_2	1.2 * 0.9 * 20 =	[illegible]	0.600	6.62
Soil	W_3	2.4 * 0.15 * 14 =	3.04	1.125	5.67
Soil	W_4	2.4 * 0.15 * 14 =	3.04	0.075	0.38
		ΣW = 85.32			ΣM_R =52.77

Eccentricity of base reaction:

$$e = 1.1 - B/2 = 1.1 - 1.2/2 = 0.50 \text{ m}$$

$$e > B/6 \; ; (0.50 > 0.2)$$

i.e. the resultant acts out side the middle third of the base.

The maximum and minimum base pressures are given:

$$R_v = \sum W + \sum P_v$$
$$= 85.32 + 6.31 = 91.63 \text{ kN/m}$$

$$q = \frac{R_v}{B}\left(1 \pm \frac{6e}{B}\right)$$
$$= \frac{91.63}{1.2}\left(1 \pm \frac{6 * 0.50}{1.2}\right) = 76.36\,(1 \pm 2.50)$$
$$= 267.25 \text{ kN/m}^2 \quad and \quad -114.54 \text{ kN/m}^2$$

From the above it can be found that the [illegible] the toe of the wall exceeded the allowable bearing capacity of th[illegible]. The presence of the tensile pressure at the base is no[illegible] the tensile pressure of the soil is very [illegible], no R.C.C. base is used.

[illegible]actor of safety against sliding:

$$S.F = \frac{[illegible]}{\sum P_h} = \frac{91.[illegible] \tan 30°}{59.9} = 0.88 < 1.5$$

Factor of safety against overturning:

$$S.F = \frac{\sum M_R}{\sum M_o} = \frac{52.77}{47.36} = 1.1 < 1.5$$

The values of the safety factor against sliding and against overturning, the minimum value at the least 1.5 usually being specified, in the present study both values are less than one, which indicated that the overall stability of the retaining wall is not calculated and checked by the designer.

1. [illegible] investigation of the site and nature of the site, which is volcanic environment slope and valley, was ignored by the designer.

2. Estimating and using higher value of allowable bearing capacity to the site, in designing of strip footing, causes in shear failure of the soil and cracks in the structure.

3. Retaining wall was used without designing and checking its overall stability. This caused failure of the retaining wall.

4. Design mistake and mistake in calculation is one of the major problems in the present study.

5. Poor construction materials used, mistake in construction and discontinuity of the construction is the second problems.

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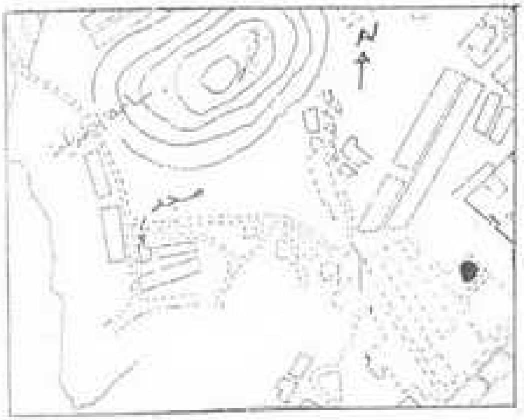

Fig.1 Location of Al-Taweela yard

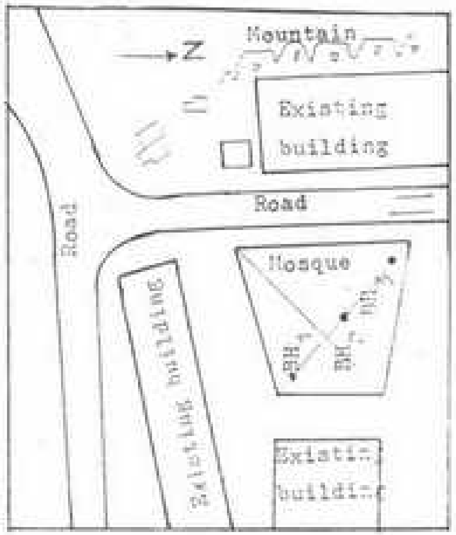

Fig.2 Site plan of Pomise Mosque.

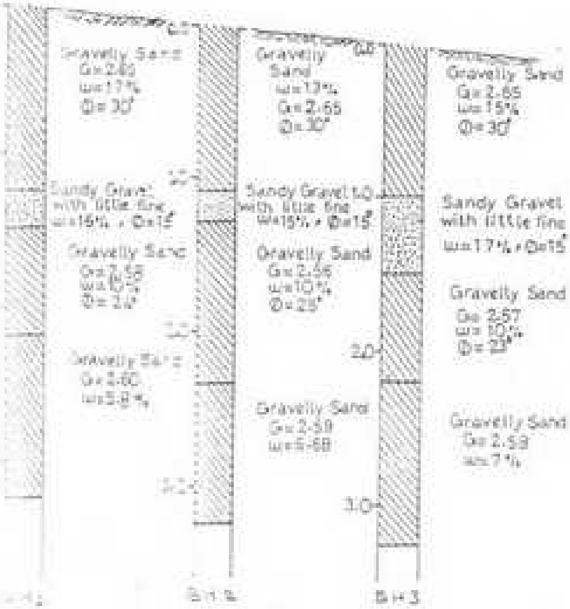

Fig.3 Soil Profiles

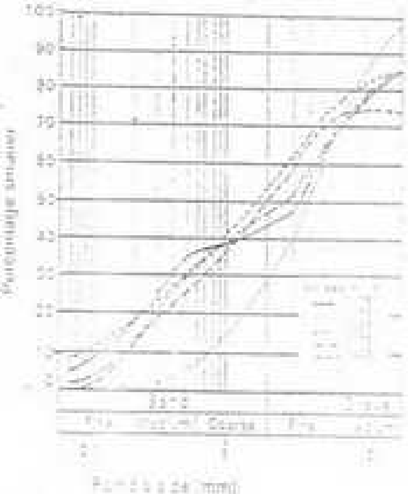

Fig.4 Grain Size Distribution of Soil.

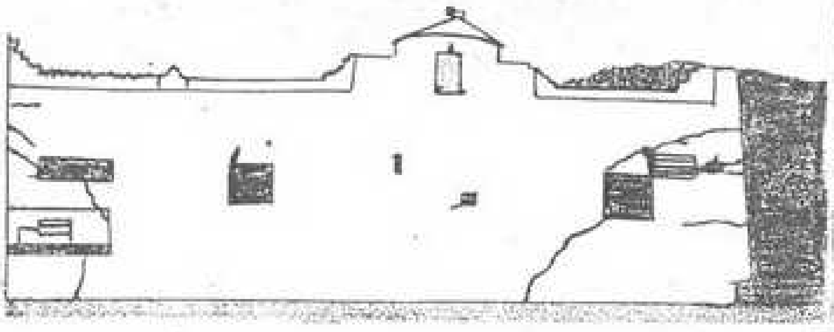

Plate 1 : Diagonal Cracks at corner.

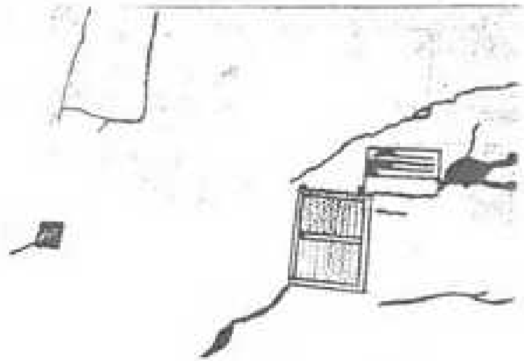

Plate 2 : Cracks at the Openings .

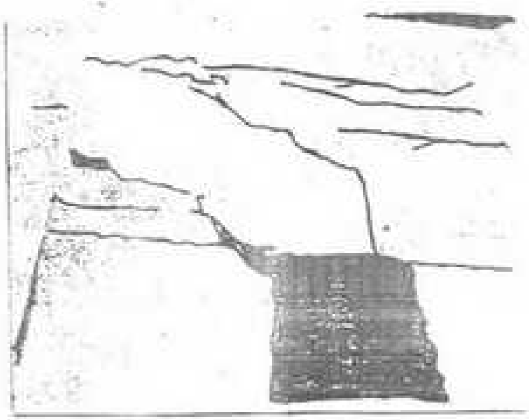

Plate 3 : Horizontal Cracks .

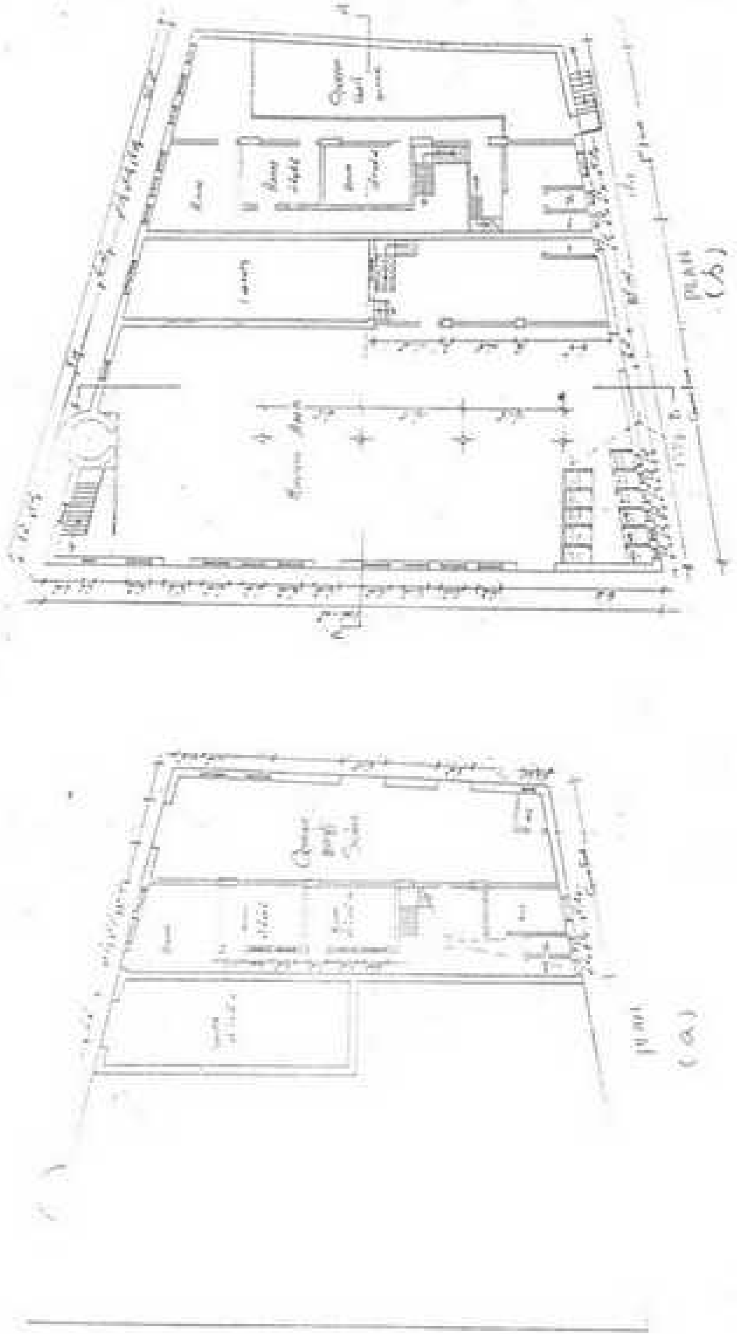

Fig.5 Plan of the Mosque with attached House.

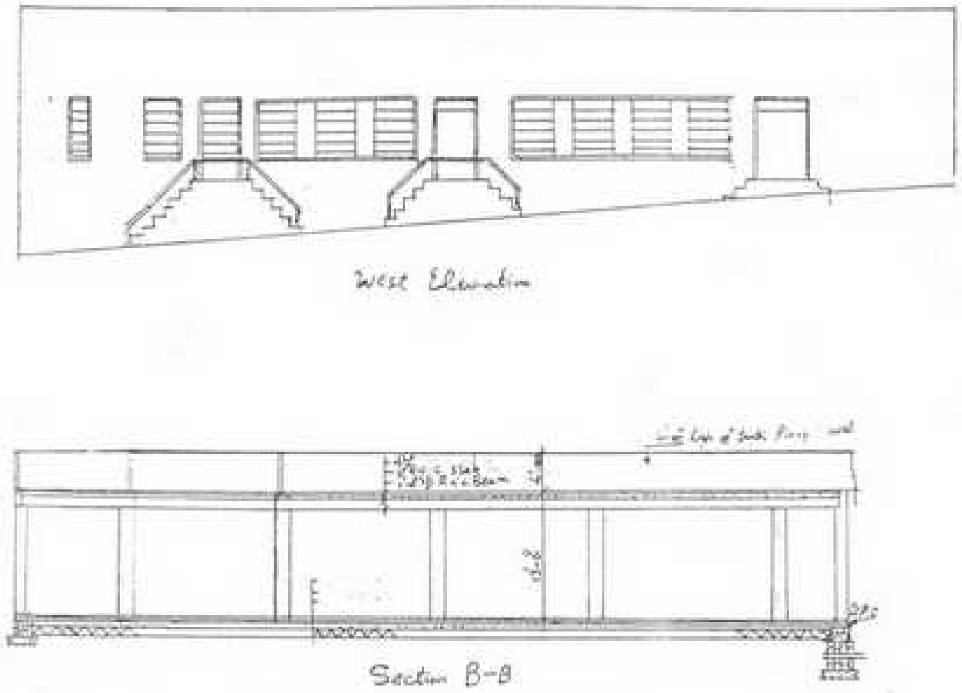

Fig.6 Cross Section of the Mosque .

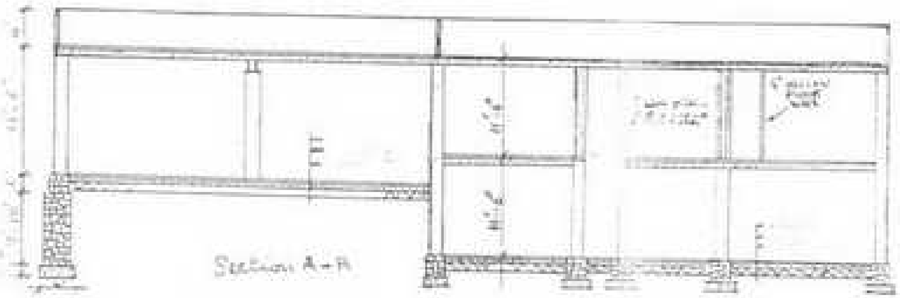

Fig.7 Footings and Wall Foundation (Retaining Wall)

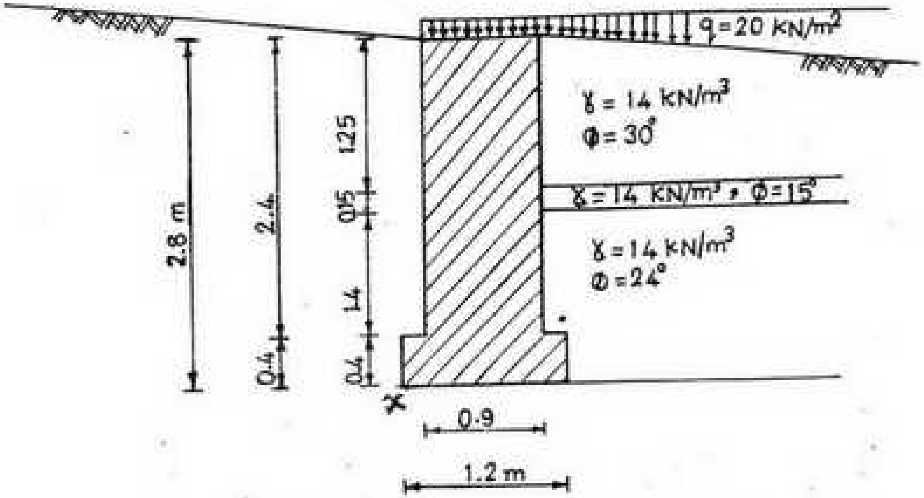

Fig.8 Distribution of Loads on the Retaining Wall .

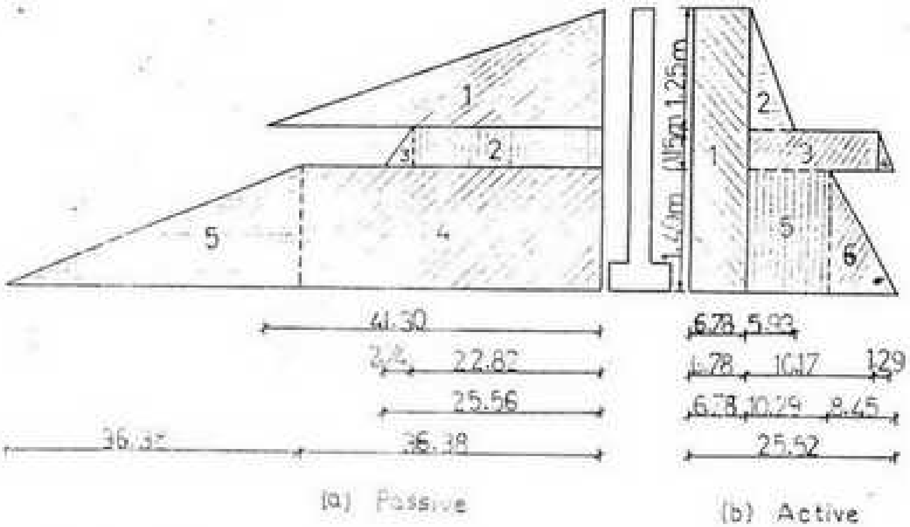

Fig.9 Pressure Distribution .

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التحليل الإنشائي لكوبري المعلق سيفرن باستخدام نظرية الترخيم المحدد أثناء التنفيذ

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ملخص : كوبري نهر سيفرن المعلق يجري عليه الطريق M48 بين انجلترا وويلز وطوله ٦٠٠٠ قدم ، يتكون من ثلاث فتحات ، الفتحة الوسطى ٣٠٠٠ قدم . هذا الكوبري كان الأول في استخدام فكرتين لتقليل عدم الاتزان الديناميكي : جسم الكوبري الانسيابي والعلاقات مثلثة الشكل . بعد تنفيذ الأساسات والبرج والكابلات الرئيسية يتم تنفيذ الكمرة الرئيسية على مراحل . الهدف من هذا البحث هو دراسة السلوك اللاخطي للكوبري أثناء تنفيذ الفتحة الوسطى . أجري التحليل الإنشائي على الفتحة الوسطى ٣٠٠٠ قدم منفذ بها ٤٠٠ قدم من الكمرة الرئيسية .

الكلمات الدالة : كباري معلقة ، تحليل إنشائي ، لاخطي ، التنفيذ .

FINITE DEFLECTION THEORY ANALYSIS OF THE SEVERN SUSPENSION BRIDGE DURING ERECTION

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Abstract: The suspension bridge over the river severn carries the motorway M48 between Wales and England, 6000 ft, comprises 3 spans, the middle span is 3000 ft. This bridge was the first to use two new ideas to reduce aerodynamics instability - a streamlined deck and triangulated suspenders. After constructing the main supports, the towers, and the main cables, the middle span main girder was partially erected. It is the aim of this paper to study the nonlinear behaviour of the bridge having the middle span partially erected. The analysis is performed for partially erected main girder 400 ft in the middle of the span and supported by triangular suspenders connected to the main cable, 3000 ft span.

Keywords: Suspension Bridges, Structural Analysis, Nonlinear, Erection.