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HIGH STRENGTH CONCRETE USAGE IN MALAYSIA THE PETRONAS TWIN TOWER PROJECT AND FUTURE PROSPECTS

by

IR. DR. KRIBANANDAN a/l GURUSAMY
B.Sc., Ph.D., C.Eng, MICE, P. Eng, MIEM

Taywood Engineering Sdn Bhd.
9th Floor, West Block Wisma Selangor Dredging
142C Jalan Ampang
50540 Kuala Lumpur
Tel: 03-2633532 Fax: 03-2633533

Abstract:

The use of high strength concrete (HSC) in structures is increasing worldwide and has begun to make an impact in Malaysia. While 30 MPa concrete is still the norm, in many recent projects particularly in high rise construction 50, 60 and even 70 MPa concrete has been specified particularly for load bearing columns. The most significant beakthrough in the use of high strength concrete in Malaysia is of course the Petronas Twin Tower project currently being developed by the Kuala Lumpur City Centre Berhad. The project is part of a massive real estate development where two adjacent towers rising 450m above street level, are being constructed with 80 MPa (characteristic cube strength) concrete for the lower level columns.

This paper outlines the pre-construction consultancy inputs which were undertaken by way of trial column construction to support the use of the high strength concrete in the Petronas Twin Tower project. The requirements for curing, insulation, striking time, strength development and concrete temperature and strain monitoring results are discussed. The future prospects for the use of HSC in construction projects is also considered.

1. HIGH STRENGTH CONCRETE - RECENT DEVELOPMENTS

A few years ago, a characteristic compressive strength of 40 MPa would have been considered high, in Malaysia but this is now becoming commonplace. Probably the most suitable definition for "High Strength Concrete" is concrete with a compressive strength in excess of the maximum grade specified in national codes and standards, up to the practical upper limit of strength for concrete made with natural aggregates. This is thought to be in the region of 150 Mpa (1). High strength concrete containing normal weight aggregate can be considered as concrete with a characteristic 28 day cylinder strength of 60-120 Mpa (75 - 150 Mpa characteristic cube strength).

The achievement of such high strength concretes has been possible primarily through the introduction of two new materials i.e. superplasticisers and Microsilica. Superplasticisers or high range water reducing admixtures were developed in the mid 1970's. These admixtures enabled very low water/cement ratios to be achieved in concretes without the need for excessively high cement contents, whilst still producing sufficient workability to enable the concrete to be placed using conventionally accepted techniques. Microsilica (or silica fume) significantly increases the strength of the cement paste and when used in combination with superplasticisers has enabled the strength of concrete to be substantially increased, to the point where the mechanical properties of the aggregate become the limiting factor (2).

Much of the development of high strength concrete has been undertaken in the United States, where a large number of high rise structures (particularly in the Chicago and Seattle areas) have been constructed with concretes of characteristic cylinder strengths above 60 MPa (3, 4). Elsewhere, the Japanese Ministry of Construction has funded a four year development programme on the development of advanced concrete buildings using high strength concrete and reinforcement (5). In Norway, extensive research into high strength concrete has been co-sponsored by the offshore oil industry and the Royal Norwegian Council for Scientific and Industrial Research. Over the last five years, this has led to recommendations for the design and use of concretes with strength of up to 105 MPa in Norwegian Standards (6). France has also recently completed a national project on "New Ways for Concrete" that included high strength concrete (7). The ready mix concrete industry in the Netherlands have undertaken their own studies on this material (8).

The use of high strength concrete throughout Malaysia to date has been limited. The barriers to the more widespread application of high strength concrete, in view of the generally positive findings elsewhere, may be ascribed to either a lack of awareness of its properties or lack of confidence by specifiers that it can be used economically and practically in the site situation. This arises both from the lack of interaction between researchers and construction professionals and from failure to absorb the extensive available information into National Standards or Codes of Practice for the use of high strength concrete.

2. PETRONAS TWIN TOWER PROJECT

2.1 Introduction

The Petronas Twin Tower project in Kuala Lumpur, Malaysia is a prestigious development consisting of 216,901m^2 of total floor space, 88 levels, (6 Basement and 82 superstructure) rising to a height of 450m above street level. It will be tallest building in the world on completion in March 1996. A plan view of the structure is shown in figure 1. The structural columns, ring beams and core are of Reinforced Concrete of 40 to 80 Mpa cube strength concrete with steel long-span floor beams. Grade 80, is specified up to level 22 for the 2.4m diameter reinforced concrete columns (see figure 2). This is the first project in Malaysia where such high strength concrete has been specified, with the previous high known to the author being Grade 65 MPa concrete for columns on the Public Bank Building in Johor Bahru (completed Dec. 1993). To achieve the projected completion in approximately 2 years 4 months every floor needs to be constructed in approximately 4.3 days thus putting great pressure on the contractor to achieve delay free construction. The need to resolve all problems prior to construction was critical and in this context full size trial columns were constructed and monitored and all potential problems identified and brought to the attention of the contractor.

2.2 Design Philosophy

The client and contractor were made aware of the unusual needs of the project and in particular the use of high strength 100 MPa (80MPa + 20MPa margin) concrete in large diameter columns (2.4m). The potential for high heat of hydration and subsequent cracking of concrete, and stringent QA/QC requirements to achieve consistent concrete were highlighted and accepted as important aspects which needed specialist inputs. Other aspects considered included the need for early age striking of form work (<15 hours), minimising cracking in corewalls and curing requirements to achieve sound concrete.

2.3 Trial Column Casting

2.3.1 Introduction

As part of the materials selection several trial columns of actual dimensions were poured and monitored for Heat of hydration, strain, cracking potential and durability. The original mix design specified for the concrete was reviewed to minimise the risk of early age thermal cracking and in keeping with the requirements for early age striking of formwork (at 10 to 12 hours) to meet the construction schedule. Advice was given on the concrete insulation requirements during casting, use of additives in concrete, the requirements for fresh concrete properties, insitu strength development particularly at early age and temperature differentials within concrete affecting cracking potential.

2.3.2 Dimensions of Trial Columns/Formwork Details

The trial columns were of dimensions 2.4m height and 2.4m diameter. Two identical columns were fabricated and cast as follows:

Column No. 1	08/02/94	12:00 hours
Column No. 2	08/02/94	13:30 hours

For this investigation 12:00 hours and 13:00 hours are considered zero hours for Column 1 and Column 2 respectively.

The same system formwork to be used for the actual column casting was used in the mock-up column casting. The forms used were 12mm steel in two separate halves bolted together on site. One half of the formwork was removed 8 hours 20 minutes after concrete casting while the other was removed after 13 hours, for both columns.

2.3.3 Concrete Mix

The concrete for the mock up columns was site batched. Two concrete mixes were considered one OPC /micro silica and the other OPC/PFA/microsilica. The Pulverised Fuel Ash (PFA) was introduced into the second mix by using masscrete supplied by Associated Pan Malaysia Cement (APMC). According to APMC product literature masscrete contains approximately 20% by wt of PFA interground with OPC. The mix therefore approximated to 460/69//35/OPC/PFA/microsilica mix, i.e. a 12% PFA replacement. The concrete 1m^3 mix designs are shown in Tables 1 and 2.

A slump test and temperature measurements were carried out for each concrete batch before the concrete was poured into the forms. The slump was between 190 - 220 mm while the fresh concrete temperature ranged from 32^{o}C - 35^{o}C.

2.3.4 Concrete Placement

The trial columns were both cast to a height of 2.4m. The column casting was undertaken using pumped concrete in a continuous pour. Both columns took 1.5 hours to pour.

2.3.5 Concrete Strength

The structural concrete strength specified was 80 MPa with a 20 MPa margin which meant a target strength of 100 MPa had to be obtained at 56 days. A water/cement ratio of 0.25 was specified for this grade. This was achieved with a combination of OPC/PFA and micro silica as discussed above. Due to the fast track construction programme form striking was required at early age (between 10 - 12 hours) at a minimum strength of 15 MPa. Tests were therefore conducted to ascertain early age strength and in this context insitu strength was measured and compared to cube strengths to consider the advantage of strength gain with temperature.

Concrete cube samples were taken for cube compression strength testing at 12 hours, 16 hours 24 hours (1 day), 96 hours (4 days) the concrete cubes were made, stored and tested at the site laboratory. Strength determination was also undertaken at 7 days and 46 days. The cube strength results are plotted in Figure 3 and indicate that the target cube strength was met.

The insitu strength of concrete as measured by taking cores were compared to standard cube testing at early age. The core sampling of the concrete for compression strength testing consisted of 100mm diameter diamond tipped coring. The depth of core sample was approximately 200mm in order to obtain a 100mm length for testing accept at 46 days when a 1.2m core was taken to also ascertain strength of the concrete with depth. Concrete cores were tested at 12hours, 16 hours, 24 hours (1 day), 96 hours (4 days), 7 days and 46 days after concrete casting. The strength data beyond 7 days are not discussed in this paper. The target times for coring are shown in Table 3. The actual coring times are shown in Tables 4 and 5 for Columns 1 and 2 respectively. The cores were taken by an independent test laboratory and tested off site in accordance with BS1881; Part 120: 1983. The cores were photographed, wrapped in plastic cling film and aluminium foil and transported to the laboratory for testing. All the core strength results are given as estimated equivalent insitu cube compression strength. These have been plotted in Figure 4. In general the insitu core strengths are higher than cube strengths up to 4 days. At 7 days there appears to be a marginal drop in strength. Concrete gains strength with age, it also gains strength more rapidly the higher the early age temperature. The results of insitu core compression strength tests at 12 to 14 hours, for both Columns 1 and 2, are considerably higher compared to standard cube compression strength as expected.

The early age strength development showed acceptable performance. The standard cube sampling and testing gives a conservative estimate of the insitu compression strength, and the 15 MPa strength requirement is exceeded by the cubes after 8 hours. Stripping of form work can therefore proceed comfortably between 10 and 12 hours for this grade (80 MPa) concrete. It was recommended that these tests be repeated for the 60 MPa and 40 Mpa concrete to be used at higher levels of the structure and that a pull off or fracture test be used to estimate insitu strength for formwork removal.

2.3.6 Curing

The concrete was cured by the side form work before formwork striking, and the concrete base below. Polythene sheeting was used to cover the top of the column primarily as a protection against rain however, effective curing is provided when secured at the edges. This polythene sheet was removed from both columns approximately 3.5 days after casting.

The columns were covered with a roll on applied curing membrane immediately after formwork removal.

2.3.7 Insulation

The steel (12mm) forms on the sides of the column provides no significant insulation. The concrete base provided some insulation. During normal construction the concrete below will still have retained heat and will therefore act as insulation for the bottom concrete in the columns.

Insulation of the column sides and top surface was not considered essential based on the trial Column 2 performance (i.e. no cracks observed). It was also concluded that inappropriate use of insulation can increase the likelihood of cracking.

2.3.8 Concrete Temperature And Strain

Concrete temperature and strain were monitored for a minimum of 7 days in the columns. The monitoring locations for the concrete temperature and strain in the columns are shown in Figures 5 and 6. Two one cubic meter hot blocks for each mix type with strain and temperature monitoring were cast at the same time as the columns to establish the free thermal coefficient of expansion and contraction of the concrete. The monitoring results were used for the analysis of restraint factors and the likelihood of early age thermal cracking of concrete. the strains were monitored automatically with data logger which measures period and apparent strain of vibrating wire gauges (VWG's). Thermocouple temperature readings were also recorded on a data logger. The monitoring started on 8th February 1994, 12:00 hours at the start of concrete casting for column 1 and this is referred to as zero hour monitoring in all the reporting for column 1. In the case of column 2, monitoring began at 13:30 hours on 8th February 1994.

Two typical graphs of temperature against time are given in Figures 7 and 8 for the OPC/Microsilica and OPC/PFA/Microsilica concrete columns respectively. The curves are for monitoring at the centre of the column (T7 ,T20) and 100mm from the outside surface at Mid Column height (T3,T16).

Significant monitoring data results (9,10,11) were :

Column 1 (OPC/Microsilica Grade 80 Mix)

a) The peak temperature recorded was 91.6^{o}C at the centre of column after 29 hours of monitoring.

b) The concrete temperature at placement was 32 and 33^{o}C. This was below the specification requirements of a maximum limit of 35^{o}C).

c) The temperature rise per 100kg cementitious materials was calculated as 11.6^{o}C.

d) After almost 8 days the concrete temperature was approaching ambient with the peak mid column temperature having dropped from 92^{o}C to 37^{o}C.

e) The maximum differential temperature recorded was 57.5^{o}C at 27.5 hours monitored during the heat up phase.

f) The recommended maximum temperature differential of 27.7^{o}C for granite concrete was exceeded at several locations.

g) The maximum differential temperature occurs at the top corner of the column where cracking initiated in Column 1.

h) The maximum bulk temperature in the mid section of the column was 82.7°C which occurred at 14.5 hours of monitoring.

Column 2 (OPC/Masscrete/Microsilica Grade 80 Mix)

a) The peak temperature recorded was 87°C at the centre of column after 26.5 hours of monitoring.

b) The concrete temperature at placement was 33°C and 35°C. This was below the specification requirements of a maximum limit of 35°C.

c) The temperature rise per 100kg cementitious material was 9.8°C.

d) After almost 8 days the concrete temperature was approaching ambient with the peak mid column temperature having dropped from 87°C to 37°C.

e) The maximum differential temperature recorded was 52.9°C, at 33 hours monitored during the cool down phase.

f) The recommended maximum temperature differential of 27.7°C for granite concrete was exceeded.

g) Although the temperature differential results exceeded the limits for granite concrete (of 27.7°C) cracking did not initiate at the exterior top corner of the column, nor had it propagated down the column. This was because the high differential temperatures developed at very early age, do not have sharp gradients, and benefited from early age creep relief. The visual examination of the column confirmed that no thermal induced cracking had occurred on the external surface of the column.

h) The additions of flyash to the new concrete mix delayed the heat development (i.e. maximum temperature differential occurred on the cool down phase rather than the heat up phase for the OPC concrete used in Column 1), and slightly lowered the critical temperature differentials within concrete; both these have resulted in a lower probability of cracking in the concrete by comparison with the column 1 OPC concrete.

i) The maximum bulk temperature in the mid section of the column was 79.7°C which occurred at 22 to 24.5 hours of monitoring.

j) The maximum bulk temperature at thermocouple positions 100mm away from the side form was 66.1°C which occurred at 10 hours of monitoring.

2.3.9 Concrete Strain And Cracking Potential

The strain profiles did not indicate any cracking strain relieve during the concrete cool down phase for Column 1 and 2. In other words no internal thermal cracks formed during the concrete cool down.

The strain results indicated heat up phase exterior cracking in Column 1 which was consistent with the visual results.

The cracking in trial Column 1 was primarily caused by differential temperature induced strain. The probability of cracking in Column 2 was reduced by the use of PFA.

Significant comments on the cracking and non cracking in trial columns 1 and 2 are :

a) The insulation used at the top of trial column 1 was one 50mm layer of polystyrene . Its early removal at 13 hours resulted in a sudden drop in temperature at the surface, while the bulk temperature was increasing.

b) The cracking in trial column 1 was due primarily to early removal of insulation and differential temperature induced strain. The probability of cracking in column 2 was reduced by the use of PFA and the non use of polystyrene insulation, and no cracking occurred.

c) The cracks in trial columns 1 would have initiated at the top corner and then propagated across the top surface and down the sides.

d) The exterior cracks which formed on column 1 will be subject to compression during the cool down phase which will tend to close the cracks.

e) Induced strain in the concrete greater than about 80 microstrain will initiate cracking in concrete. Analysis of trial column 1 indicates the monitored temperatures were consistent with the formation of cracks.

f) The general comment on structural significance of early age thermal cracks by CIRIA (Report No 91 'Early Age Thermal Crack Control in Concrete') is that they do not affect the structural integrity.

g) The cracks formed in trial column 1 are not considered to be a durability risk (i.e. no widespread premature durability failure) to the building structure in the future.

2.4 Conclusions

The trial column casting, monitoring and assessment indicated that concrete used in the column which included masscrete (i.e. PFA replacement) had a marginal benefit as regards early age thermal cracking due to lower temperature rise. PFA, as used in trial column 2, reduced the risk of early age thermal cracking occurrence and propagation by:

i) slowing down the heat of hydration heat rise
ii) reducing the peak heat of hydration temperature
iii) reducing and delaying the maximum differential temperature

It was recommended that the following be considered prior to full scale production:

a) check on the consistency of PFA supply and quality in Malaysia
b) additional cost with the use of masscrete
c) reducing fresh concrete temperature prior to placement

The steel formwork stripping can be carried out comfortably between 10 and 12 hours for this grade (80mpa) concrete for both concrete mix designs investigated. Significant considerations are :

a) in situ concrete compression strength exceeds 15 Mpa
b) standard cube sample compression strength exceeds 15 Mpa
c) a relationship of in situ to standard cube compression strength was developed which showed the extent of increase in situ strength gain at early age
d) the standard cube compression strength testing can be used to predict the in situ strength during construction
e) steel formwork removal does not influence thermal crack occurrence as the steel gives no insulation
f) the formwork removal will need to prevent excessive surface concrete tearing during removal particularly if removed too early

Insulation of the column sides and top surface is not considered essential based on the trial column 2 performance (i.e. no cracks observed). Inappropriate use of insulation can increase the likelihood of cracking.

3. Future Developments

High strength concrete is being successfully used in the central core, perimeter columns and perimeter ring beams of the Petronas Towers in a Kuala Lumpur City Centre development. High strength concrete permits vertical core and column elements to be economical and of reasonable size saving rentable space. It permits construction using relatively simple equipment and skills of the local work force.

As economic pressures increase in the centre of the major cities of Malaysia and rentable space increases in cost, the use of high strength concrete is likely to provide an attractive alternative in the medium term. It is therefore necessary to increase the exposure of local construction professionals to HSC and consider incorporating the existing international experience into our codes.

References

1. FIP-CEB, High Strength Concrete - State of the Art Report. FIB-CEB Bulletin d'Information No. 197, Aug. 1990.

2. W.F.Price, P.B. Bamforth, Brite Euram Project 5480, economic Design and Construction with High Strength Concrete, TEL Report 1303/93/6610, July 1993.

3. J.Moreno,225 W.Wacker Drive - State of the Art High Strength Concrete in Chicago. Concrete International, Jan 1990, 35-39.

4. AMERICAN CONCRETE INSTITUTE, State of the Art Report on High Strength Concrete, ACI 363R-92, 1992.

5. T.Murota et al, Development of advanced reinforced buildings using high strength concrete and reinforcing. Buildings Research Institute Ministry of Construction, Ibarakiken Japan 1990.

6. I.Holland, High Strength Concrete- A major research programme. Proc. Symp. "Utilisation of high strength concrete" Stavanger 1987,135-148.

7. Y.Malier (Ed), High Performance Concrete from material to structure , Spon 1992.

8. Netherlands Centre for Civil Engineering Research and Codes (CUR), "High Strength Concrete: CUR Report 90-9, 1990.

9. Dr. Kribanandan Gurusamy, R. J. Paull, Petronas Tower 2 Project (KLCC), Mock Up Columns, Temperature and Strain Monitoring, Report 1303/94/6827, February 1994.

10. Dr. Kribanandan Gurusamy , R. J. Paull ,Petronas Tower 2 Project (KLCC), Mock up Columns Cracking Assessment - summary Report, TEL Report 1303/94/6845 March 1994.

11. Dr. Kribanandan Gurusamy, Petronas Tower 2 Project (KLCC) Strength Development of Masscrete vs OPC, TEL Report 1303/94/6846 April 1994.

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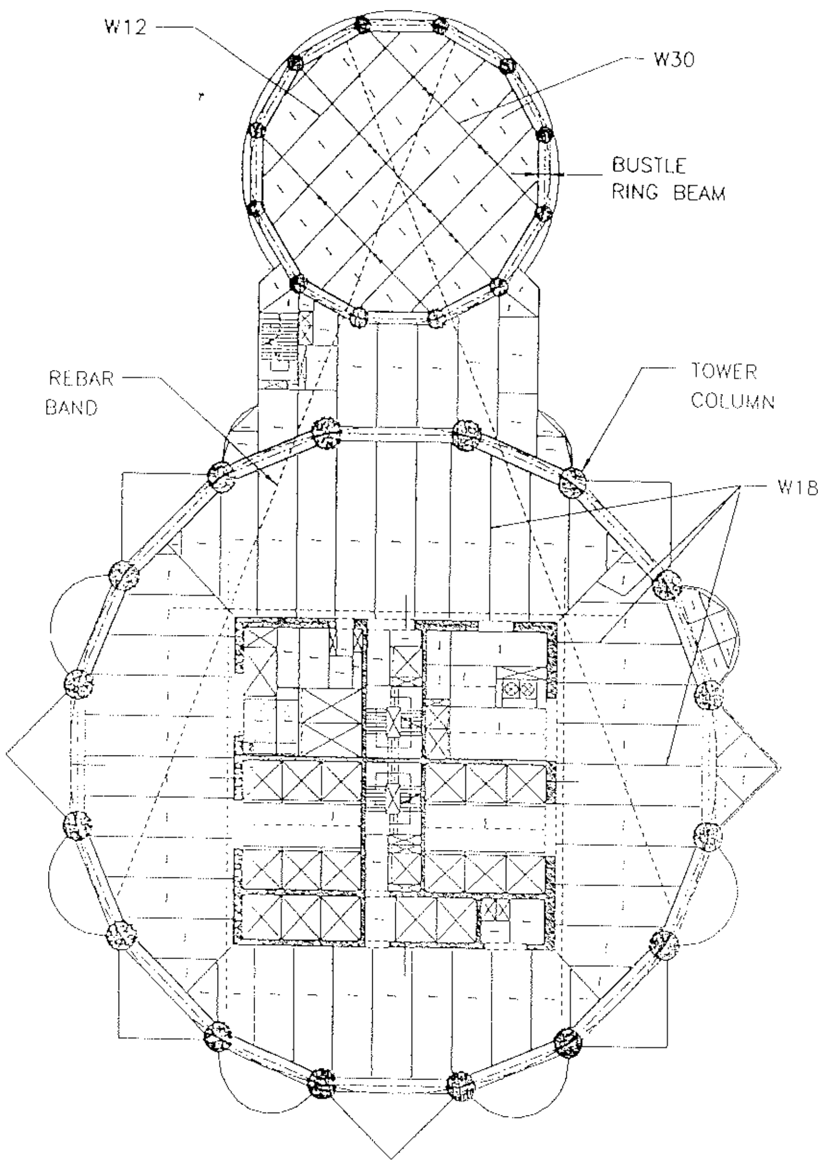


FIGURE 1: PLAN VIEW OF THE PETRONAS TWIN TOWER BUILDINGS

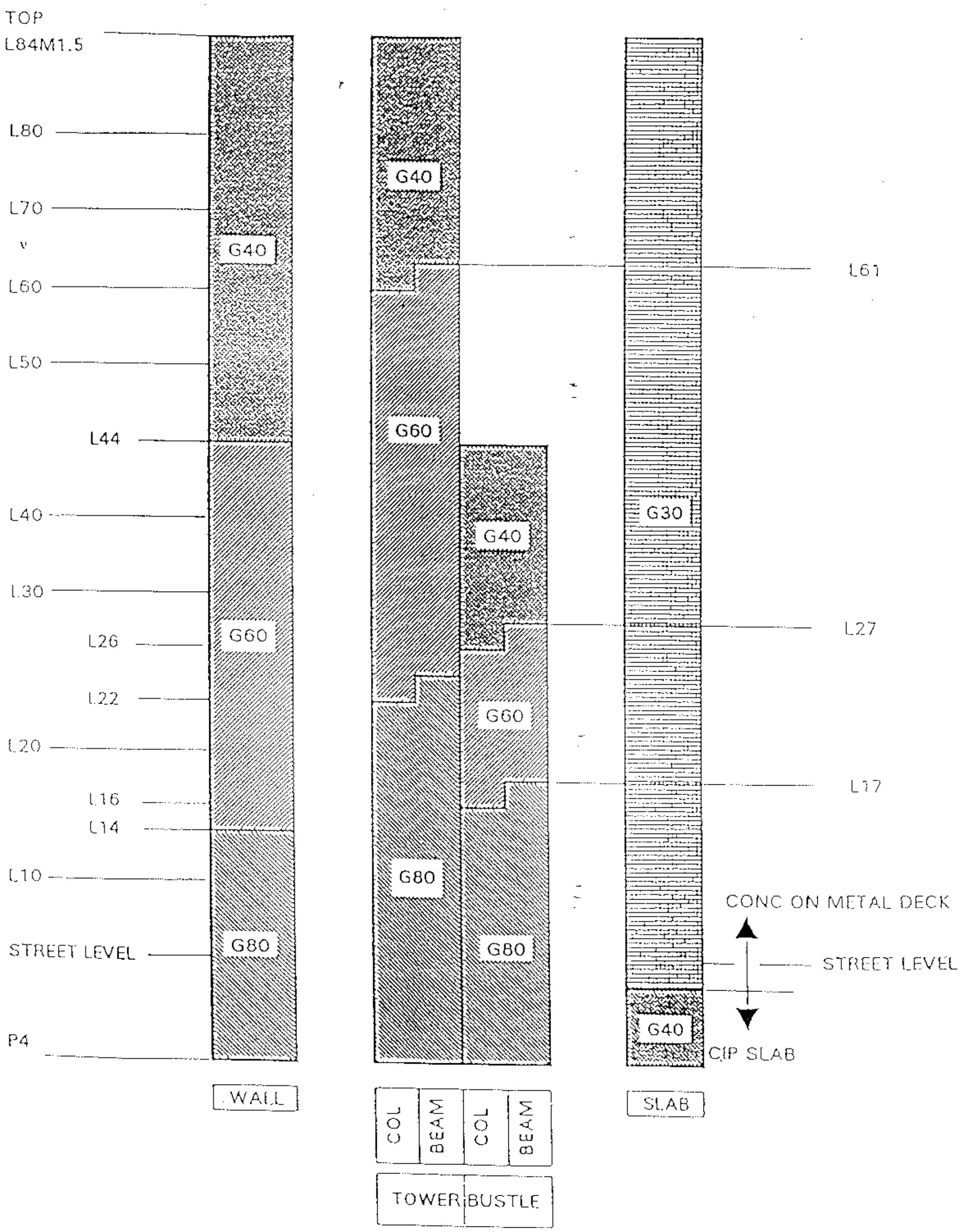
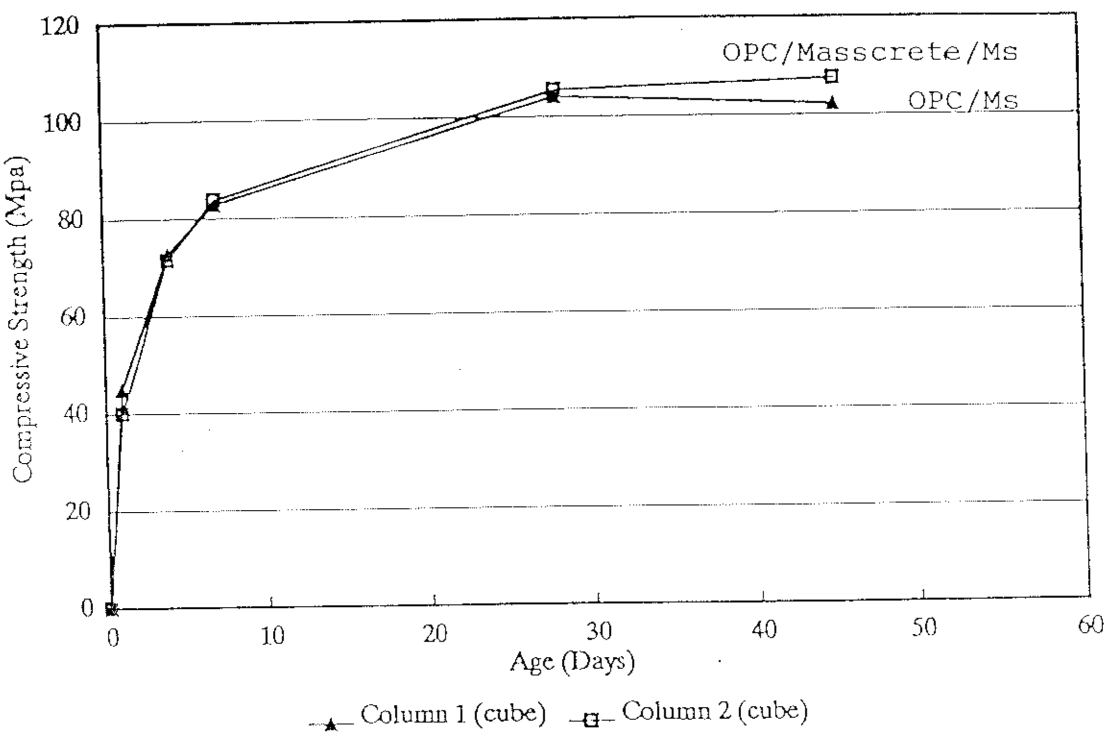


FIGURE 2: CONCRETE GRADE FOR FULL HEIGHT TOWER & BUSTLE

FIGURE 3: CUBE STRENGTH RESULTS



Column 1 (Grade 80, OPC/Microsilica)
Column 2 (Grade 80, OPC/MASSCRETE/Microsilica)

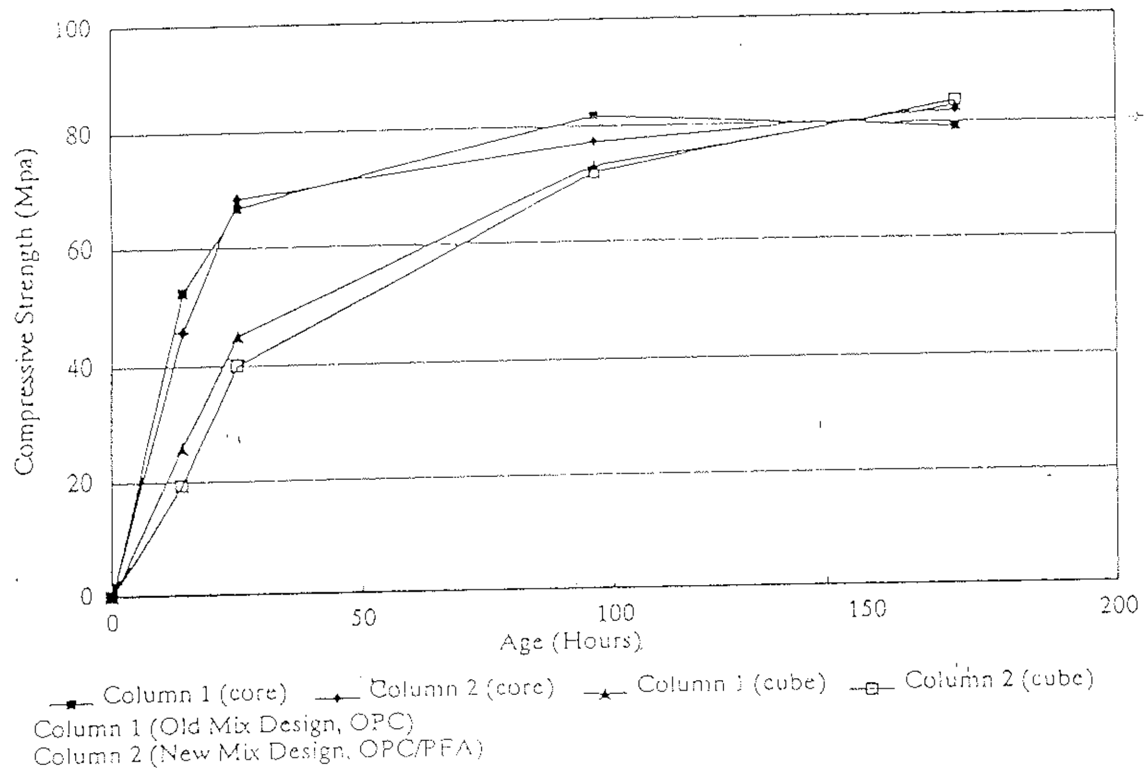


FIGURE 4: CORE AND CUBE STRENGTH RESULTS FOR GRADE 80 CONCRETE MOCKUP COLUMNS,, PETRONAS TWIN TOWER PROJECT

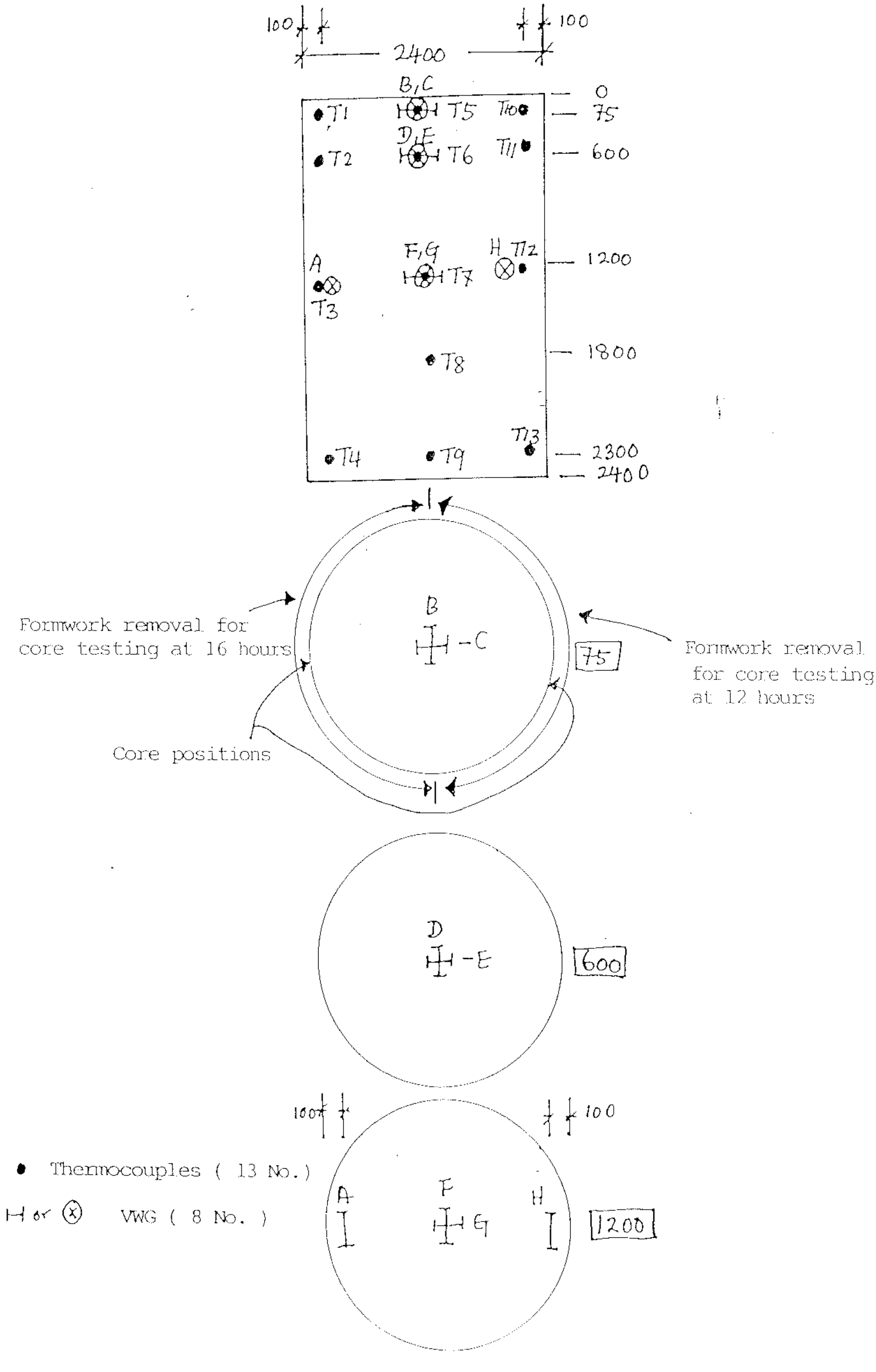


FIGURE 5: ACTUAL THERMOCOUPLE AND STRAIN GAUGE POSITION COLUMN 1

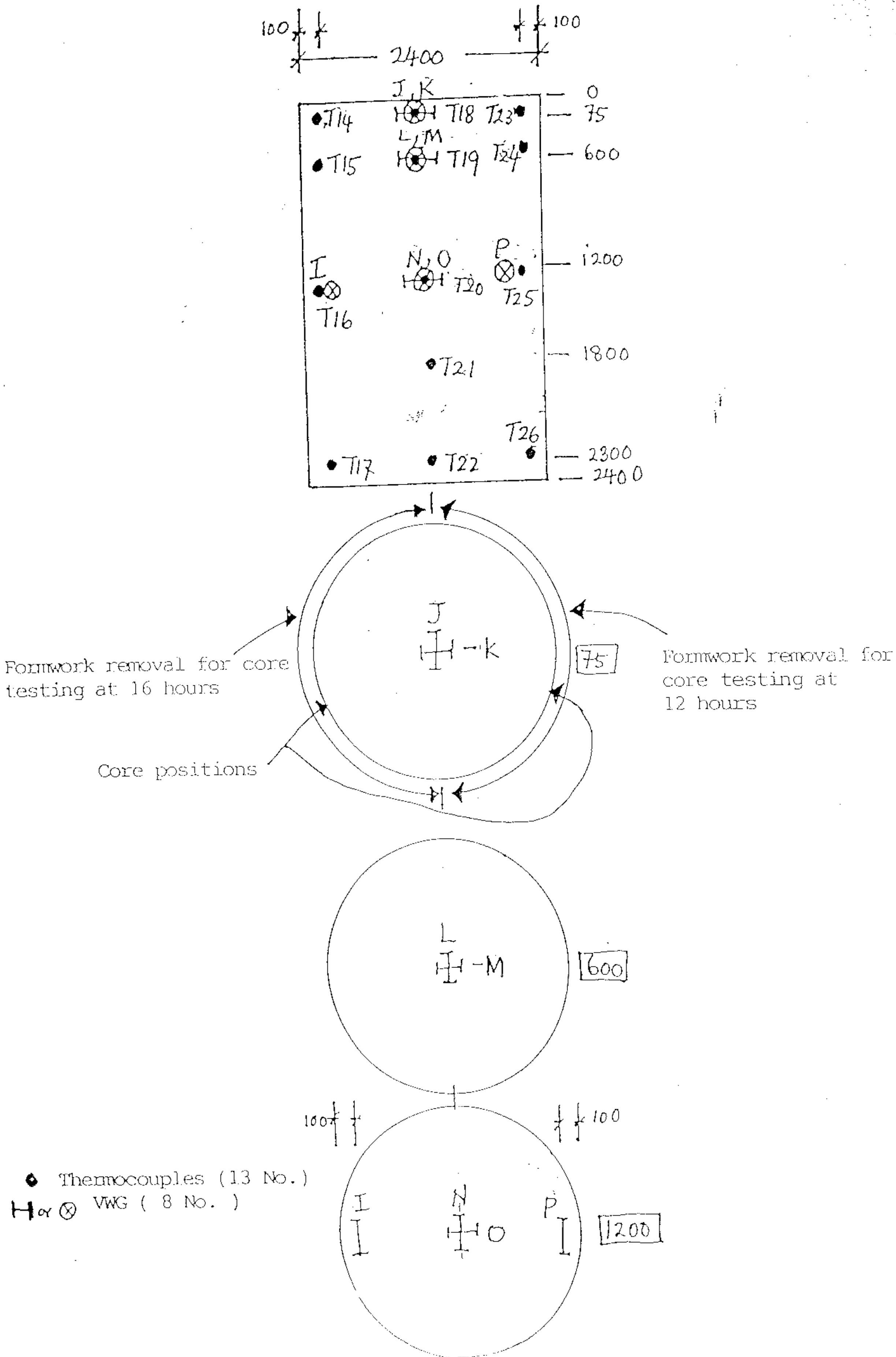
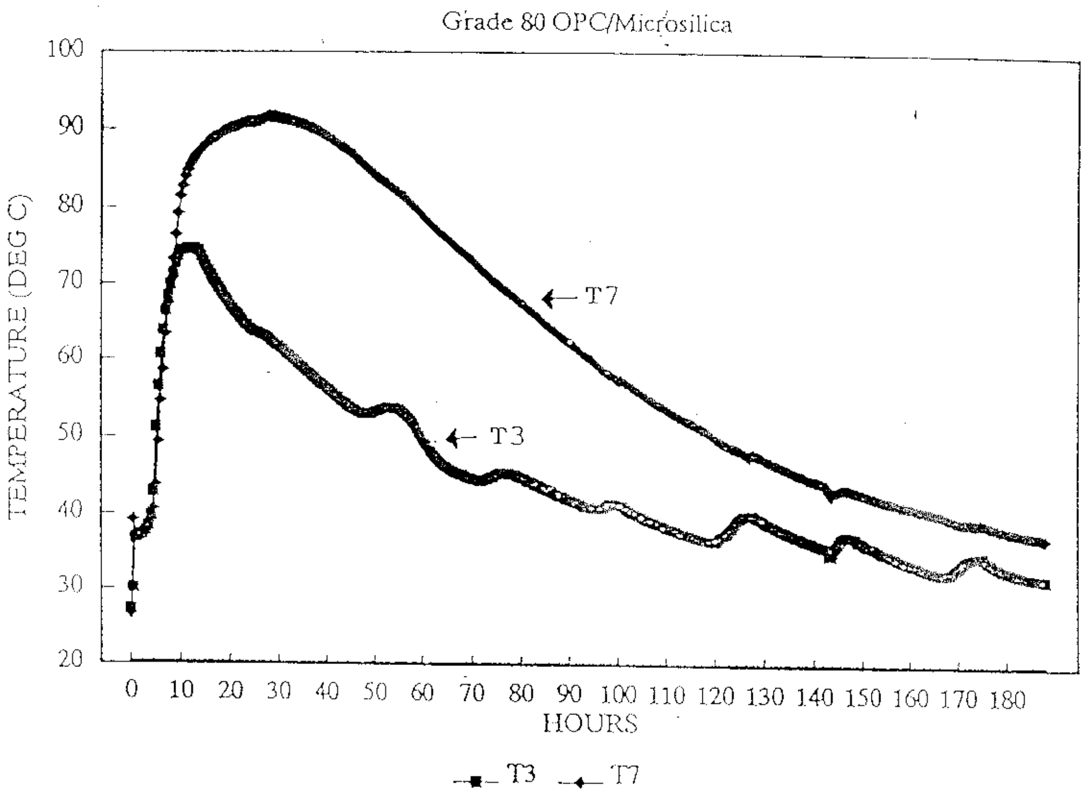


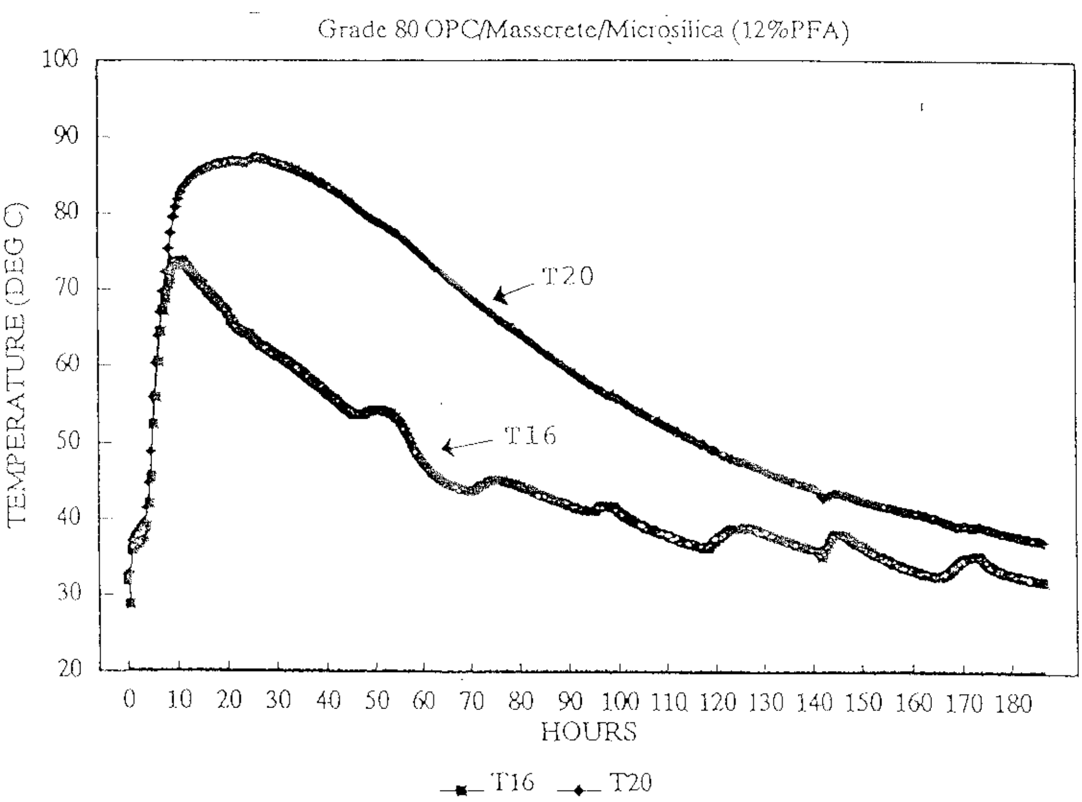
FIGURE 6: ACTUAL THERMOCOUPLE AND STRAIN GAUGE POSITION COLUMN 2

FIGURE 7: TEMPERATURE PROFILE



Note: Refer Figure 5 for thermocouple positions

FIGURE 8: TEMPERATURE PROFILE



Note: Refer Figure 6 for thermocouple positions

TABLE 1: COLUMN 1 - OPC MIX

ITEM	DESIGN MIX	ACTUAL MIX BATCH 1	ACTUAL MIX BATCH 2
OPC (kg/m^3)	505	503	503
MASSCRETE (kg/m^3)	–	–	–
SILICA FUME (kg/m^3)	30	29	30
WATER (litres)	134	133	133
C. AGG (kg/m^3)	1000	990	1000
F. AGG (kg/m^3)	750	738	737
P300N	1.00	1.0	1.0
R1000	9.06	9.08	9.08
SLUMP (mm)	220	195	200
CONC. TEMP (^0C)	–	32	33

TABLE 2: COLUMN 2 - OPC-MASSCRETE MIX

ITEM	DESIGN MIX	ACTUAL MIX BATCH 1	ACTUAL MIX BATCH 2
OPC (kg/m^3)	184	185	186
MASSCRETE (kg/m^3)	345	343	343
SILICA FUME (kg/m^3)	35	34	34
WATER (litres)	152	152	152
C. AGG (kg/m^3)	1006	1003	1000
F. AGG (kg/m^3)	728	715	725
P300N	0.8	0.8	0.8
R1000	8.48	849	849
SLUMP (mm)	220	220	190
CONC. TEMP (^{o}C)	–	33	35

Note: This approximates to a OPC/PFA/Silica Fume Mix of 460/69/35 or a 12.2% PFA replacement mix (masscrete is nominally a 20% PFA replacement mix).

TABLE 3: CORE RETRIEVAL - TARGET TIMES

Time After Casting	Date and Time for Coring	Time for Testing	Core No
9 hrs	21.00 08/02/94 (Tue)	12 hrs	1 , 2
13 hrs	01.00 09/02/94 (Wed)	16 hrs	3 , 4
21 hrs	09.00 09/02/94 (Wed)	24 hrs	5 , 6
93 hrs	09.00 12/02/94 (Sat)	96 hrs	7 , 8
7 days	09.00 15/02/94 (Tue)	7 days	9, 10
43 days	10.00 22/03/94 (Tue)	46 days	21, 22

Column 2 (Casting 13:30 on 08/02/94)

Time After Casting	Date and Time for Coring	Time for Testing	Core No
9 hrs	22.30 08/02/94 (Tue)	12 hrs	11, 12
13 hrs	02.30 09/02/94 (Wed)	16 hrs	13, 14
21 hrs	10.30 09/02/94 (Wed)	24 hrs	15, 16
93 hrs	10.30 12/02/94 (Sat)	96 hrs	17, 18
7 days	10.30 15/02/94 (Tue)	7 days	19, 20
44 days	10.00 23/02/94 (Wed)	46 days	23, 24

TABLE 4: CORING DETAILS FOR COLUMN 1

CORE NO	CORING DATE	CORING TIME		LENGTH/mm	REMARKS
		START	FINISH		
2	08/02/94	21:45	22:20	210	No visual defects (NVD)
1	08/02/94	22:34	23:50	280	Steel bars on both sides
3	09/02/94	00:25	01:45	227	No visual defects (NVD)
4	09/02/94	01:45	02:10	250	Steel bars on both sides
5	09/02/94	09:00	09:35	211	Steel bars on both sides
6B (Note 1)	09/02/94	10:05	10:45	215	Steel bars on both sides
7	12/02/94	09:00	10:30	256	Steel bar on one side
8	12/02/94	09:00	10:30	265	Steel bar on one side
9	15/02/94	09:00	10:00	267	No visual defects (NVD)
10	15/02/94	10:20	11:30	260	Steel bar on one side
21	22/03/94	11:45	14:45	1250	Steel bars on both sides
22	22/03/94	15:50	19:45	1200	Steel bar on one side

NOTE: 1 The first core taken at this location (6A) was too short for testing purposes and a second core 6B was taken.

TABLE 5: CORING DETAILS FOR COLUMN 2

CORE NO	CORING DATE	CORING TIME		LENGTH/mm	REMARKS
		START	FINISH		
12	08/02/94	23:00	23:35	264	No visual defects (NVD)
11	08/02/94 09/02/94	23:40	00:15	250	Steel bar on one side
13	09/02/94	02:35	03:15	280	Steel bar on one side
14	09/02/94	02:50	03:00	260	Steel bar on one side
15	09/02/94	11:00	11:40	217	Steel bar on one side
16	09/02/94	12:00	12:40	212	Steel bar on one side
17	12/02/94	10:30	11:30	275	Steel bar on one side
18	12/02/94	10:30	11:30	266	Steel bar on one side
19B	15/02/94	14:05	14:30	275	Steel bar on one side
20B	15/02/94	14:35	15:10	280	Steel bar on one side
23	23/03/94	09:10	12:15	1450	Steel bar on one side
24	23/03/94	12:50	16:40	1330	Steel bar on one side