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THE PETRONAS TOWER: THE WORLD'S TALLEST BUILDING

by

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Abstract:

The 450 m Petronas Twin Towers under construction in Kuala Lumpur, are part of a massive real estate development set to transform the city centre into a bustling metropolis. The world's tallest building is being constructed with concrete columns, ring beams and a core of 40 to 80 Mpa cube strength concrete and steel long span floor beams.

The paper discusses the general foundation and structural system of the towers and related benefits of high strength concrete. In particular the pre-construction Consultancy inputs undertaken by the Author's company by way of trial column construction to support the use of the high strength concrete, and the requirements for curing, insulation, striking time, strength development, concrete temperature and strain monitoring are outlined. The concreting logistics , construction approach and quality assurance of concrete supply are also dealt with.

1. INTRODUCTION

The Petronas Towers are part of a massive real estate development on a 100 acre site in Kuala Lumpur city centre and will include office buildings, a retail centre, hotels, residential buildings and substantial public parks, gardens and lakes. The Petronas Towers linked by a skybridge at mid height and associated retail base and parking facilities are the first developments on the site and due to be ready in the middle of 1996. It consists 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 the tallest building in the world on completion in 1996. A plan view of the structure is shown in figure 1. This is the first project in Malaysia where such high strength concrete has been specified. To achieve the projected completion in approximately 28 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 main structural system for the superstructure and foundation design were selected after a rigorous study and evaluation by the Design and Project Management team. The structural approach in the tower frame combines the most favourable aspects of concrete and steel construction. Structural Steel is used for long-span typical floor beams supporting metal deck slabs. Structural concrete is used in foundations, in the central core, in sixteen tower perimeter columns and variable depth perimeter beams and in twelve smaller columns and ring beams around the bustle (half height mini tower attached to the main tower). Outrigger beams link the core and perimeter at levels 38 to 40 for additional efficiency (1). The production and delivery of high strength concrete was of particular concern for the structural elements. Rigorous trials were undertaken prior to construction to confirm the project specification requirements could be met.

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2. FOUNDATIONS AND SUPERSTRUCTURE

The foundation system of the Towers consists of a 4.5m thick piled raft supported on rectangular friction piles (barrettes) varying in depth from 40m to 110m. The variation in pile lengths is to control predicted settlement under differing thickness of Kenny hill formation underlain by limestone (see figure 2). The 13,200 cubic meters of concrete in each raft was cast in one continuos 50 hour operation which therefore avoided any construction joints. Concrete grade 45 was specified for the compression piles. This is a low heat mix and allowed the use of a mix with good workability and slow setting time needed for tremie concreting. The raft concrete is 60 Mpa cube strength while the lower level columns in the tower are 80 Mpa.

In the case of the Raft concrete which is really a mass foundation, a highly workable concrete was required to facilitate the pumping, placing and compacting operations. The 60 Mpa raft concrete contained 9% silica fume to achieve the required strength, workability and cohesion. The initial temperature of the concrete at pour was reduced by using chilled water for production of concrete at the batching plant, cooling the aggregate by spraying with water and sheltered as feasible and stockpiling cement for several weeks so as to cool rather than being used warm from the mill. This allowed the peak temperature of the concrete to be maintained below 90^{o}C. (1)

In mass concrete pours, significant cracking can occur due to temperature differentials between concrete core and the surface of concrete. To prevent a large heat loss and therefore large temperature differentials, the top of the raft was insulated using 50 mm thick polystyrene and with the pre-cast formwork panels providing insulation to the sides. The temperature gradient was continuously monitored and measured by means of thermocouples placed at several depths in the raft and limited to 25 ^{o}C.

Various approaches were explored in developing the overall structural system of the Petronas towers. The scheme being implemented consists of cast in-place perimeter frame with sixteen columns and cast in-place concrete core. Outrigger beams at mid-height of the structure provides additional stiffness to the structure. The concrete used varies in three steps from grade 80 at the lower floors to grade 40 at the upper floors. Grade 80 is specified up to level 22 for the 2.4m diameter reinforced concrete columns. The floor system consists of cast in-place concrete slab on ribbed metal deck to act compositely with filled concrete, supported on steel beams.

The perimeter ring beams at tower and bustle were of a tapered construction to overcome the problem of the limited space available. Concrete grades for the ring beams follow the grades in the columns to avoid confusion in the field and possible waste in the concrete pump lines. Each Tower has one central core for all lifts, tower exit stairs and mechanical services. Core design resulted in two virtually solid walls running north-south and one running east- west making the core quite stiff and efficient. Concrete grade varies in three steps from 80 Mpa to 40 Mpa. Due to the premium in cost for high strength concrete grades it was determined that concrete grade reduction should be made first, and then reduction in wall thickness, even though this results in greater total concrete volume.

2. HIGH STRENGTH CONCRETE THE LOGICAL CHOICE

Various approaches were considered for the structural framing system of the Petronas towers (1,2). This included the all concrete option, various mixtures of composite steel and concrete structures. In a detailed study of cost, constructibility and practicality it was confirmed that the concrete option was the correct solution. The benefits of the high strength concrete option include:

Structural Efficiency - Columns of concrete and particularly high strength concrete carry vertical loads at a cost per unit load which is a small fraction of that of steel. Using high strength concrete further improves efficiency and adds to the advantage of reductions in member size at lower levels and therefore saving on rentable space. In addition the core and frame provide adequate lateral stiffness without the need for additional structural materials while the core walls serve as fire rated structural members as well as carrying vertical and lateral load.

Constructibility - Cast in-situ concrete can be placed by conventional means and avoids heavy craneage or special rigging to lift large prefabricated building frame elements. This has allowed considerable flexibility to the contractors and maximises use of the skills of the local labour pool.

Occupant comfort - The high average mass density of the towers, lengthens the building period, reducing perceptions of acceleration and improving comfort under windy conditions. In addition the concrete core, columns and ring beams contribute to the damping values providing occupant comfort without the cost and space penalty of special damping devices.

3. PRE-CONSTRUCTION CONSULTANCY

3.1 Introduction

Due to the nature of this project being the first super tall structure of its kind and the very limited experience with the use of high strength concrete in Malaysia the contractors were required to demonstrate that the requirements of the project could be achieved prior to actual construction of the structural elements. In this context the author’s company were involved in the construction of full size trial columns and rigorous monitoring of concreting materials. All potential problems were identified and brought to the attention of the contractor and relevant changes made.

3.2 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 formwork (<15 hours), minimising cracking in corewalls and curing requirements to achieve sound concrete.

3.3 Trial Column Casting

3.3.1 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, in-situ strength development particularly at early age and temperature differentials within concrete affecting cracking potential.

3.3.2 Dimensions of Trial Columns/Formwork Details

The trial columns were of dimensions 2.4m height and 2.4m diameter. Two identical columns were fabricated with the same system formwork to be used for the actual 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.

3.3.3 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. Pulverised Fuel Ash (PFA) was introduced into the second mix by using mascrete supplied by Associated Pan Malaysia Cement (APMC). According to APMC product literature, mascrete 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 1m^3 concrete 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.

3.3.4 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.

3.3.5 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 in-situ 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) and 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 indicated that the target cube strength was met.

The in-situ strength of concrete as measured by taking cores were compared to standard cube testing at early age details of which are given elsewhere (3) The results of in-situ 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 in-situ compression strength and the 15 MPa strength requirement is exceeded by the cubes after 8 hours. Stripping of formwork 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 in-situ strength for formwork removal.

3.3.6 Curing

The concrete was cured by the side formwork 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.

3.3.7 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 would still have retained heat and would 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 i.e. removal before the internal temperature of concrete had peaked can increase the likelihood of cracking.

3.3.8 Concrete Temperature And Strain

Concrete temperature and strain were monitored for a minimum of 7 days in the columns. The monitoring locations and detailed results are given elsewhere. (3)

Significant monitoring data results (4,5,6) 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^{o}C 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^{o}C which occurred at 14.5 hours of monitoring.

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

a) The peak temperature recorded was 87^{o}C at the centre of column after 26.5 hours of monitoring.

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

c) The temperature rise per 100kg cementitious material was 9.8^{o}C.

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

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

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

g) Although the temperature differential results exceeded the limits for granite concrete (of 27.7^{o}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^{o}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^{o}C which occurred at 10 hours of monitoring.

3.3.9 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.

3.4 Conclusions

The trial column casting, monitoring and assessment indicated that concrete used in the column which included mascrete (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

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 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.

4 MATERIALS SUPPLY AND QA/QC

4.1 Introduction

The potential problems of materials supply and the stringent QA/QC requirements to achieve the desired concrete were recognised by the client Kuala Lumpur City Centre Berhad (KLCC). In this context part of the contractual requirements put the emphasis on the contractor to establish a comprehensive QA/QC plan for the concreting operations.

To avoid problems of concrete supply to a city centre site, a concrete ready mix company was given the contract to erect and operate an on-site concrete plant. Initially two wet-mix plants were established and a third added later. All the concrete could be distributed around the site on internal site roads which meant negligible delay between the plant and delivery locations.

4.2 Materials used

All cement came from APMC plant in Rawang including mascrete which is an interground blend of OPC with PFA (20%). The PFA is from the TNB power station in Kapar. The coarse aggregate was a 20-25mm crushed granite which came mainly from the Golden Plus quarry in Ampang about 10km from site. The sand was obtained from Puchong, extracted from a large stock of tin mine sand and delivered after processing. All chemical admixtures were supplied by Master Builders Technologies (MBT), this included Silica Fume, a Conventional retarder (P300) and A conventional Superplasticiser (R1000).

4.3 Mix Design

The concrete mix design was aimed at achieving a cohesive pumpable mix with a target slump of 200mm and a characteristic 56 day strength of 80 Mpa. The contract specification limited the water cement ratio to 0.25 for grade 80 Mpa concrete. This was later relaxed to 0.27. The requisite mix was achieved by incorporating PFA (mascrete) and chemical admixtures. Strict control on all materials ensured a consistent concrete which in general met the specification requirements. The Grade 80 concrete was supplied to Tower 1 and 2 from April to December 1994.

4.4 Quality Assurance

Each contractor was required to operate a quality plan approved by the client. Taywood Engineering helped establish the onsite quality plan for Tower 2 over the period March 1993 to February 1994 which included checks on the materials suppliers, the concrete producer and the contractors own supervision.

Aggregates and Sand : Initial approval including petrography. and Routine grading measurements for organic impurities (sand only)

OPC and Mascrete: Routine British (BS) and Malaysian Standard Tests (MS)
24 hour strength tests
Alkali content.
Temperature checks on loading
Carbon Content (pfa portion of mascrete)

Admixtures and Silica Fume : Routine BS/MS tests and manufacturing consistency tests.

Concrete Production:	Routine Strength and workability tests Production records check Water temperature checks Concrete temperature checks Tests of elastic modulus, shrinkage and creep General Production Supervision
Concrete Delivery:	Check of delivery docket Re-verification of temperature and slump Strength verification for formwork removal Inspection of finished surfaces

5 CONCLUSIONS

High strength concrete is being successfully used in the central core, perimeter columns and perimeter ring beams of the Petronas Towers in the 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 workforce.

As economic pressures increase in the centre of major cities 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 national codes.

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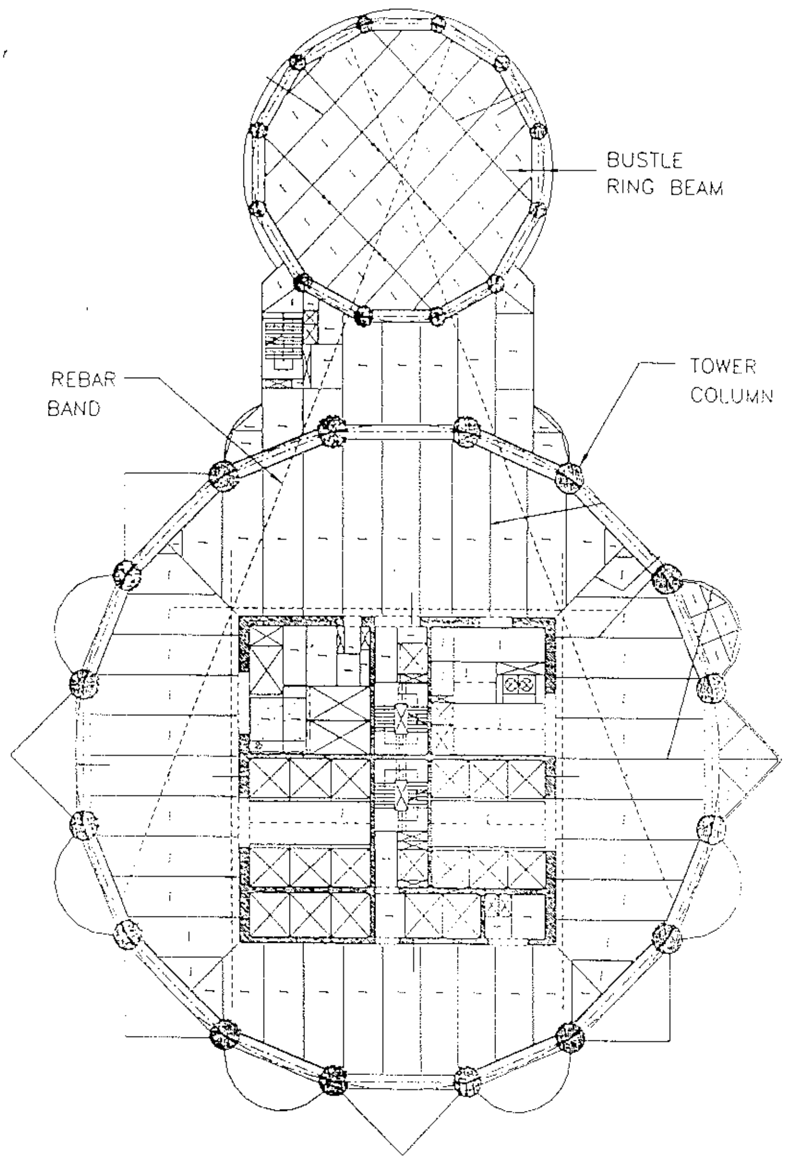

Figure 1 Typical Floor below level 38

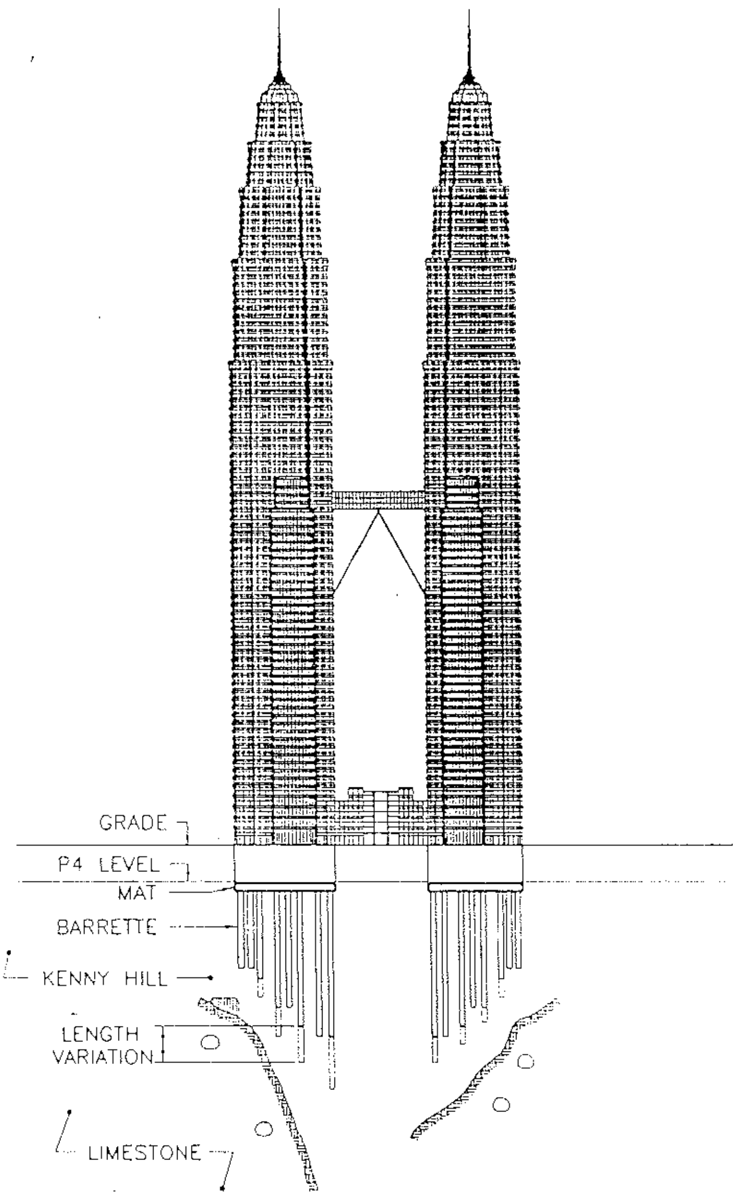

Figure 2 Elevation including foundation details

TABLE 1: OPC CONCRETE MIX (column 1)

ITEM	DESIGN MIX	ACTUAL MIX	
		BATCH 1	BATCH 2
OPC (kg/m³)	505	503	503
MASSCRETE (kg/m³)	-	-	-
SILICA FUME (kg/m³)	30	29	30
WATER (litres)	134	133	133
C. AGG (kg/m³)	1000	990	1000
F. AGG (kg/m³)	750	738	737
P300N	1.00	1.0	1.0
R1000	9.06	9.08	9.08
SLUMP (mm)	220	195	200
CONC. TEMP (°C)	-	32	33

TABLE 2: OPC MASCRETE CONCRETE MIX (column 2)

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 (°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).