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ISSE

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Contents

(VIEWS APPEARING IN THESE ARTICLES ARE THE AUTHORS' OWN VIEWS AND ARE NOT NECESSARILY THE OFFICIAL VIEWS OF ISSE)

ISSE FRATERNITY

WELCOME TO NEW MEMBERS

We welcome following new members, who joined after 1- 7 - 2006

M-753 Lalpuria Kantilala D.
M-755 Sakthivel Ramanathan
M-757 Patel Kiran R.
M-759 Dingane Rahul Ashok
M-761 Somaiya Dilip H.
M-763 Tambe Ujwala Eknath
M-765 Khodade Tanaji Shankar
M-767 Redekar Madhukar Krishna
M-769 Shukla Mahendra Prasad
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M-777 Kulkarni Kiran Ramkrishna
M-779 Khot ArunkumarAppasaheb
M-781 Soman Mrudul Sharad
M-783 Thombare Vishal Ramesh
M-785 Chaudhari Sanjeev V.
M-787 Patel Manojkumar Kantilal
M-789 Bhise Jitendra Mangesh
M-791 Chikodi Kedar Arvind
J - 21 Newalkar Naynesh Sharad
M-754 Tinwala Juzer A.
M-756 Brahmbhatt Rajendra B.
M-758 Patane Shivadatta S.
M-760 Deshpande Sachin Vilas
M-762 Golam Uday Hanumant
M-764 Rewale Jayanti S.
M-766 Nagda Hitendra Sunderji
M-768 Pachchigar Divyakant V.
M-770 Anilkumar Hanumanthappa
M-772 Jayarm M. V.
M-774 Naik Deepak Dinanath
M-776 Tipnis Rajeev Shivaji
M-778 Patel Bhavesh Jagdishchandra
M-780 Chavan Ravindra Shankar
M-782 Deora Durga Singh
M-784 Chopda Rajendra B.
M-786 Anwala Kailash G.
M-788 Shah Rajesh Ghanshamdas
M-790 Chougule Dhanapal B.
M-792 Shah Rajesh Natvarlal

WITH THE NEW ADDITIONS, FRATERNITY (AS ON 30.9.2006) IS

MEMBERS : 792 ORGANISATION MEMBERS : 12 JUNIOR MEMBERS : 5

PATRONS : 29 SPONSORS : 8

REVISED STRENGTH : 846

OUR INTENTIONS

1. To restore the desired status to the Structural Engineer in construction industry and to create awareness about the profession.
2. To define Boundaries of Responsibilities of Structural Engineer, commensurate with remuneration.
3. To get easy registration with Governments, Corporations and similar organisations all over India, for our members.
4. To reformulate Certification policies adopted by various authorities, to remove anomalies.
5. To convince all Govt. & Semi Govt. bodies for directly engaging Structural Engineer for his services.
6. To disseminate information in various fields of Structural Engineering, to all members.

FIELDS CONSIDERED AS ASPECTS OF STRUCTURAL ENGINEERING :

- Structural Designing & Detailing
- Computer Software
- Materials Technology, Ferrocement
- Teaching, Research & Development
- Rehabilitation of Structures
- Construction Technology & Management
- Geo-Tech & Foundation Engineering
- Environmental Engineering
- Non Destructive Testing
- Bridge Engineering

& Other related branches

OUR ESTEEMED SUPPORTERS

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Rehab Consultants Pvt. Ltd.
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Dalal Consultants And Engineers Limited
A. A. Nayak & Co.
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B] PATRON MEMBERS :

Refab Construction
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Orient Timber Industries Pvt. Ltd.

INDIAN SOCIETY OF STRUCTURAL ENGINEERS LIST OF PUBLICATIONS AVAILABLE AGAINST DONATION AT ISSE HEAD OFFICE

	TITLE	Amount of Donation
1)	Proceedings of National Conference on - Corrosion Controlled Structure in New Millenium	Rs. 400/-
2)	Design of Reinforced Concrete Structures for Earthquake Resistance	Rs. 600/-
3)	Workshop & Seminars-Seismic Design Book	Rs. 150/-
4)	Proceedings of One Day Course of ISO-9001	Rs. 150/-
5)	Brain Storming Session On Use Of Speciality Products In Structures.	Rs. 200/-
6)	Proceedings of Workshop on Software Tools for Structural Design of Buildings with CD.	Rs. 500/-
7)	Proceedings of Workshop on Structural Audit	Rs. 150/-
8)	Professional Services by Structural Design consultant - Manual for practice	Rs. 150/-
9)	Guidance for Effective Use of Structural Software.	Rs. 150/-
10)	Guidance for Effective Use of Structural Software - CD	Rs. 100/-
11)	Shear Walls in Highrise Buildings	Rs. 150/-
12)	Seminar Papers of “Innovative Repair Materials / Chemicals”	Rs. 200/-
13)	Foundations for Highrise Buildings	Rs. 150/-

AKSHARDHAM TEMPLE- NEW MASTERPIECE IN DELHI

Akshardham Monument (Height: 141 ft; Width: 316 ft; Length: 356 ft)
is a creation of divine inspiration and many hands

CONCEPT & PLANNING

Swaminarayan Akshardham has 234 ornately carved pillars, 9 domes, 20 *samvaran shikhars* and 20,000 *murtis* inside and outside of the monument.

How did the colossal Swaminarayan Akshardham take shape in only five years? The story of its creation is as fascinating and phenomenal as its magnificent character and impact.

Swaminarayan Akshardham is not the creation of one architect who designed and planned it. The story is one of divine inspiration and human endeavour, out of which, like the unfurling of the petals of a flower, Akshardham blossomed in all its beauty and magnificence. Through the grace of Bhagwan Swaminarayan and the inspiring force of Pramukh Swami Maharaj, the Akshardham project team members were inspired with ideas. Every part of Akshardham reflects this divine inspiration that worked through the minds of experts in different fields. The facts provided below are a testimony to the cooperative efforts and divine inspirations.

In December 1992, after the opening ceremony of Akshardham at Gandhinagar had concluded, Swamishri started the Akshardham project for New Delhi.

In August 1994, when Swamishri was in a small town in the Czech Republic, he discussed with Ishwarcharan Swami and a team of sadhus about the architectural design of Akshardham. After studying photographs of India's ancient mandirs of Angkor Vat, Konark, Jagannathpuri, Bhuvaneshwar and others Swamishri gave his decision, "The monument (in Delhi) should be made of pink stone from Bansipahadpur. It should be unique by itself." After several discussions with Swamishri, Ishwarcharan Swami declared that the monument would have nine *ghummats* (domes) and four *roop chowkis* (porches). Out of the nine domes, four would be profusely carved and saucer-shaped from inside and five would be in-depth with carvings. This was a novel decision. It was decided to make the stone monument in accordance with the pristine Indian traditional style of architecture. Thereafter, Ishwarcharan Swami and Harshadbhai Chavda discussed the plans with architects in Amdavad, who prepared 12 different sketches. In 1995 Swamishri saw these sketches and plans in London and made some important suggestions.

In the meantime, the process for acquiring the proposed land was under way. Also, at Swamishri's suggestion, in 1997, Ishwarcharan Swami and a team of six sadhus started discussions on the Akshardham project. On the basis of these discussions, Shrijiswarup Swami designed the Akshardham complex.

With inputs from Swamishri and the distillation of discussions held by the team of sadhus, two models were made in 1997. When Swamishri saw them in Bochasan he chose one and suggested some important changes. The models had galleries that made it look like a royal palace. Swamishri wished that the monument should have a spiritual look. With this in mind he commissioned the Akshardham team of sadhus to study the architecture of ancient mandirs of India and design a unique and impressive monument. Swamishri declared that the monument should not be less than 108 feet high.

At Swamishri's word the Akshardham team of sadhus visited and studied India's ancient mandirs that dated to before the 12th century. In June-July 2000, Ishwarcharan Swami and the team of sadhus arrived at some important decisions regarding the design of the Akshardham monument. The decisions were as follows:

1. ANCIENT STYLED MANDOVAR (OUTER WALL)

Instead of Akshardham having a plain outer wall *(mandovar)* with galleries, as designed earlier, it was decided to have it designed according to the ancient Nagradi style with ornately carved layers. The *kanpith, kumbhs, kalashes, jangha, chhaja* would also be carved profusely.

2. MANDOVAR WITH MURTIS

Besides having carved layers of the *mandovar* another decision was to have *murtis* installed in the *jangha* (middle part of wall). The recesses for the *murtis* would be decorated with ornate pillars. After due research the *murtis* of India's rishis, *acharyas,* avatars and great personalities would be made and installed.

3. PROFUSELY CARVED PILLARS

In the design by the *sompura* there was only one type of stereotyped carved pillar. Instead, in the *mandapams* of the nine domes, six types of different carved pillars were designed by the Akshardham team to provide different experiences. The team of sadhus decided upon the designs in alliance with ancient Indian architecture, namely: (1) the small dome by the main entrance to the monument would have octagonal pillars and the recesses of the pillars would have beautiful *murtis,* (2) the central dome area (Swaminarayan Mandapam) and porch of the main entrance would have quadrangle pillars and its recesses would have ornate *murtis,* (3) the dome behind the central dome would have ornately carved octagonal pillars with no *murtis* carved into them, (4) to the right of the central dome there would be thin pillars and quadrangle pillars with delicately carved *murtis* installed in them, (5) to the left of the central dome the pillars would have less broad layers with small carved *murtis* in them. The 25 ft high pillars would be ornately carved at their corners and middle parts.

4. Main Entrance with Carved Pillars

In the original design there were four doors and porches to the monument. A change was made in which it was decided to have two tall profusely carved pillars in front of the main entrance with *samvarans* on top. There would also be pillars and *samvarans* on the two side porches of the monument. The pillars would be carved similarly to the pillars at Oshiya and Modhera mandirs.

5. Intricately Carved Kakshasan (rear wall of the monument)

Instead of having a porch at the rear side of monument, it was decided to have an ornate *kakshasan*, or wall.

6. 500 Paramhansas

No decorative arches (*kamans*), stone garlands (*torans*) or brackets were to be placed on top of the upper base of the pillars. Instead, *murtis* of the 500 *paramhansas* would be installed.

7. Ornately Carved Ceilings and Domes

It was decided that the inside of four of the domes be saucer-shaped with traditional ornate designs. And the inside of the other domes be decorated with in-depth carvings of the symbols of Sanatan Dharma.

8. Marble Inside the Monument

The pink sandstone monument would have marble stone inside. The pillars and its upper bases would be made of marble. The floors would also be of marble, while the beams would be made of pink stone. Later Swamishri suggested that the beams and all the domes be made out of marble.

9. Monument with Two Peeths (plinths)

Instead of the norm of having one plinth in ancient mandirs it was decided to have two plinths *(peeths)*. The lower plinth would comprise of stone elephants. After extensive research the Akshardham team of sadhus designed a creative and unique plinth supported by stone elephants.

10. Height of Monument

Everyone felt that instead of having a low monument with a royal palatial look it would be better to have a higher structure. Initially, with Swamishri's consent, it was decided to have the monument at a height of 108 ft. Thereafter, following several detailed discussions, the Akshardham team of sadhus presented a monument design with the alternative heights of 121 ft, 123 ft, 127 ft, 129 ft and others. Shrijiswarup Swami had all the heights designed on a computer, from which Swamishri chose the design with its present height of 141 ft.

11. Two-storey Parikrama

After due research on ancient mandirs it was decided to have a 30 ft high *parikrama* (colonnade) bearing in mind the height of the Akshardham monument. From the observation of a portion set up at the workshop in Pindvada it was decided to have galleries and *samvarans* to crown it.

With reference to the strength of stone and the weight it would have to bear he made important changes in the width of the pillars and beams. He also suggested to add 68 more pillars in order to enhance the life of the structure.

From the deep studies and observations by Swamishri and the Akshardham team of sadhus, significant changes were introduced into the original monument design. Due to the inspiration and blessings of Swamishri the project's conceptual and creative development was shouldered by the sadhus. The research and responsibility for the monument's ornateness was also taken up by the sadhus and volunteers. Through the colossal efforts of the sadhus the once royal, palatial design of the monument was transformed into a breathtaking structure that would spiritually impress and inspire people for thousands of years. Thereafter the task of the *sompura* became easier and less burdensome. The details of the designs in stone were an outcome of Harshadbhai Chavda's years of experience and skill in stonework.

The concept and planning of Akshardham was a concerted effort by Swamishri, sadhus, volunteers and experts.

FOUNDATION FOR AKSHARDHAM (Solid foundation that would last for thousand years)

"We want to build Akshardham so that it survives for thousands of years." Pramukh Swami Maharaj echoed these words to the Akshardham team of sadhus, engineers and architects.

For thousands of years, the Yamuna had flowed on this land. Now, the challenge was to build a stone edifice that would stand for thousands of years on its soft bed.

The first step was the scientific analysis of the soil: the weight of structure that the soil can bear, seismic forces, sand liquefaction, soil density, ground water levels and other detailed scientific tests. For this, tests were performed at different levels, upto 100 ft below the surface. The soil bearing capacity was tested at India's leading laboratory.

Since New Delhi is located in Earthquake Zone 4 (meaning, there are high chances of large earthquakes), geologists advised for the need to incorporate earthquake-resistant features. With this in mind, Tehri Dam, Uttarkashi, Koyana Dam and others, that have had earthquakes, were studied. Also, the water table in this region is very high, such that water can be tapped at a mere 15 ft below ground level. These and many other factors were considered in consultations with leading experts in foundation technology.

Much thought was given to using concrete piles, geo-piers and other modern methods. But finally, it was decided to use the 'steel-less' method to prepare the foundations for the enormous 141 ft high, 316 ft wide and 356 ft long monument.

After scientific soil testing, the foundation of Swaminarayan Akshardham was made in accordance with an ancient Indian technique. Layers of sand, stone and geomatresses were laid to create a strong foundation.

The land was with large 10 ft deep craters. To fill up the craters and level the land would have required one year to accomplish. But it was accomplished in only six months by channeling the excess silt of river Yamuna.

An iron-free foundation with a 42 feet high plinth of 5 million bricks was built. The internal connections of the hexagonal shaped brick plinth added to its strength

With the possibility of earthquakes care was taken to build a robust foundation, by considering soil porosity and the seismic forces.

Thus, the technique used to build the foundations for the mandirs at Konark, Somnath, Dwarika, Jagannathpuri, Bhuvaneshwar, Mahabalipuram and other coastal mandirs was chosen.

For this, a leading foundation technologist and devotee, Shri Maheshbhai Desai, and structural consultant for the project, Shri B.V. Chaudhary, designed the foundation.

First, a 961,874 cu. ft (285 ft wide x 225 ft long x 15 ft deep) foundation pit was dug in which, using gabions of geomatrix and geofilters, 11 separate layers of sand and stones were laid. In the event of any earthquake or other geological movement, the fluidity of the foundation would enable it to absorb the disrupting waves of ground movement.

On top of these layers, a five-feet thick raft of plain cement concrete (P.C.C.) was laid. Then, numerous concrete cubicles, measuring 36 sq. m, were arranged like a jigsaw puzzle around the raft. Bearing in mind the expansion and contraction properties of concrete, a special concrete curing process was carried out. The special feature of this entire process was that not a single piece of metal was used.

On top of this foundation, the monument's main plinth was built using five million specially prepared and tested Meerut bricks. Normally, these bricks have a weight bearing capacity of 35 kg/cm², but here bricks with a weight bearing capacity of 100 kg/ cm² were rigorously tested and used.

In some sections, walls measuring 7.5 ft in thickness and 21.5 ft in height were built, while in other areas, the walls were 10.5 ft thick. Between these walls, the open spaces were filled with sand and round white stones and then the entire area was subjected to compacting.

On top of this, a one metre thick raft of P.C.C. was spread to further strengthen the plinth.

Also, experts estimated that it would take one year to prepare such a foundation, but this task was completed in just six months.

The Swaminarayan Akshardham site is 205 ft above sea-level and at first there were many areas which were 12 to 15 ft below ground. About ten acres were 10 to 12 ft below ground level. All these would have required thousands of trucks of soil to bring them up to ground level.

Fortunately, at that time, work was in progress to clean the river Yamuna. To clear excess soil from the Yamuna, the government had placed giant dredging machines on the river's banks. The soil extracted with the water was taken by pipeline to areas where bridges were being built. The same process was used to bring the soil to the Akshardham site. It would have taken over a year to bring the 400,000 cu. ft of soil necessary, but, using this technique; the task was completed in only six months.

In addition, 3500 concrete piles, 60 ft deep, were used to prepare the foundations for the exhibition halls, giant screen theatre, Premvati restaurant, visitors' centres and other buildings on the site. The concrete piles used in each building were joined together using a pile cap to form a special net.

In some places, the old bali (wooden) piling technique was also used.

Thus, the foundations of Swaminarayan Akshardham are such that they will truly stand the test of time.

URGENT

IS YOUR EMAIL ID & ADDRESS UPDATED IN DATABASE OF "isse.org.in ?

Inspite of repeated reminders, it is noted that many members have not updated their details in the database of our web site "isse.org.in".

Therefore when we sent emails by using the facility of "Batch Emailing", only 335 mails could be sent out. That means that only 335 members have communication channel with us and rest of the members cannot be contacted by us for providing them information about forthcoming events, issue of journals etc.

Similarly many journals are returned undelivered because of wrong / changed address

We earnestly want EVERYONE to have his email id and updated address registered in our database of "isse.org.in".

PLEASE GO TO OUR WEB SITE, ENTER YOUR NAME, DATE OF BIRTH, OPEN THE FACILITY OF "MODIFY MEMBER DETAILS", ENTER YOUR EMAIL ID AND ALSO UPDATE YOUR OTHER DETAILS, IF NECESSARY.

GOOD NEWS ABOUT OUR EARTHQUAKE BOOK WHICH HAS BECOME OUT OF PRINT.

Since its new edition is yet awaited, We decided to meet urgent demands by supplying nicely Xeroxed copies of the book and accordingly same are now available for Rs 600/- plus forwarding charges, outstation charges etc.

If you need any copy please contact our head office.

Be an ISSE Author

We invite our readers to contribute articles and be a part of a big family at ISSE.

In particular, we will appreciate receiving contributions on the following :

- Articles bearing on innovative design and construction.
- Articles dealing with challenging construction problems and how they were solved.

Authors of published articles will be felicitated at ISSE Annual Day Function.

EVALUATION OF DISTURBED AND UN DISTURBED CONCRETE CORES

(PROF G. B. Chaudhari.- Geotech. Consultant)

Green concrete consists of cement, sand, metal aggregates and water. When all these materials are mixed properly, placed properly and cured, then it gets hardened with time. About 28 days gains required to gain characteristic strength. Latter on also strength goes on gaining up to about 10 to20 percent more than characteristic strength within 3 to 6 months time period .This concrete has a heterogeneous grain structure arrangement in which some amount of micro cracks do exist . This structure of concrete is considered as the undisturbed state of concrete

When a concrete core is drilled through this an undisturbed state, some disturbances do occur to the structural arrangements of grains due to drilling operations etc. and more and more micro cracks can develop in the concrete of the drilled core .

A new concept of determining the degree of disturbance to the concrete grain structure was developed by me and the degree of disturbance, Dd, is defined as stated below-

Dd = <u>modulus of elasticity of undisturbed concrete = 5000vfck</u>

modulus of elasticity of disturbed
concrete core by
platen method(explained at the end)

=< 2.5 undisturbed concrete

>2.5<4 moderately disturbed concrete

>4 severely disturbed concrete

Using this concept, the compressive strengths of drilled concrete cores were certified. When ever Dd value was less than 2.5, then the compressive strength values were found satisfactory i.e. 28 days hardened concrete compressive strength was reflected in concrete cores. Whenever Dd values were greater than 2.5, then compressive strength values were found much less than the required values

Another parameter which needs attention is the ratio of minimum diameter of drilled core to maximum size of metal aggregate. In general, this ratio shall not be less than 4 to get an appropriate value of compressive strengths of drilled cores. For example, if maximum size of metal aggregate is 20 mm then, the minimum diameter of drilled concrete core should be about 80 mm. If maximum size of metal aggregate is 40 mm then, the minimum diameter of concrete core shall not be less than about 160 mm. In case, this ratio is less than 4, then many a times the compressive strength results were found less than the required values and this may be due to drilling operation disturbances and grain size effect. Platten method is explained below.

PLATTEN METHOD OF DETERMINIG MODULUS OF ELASTICITY OF HARDENED CONCRETE CORE AND ROCK CORE.

The concrete core is placed in UTM machine and axial deformation measuring dial gauge is positioned below UTM head as shown in Fig.1.

When reaction load is applied on concrete core , then axial deformation dial gauge measures total axial deformation over length Lt. Note the observations of loads on UTM machine at every 10 divisions of dial gauge (least count=0.01 mm), till the concrete core gets crushed. Then graph is plotted of stress versus total strain as shown in Fig,2

From Fig,2, first obtain maximum stress increment at about 50 % of crushing strength value and corresponding total strain value. Then, determine strain in plattens as stated below-

Strain in platten = maximum stress increment / E of steel (=2000000 kg/cm2)

Then, determine strain in concrete core = total strain at maximum stress increment - strain in platten

Now, modulus of elasticity of concrete core = maximum stress increment / strain in concrete core, in kg / cm 2

Platten method gives appropriate values of modulus of elasticity of harden concrete core and rock cores.

Congratulation !

Eng. Dilip Pradhan

and

Eng. Shantilal Jain

(Advisory trustee of ISSE)

were elected unanimously as Chairman and Honorary Secretary respectively, of IEI MSC for the session 2006-08 in the meeting of the Committee held at Nagar on 8th October, 2006.

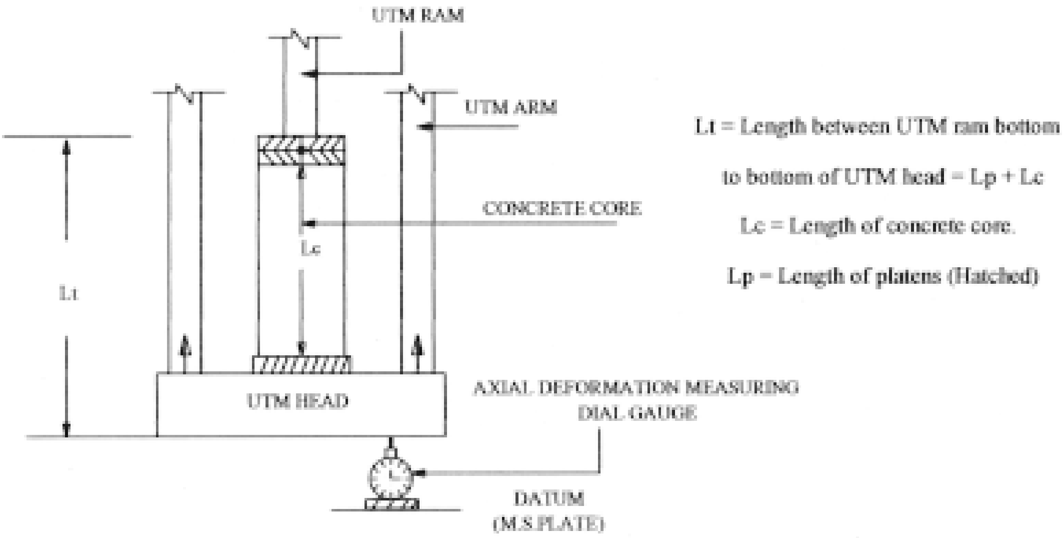


FIG. 1- SHOWS POSITION OF CORE & DIAL GAUGE.

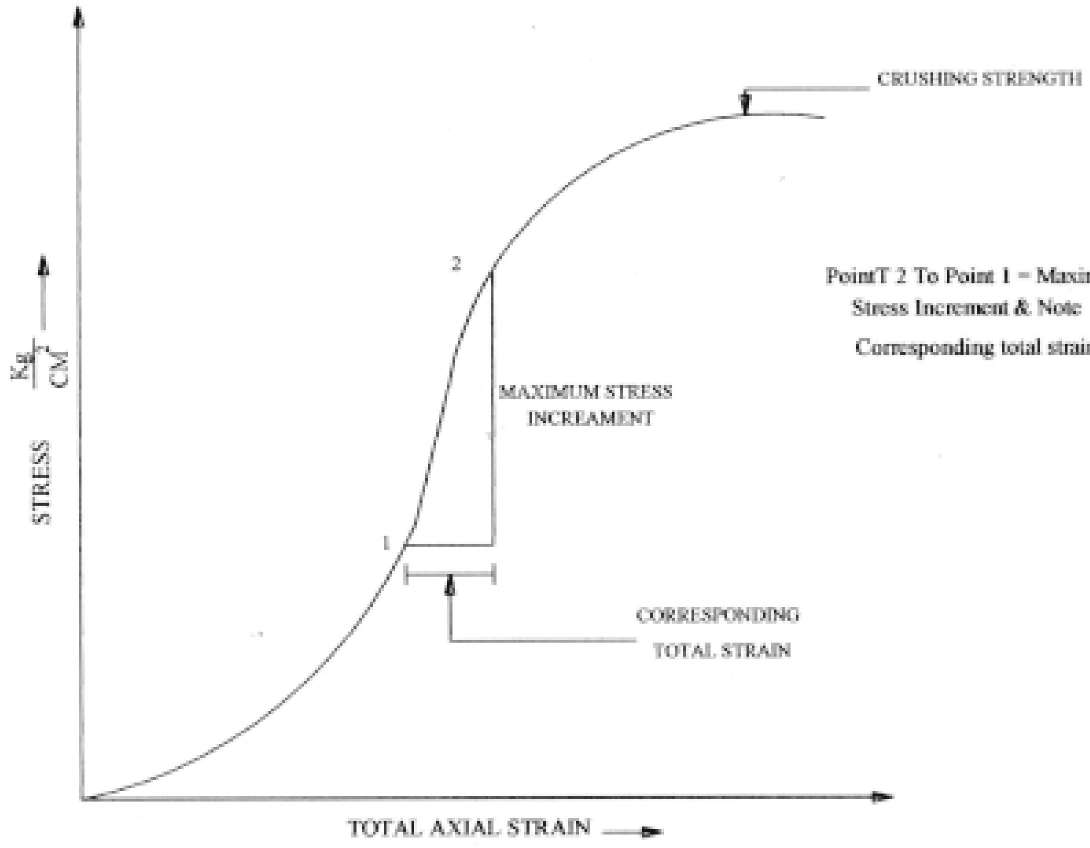


FIG. 2- SHOWS STRESS VERSUS TOTAL STRAIN GRAPH.

CRANE FOR ERECTION OF LARGE SPAN SHED ABOVE WATER

(Eng. A. B. Karnik - Consulting Engineer)

Shed admeasuring 41m wide x 140 m long x 30 m high was to be erected ABOVE WATER. This situation was to prevail all through its life time.

Main purpose of this shed with permanent Gantry Crane etc. was for maintaining the SHIPS AFLOAT.

This shed was designed by other Consultants with twin leg columns and built-up rafters together as Portals. One row of columns rested on Jetty and other row of columns rested on 3.80 m wide R.C.C. platform. This platform was supported on twin piles of 1000 mm dia. at 8 m c/c coinciding with column spacing. Clear width of platform available beyond outer column leg was only 500 mm.

Normally, erection of shed on land follows a sequence of erecting columns and maintaining them in plumb vertical position by means of temporary GUYS or MAST.

This methodology was just not possible here over water.

Conventional floating crane for use throughout the erection time of shed was prohibitively costly.

The author believes in "never say die approach", and therefore with some ingenuity, efforts were made to evolve workable solution.

Crane to be designed had difficult task to perform. The task was to erect a Shed of above dimensions TOTALLY OVER WATER.

All the structural steel members had to be floated over barge and assembled piece by piece in desired position.

Innovation was necessary ingredient in the design of tailor made Crane which efficiently erected above shed in record time.

The required crane was erected with floating crane and main shed erection by this crane itself.

Crane dimensions were decided such that main shed erection was an easy affair. Following logic gave best results.

a) Inside to inside of Crane Columns was 43 m to allow clearance of 1 m beyond outer face of Shed Columns.

b) Bottom chord of the crane member connecting side columns was made parallel to the top chord of the shed member with clearance of 500 mm.

c) Along the length, crane columns were two at 8 m c/c matching exactly with the shed column centre lines.

d) Base of the side columns was further extended by 8 m on either side of these two columns for comfortable stability of the crane while moving on wheels along length.

e) Vertical and top inclined members were designed as open web members with 2500 mm distance between chord members and side width of 2000 mm for rigidity against deflections/displacements. 2000 mm wide x 2500 box was intentionally cre-ated for workers to walk safely within on inclined members and attend to work on shed assembly which was just 500 mm below crane bottom chord.

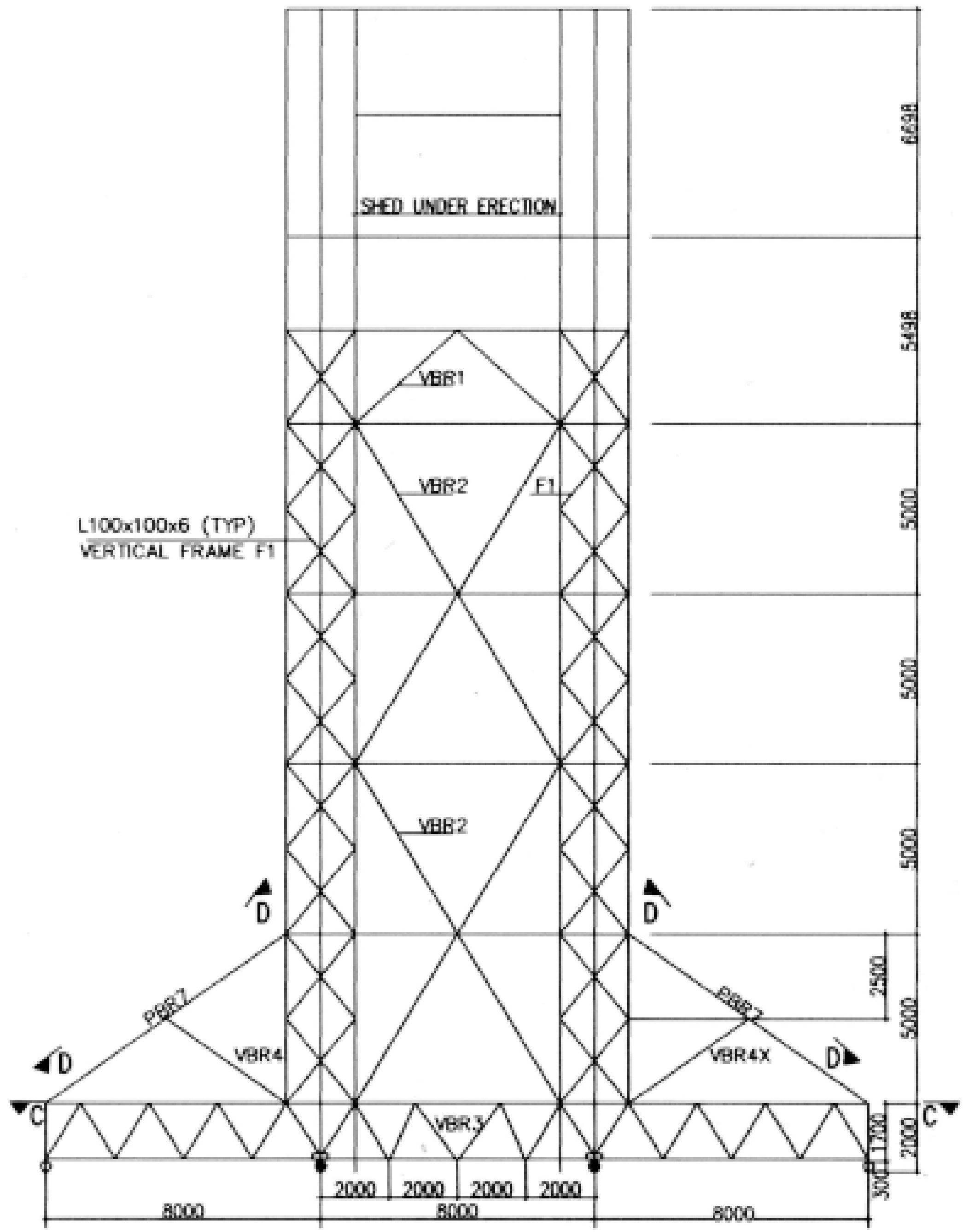
f) Steel sections used were MC 125, Angles 100 x 100 x 10 and down with total weight of Crane for above dimensions not exceeding 70 T.

g) Lifting tackles were hooked up to the bottom chords of the top inclined box members and the assembly of the main Shed was done with speed and great ease.

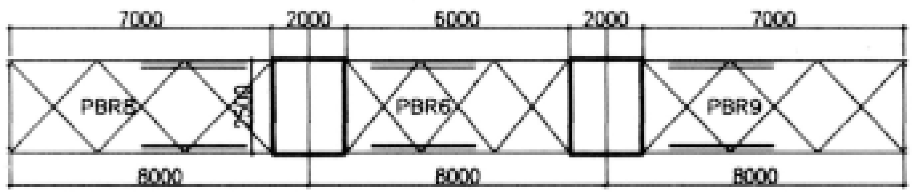
EDITOR'S APOLOGY

While printing this article in the last issue of this journal, by inadvertence, it was printed without its supporting sketches etc. We very much regret the serious lapse.

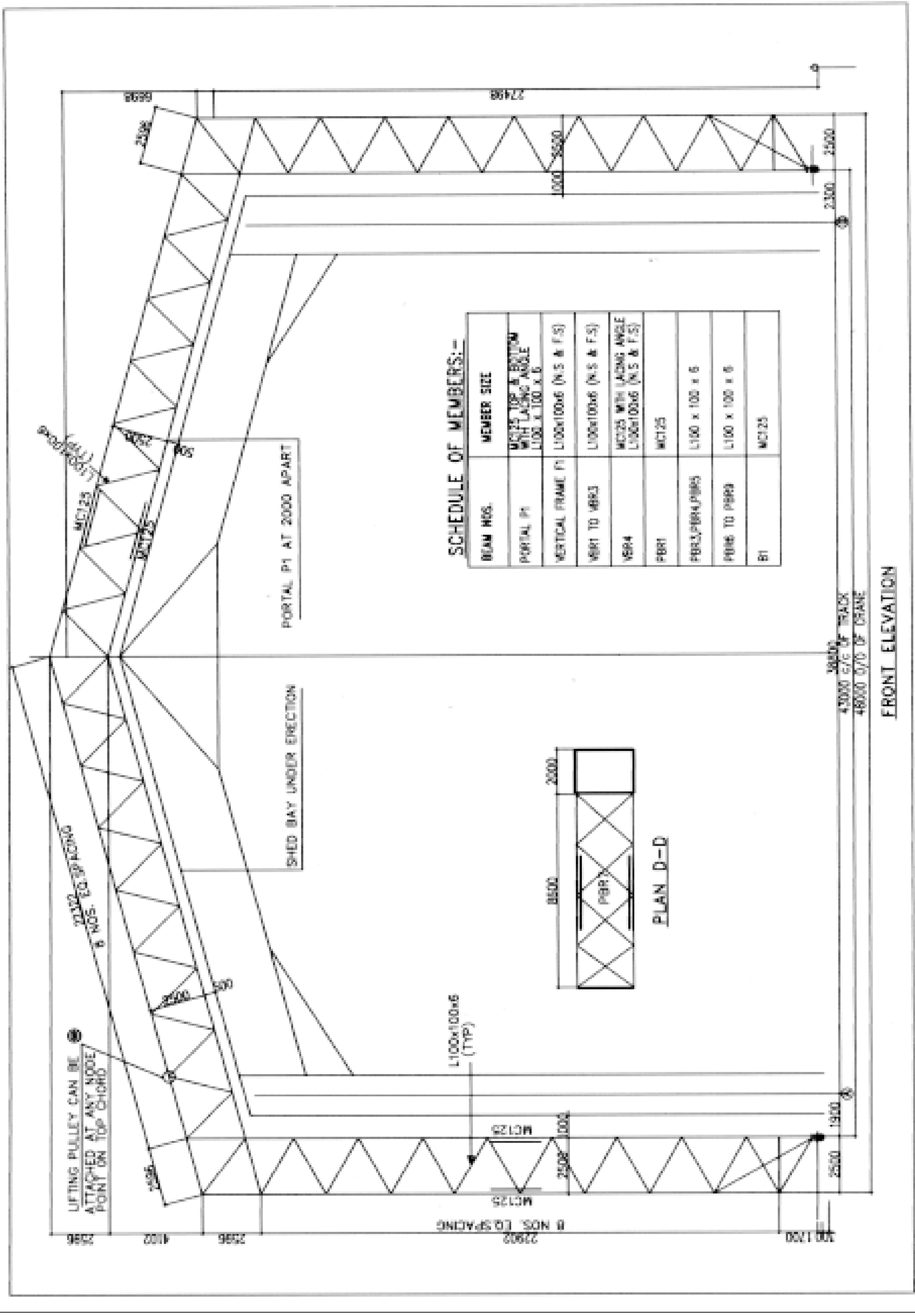
We are therefore reprinting this article, but this time with all supporting sketches etc. We certainly believe that Eng A.B. Karnik will pardon the lapse, and will continue contributing his articles as before.

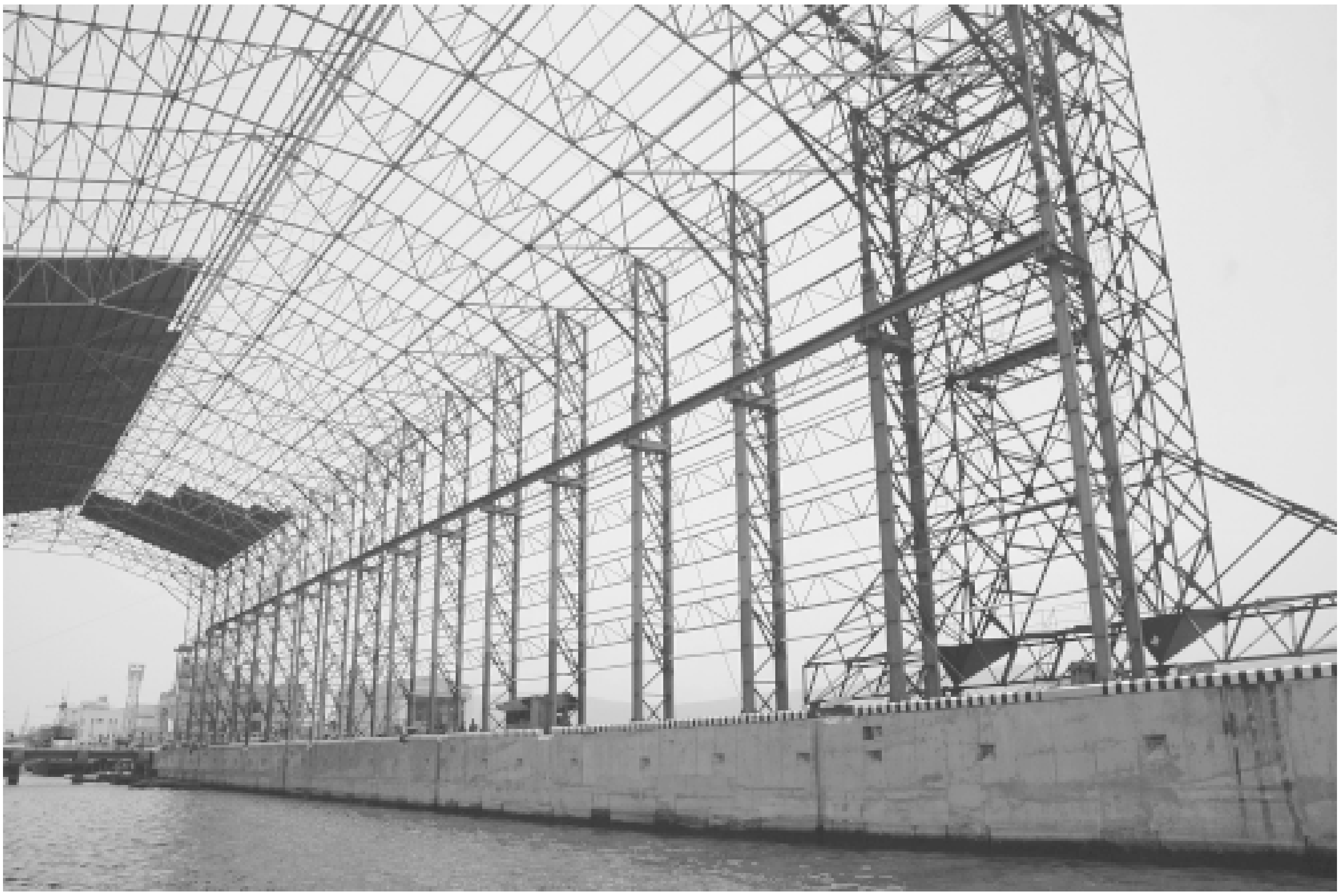


SIDE ELEVATION



PLAN C–C


SCHEDULE OF MEMBERS:-
BEAM NOS. | MEMBER SIZE
PORTAL P1 | MC125 TOP & BOTTOM WITH LACING ANGLE L100 x 100 x 6
VERTICAL FRAME F1 | L100x100x6 (N.S & F.S)
VBR1 TO VBR3 | L100x100x6 (N.S & F.S)
VBR4 | MC125 WITH LACING ANGLE L100x100x6 (N.S & F.S)
PBR1 | MC125
PBR3,PBR4,PBR5 | L100 x 100 x 6
PBR6 TO PBR9 | L100 x 100 x 6
B1 | MC125
PORTAL P1 AT 2000 APART
SHED BAY UNDER ERECTION
LIFTING PULLEY CAN BE ATTACHED AT ANY NODE POINT ON TOP CHORD
8 NOS. EQ.SPACING
L100x100x6 (TYP)
MC125
PLAN D-D
PBR1
43000 c/c OF TRACK
48000 O/O OF CRANE
FRONT ELEVATION



GUIDELINES FOR OPEN FOOTINGS ON ROCK

(Eng V.T. Ganpule – Geotechnical Consultant)

ABSTRACT:

With advent of excessive demand for Car parking many projects with double basement have come up in the city while many may be in the pipeline. As a result the raft /open footings resting on rock are deployed to transfer heavy column load of multistoried structure. It is observed that many structural consultants are tending to be over conservative while designing the foundation on rock as result of lack of adequate information about the methods of evaluation of strength of the rock masses from U.C.S. of intact cores and difficulties in estimation of settlement. The purpose of the article is to discuss the engineering and geological aspects which will enable the structural consultants to get a feel of the behavior of rock subjected to loads. The methods for estimating settlement shall be presented in a separate article.

INTRODUCTION:

Rock would seem to be the ultimate excellent reaction for engineering loads and often it is. But the term 'rock' includes a variety of types and conditions of material, some of which are surely not 'excellent' and some that are potentially dangerous. Moreover, the experience record of construction in rocks includes numerous examples of economic difficulties revolving around mistaken or apparently malevolent behaviour of rock foundations. Such cases have involved excavation-overbreak, deterioration of prepared surfaces, flooding or icing by groundwater seepage, accumulation of boulders from excavation, gullying or piping or erodible banks, and mistaken identity of materials in the weathered zone.

Foundations on rock can be classified into three groups spread footings, rock socketed piles and tension foundations. The basic geotechnical information required for the design of all three types of foundation consists of the structural geology, rock strength properties, and the ground water conditions. The open foundations resting on rock are dealt in the following paragraphs.

DESIGN CRITERIA :

Foundation in rock must satisfy the same design criteria as any other type of foundation; namely: adequate stability, tolerable deformation, and cost- effectiveness. The primary difference between foundation in soil and in rock is that rock masses can be exhibit wide variation and can surprise the designer who has not developed a thorough understanding of the site geology. This variability often makes rock mass characterization difficult because one must deal not only with the rock material, but also must be concerned with the discontinuities which are pervasive throughout a rock mass.

APPROACHES TO DESIGN :

In other than 'good'-quality rock masses, the geotechnical considerations will normally control the design, barring some construction limitations. The question then is how to approach the geotechnical design. Four different general approaches can be adopted: full-scale load test, building code criteria, empirical rules, and analytical methods. Full-scale foundation is constructed in accordance with the proposed construction techniques and is then tested, preferably to failure. Proper interpretation of these data should lead to a sound design. However, load tests are costly and are not warranted on most projects. Therefore, alternative approaches must be adopted for most designs.

One approach is to allowable stresses given in building codes. Little, if any, geotechnical information is required to design on this basis of codes, other than knowledge of the rock type. The nature and function of the structure, loading conditions, tolerable deformations etc. are not included. Therefore, building code criteria tend to be very conservative. A further problem is that foundation settlements will be unknown. Critical evaluations of these code values suggest that actual design values can be significantly larger. Depending on particulars, increases by factors of two to more than ten may be appropriate. However, for simple and lightly loaded structures on relatively good rock masses, it may be reasonable to use code values, primarily because they are sufficiently high that little economy may be realized by increasing the design values, given the structural limitations.

Empirical rules present one alternative. One of the more useful of method was suggested by Peck who presented an empirical correlation between the allowable contact stress and the RQD as shown in the graph. The recommendations by Prof. Peck are as under.

RQD > 90 % - no reduction.

90 % > RQD > 50 % reduce bearing pressure by factor of about 0.25 to 0.50.

RQD < 50 % - reduce bearing pressure by a factor of about 0.25 to 0.1and reduce

bearing pressure further if extensive clay seams present.

Although building codes and empirical rules provide some insight into the design process, neither addresses the problem directly outline procedures for computing both the bearing capacity and settlement. In the remainder of this paper, a general procedure is presented to focus directly on these factors.

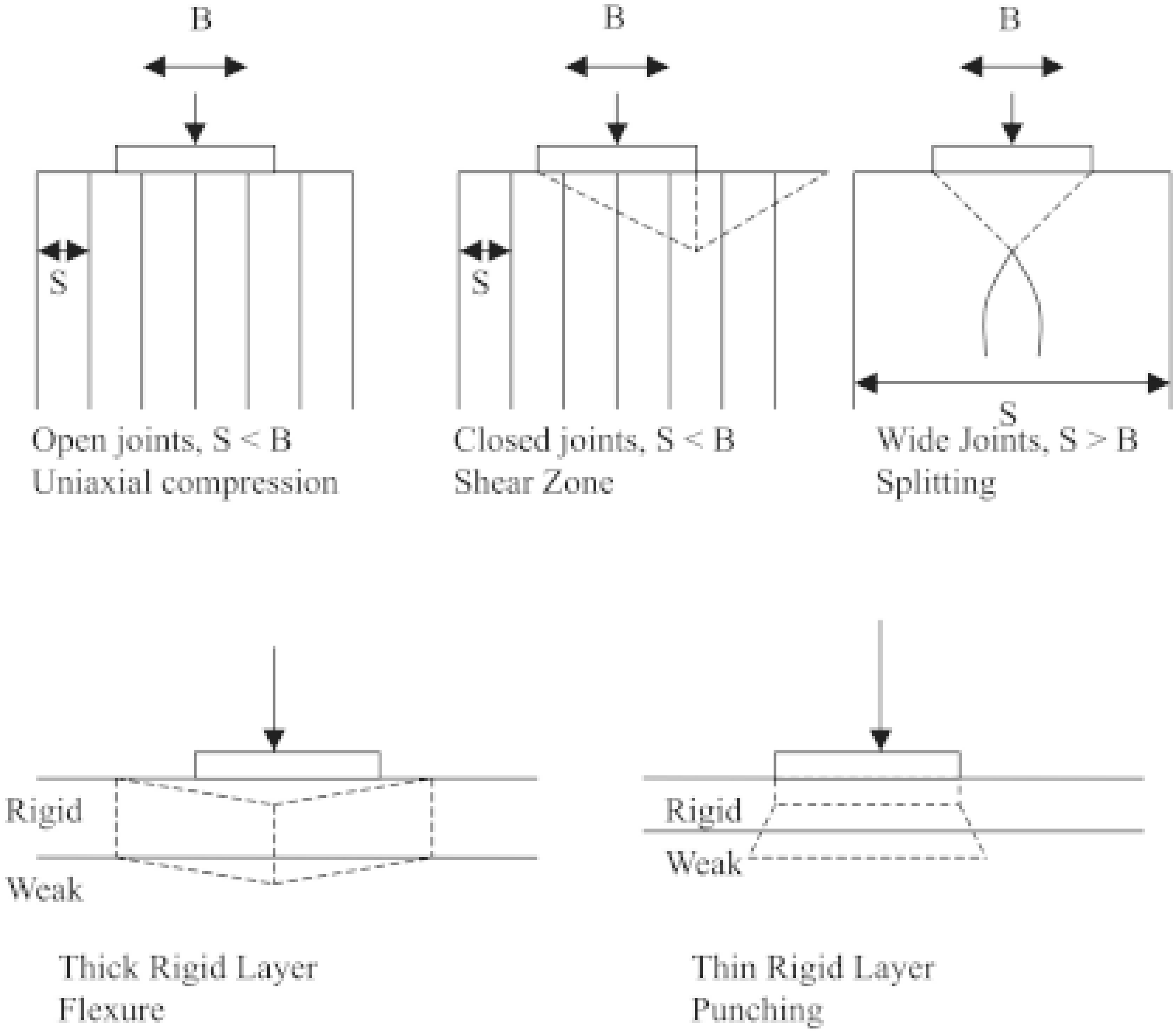
BEARING CAPACITY OF FOUNDATIONS ON ROCK :

The bearing capacity of the rock mass must be computed to assess the degree of safety of the foundation. Although settlement criteria often control the design, there are situations where the bearing capacity governs. Again, the rock mass must be idealized, based on bearing capacity.

BEARING CAPACITY MODELS :

Sowers has proposed several possible bearing capacity failure modes. Depending on the geologic data, one of these modes can be selected as being representative of the likely field behavour. For a thick rigid layer overlaying a weaker one, failure can be by flexure. The flexure strength is approximately twice the uniaxial tensile strength which is of the order of 5 – 10 % of the uniaxial compressive strength. For a thin rigid layer overlying a weaker one, failure can be by punching, which is effectively a tensile failure of the rock material. In both of these cases, failure in the underlying layer could occur first by one or the other, failure modes

For open vertical joints, failure would occur by uniaxial compression of the rock 'column'. In this case, the ultimate bearing capacity would be given Mohr-Coulimb theory as



Bearing capacity failure modes(Sowers 1979)

$$q_{ult} = q_u = 2c\tan(45+\phi/2) \qquad (1)$$

In which

c = cohesion

ϕ = friction angle.

All three are rock mass properties

For closed joints, a general wedge shear zone develops. The ultimate bearing capacity for this case is given by the Bell solution. (based on Mohr's envelopes)

$$q_{ult} = cN_c\zeta_c + \frac{B}{2}\gamma N_\gamma \zeta_\gamma + \gamma D N_q \zeta_q \qquad (2)$$

in which

B = foundation width

D = foundation depth

γ = effective unit weight of the rock mass

N_c, $N\gamma$ and N_q = bearing capacity factor

ζ_c, $\zeta\gamma$, and ζq = collective terms representing modification factors for shape, inclination, etc.

For a foundation on the rock surface, D = 0 and even if the open foundation is embedded, the $\gamma D N_q$ term normally is very small by comparison with the cN_c term. Similarly the $B\gamma N\gamma/2$ term is very small by comparison. For these reasons, a common and conservative design assumption is made as.

$$q_{ult} = cN_c \zeta_c \qquad \text{------------ (3)}$$

For the Bell solution, Nc is evaluated as

$$Nc = 2N\phi^{0.5} \ (N\phi + 1) \qquad \text{------------ (4)}$$

Where

$$N\phi = \tan^2 (45 + \phi /2) \qquad \text{------------ (5)}$$

For a rigid rock mass, vertical concentric load, horizontal ground surface, and D = 0, ζ_c is given by

$$\zeta_c = 1 + (B/L) (Nq/Nc) \qquad \text{------------ (6)}$$

as a function of the foundation width (B) to length (L) ratio and

$$\frac{N_q}{N_c} = \frac{N_\phi^{\ 3/2}}{2(N_\phi^{\ 2} + 1)} \qquad \text{------------ (7)}$$

For widely spaced joints, failure occurs by splitting beneath the foundations, Equation is multiplied by 0.85. as proposed by Goodman who made a somewhat more conservative assumption and considered the case of no stress transmitted across the vertical discontinuity adopted by Bell , the bearing capacity factor is

$$N_c = \frac{2 N_\phi^{\ 0.5}}{N_\phi - 1} \left[N_\phi \left[\frac{S}{B} \right]^{\left(1 - \frac{1}{N_\phi}\right)} - 1 \right]$$

However such situation of joints wider than the width of footing is not likely to be met while designing foundations for a tall structure in Mumbai hence is not dealt in detail.

STRENGTH PARAMETERS :

When evaluating the bearing capacity, it is important that rock mass strength parameters be used. In much the same way that the compressibility or modulus of the rock mass is lower than that of the intact rock, the strength is also lower. However, because of discontinuity interlocking and roughness, the reduction for strength is not **as great as that for the modulus**. Heuze has presented a compilation of some of the available data on uniaxial compressive strength reduction as sample size increases. The data show that the uniaxial compressive strength for the rock mass ranges from about 25 % to upwards of 100 % of that for the rock material.

The rock mass is the in situ fractured rock which will almost always have significantly lower strength than the intact rock because the fractures divide the rock mass into blocks. The strength of the rock mass will depend on such factors as the shear strength of the surfaces of the blocks, their continuous length, and their alignment relative to the load direction. Further more, if the loads are great enough to extend fractures and break intact rock or if the rock mass can dilate resulting in loss of interlock between the blocks, then the rock mass strength may be diminished significantly from that of the insitu rock. Foundations located in fractured rock which are designed using the strength values of intact samples tested in the laboratory are likely to be significantly under-designed.

Other conditions that may be encountered are foundations containing potentially unstable blocks, formed by single or intersecting fractures that may slide from the foundation. In these circumstances, the shear strength parameters of the fractures themselves must be used in design rather than the shear strength of the rock mass. This shows the importance of carrying out careful geological mapping to identify such critical geological features and ensure that the strength testing programme is appropriate for the likely mode of failure.

From the above it can be observed that the properties of the rock mass depend on its structural geology. The design therefore shall include a thorough examination of the structural geology of the site. Even the strongest rock may contain potentially unstable blocks. It is therefore suggested to map the site from geological point of view. The geological mapping is invariably done for dams. Bur other structures like bridges and high rise buildings on piles except investigation data no other geological information is available.

The designer dealing with high rise buildings resting on rock can only map the site after complete excavation while has to make the design with the help of soil investigation data normally available. The bore-log generally gives recovery and Rock quality designation of the formation depth wise. The laboratory tests on rock cores include U.C.S., water absorption, porosity, and density. With the help of the above data, designer has to design his foundation system. The four methods which enable to get properly of rock mass are given below.

1) Author's Method :-

The term recovery represents the length of solid rock cores recovered per metre run.

The term RQD represents the total length of pieces having length of each pieces more than 10 cm per metre run.

RQD of the rock represents joints and cracks in X and Z direction, the mechanical joint i.e piece length or a break in the length of rock due to mechanical efforts or recovering the core. The unconfined compressive strength test is conducted on a core which is more than 10 cm in length as that such in reality it represents the properties of the best portion of rock. Using these three parameters, the author has proposed a simple relation to get rock mass properties using the data available from investigation. The characteristics of the rock mass at particular depth or in a range is represented by following relation

Characteristics strength = U.C.S. of intact rock x average of

$$\frac{\text{Recovery \%} + \text{RQD \%}}{2}$$

The use of relation in assuming bearing capacity is explained in the later paragraphs.

2) <u>Kulhaway and Goodman method</u> :-

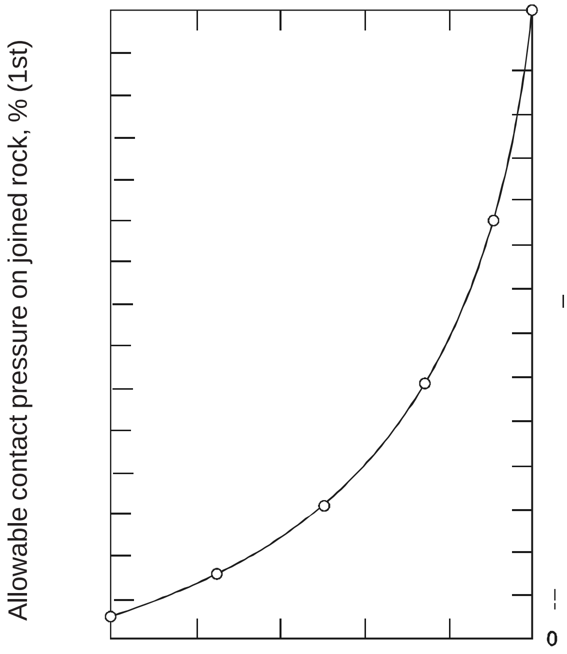
Kulhaway and Goodman have proposed following table for evaluating the mass properties from the field data. However they have taken into consideration RQD only.

Suggested design values of rock mass - (Kulhaway – Goodman) Table -1

RQD (%)	Rock Mass Properties		
	Uniaxial compressive strength	**Cohesion**	**Angle of friction**
0 - 70	0.33 q_u	0.1 q_u	30^0
70 - 100	0.33 -0.8 q_u	0.1 q_u	30-60^0

3) Peck's Method

The recommendations by Prof. Peck are stated earlier for very good rock. The same can be used however for other type o either method proposed by undersigned or Kulhaway may be used.



Allowable contact pressure on joined rock (based on Peck et al1). If $q_a > q_u$ (uniaxial compressive strength), use qu instead of q_a. If RQD is fairly uniform, use average RQD within depth (D) = foundation width (B). If RQD within D = B/4 is lower use lower RQD for design or remove this zone.

4) Hoek –Brown Method (based on geological mapping)

The strength criterion for fractured rock has been developed by Hoek (1983) and Brown (1988) which can be applied to the design of foundation. It is an important criteria developed by trial and error and based ob observed behavour of rock masses , model studies to failure mechanism and tiaxial compression test of fractured rock Hoek expression is given below.

$$q_{ult} = \left[s^{0.5} + (ms^{0.5} + s)^{0.5}\right] q_u$$

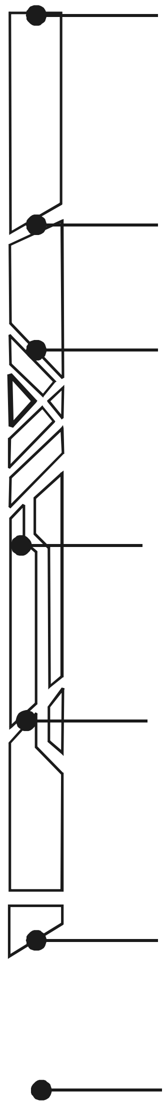
The method proposed by Hoek and Goodman necessitate the detailed geology mapping which may not be available for professional dealing with building foundation. Incase of building with basement, deep basement and foundation resting on roc, such mapping can be done by dividing entire plinth area with exposed rock into a grid of 3 m x 3 m grid but only after the excavation upto desired depth. The method proposed by Hoek and Brown is conservative. It is very useful incase of dam foundation and tunnels where stress distribution is nonuniform over the bearing surface as well as in foundation located on the crest of stiff slope or building foundation the S.B.C. obtained by Hoek and Brown method on excavating for the basement may be taken as a lower bound.

Table Value of s and m based on Rock classification (Brown and Hoek)

Quality of Rock Mass	Joint Description and spacing	s	Value of m as Function of Rock Type (A-E) From Table B.3				
			A	**B**	**C**	**D**	**E**
Excellent	Intact (closed) Spacing > 3 m (10 ft.)	1	7	10	15	17	25
Very good	Interlocking Spacing of 1 to 3 m (3 to 10 ft)	0.1	3.5	5	7.5	8.5	12.5
Good	Slightly Weathered Spacing of 1 to 3 m (3 to 10ft)	4 X 10^{-2}	0.7	1	1.5	1.7	2.5
Fair	Moderately Weathered Spacing of 0.3 to 1 m (1 to 3 ft.)	10^{-4}	0.14	0.2	0.3	0.34	0.5
Poor	Weathered with Gouge (soft material) Spacing of 30 to 300 mm (1 in to 1 ft.)	10^{-5}	0.04	0.05	0.08	0.09	0.13
Very Poor	Heavily Weathered Spacing of less than 50 mm (2 in)	0	0.007	0.01	0.015	0.017	0.025

(Where S and m represents rock mass properties to be taken from table)

The rock mass properties S and m are crudely equivalent to c (cohesion) and N while the expression in bracket corresponds to 'Nc'.



Total length of core run =200 cms

$$\frac{\Sigma \text{ Length of core pieces} > 10\text{cm length}}{\text{Total length of core run}} \times 100$$

$$\left(\frac{38 + 17 + 20 + 35}{}\right) \text{X } 100 = 55\%$$

What are we calling a rock ?

Grade	Description	Lithology	Excavation	Foundations
VI	Soil	Some organic content, no original structure	May need to save and re-use	Unsuitable
V	Completely weathered	Decomposed soil, some remnant structure	Scrape	Assess by soil testing
IV	Highly weathered	Partly changed to soil, soil > rock	Scrape NB corestones	Variable and unreliable
III	Moderately weathered	Partly changes to soil, rock > soil	Rip	Good for most small structures
II	Slightly weathered	Increased fractures and mineral staining	Blast	Good for anything except large dams
I	Fresh rock	Clean rock	Blast	Sound

Engineering classification of weathered rock

Table :- Description of Rock Types for use of Table – Hoek

Rock Type	Description
A	Carbonate rocks with well-developed crystal cleavage (dolostone limestone, marble)
B	Lithified argillaceous rocks (mudstone, siltstone, shale,slate)
C	Arenaceous rocks (sandstone, quartzite)
D	Fine-grained igneous rocks (andesite,dolerite,diabase,rhyolite)
E	Coarse –grained igneous and metamorphic cry

However the S.B.C. values obtained are conservative and represent lower bound. For the categories Excellent, very good and good (refer table enclosed)the method gives reasonably realistic values. For the other three classes the values are very conservative and S.B.C. may be evaluated using 'N' values or on the principles of soil mechanics.

The geological mapping of the area shall be done on dividing the area into a grid of 3 M. The geological compass shall give strike and dip direction. Similarly the orientation extent depth and roughness of cracks can be noted and plotted. The table can then be referred for estimating the lower bound S.B.C.

Example :-

The following example shows how to work out S.B.C. on resorting to Author's method and Prof. Kullhaway-Goodman method.

The S.B.C. by both methods shall be worked out and selecting the lesser of the two the foundation system can be designed

On excavating the entire area the geological mapping shall be done to get S.B.C from Hoek and Brown's method. It serves as a lower bound.

Rock type basalt (weathered) U.C.S. = 100 kg/cm^2

Recovery = 60 %

RQD = 15 %

1) From the table suggested by Kulhaway and Goodman the value of c and q_u can be worked out on knowing U.C.S.. Similarly the 'ϕ' value to be taken in evaluating N_c and N_q can be assessed. The values of N_c and N_q for $\phi = 30^0$ are worked out below.

$N_c = 2\, N_q^{0.5}\,(N_q + 1)$

Where $N\phi = \tan^2 (45 + \phi /_2)$

$= \tan^2 (60)$

$= 3$

$\therefore N_c = 2 \times (3)^{1/2} \times (3+1)$

$= 8 \times 1.7 = 13.8$

As regards N_q

$$N_q = N_c \times \frac{N_\phi^{\,3/2}}{2(N_\phi^{\,2}+1)}$$

$$N_q = 13.8 \times \frac{3^{3/2}}{2(3^2+1)} = 3.5$$

But for the cases foundations placed on rock surface the N_q is neglected

Example - I

Now suppose U.C.S.= 100 kg/cm^2 and RQD 50 %

$\therefore$ q_{mass} = 33 kg /cm^2

c = 10 kg/cm^2 and $\phi = 30^0$

$\therefore \underline{N_c = 13.8}$

$\therefore$ U.B.C. = 13.8 x 10 = 138 kg/cm^2

Adopting a factor of safety = 3

S.B.C. = 46 kg/cm^2 = 460 t/m^2

But q_{mass} = 33 kg/cm^2 (show strength of rock)

$\therefore$ cohesion = 16.6 kg/cm^2

$$\therefore \text{S.B.C.} = \frac{16.6 \times 5.7}{3}$$

= 31.54 = 315.4 T/m^2

Author's Method :-

Characteristics U.C.S(value for design).

$$= \left[\frac{\%\,\text{recovery} + \%\,\text{RQD}}{2}\right] \times \text{U.C.S. (Lab)}$$

The lowest U.C.S. in the zone of significance shall be taken for the purpose of evaluating the characteristic strength of rock mass.

$$\text{S.B.C.} = \frac{cN_c}{\text{Factor of safety}} \qquad \text{Where } c = \frac{\text{U.C.S.}}{2}$$

and $N_c = 5.7$

Example U.C.S. = 100 kg/cm^2 Recovery 60 % , RQD 15 %

$$\text{Characteristics U.C.S.} = \left[\frac{60\% + 15\%}{2}\right] \times 100$$

$$= 37 \text{ kg/cm}^2$$

$$\therefore \text{ S.B.C.} = \frac{37}{2} \times \frac{5.7}{3} = 35 \text{ kg/cm}^2 = 350 \text{ T/m}^2$$

(using f.s. = 3)

= 263 t/m^2 (using f.s. = 4)

The factor of safety 3 is proposed for rock masses having average (RQD + Recovery) more than 60 % and 4 for less than 60 % .

SUGGESTED PROCEDURE FOR DESIGN :

1) The initial design may done adopting lower of the two values as obtained from Kulhaway –Goddman's method and 'Authors'method.

2) Later when the profile is exposed the geological mapping as explained earlier can be done.

3) From the table proposed by Hoek and Brown the safe bearing capacity can be worked out.

4) The two value obtained BY Hoek and Brown's method thus represent the lower bound.

5) For weak rocks, Smid hammer test at few places on the exposed area can give better idea about the rock mass strength.

6) The S.B.C. as per I.R.C. recommendation for soft rock is between 100 T/m^2 to 200 T/m^2. It may please be remembered that I.R.C. recommendations are for bridges where the spans will be large (more than 20 metres) and as such the pressure bulbs for each pier footing are not fouling with any other pier. Incase of building foundations the span may not be this large and pressure bulbs may faul.

7) Finally the minimum S.B.C. obtained from semi analytical methods may be compared with I.RC. recommendations and lesser of the two may be adopted. In reality such adoption is not so rational but to be consistent with the IRC code the S.B.C. reduction may be made.

ISSE Seminar on Foundations for Highrise Buildings (Report)

Indian Society of Structural Engineers had organized one day seminar on “Foundations for High-rise Buildings” at the Institution of Engineers Auditorium on 23 Spet.2006. It was well attended by more than 225 delegates. Delegates from outside Mumbai as far away from Rajkot attended the seminar. **The proceedings of this seminar (hard copies) are available from ISSE office for Rs 150/- per copy.**

The first speaker was **Dr. S. Y. Mhaiskar** who is well known for his contribution to the Indian Geotechnical Society & is currently the Principal of S. P. College of Engineering, Mumbai. His topic was “ Geotechnical investigations for Raft and Pile foundations”. He discussed the purpose and scope of investigations which include type and depth of foundation, data to determine allowable load / pressure, ground water location, data to predict lateral pressures and uplift forces, identify and resolve construction problems. He talked about the design data for rigid and flexible raft foundations. The main parameter “modulus of subgrade reaction “ (K) is required by the design engineers. Suggested values of K for different soil types were presented. He showed the pressure distribution below the rafts for different type of strata.

He then talked about calculation of the pile capacities for carrying vertical loads, horizontal load and uplift forces. He emphasised that the depth of investigation should be below the zone of influence to obtain reliable data. Core drilling should be done using triple core barrel. The expenditure for such investigations is a fraction of the total cost of the project. No efforts should be spared to obtain reliable parameters which can be used in design.

The second speaker was **Dr. N. Y. Nayak**, whose book “Foundation design manual 5th Ed” is referred by the engineers. He emphasized the need for adequate soil investigations under the guidance of expert geotechnical engineer. Adequate number of bore holes, up to zone of influence should be taken. Essential field testing like SPT, collection of UD samples, core recovery & RQD, field permeability test, collection of soil and water samples for chemical analysis should be carried out. He cautioned that CR/RQD results are affected by various factors like type of machine used for boring, driller experience, single /double/ triple core barrel used, type of core catcher used, logging person’s experience. So, the results should be carefully interpreted. He then talked about the selection of foundation type like shallow and deep foundations. Shallow foundations can be isolated foundations or raft foundation depending on the permissible bearing pressure. Deep foundations can be pile foundation, well foundation and pile raft foundations. He discussed design criteria based on shear strength and settlement considerations. He also informed about the revised draft of IS2911 Par IV in which permissible total settlement at 1.5 times design load is linked to pile diameter. He gave some tips for good engineering practices for pile foundations -

1. Cleaning of bore hole bottom is very important, otherwise soft toe will lead to excessive settlement
2. Avoid L bends to pile reinforcement cage which results in poor concreting at toe.
3. Proper lowering of long reinforcement cage, otherwise it results in side scrapping muck falling into borehole.
4. Proper sealing of tremie joints is essential for good concreting
5. Chipping of concrete above the pile cut-off, should be avoided which damages the pile. The concrete can be removed when it is green
6. Proper cover blocks should be provided
7. Blended cement is preferable and water cement ratio should not be more than 0.4
8. It is desirable to use 40mm down aggregate which results in saving in cement, but the tremie pipe of 250mm should be used instead of 200mm.

The next speaker **Mr. Jaydeep Wagh** discussed some case studies.

He discussed one case study of “pressure reducing piles”. Weathered bed rock was encountered at shallow depths with SBC of 45 t/m2 whereas minimum allowable pressure of 80 t/m2 was required. Hard basalt was available at 20m depth. Detail FEM analysis was carried out with various combinations of pile arrangement using the concept of settlement, thereby reducing number of piles for the project. The aim was to reduce the pressure beneath the raft to less than 65 t/m2. The analysis results indicated 80% reduction in number of piles compared to full pile foundation.

Second case study was discussed about the geotechnical investigations for the proposed tallest building in India. Sufficient bore holes were taken up to 30m in the bedrock under supervision of geotechnical consultant. Only hydraulic rigs were utilized for the investigations. Double and triple tube core barrel was used. Observation wells were installed to monitor ground water fluctuations. Pressure meter tests were conducted in the boreholes. The detailed investigation helped identify a 4m thick inter-bedded layer of hard basalt bedrock. The slope in the bedrock was also identified. Bored cast in situ piles were finalised terminated on hard basalt with allowable bearing pressure on each pile of the order of 1000 t/m2. Integrity test on all piles and routine load tests for 2% piles were to be carried out.

The post lunch session began with the discussion of “Analytical modelling for foundation systems”. The speaker was **Mr. Hemant Vadalkar**. He is an expert in various structural analysis software packages. He presented the results with the help of the software along with simple hand calculations. His lecture was laced with humour and he kept the audience awake after the heavy lunch. He complemented his wife for adding floral background to all the slides to make them more attractive.

He talked about analytical models for -

1. Analysis of piles subjected to lateral loads
2. Analytical model of pile caps
3. Analytical model for raft foundation.

He explained the use of soil spring supports based on the value of sub-grade modulus. The soil spring supports should be compression only support. That means they are active only under compression. For this condition, iterative analysis is required to check the tension in soil spring. The spring going in tension is to be removed in the next iteration. Iteration should continue till all the springs are in compression. A load case having horizontal loads along with dead loads should be considered for iterative analysis.

For analysing pile caps , piles can be modelled as springs which can carry compression and tension. Various pile cap models were discussed using simple beam elements and plate elements. If pile supports are defined as pinned, the pile cap analysis will not be correct as the vertical load distribution will not be based on the spring stiffness. This point was highlighted by a sample example.

Various analytical models of strip foundation and raft foundation were discussed for various values of sub-grade modulus K for soft, medium and hard strata. This was done to study the pressure distribution below the raft for various soil conditions. Compression only spring support should be used to simulate soil support which will take into account the loss of contact and negative spring reactions are avoided. It can be observed from the results that for the hard strata with higher values of K, the pressure is much higher below the point of application of column loads. It was shown through FEM analysis that Bending moments in the raft supported on hard strata are less compared to those on soft strata, for similar loading.

Comparison of finite grid and finite element method was presented. Using finite grid method, bending moment diagrams can be viewed where as in finite element method, stress contours are obtained. It was emphasised that results of finite element analysis needs to be interpreted cautiously and correctly.

Ms. Alpa Sheth from VMS consultants was the next speaker. She discussed about the optimization of thickness of mat foundations. She talked about the parameters in design of mat foundation, settlement , sub-grade modulus etc. She discussed various methods like rigid plate method and flexible plate method, finite difference method, finite grid method and sophisticated Finite Element method. Various types of mat foundations were discussed. She emphasized that since it is very difficult to predict the sub-grade response accurately, we have to carry out analysis that is approximate. She explained the detailing of the raft. She also discussed the case study of the actual project executed with very odd shape of raft due to site constraints.

Iin the post tea session, first speaker was **Mr. S. Ray**. He is the Vice-President of Simplex Infrastructure Ltd. He discussed about the pile foundation for one of the high rise building being constructed at Prabhadevi, Mumbai. Detailed soil investigation was necessary to avoid surprises at the later date. Bored cast in situ Piles of 1000 and 1200mm diameter were used in the project. He explained the problems faced at site during piling. The pervious layer of sand gravel mixture at a depth of 9m was missing in the soil report. He stressed that carrying out trial bores close to soil investigation boreholes is essential in every project for reconfirmation of soil strata as pile bores are of much larger in diameter than soil investigating bores and can give better identification. Trial bores were taken to determine construction methodology and to study the problem. It was observed that the bore hole was collapsing due to sand flowing in due to water jetting through highly pervious layer. Finally, it was decided to provide permanent steel liner up to the rock. A correlation of the penetration of the tool with shear parameters of various strata helped in deciding the termination of piles in weathered rock.

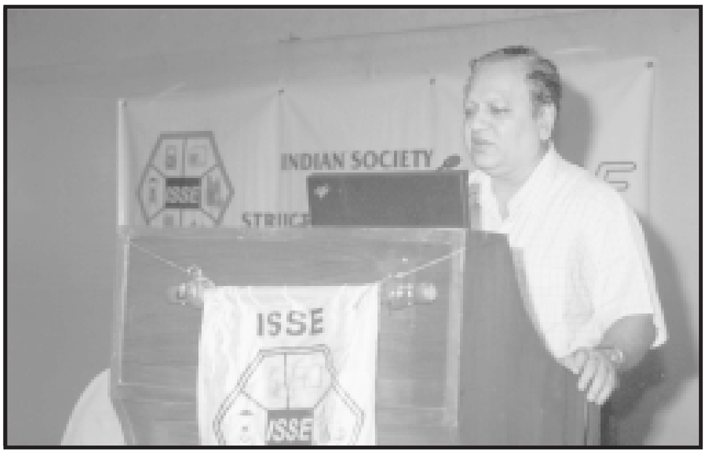
The last speaker of the day was **Mr. Kamal Hadkar**. He presented the information about some of projects executed by him.

He discussed about type of foundations adopted for various high rise buildings of 35 to 50 storey heights. This includes pile foundation, raft foundation, piled raft foundation. He elaborated on specifications for laying large volumes of concrete in the massive foundations. He touched upon the issue of heat of hydration and precautions to avoid thermal cracking inside the concrete mass. Concrete temperature at the time of poring should be kept as low as possible 25- 27 degrees by using chilled water, cooling aggregates, using fly ash. Slump should be between 100-125mm. The temperature should be monitored for the next 72 hours by using thermocouples placed inside the body of concrete. The interesting part was that to avoid the thermal cracking, the differential temperature between the core and outside surface should not be more than 20 degree centigrade. The core temperature rises to 70 degrees during the 72 hours after concreting. Therefore to avoid large temperature differential, the raft surface needs to be covered with insulation material like thermacole sheets. He showed recorded results of temperature of the raft core and surface concrete for one of the projects.

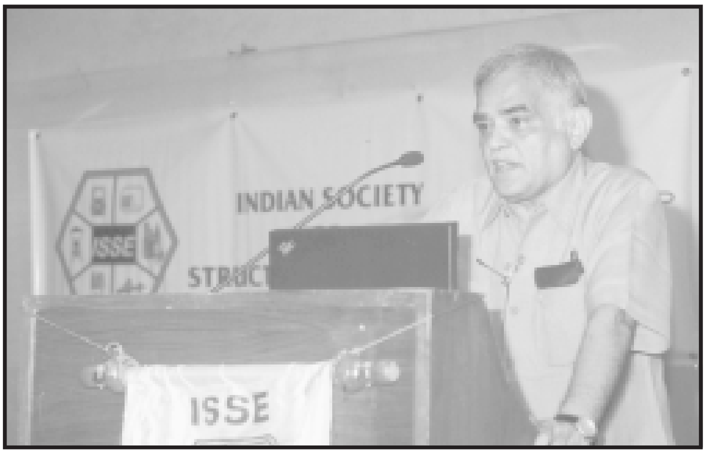
This was very interesting and new for many delegates. Generally, this kind of temperature monitoring is not usually done which is very important for durability consideration. He also talked about the location of construction joints and planning of concrete pour with minimum number joints.

He discussed the foundation systems for Dubai Marina tower, Planet Godrej at Mahalaxmi (Piled raft foundation) , Beaumonde tower at Prabhadevi (Piled raft foundation), Seasons Hotel, Worli (Raft foundation) and ITC Grand Central project (Raft Foundation). He projected the photographs of one of the projects in Dubai showing un-imaginable very neat and clean construction project site with excellent quality of construction. Almost all the delegates remained present till the end, up to 7:00 PM. This shows how interesting the presentations must have been.

Mr. Shantilal Jain proposed the vote of thanks. **Mrs Kirty Vadalkar** conducted the day's proceedings. Overall, it was an informative event & well attended, too. We the ISSE team look forward to more such events.

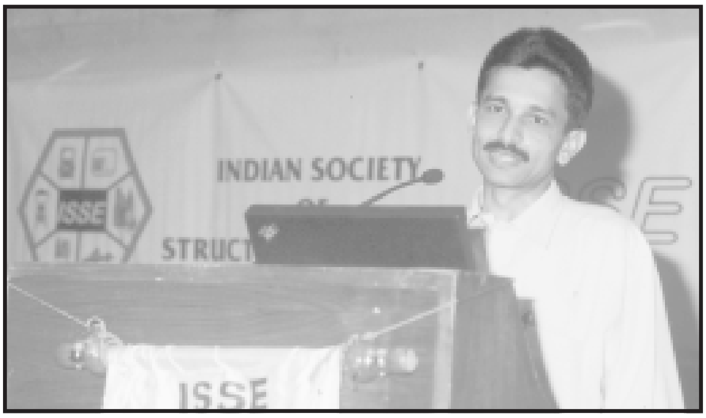


At Mike : Dr. S. Y. Mhaiskar



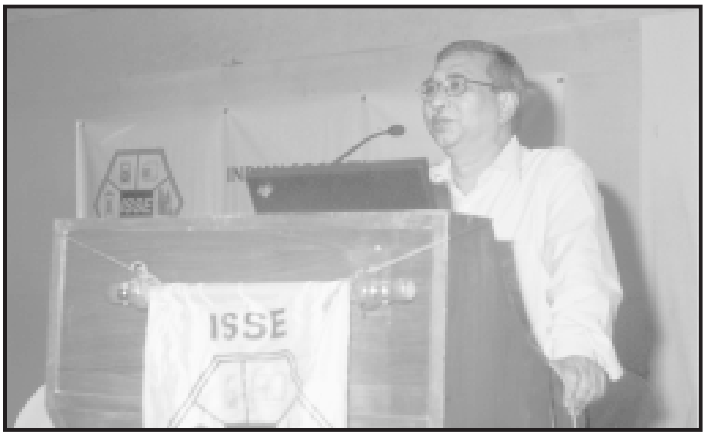
At Mike : Dr. N. Y. Nayak

At Mike : Mr. Jaydeep Wagh



At Mike : Eng. Hemant Vadalkar

At Mike : Ms. Alpa Sheth



At Mike : Eng. S. K. Ray

At Mike : Eng. Kamal Hadkar