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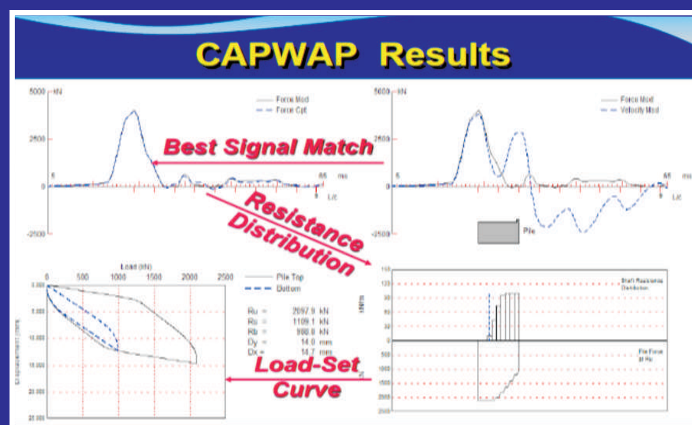

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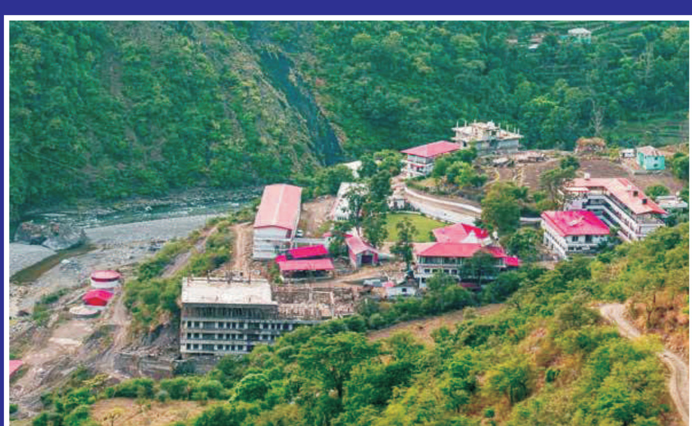

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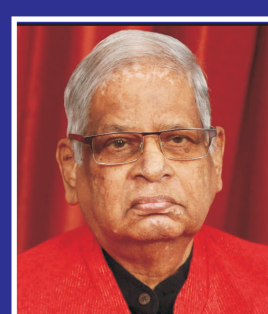

SHIVAJI G. PATIL

(Janaury 01, 1942- November 20, 2022)

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Correspondence Address : C/O, Maansi Nandgaonkar, 101, Sunflower, Sakharam Keer Road, Shivaji Park, Mahim, Mumbai - 400016

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Editor : Hemant Vadalkar

AIMS & OBJECTIVE OF ISSE

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 organizations 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.

FIELD OF INTEREST

- 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

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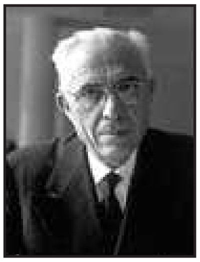
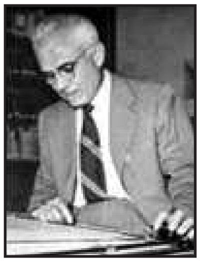
2251 Mushahid Gulamkadar Shaikh
JM 70 Bhagyashree Prakash Girme

Patrons : 38
Members : 2251
Student Members : 370
Organisation Members : 35
Junior Members : 70
Sponsor : 8
IM : 05

TOTAL STRENGTH : 2,777

GEM 34 PROF. PIER LUIGI NERVI-EXTRAORDINARY STRUCTURAL ENGINEER AND ARCHITECT

Dr. N. Subramanian, Ph.D., FNAE

Prof. Pier Luigi Nervi (June 21, 1891 - January 9, 1979)

Pier Luigi Nervi, Italian architect and structural engineer, was internationally renowned for his technical ingenuity and aesthetic sense of design, especially as applied to large-span structures built of reinforced concrete. He explored the various possibilities of using reinforced concrete and ferrocement by creating a variety of innovative structural systems. His innovative projects include a prefabricated 942 m span arch for the Turin Exhibition (1949–50) and the first skyscraper in Italy, the Pirelli Building (1955) in Milan, a collaborative design. During his long and successful career, Nervi combined his activity as an engineer with that of an architect, theorist, and teacher. He worked during 1945 and 1962 as an adjunct professor of material technology and construction technique at the Faculty of Architecture of the Sapienza University in Rome.

In 1961 Harvard University, USA, appointed Nervi to the Charles Eliot Norton Chair of Poetry and in 1963 awarded him an honorary degree. He later received the Gold Medal from the American Institute of Architects in recognition of his work. He was frequently asked to deliver lectures in universities and institutions in Italy and worldwide. In addition, he was also a prolific writer and wrote several books on architecture and published articles in some of the most important Italian and international journals of his time, such as Domus, Casabella, L'Architecture d'aujourd'hui, and Concrete. Although Nervi's primary concern was never aesthetic, many of his works, nonetheless, reached the realm of poetry.

Early life

Pier Luigi Nervi was born on June 21 1891in Sondrio, a town in the Italian Alps, as the only son of the local postmaster, Mr. Antonio and his wife Mrs. Luisa Bartoli. Due to the work of his father, Nervi spent his youth in several cities of Italy. He was obsessed with mechanical things as a child, and when he attended the University of Bologna, his interests expanded to the mechanics of large scale engineering projects. He graduated from the university on July 28 of 1913.

Civil Engineering Practice

Nervi, after completing his degree, joined the office of the Società per Costruzioni Cementizie (Society for Concrete Construction), based at Bologna, where his former Professor Muggia was the director. The First World War broke out at that time and hence young Nervi was forced to work in the Italian Army from 1915 to 1918, serving in the Corps of Engineering as a lieutenant. After 1918, he returned to work and was sent to the office at Florence, with considerable autonomy. In 1926–27 he designed his first significant work, a cinema in Naples, and followed it with the municipal stadium (Berta Stadium) in Florence, built in 1930–32.

In 1932, Nervi and his cousin in Rome formed a contracting firm of Nervi and Bartoli, with which he was associated till the end of his career. Nervi was commissioned in 1938 to design and build a set of six huge airplane hangars for the Italian air force. He conceived them as concrete vaults, spanning areas of 100.5 m by 39.6 m with no supporting pillars (see Fig.1). The hangars proved to be economical and functional, and their high vaults, where Prof.

Nervi used his own system of prefabricating for the first time, were aesthetically pleasing. All of these structures were destroyed during the World War II. He also designed several large underground tanks for Nafta, with capacities up to 30,000 m3. During 1940s, he developed novel reinforced concrete systems, which helped in the rebuilding of many buildings and factories throughout Western Europe. Nervi even designed/created a boat hull that was made of reinforced concrete as a promotion for the Italian government.

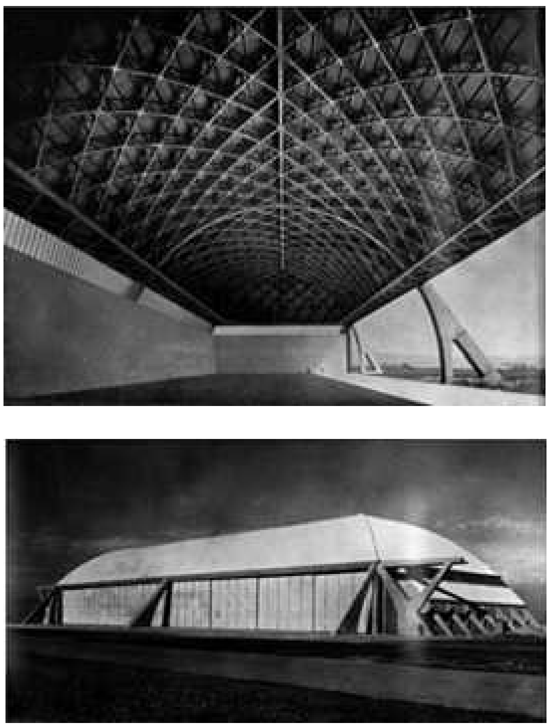


Fig. 1 Airplane Hangar by Nervi, 1938
(a) Inside view, (b) Outside view

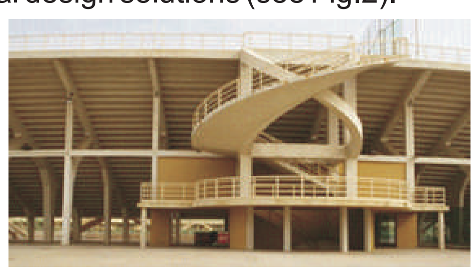
Nervi also stressed that intuition should be used as much as mathematics in design, especially with thin shelled structures. He borrowed from both Roman and Renaissance architecture to create aesthetically pleasing structures, yet applied structural aspects such as ribbing and vaulting often based on nature. This was to improve the structural strength and eliminate the need for columns. He succeeded in turning engineering into an art bytaking simple geometry and using sophisticated prefabrication to find direct design solutions in his buildings.

Engineer and Architect

Pier Luigi Nervi was educated and practiced as ingegnere edile (translated as "building engineer") - in Italy, at the time (and to a lesser degree also today), a building engineer might also be considered an architect. After 1932, his aesthetically pleasing designs were used for major projects. This was due to the booming number of construction projects at the time which used concrete and steel in Europe and the architecture aspects took a step back to the potential of engineering. Nervi successfully made reinforced concrete the main structural material of the day. Nervi expressed his ideas about buildings in his four books and many papers. Throughout his career, he collected images and photographs of buildings, ranging from ancient Egyptian and Greek temples to Gothic cathedrals, from great Renaissance domes to modern structures like the Galerie des Machines in Paris and the Twin Towers in New York (Antonucci and Nannini, 2019). Nervi used his monumental collection of architectural images constantly in his lectures, conferences and publications. For Nervi, architecture was fundamentally a structural fact; therefore, he saw its aesthetic features as logical consequences of static and building solutions. The architectural features used by him, such as the pleated or ribbed domes, transitional or variable section pillars, isostatic ribbed floor slabs, etc., were very innovative.

Notable Projects

Most of his structures are in his native Italy, but he also worked on projects abroad. Nervi first garnered major public attention with this work of cantilevered roof and elegantly winding stairs of the stadium in Italy-which showed his affinity for visually dramatic structural design solutions (see Fig.2).



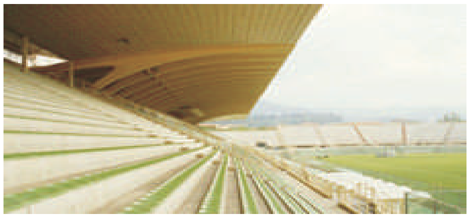
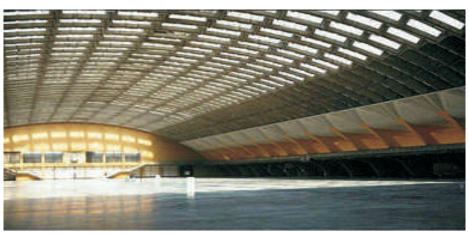


Fig.2 Artemio Franchi Municipal Stadium, Florence, Italy, 1932, Nervi with Alessandro Giuntoli (Photo:Yoshito Isono)

Nervi designed a large 96m x 75m Exhibition Building, at Turin (Fig. 3), with a thin, corrugated vaulted roof made of a series of prefabricated ferrocement elements. He constructed it by using special scaffolding and with reinforced concrete ribs along the ridges and valleys of the roof section. The 4.4 m long, fifteen prefabricated thin ferrocement 'wave like' elements, weighed 1500 kg each, and had thickness of 40 mm. Their ends were stiffened by diaphragms leaving an empty space of 40 mm to be filled with cement mortar. These ferrocement elements were connected by reinforced cement ribs cast on site, arranged in the valleys and ridges of the waves. Each of the three arches of the undulating roof was connected to the inclined columns by means of fanned beams, as seen in Fig. 3(b) (Antonucci and Nannini, 2019).



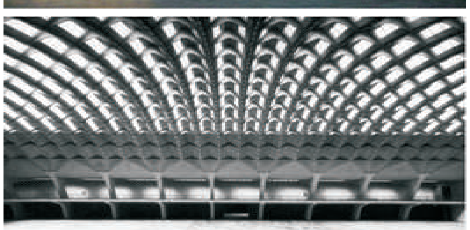


Fig. 3 Exhibition Building, Turin, 1949, Italy (a) Longitudinal view (b) Transverse view (Photo: http://www.greatbuildings.com/).

For the adjoining rotunda's 40 m diameter half-dome Nervi used a system of prefabricated rhomboid-shaped elements, connected by RC ribs which were used as permanent formwork (see Fig. 4). On this system, ferrocement slabs having a thickness of approximately 30 mm were laid. The overall depth, taking into account the dimension of the ribs, was about 70 mm only. This Nervi's patented system of precast ferrocemento (ferrocement) formworks, named as Sistema Nervi (the Nervi System) can be found in many building designed and built by him. This system enabled Nervi to build architectural structures with structural ribs, which were comparable to that of Gothic structures (Antonucci and Nannini, 2019).

Fig. 4 Hall B of the Torino Esposizioni in Turin, 1954 (Photo: Sofia Nannini)

Nervi also studied and patented a new type of structure- a ribbed floor slabs, in collaboration with Aldo Arcangeli, an engineer employed in his construction firm. This concept was based on the idea of placing ribs in the slab which follow the 'isostatic' lines of stresses. These lines define the main directions of tension and are tangent to the trajectories of the bending moments, on which the torques are zero. This invention was first experimented by Nervi in the tobacco factory in Bologna, Rome (1951), as shown in Fig.5(a), and then in the Lanificio Gatti (Gatti Wool Factory) in Rome (1951) [Fig.5(b)]. Both these projects allowed him to go beyond traditional structural forms, mirroring his attraction to Gothic structures (Antonucci and Nannini, 2019).

Fig. 5 Ribbed floor slabs (a) Tobacco factory in Bologna (Photo: M. Antonucci), (b) Gatti Wool Factory in Rome

In 1950, Nervi, American Marcel Breuer, and the Frenchman Bernard Zehrfuss collaborated on the design of the new headquarters for the United Nations Educational, Scientific and Cultural Organization (UNESCO) in Paris (Fig.6). This project was validated by a committee of five architects including Le Corbusier, Walter Gropius and Lucio Costa. Consisting of a main unit of seven floors forming a star with three branches to which is added an 'accordeon like' building (a conference center) and a cubic building, it was inaugurated in November 1958.

Fig. 6 UNESCO headquarters, Paris (1958) (Photo: www.flickr.com/photos/french-disko/3712216223)

In 1955, in association with Gio Ponti, Nervi helped design the first skyscraper in Italy, the 33 storey, 127 m tall Pirelli Tower (Fig.7); it was the first office building to use a long-span (25 m) structure. It is notable that at that time, the architects and engineers in the United States, who had long experience in the design and construction of skyscrapers, used frameworks with smaller spans only. The tower featured a tapered plan, as opposed to the conventional rectilinear skyscrapers of that time in the USA. It featured concrete columns that become gradually thinner as they approached the top of the building (from 2 m at ground floor to 0.5 m at the top). For the Pirelli Building, Nervi used experimental models-as he often had-which he tested in the laboratory at Bergamo. It remained as Italy's tallest tower from 1958 to 1995.

Fig. 7 The Pirelli Tower, Milan (1958) (Photo: www.flickr.com/photos/ikkoskinen/4824881170/)

Nervi's second skyscraper was built in Montreal in 1964, in collaboration with Luigi Moretti, [See Fig. 8 (a)] which is a 190 m tall 48 storey Stock Exchange Building. He then worked with the Australian architect Harry Seidler on the Australia Square Tower and the MLC Centre in Sydney. The MLC Center in Sydney, completed in 1973, was a cylindrical tower of 50 stories, and is shown in Fig. 8(b). At the time, this was the tallest concrete structure in the world.

Fig. 8 (a) The Tour de la Bourse in Montreal (1964)
(Photo: Nicolas Janberg, www.structurae.net/en/structures/)
(b) The MLC Center, Sydney, Australia (1973)
(Photo: Adam J.W.C.)

The 11,500-seat Palazzo dello Sport was constructed for the 1960 Summer Olympics (see Fig.9). This multi-purpose sports and entertainment arena in Rome, was designed by architect Marcello Piacentini, and its reinforced concrete dome was engineered by Nervi. This facility was designed and built along with a smaller facility, the Palazzetto dello Sport, its dome was also engineered by Nervi (Fig.10).

Fig. 9 Palazzo dello sport EUR
(now PalaLottomatica), Rome (1957)

The Palazzetto dello Sport (literally "Small Sport Palace") is an indoor arena located in Piazza Apollodoro, in Rome, Italy, built for the 1960 summer Olympics, and inaugurated in 1957(See Fig. 10). It has a 3,500 seating capacity for basketball games. It was designed by architect Annibale Vitellozzi and its reinforced thin-shell concrete dome was engineered by Nervi under the direction of Engineer Giacomo Maccagno. The arena is a 61 m diameter ribbed concrete shell dome and is constructed out of 1,620 prefabricated concrete elements. The exterior consists of a set of Y shaped piers supporting a low, fluted ferrocement dome. Inside the dome, ribs of concrete make a flowered pattern of extreme lightness and delicacy, as seen in Fig. 10(a). Much of the structure was prefabricated, and hence made the dome to be erected in 40 days.

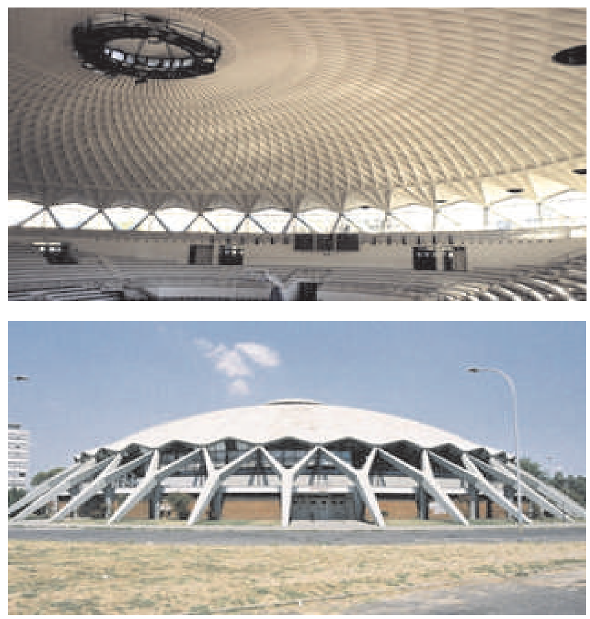


Fig. 10 Palazzetto dello sport, Rome
(Photo: Nicolas Janberg, https://structurae.net/)

The competitive tender for the construction of the 47,000 m² pavilion for the Centenary of Italian Unity, was presided over by Giovanni Agnelli and designed by Giò Ponti. The jury awarded the project to Nervi & Bartoli in 1957. In addition to Nervi, his son Antonio and one of the primary Italian engineers of steel structures, Gino Covre, worked on this project. The project revolved around the subdivision of the square roof into sixteen independent 'umbrellas', each having a side of 40 m, separated by a continuous strip of skylights, made from a ‘sunburst’ pattern of steel beams fixed to a central column having a variable geometry (see Fig. 11a). The perimeter gallery is instead covered with Nervi’s typical ‘isostatic’ ribbed slabs [Fig. 11(b)], constructed using moveable ferrocement formwork, based on a process widely tested by Nervi in various buildings, including the Gatti Wool Mill (1951-53).

Fig. 11 (a) Palazzo del Lavoro, Turin (1961)
(b) The perimeter gallery with isostatic ribbed slabs
(Source: https://www.atlasofplaces.com/architecture/palazzo-del-lavoro/)

Nervi’s first building in the United States; the George Washington Bridge Bus Terminal, in Manhattan was commissioned by the Port of New York Authority, and was built during 1961–62 as shown in Fig.12. Nervi's design became famous for the butterfly-like wings of its concrete roof, and helped him acquire fame in America. Subsequently he designed a precast, vaulted field house for Dartmouth College in New Hampshire (1961–62). In 1967, he designed the Cathedral of San Francisco, in collaboration with Pietro Belluschi, which has four vertical-warped surfaces dramatically enclosing the vertical space of the main nave (See Fig.13).

Fig. 12 George Washington Bridge Bus Station, New York City (1963),
(Photo: Nicolas Janberg, https://structurae.net/)

Fig. 13 Cathedral of Saint Mary of the Assumption, San Francisco, California (1967)
(collaborating with Pietro Belluschi)

Nervi was the consulting engineer for the Australian Embassy constructed in Paris in 1973 (Fig.14). The Embassy was built from precast modularized concrete, with quartz and granite faced exterior and prestressed precast floors. The two buildings are curved to form two quarter circles, the two arcs of an "S"-shaped complex, with the radii of the circles lined up to match the axes of the Eiffel Tower and the Champ de Mars of Paris.

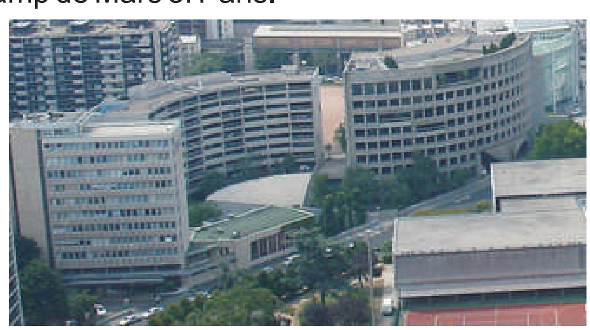


Fig. 14 Australian Embassy, Paris (1973)

Built between 1968 and 1971, the design of the Scope arena in Virginia, USA, as shown in Fig.15, resembles closely Nervi's Palazzetto dello Sport in Rome, built for the 17th Olympics which took place in 1960. Designed by Nervi in collaboration with US architectural firm Williams and Tazewell, its soaring struts lift the dome and paradoxically let light in from below, not above as in traditional domed structures. Scope exploited the extraordinary strength and flexibility of the astonishing new material of mesh-reinforced concrete and remains the largest thin-shell dome in the US.

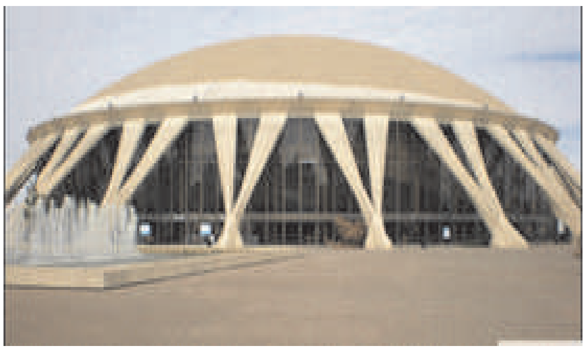


Fig. 15 Norfolk Scope Arena in Norfolk, VA, USA, 1971

The Abbey Church, also known as the Church of St. Mary and St. Louis and the Priory Chapel, was designed by Gyo Obata of Hellmuth, Obata and Kassabaum (HOK), with Nervi serving as consultant (Fig.16). The church's circular facade consists of three tiers of whitewashed, thin-poured concrete parabolic arches, the top one forming a bell-tower; the arches appear to float upwards from their grassy base. They are faced with dark insulated-fiberglass polyester window walls which create a meditative translucency when viewed from within.

Fig. 16 Priory Chapel at Saint Louis Abbey in Creve Coeur, Missouri (1962) (Photo credit: Magdalena424)

Other notable structures include Orvieto Aircraft Hanger (1935), Sacro Cuore (Bell Tower), Firenze (1962), Burgo Paper Mill, Mantua, Italy (1962), Leverone Field House at Dartmouth College (1965), Thompson Arena at Dartmouth College (1973–74), and Paul VI Audience Hall, Vatican City (1971),

Awards and Recognitions

Pier Luigi Nervi was awarded Gold Medals by the Institution of Structural Engineers, the AIA, Royal Institute of British Architects (RIBA) and the Académie d'architecture. He also received the Frank P. Brown Medal of The Franklin Institute in 1957. He was elected as honorary member of the American Academy, Institute of Arts and Letters in 1957 and Foreign Member of the Royal Swedish Academy of Art at Stockholm in 1957.

Several universities around the world conferred honorary doctorate degrees to him, which include, Buenos Aires (1950), University of Edinburgh (1960), Technischen Hochschule, Munich (1960), University of Warsaw (1961), Harvard and Dartmouth College (1962), Praga (1966), and Londra (1969). He was elected as honorary member of the American Institute of Architects in 1956.

Nervi held two conferences on April 10th and May 10th, 1962, at Harvard University, as he was awarded the Professorship of Poetry named after Charles Eliot Norton for the academic year 1961–62, together with architects Felix Candela and Buckminster Fuller. He was the first Italian to receive this honour.

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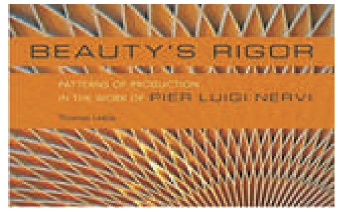

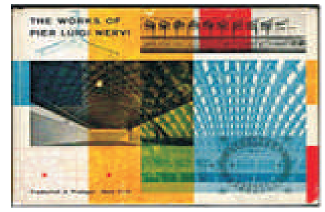

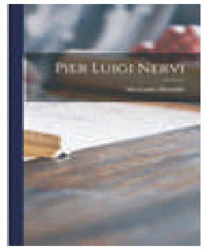

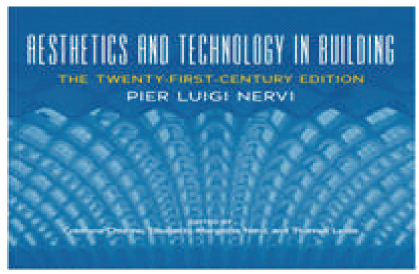

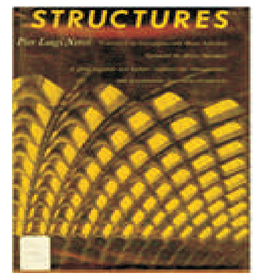

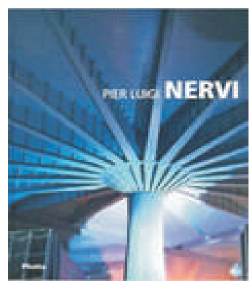

At the end of the 1960s, Nervi assumed the leadership of an innovative editorial project: the series Storia universale dell'architettura (Universal History of Architecture). The series containing 15 volumes was published in several languages by Electa, between 1971 and 1977. These volumes, each edited by a different expert, cover the whole history of architecture. Many of his books on design and engineering have been translated into English.

Family

In 1924 he married Irene Calosi and she gave birth to their four children (Antonio, Mario, Carl, and Vittorio), three of whom accompanied worked in his office. After years of intense practice in Italy and abroad, Nervi reduced his activities as a builder in the late 1960s. Assisted by three of his sons, Antonio and Vittorio, both Architects, and Mario, an Engineer, Nervi began to confine his activities largely to designs. Prof. Nervi died on Jan. 9, 1979, at Rome at an age of 87.

Acknowledgements

The author wishes to acknowledge that photos used in this article have been extracted from different sources in the Internet.

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About The Author

Dr. N. Subramanian,
Ph.D., FNAE
is an award winning Author, Structural Engineering consultant and Mentor, currently based at Maryland, USA, with over 45 years of experience in Industry (including consultancy, research and teaching). He was awarded with a 'Life Time Achievement Award' by the Indian Concrete Institute and many other awards for his contributions towards Structural Engineering. He is the author of 26 books, including the famous books on 'Design of Steel Structures', 'Design of RC Structures' and 'Principles of Space Structures' and the recent 'Building Materials, Testing and Sustainability'. (email - drnsmani@yahoo.com)

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GEOTECHNICAL CHALLENGES IN BASEMENT RETAINING METHODS

By Shekhar Vaishampayan

At the outset of any project, geotechnical investigation is conducted and depending on the findings of geotechnical data at project location and structural loading, appropriate foundation system is selected. Depth of foundation is thus selected. So, for construction of foundations, it is required to excavate to recommended depth below ground level.

To decide whether excavation is shallow or deep, accessibility of excavation depth is used as criterion. Traditionally, one man height was considered as shallow excavation. With technology upgradation, excavators have increased reach upto 4.00 m for ground level. Therefore, in general, excavation deeper than 4.00 m can be termed as deep excavation.

In urban setting today, multiple basements are required for housing parking and different services. Depth of excavation upto 20 m below ground level is required in many projects in metro cities today.

In most of the approved projects in metro cities, distance of property boundary to outer face of basement is only 1.50 m. This distance is too less for making free standing excavation for foundation construction. To retain the basement excavation without transgressing in adjoining property, adequate retaining system has become essential. In congested localities where ground water is close to ground level particularly during monsoon, retaining methods are must.

Retaining walls are basically vertical cantilever beams embedded to adequate depth and have sufficient reinforcement. Retaining walls are of two types, non-embedded and embedded. Non-embedded retaining wall derives stability primarily by gravity. So, it requires base width between 0.70 to 1.0 times retention height. It does not fit in available width of 1.50 m. In addition, other services are also accommodated in this width.

In such conditions, embedded type retaining walls are only possible. Embedded type retaining walls achieve stability by mobilizing passive resistance of soil/rock below excavation depth. Three main embedded retaining systems in common usage are RCC Retaining Pile, Diaphragm Wall and MS Sheet pile.

Diaphragm walls (including meter panels) and steel sheet piles require specialist driving equipment and require more working space. For drilling in rock additional higher technology fittings are required. On the other hand, conventional or hydraulic rotary rigs are available more easily, equipped to handle drilling in rock and are more competitive. It is also more convenien and feasible to adopt circular sections in field. Therefore, RCC retaining piles are used most frequently. Since, pile foundation equipment is used to install, so these retaining elements are named as retaining or shore piles.

RCC Retaining Piles

Depending on the subsurface profile, types of retaining piles are subdivided as below.

A) Cantilever Piles are stiffer elements with higher bending moments.

- a1) Contiguous (Touch) Piles
- a2) Secant Piles

B) Propped Cantilever Piles are flexible retaining elements with lower bending moments and controlled lateral deflections.

- b1) Contiguous (Touch) Piles
- b2) Secant Piles

Contiguous piles are generally used wherever soil stratum is predominantly cohesive. This allows piles with small clear gap in between. Cohesive soil between piles arches over clear gap. it depends on particle size of soil retained. Typically, clear gap varies from 75 mm to 150 mm. This also prevents buildup of hydrostatic pressure in retaining wall. In rainy season, if ground water table rises then there is tendency of cohesive soil to erode through the clear gap. If fine fraction of soil contains more silt then erosion happens very easily.

Secant piles become necessary when high ground water condition and non-cohesive strata is to be retained. Then use of alternate PCC and RCC piles is adopted as solution. PCC piles with fresh green concrete are drilled with minimum of 100 mm overlap and Main load carrying RCC piles are cast. These piles are drilled well below, excavation depth and are reinforced. Therefore, secant piles are designed for soil plus hydrostatic pressure.

Diaphragm walls, meter panels or sheet piles also can be designed using the same principles. They can also be designed as cantilever or propped units. However, Diaphragm walls, meter panels and sheet piles require more friendly soil conditions. These can be installed more easily when rock is deep and will not be within design depth of these retaining systems. Homogenous soil stratification without obstructions like filled up boulders or construction debris or vegetation roots is best suited for these walls.

Forces Acting on Retaining Walls

Retaining walls means there is difference in ground level on two sides of wall. This difference is the retaining height. In embedded retaining walls, retaining elements are installed first and excavation follows as in case of basement excavation. Naturally soil layers on higher side exert lateral loads. These will tend to destabilise the retaining wall.

Destabilising Forces are listed below

A) Active Earth pressure exerted by different soil layers till foundation depth of wall. These are derived from geotechnical investigation data of the project site.

B) Hydrostatic Pressure due to ground water are considered from likely highest ground water level using investigation data and information about project site during monsoon.

C) Active pressure exerted by Surcharge Load is also considered. Intuitively, this is not considered as governing force. It has been noted that surcharge loads affect the bending moments in the retaining walls significantly. Therefore, this factor should be studied diligently.

Surcharge load will be caused by sloping ground surface above retaining height. It can also be due to stacking of excavation spoils or construction materials adjacent to excavation face. This could be also due to an existing structure next to excavation. In such case bearing pressure on footing of adjacent structure will be surcharge load. Loads from vehicles/cranes which bring in construction materials or cart away spoils will also cause surcharge loads.

Embedded part of retaining wall derives resistance from soil or rock in which it is embedded. Stabilizing forces acting on retaining wall are listed below.

D) Passive Earth Pressure exerted by different soil/rock layers till termination depth of wall. These are derived from geotechnical investigation data of the project site.

E) Horizontal Load resulting from supports applied to control deflection of retaining walls and thereby to enhance stability of retaining walls. Dead man anchors, Passive Anchors, Raker supports, and internal struts and prestressed anchors are the alternatives.

Use of supports at intermediate level in retaining wall allow for more flexible structure. It controls lateral deflection of retaining wall. It will mean reduction bending moments and so lesser reinforcement steel.

Traditionally, dead man anchor which was placed at sufficient distance outside failure wedge. But today that much of land is not available in urban projects. So, these are replaced by anchors. These allow ease of working during excavation. But installation of passive anchor or active anchor will intrude into adjoining property. If it creates legal issue then one may have to resort to raker supports or internal strutting.

Rakers are generally not first choice as they derive supports from bottom raft and till then slope may not remain stable. It is temporary solution in case of collapse of unsupported excavation slope. Therefore, internal struts become only alternative. But it reduces the working speed of excavation or construction considerably and it should be accounted for in the project schedule.

Use of intermediate supports also mean excavation is done in stages. Excavation must be stopped at every support level to install waler beam and anchors/structs. Only then further excavation is permitted. Excavation without installation of support will destabilize the retaining wall.

Design Criterion Of Retaining Walls

While designing the retaining pile, normally first part is to determine minimum embedment depth below retaining height at which summation of bending moments will be zero. While doing this, stabilizing forces are reduced by factor of 2.0 to ensure compatibility of deflection in active and passive pressure conditions.

Maximum bending moment location is then computed by finding depth at which sum of shear forces becomes zero. Usually, reinforcement steel is provided for this magnitude of bending moment.

Important and non-negotiable parts of design are embedment depth and reinforcement steel. In field both aspects must be achieved without exception. Field teams must be made aware that these are not bearing piles used to support the structure. If embedment depth is not achieved, then retaining system can collapse, which has been observed as most common cause of failure.

Another very important aspect is to know magnitude of permissible lateral deflection of retaining wall. IS 2911 allows 5 mm as maximum permissible lateral deflection for bearing piles. Many designers also refer to IRC 78, which permits lateral deflection of 1 percent of pile diameter for bridges. Both of these criteria are for vertically loaded bearing piles. Therefore, these values are not applicable for retaining piles.

Theoretically, maximum magnitude is limited to 1/250 of the span or height retained as per IS 456. But practically, lateral deflection upto 25 mm seems to work in most cases. it is recommended to ensure that maximum deflection of pile head or top of retaining wall shall be limited to15 mm and in no case, it should exceed 25 mm.

Retaining piles will deflect after excavation is carried out for basement. Despite numerical limits listed above adjoining services can get affected. Adjoining services such as storm water drains, sewer lines, water pipelines or gas pipelines may not have sufficient flexibility. Sometimes if storm water drains crack then water flowing through the opening can cause a lot erosion into excavation pit and cause distresses to services listed above. In such cases, permissible lateral deflection shall be reduced to acceptable value.

In most of the situations, retaining piles are used only as enabling works. Therefore, durability of these piles is not very critical. In case of severe space constraint, retaining piles are made part of basement walls, then durability aspects shall be strictly followed.

Constructability Factors

There are number of practical aspects which are considered before selection of piling equipment. These are access to site, machinery available in local market, likely response of soil and rock stratum to drilling tools, vibrations that may be induced while drilling. A few main aspects are discussed below.

Sub surface stratification at the project location is the most important factor in selecting piling rig. Likely response of different soil and rock stratum to drilling tools is judged based on the shear strength of soil and unconfined compressive strength of rock. If strength of rock is more than 30 MPa and rock core recovery is more than 50 percent, drilling becomes very time consuming and expensive. In some case, if there are pockets of rock with high strength, use of core barrel becomes necessary. Piling rig of adequate rating shall then be employed. Practical rating of piling rig is decided by torque generated at Kelly bar rpm of 10. For more details, please refer to Pile Termination Criterion by the same author in previous issue of this journal.

Whenever project location is easily accessible, any drilling rig/machine can be taken to site. In congested locality access road to plot may be too narrow for rig to reach site. Sometimes plot dimensions could be too small to accommodate movement of rig and concreting unit. In such situations, availability of compact machine of adequate capacity can be critical.

If conventional piling rig is employed at a given site, percussion action of chisel can induce vibrations in the vicinity of pile location. Hydraulic drilling piling machines operate in rotary mode and cut the rock. So hydraulic piling rig does not cause significant vibrations and work very close to adjacent property can be executed. But the location of pile nearest to existing structure is affected by shape of drilling rig. Minimum required distance from structure is usually 750 to 800 mm

About the Author:

B Tech (Civil), M Tech (Geotech), MIGS, MIE

Shekhar Vaishampayan is a geotechnical consultant having four decades of experience in the field and is a director of M/s Sub Surface Consultants Private Limited,Thane.

Email : shekhv@gmail.com

HIGH STRAIN DYNAMIC TEST

By Mehfooz Ahmed Khan

1.0 TEST METHOD: ASTM D 4945:2017

1.1 REFERENCE STANDARD: IS 2911 PART-4:2013

1.1 TERMINOLOGY:

Initial Test Pile : A pile that is not a part of the foundation system, is installed to verify the design load-carrying capacity of a pile.

Working / Routine Pile : A pile forming part of the foundation system of a given structure.

Working Load / Design Load : The load assigned to a pile as per design.

Ultimate Load Capacity : The maximum load which a pile can carry before failure, that is when the founding strata fail by shear as evident from the load settlement curve or the pile fails as a structural member.

Gross Displacement : The total movement of the pile top under a given load.

Net Displacement / Permanent Set : The net vertical movement of the pile top after the pile has been subjected to a test load and subsequently released.

The factor of Safety : It is the ratio of the ultimate load capacity of a pile to the safe load on the pile.

Impact Force : A transient force applied to the top of the deep foundation element.

Hammer Cushion : The Material inserted between the hammer striker plate and helmet

on the top of the deep pile foundation.

Particle Velocity : The instantaneous velocity of a particle in a deep foundation as a

strain wave passes by.

Wave Speed : The speed with which a strain wave propagates through the deep.

2. OBJECTIVE OF TEST: Dynamic Load Testing is a fast, reliable, and cost-effective method of evaluating foundations. This Test gives more information in less time as follows.

- Pile Load Bearing Capacity / Shaft resistance / Shaft resistance along with the shaft / Toe

Resistances / Max Tensile Stresses & Compressive Stresses / Tensile Stresses & Compressive Stresses along with the shaft.

2.1 PRINCIPLE OF TEST:

- Hammer causes a downward traveling stress wave to enter the pile.
- Soil resistance causes stress-wave reflections.
- Stress in piles can be represented by 1-dimensional Wave Theory.
- These stress waves can be measured and identified with the measurement of force and velocity near the pile top.

High strain dynamic testing consists of estimating soil resistance and its distribution from force and velocity measurements obtained near the top of a foundation impacted by a hammer or drop weight. The impact produces a compressive wave that travels down the shaft of the foundation.

A pair of strain transducers obtains the signals to compute force by multiplying strain, cross-sectional Area & Modulus of Elasticity of concrete, while measurements from a pair of accelerometers are integrated to velocity by formula $a = \Delta v / \Delta t$. These sensors are connected to an instrument (such as a pile driving analyzer), that records, processes, and displays data and results.

As long as the wave travels in one direction, force and velocity are proportional and related by the expression F = Zv, where: = EA/c is the pile impedance

Where E is the pile material modulus of elasticity

A is the cross-sectional area of the pile

C is the material wave speed at which the wavefront travels

The wave assumes an opposite direction (a reflection) when it encounters soil resistance forces along the shaft or at the toe. These reflections travel upward along the shaft and arrive at the pile top at times that are related to their location along the shaft. The sensors near the pile top take measurements that translate what is happening to the traveling waves and make it possible to estimate soil resistance and its distribution. The data obtained in this fashion permits the computation of total soil resistance, which includes both static and viscous components. The dynamic component is computed as the product of the pile velocity times the damping factor (a soil parameter related to soil grain size). The static component is the total soil resistance minus the dynamic component). Dynamic load testing takes a further step in analyzing the data and computing static capacity and resistance distribution & Pile integrity.

2.2 CASE METHOD

The Case Method Equation: RTL = Fd,1 + Fu,2

RTL is the total pile resistance: Dynamic (RD)+ static (RS) (shaft resistance + end bearing)

RTL is mobilized during time 2L/c following time t1

The Static Resistance is Total Resistance minus Dynamic resistance

Rstatic= RTL - Rdynamic

Assuming Rdynamic= Jv* v [kN/m/s][m/s]

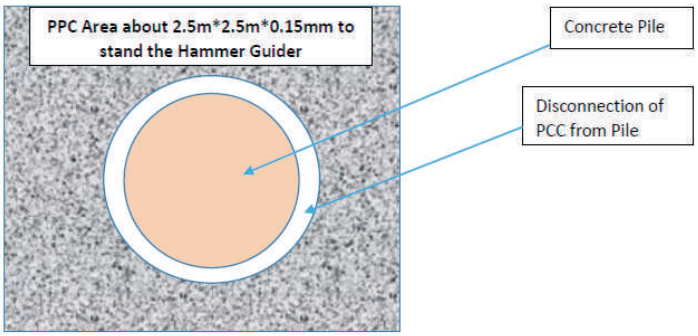
Introducing: Jc = Jv / Z ..... Case Damping Factor

Then Rdynamic = Jc* Z *v Where Jc= Dumping Factor, Z=Impedance, v= Pile velocity

3.0 PROCEDURE OF TEST:

3.1 CRITERIA OF PILE FOR TESTING IN CUSTOMER SCOPE?

- Age of Pile at the Test > Pile concrete should attain at least designed Grade of concrete (Preferably 28days).
- Pile shaft must be chipped off till sound concrete level (ie removed soil mixed concrete from pile shaft).
- Pile head must be smooth, leveled and clean, It is expected that the capping of the pile will be mature enough to withstand the impact of hammering.
- The pile's surrounding area must be sufficiently PCC(thickness approx.150mm) to support the guider of the hammer.
- The surrounding PCC should not be connected to the test pile. Please refer below sketch.



3.2 NO/S OF PILE FOR TEST?

For Initial Test : As per Clause No 5.1.1 of IS 2911 Part-4 for respectively designed piles.

For small-size projects (for piles less than 1000 numbers), a minimum of two tests.

For large-size projects (for piles more than 1000 numbers), a minimum of two tests for the first 1000 piles and an additional one test for every additional 1000 piles and part thereof.

For Routine Test : As per Clause No 5.2 of IS 2911 Part-4.

The number of tests shall be 0.5 percent of the total number of piles, subject to a minimum of one (1) test. The number of tests may be increased up to 2 percent in particular, cases depending upon nature, type of structure and sub-strata condition. or as per the decision of the engineer in charge of both types of piles.

3.3 HOW TO SELECT THE PILE FOR THE TEST?

If Engineer Incharge is not specified the piles for the test then generally, the pile to be selected for the test which has variable the impedance variation / the magnitude of defected piles or pile toe is not evident in Pile Integrity Test Report.

3.4 WHAT INFORMATION IS REQUIRED FROM THE CUSTOMER PRIOR TO THE TEST?

Project Identification, Pile Identification, Grade of Pile, Pile Dia, Pile cut-off Length, Casting Date, Soil investigation Report, Design Load, Type of Pile, Splices / Cold Joints, Liner if any with the depth etc.

4.0 APPARATUS?

Impact Hammer, Hammer Cushion, Striker Plate (Helmet), Crane, Hammer Guider, Pile Dynamic Analyser, Accelerometer, Strain Gauge, Level Pipe & Measuring Tape.

4.1 HOW TO SELECT IMPACT HAMMER / MASS?

In general, The Weight of the Hammer / Impact Device should be at least 1-2% of the desired Ultimate Test Capacity / Test Load to get adequate mobilization of the pile shaft as per clause No 5.1 of ASTM D 4945.

This desired Ultimate capacity, if not given then it can be computed as the design load and multiplied by safety factors.

The Safety factor depends on the use of a pile. For the Working pile, It will be 1.5, & for Initial Test Pile it will be 2.5 as per IS 2911 Part-4.

4.1.1 HOW TO SELECT CRANE FOR DROPPING HAMMER / MASS?

Crane or suitable equipment capable to free fall of the suitable weight of Hammer per

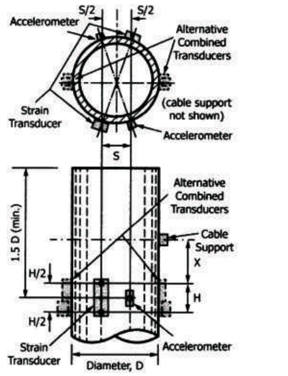
clause no 4.1 from adequate height safely.

4.1.2 HOW TO PLACE THE TRANSDUCER ON THE PILE?

The Transducer shall be placed at least 1.5 times the diameter of the pile below the pile top to avoid irregular stress concentration. Align the Transducer with their sensitive direction parallel to the long axis of the pile. A similar type of transducers will place on the circumference of the pile so that they are symmetrically opposed and equidistance from the pile centroid in a plane perpendicular to the pile axis.

4.1.2.3 HOW MANY NOS OF THE TRANSDUCER TO PLACE ON THE PILE?

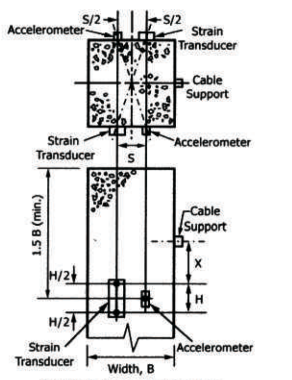
2 Nos Strain gauges & accelerometer to place to cancel bending.



Drill and Tap Holes for Strain Transducers, Accelerometers, and Cable Supports as Shown. Typically use 6 mm Bolt Size.

NOTE 1—Shown as separate transducers or alternative combined transducers.

FIG. 4 Typical Arrangement for Attaching Transducers to Pipe Piles



Set Bolt Anchors or Studs for Strain Transducers, Accelerometers, and Cable Supports as Shown. Typically use 6 mm Bolt

NOTE 1—Shown as separate transducers.

FIG. 5 Typical Arrangement for Attaching Transducers to Concrete Piles

4.1.3 HOW TO DETERMINE THE WAVE SPEED?

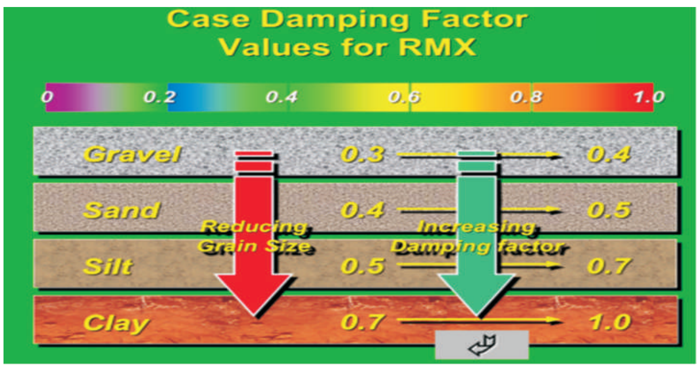
The wave speed is obtained by adjusting the rise to the rise of tensile reflections.

4.1.4 HOW TO ASSUME THE DENSITY OF PILE / FOUNDATION?

Concrete Pile is Often assumed at approx 2450kg/m3 or can be computed by mass and volume of the foundation or indirectly through their effect on impedance and proportionality as per ASTM D4945 clause no 6.3.

4.1.5 CASE DAMPING FACTOR(Jc)?

This value is computed based on the size & type of soil strata of pile / s as per the soil investigation report.



4.2 HOW DOES THE IMPACT AND TILL WHEN?

Prior to taking an observation, the impact of the hammer/ mass from a height recommended of 0.2-0.5m to verify the performance of the gauges. The height of the fall can not be limited prior to the test. It's a trial method depending on the response of strata & concrete properties to decide the height of the fall during the impact output through CAPWAP.

Increase the impact height until one of the following conditions is observed:

- A permanent set ≥2 mm per impact may indicate that sufficient movement has ccurred during the impact event to fully mobilize the capacity as per ASTM D4945 Note-1.
- Tensile stress allows (Tensile stress <0.15 times of Fck of concrete used in pile shaft)
- Compressive stress <0.85 times of Fck of concrete used in pile shaft as per ASTM D4945 Clause No 5.1.
- Test Load Achieved

Note : Re-striking duration would be at least 15 min, as per ASTM D4945 clause No. 3.2.13

If the above condition is not observed then Increase Hammer Weight.

4.3 HOW TO CHECK THE QUALITY OF DATA?

The Force & velocity graph returns to zero after impact as per ASTM D4945 clause no 5.4.3.1 & 5.4.3.2 respectively and at the same time complying with the following condition:

- A permanent set 2 mm per impact or Test Load Achieved (whichever earlier) having Tensile stress & compression Stresses within the tolerance as specified in the standard or test load achieved by keeping the Force Velocity ratio recommended as 1.0 (Permissible between 0.8-1.3) of PDA results.

5.4 WHAT IS PDA TESTING?

To acquire data of Pile Driving Analyzers from accelerometers and strain gauges attached to the pile shaft near the pile head with a set attached on opposite sides of the pile to monitor and minimize the effects of eccentric hammer impacts.

The Force and velocity are measured at the Pile Top during the impact by a pile driving hammer or other suitable drop hammers. This process is called High Strain Dynamic Test. In this Process Force and Velocity are experienced at the top of the pile and were evaluated through CAPWAP to obtain the simulated static Capacity of the pile.

5.2 WHY IS CAPWAP ANALYSIS REQUIRED?

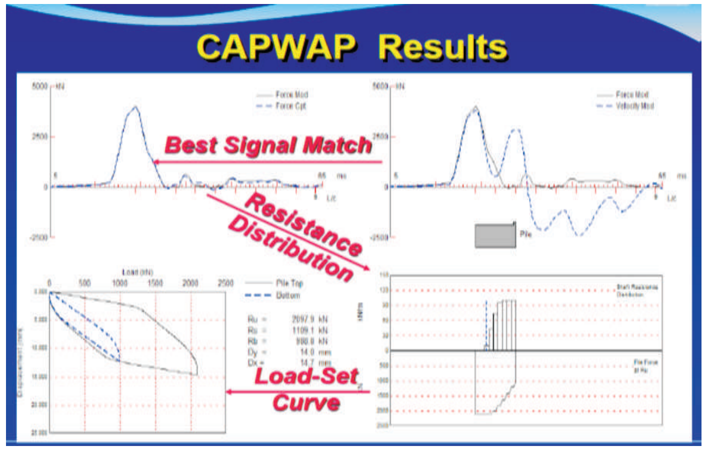
The Case Pile wave Analysis Program (CAPWAP) is a rigorous analytical procedure that was developed to compute soil resistance forces and their distribution from measured pile head force and velocity records. The analysis is usually performed inter-actively by the engineer using a micro-computer, although the program in its expert system mode can also obtain automatic solutions. The Pile is represented as a continuous wave transmission model. The Soil reaction is assumed to be constant primarily of static (elasto-plastic) and dynamic (linear dumping) components. The analysis procedure consists of signal matching pile Force or Velocity histories given one measurement as input and the other as a boundary condition by manipulating the soil model along the pile shaft and under its toe. Analysis results include Static pile capacity, shaft resistance distribution, end bearing, quake, damping, pile head and toe load versus displacement relations, and the pile-soil load transfer curve.

5.3 WHAT IS SIGNAL MATCHING?

The CAPWAP Results are most reliable if the Signal match value is ≤5.

Signal matching provides the most reliable means of predicting the performance of a pile tested by dynamic methods. The pile and the soil data are modeled according to the best estimates made by the operator performing the analysis, and a calculation is made using wave-equation methods. The calculated signals are displayed on the computer screen along with the measured signals. The operator then performs a number of iterations, varying the input data until a satisfactory match between the measured and calculated signals is obtained. Once a satisfactory match is obtained, a plausible model of the pile-soil system is deemed to be established.

A further advantage of signal matching methods is that the distribution of the resistance of the pile down the pile shaft and the pile toe is predicted



5.4 WHAT TO REPORT THE TEST RESULTS?

- Pile shaft Capacity from force and velocity record
- Pile Toe resistance, Shaft Resistance & distribution of shaft resistance along the pile shaft.
- Pile Net Settlement & Max Displacement during impact.
- Max Compressive stress & Compressive stresses along the shaft length during Impact.
- Max Tensile stress & Tensile stresses along the shaft length during Impact.
- Pile Integrity Status.

About The Author

Mehfooz Ahmed Khan
Email id:mehfooz@globallab.co.in
GENRAL MANAGER

GLOBAL LAB ,Mumbai since last 8 years
Founder:PYB foundation ,Mumbai

REHABILITATION OF SPIRITUAL MONASTERY : A CASE STUDY

By Saurabh Samant

1.1. Abstract

This study investigates key challenges of retrofitting and rehabilitation in the holy abode of Sri Badrika Ashram located in the deep valleys of Himachal Pradesh, India. Buildings on hills behave dynamically, very different from buildings on flatlands. Due to irregularity in horizontal and vertical planes, they inhibit non-uniform mass and stiffness distribution and are subjected to torsional forces. hilly terrains have highlighted serious concerns about existing construction practices. Lack of adequate planning and design has resulted in haphazard development in hilly regions. Hence soon after aperiod of time structure requires rehabilitation.

Fig 1.1: Sri Badrika Ashram, Himachal Pradesh, India

This state-of-the-art review investigates the factors that influence the execution of retrofitting of buildings on slopes while explaining reasons that have caused enormous difficulties during the process of retrofitting and rehabilitation of buildings in recent events.

1.2. Background Information

Sri Badrika Ashram at Sirmour, Himachal Pradesh, was constructed around the year 2018. The building consists of 3 floors of sadhak rooms at the lower ground level, an auditorium at the upper ground level, and 3 floors of suits room above the upper ground level, Key section of the same is as below.

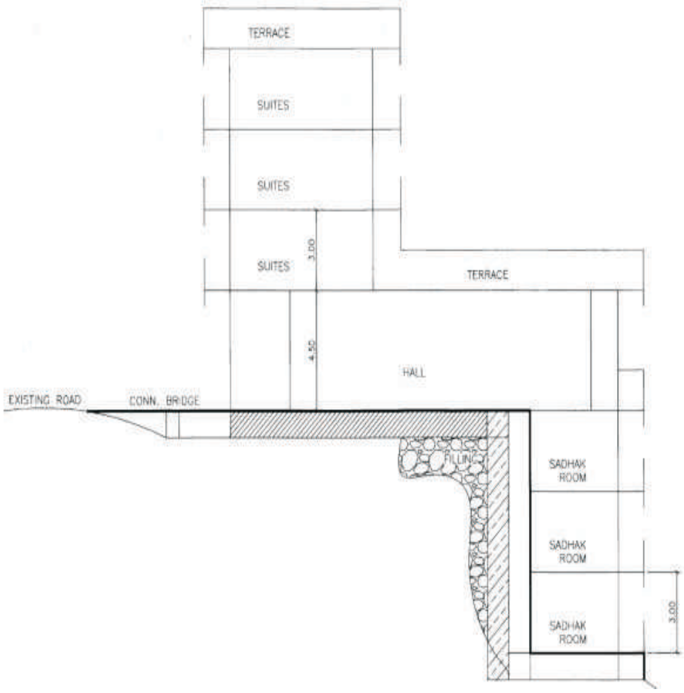


Fig 1.2: Key Section

The Existing structure of the Ashram Building is the RCC Moment-Resisting frame. Infill walls are of brick masonry and the Floor slab is acting as a rigid Diaphragm. A counterfort retaining wall with weep holes has been constructed to retain the earth pressure from the upper ground side and connected with floors at the upper ground floor level (Third slab level of sadhak rooms). As per the original structural drawings, the foundation system is Isolated footing at different levels.

1.3. Distress in Structure

The building started showing visible distress within a few years of construction. Interior partition walls had diagonal cracks as shown in Figure 1.3.1.

Fig 1.3.1: Cracks in structural members

Reinforcements were exposed in the lower plinth level and there was extensive honeycombing observed in the structure.

Fig 1.3.2: Rebar exposure and honeycombing in concrete

1.4. Case Evaluation.

In the first stage, a structural audit of the building was carried out to assess the quality of concrete, and grade of concrete construction using the CAPO test as per ASTM standards. Vertical alignment of the building was also assessed. Based on the structural audit, it was found that the building was constructed with a lower grade of concrete as compared with theoriginal design grade. Further, the investigation revealed that the building is tilted from the back side in both directions in the order of 1.5 degrees.

Using the details obtained from the audit, the dynamic behavior of the building was analyzed under as-built conditions and deficiencies in the structural frames were obtained.

1.5. Course of Action: Retrofitting

1) The columns below the lower plinth level had lower strength hence jacketing of the columns below the plinth has been undertaken.

Fig 1.5.1: Column Strengthening

2) The subgrade at the foundation level of columns of the sadhak rooms was stabilized by pressure grouting of cement.

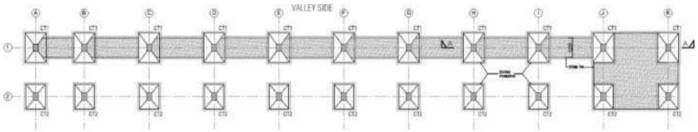


Fig 1.5.2a: Foundation strengthening layout

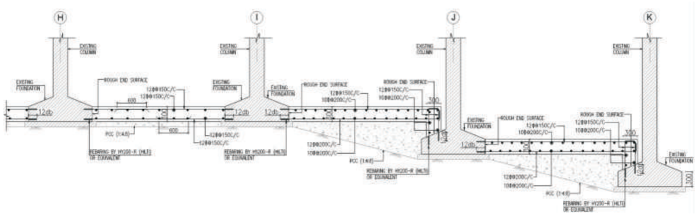


Fig 1.5.2b: Sections A-A

3) The foundation of the sadhak rooms is isolated footings at different levels and sizes such that the bearing pressure may reach a very high value and may cause uneven settlements. Column foundations of the sadhak room were connected through a raft between Grid J-1, K-1, J-2, and K-2. Other foundations of a column of the outer grid (valley side) were connected through a 1200mm wide strip.

Fig 1.5.3: Raft Provision

4) Beams supporting stub columns were implemented with advanced carbon fiber-reinforced polymer wrapping.

Fig 1.5.4: Beam Strengthening with CFRP wrapping

5) Beam-column junction of the lower terrace level was repaired and strengthened by Epoxy grouting and carbon fiber-reinforced polymer wrapping.

Fig 1.5.5: Column-Bean junction confinement with CFRP

1.6. Conclusion

The execution of rehabilitation in a hill settlement in a constrained hilly area is a tedious and difficult task. In spite of the difficult terrain, extreme weather, material availability, and transportation issues, the project was successfully completed and handed over to clients by DGC Infrastructure Pvt. Ltd. in a short period of 90 days.

1.7. Acknowledgement

We would like to express our gratitude to M/S creative design solution Pvt. Ltd. for their assistance with analysis and execution.

About The Author

Saurabh Samant
graduated from IIT Madras with M.Tech in Civil Engineering (Structural Engineering) in the year 2014. He has over 8 years of experience in high rise building design and rehabilitation of structures.
Email : saurabhs@dgc24.com

Shivaji G. Patil

(Janaury 01, 1942- November 20, 2022)

Late Shri S. G. Patil was a founder Trustee of ISSE, the organization formed under the guidance of Late Shri R. L. Nene.

I had long association with Shri S. G. Patil right from my first job. He guided me initially for design procedure to be followed. He was a senior engineer where I started my job. He had in depth knowledge of structural analysis.

After some years of practice he had jobs abroad also. After coming he started his own office as Consulting Structure Engineer. During his practice he designed many buildings, colleges, Educational Institutions, Industrial Structures. He had office in Mumbai and Belgam.

In ISSE Meeting he was very bold in expressing his opinions about all structural and non-structural problems. He was a part of ISSE Committee to discuss about engineer's problems with Municipal and state Government authorities. He was President of ISSE for some time and then parent advisor. He participated in many seminars on various technical subjects and his lectures were a technical treat to engineers.

ISSE has lost a very knowledgeable engineer. His absence will be felt for long time.I personally lost a very close friend and guide to whom I would go for any advise. Apart from regular activities he was keeping personal relations with ISSE Trustees. We all pray to god to give strength to his family members to bear the loss.

Om Shanti!

by Suresh G. Dharmadhikari

NEWS AND EVENTS DURING OCT – DEC 2022

by Hemant Vadalkar

8 OCT 2022 :

Indian society of structural engineers, Aurangbad, conducted an event on ‘Performance based structural design of structures” in colaboration with MP Birla group on Saturday- 8th October-2022.

Dr. Vasant Matsagar has given a wonderful presentation on the topic, Mr. Vispute & Mr. Dixit from M P Birla group presented their products during the event. Er. Karim Pathan (Chairperson-ISSE), Er. Shilpa Danekar (Secretary- ISSE), Er. Naveed Akhtar (Jt. Secretary -ISSE), Er. Sohail Shaikh (Treasure -ISSE) and Er. Vaibhav Dixit (Technical Head-MP Birla Group) and Mr. Suneet Gupta, & Mr. Sachin Sakharkar(MP Birla Cement) took the efforts for the event.

13 Dec 2022 :

IEI in association with ISSE arranged technical lecture on Retrofit and Rehabilitation. The presentation started with Saurabh Samant who talked about retrofit and rehabilitation. He presented various rehabilitation techniques available in the industry and showcased practical execution at site.Er Sahil Mhatre gave a talk on wireless health monitoring system and showcased 2 case studies where DGC has performed health monitoring services.The conference ended with Dr Gopal Rai who presented on various case studies of building and bridge rehabilitation projects done by DGC using Fibre reinforced polymer, reinforced concrete jacketing and externally bonded steel plates.

Publications For Sale

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2	Workshop on ISO-9001 for Construction Industry	250/-
3	Workshop on- seismic Design of Building – 23^{rd} February,2002	250/-
4	Workshop on Effective Use of Structural Software, 6th March, 2004	250/-
5	One Day Seminar on "Shear Walls In Highrise Building",30th Octobar, 2004	250/-
6	Seminar on "Innovative Repair Materials / Chemicals", 1st October, 2005	300/-
7	Seminar on "Foundations For Highrise Buildings", 23rd September, 2006	250/-
8	Seminar on structural Detailing in RCC Buildings- 26th Ma y,2007	300/-
9	One Day Work Shop on "Pile Foundations", 20th February, 2010	250/-
10	One Day One Day Seminar on “Pre - Engineered Structures", 29th January, 2011	250/-
11	One Day workshop on “Insight into Wind Loading using IS875, Part 3 : 2015”, 27th April 2019	300/-
12	One day workshop on “Structural Health Evalution Vis - A - Vis Prescriptive "Mandatory Format Of Structural Audit” On 18^{th} Jan ,2020	300/-
13	"Performance Based Seismic Design of Buildings" by Er. Vatsal Gokani released on 5th August, 2022	600/-
14	Any ISSE Journal Copy	100/-

Note : Additional courier charges for Mumbai Rs. 50 for outstation Rs. 100).

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