STRUCTURAL ENGINEERING

QUARTERLY JOURNAL OF

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

ISSE

VOLUME 20-3

July - Aug - Sept 2018

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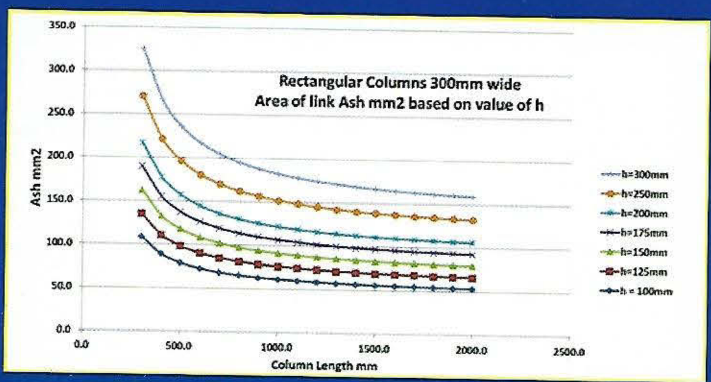

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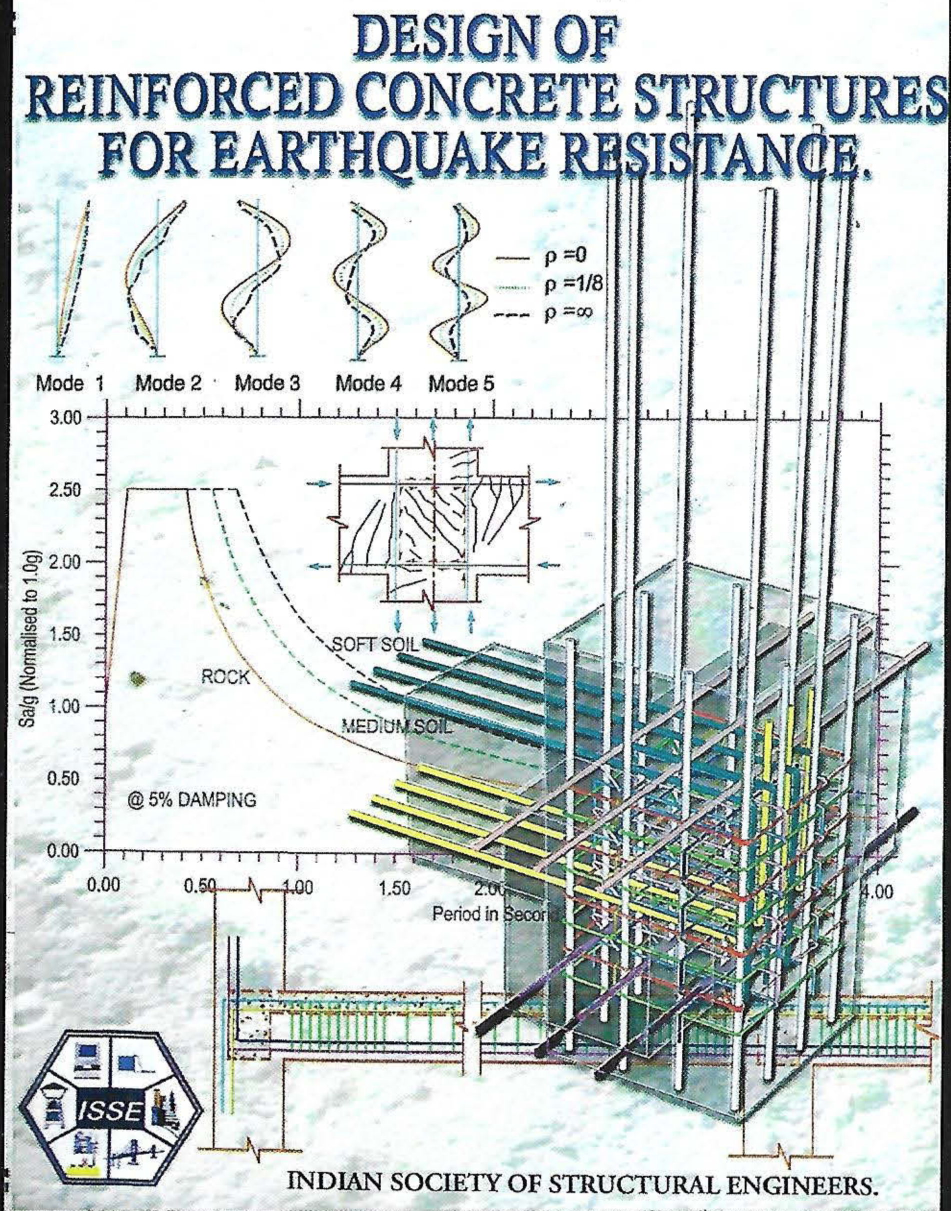
LET US BUILD A STRONG STRUCTURE OF INDIAN SOCIETY



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TO

DESIGN OF REINFORCED CONCRETE STRUCTURES FOR EARTHQUAKE RESISTANCE
by Mr. D. S. JOSHI et al, MUMBAI

The book *Design of Reinforced Concrete Structures for Earthquake Resistance* is written by a team consisting of Mr. D. S. Joshi, Mr. R. L. Nene, Mr. M. D. Mulay, Mr. S. Salgaonkar and Mr. N. D. Joshi. It is published by Indian Society of Structural Engineers (ISSE), Mumbai, in the year 2001. All the authors are structural engineers and are members of ISSE committee for Standards and Codes. The team leaders, viz. Prof. D. S. Joshi and Mr. R. L. Nene are highly experienced and eminent structural consultants.

In the 12 sections that the book has, the subject has been dealt with very ably from the point of view of structural consultants and the book aims to provide in a consolidated form, information available in India and abroad. The book discusses mainly the IS codes by pointing to salient features concerning the seismic design with figures, pictures, details and references along with the filtered recommendations by the ISSE as a summary. Starting from a section on causes of earthquakes in the context of Hindu Mythology, the book covers in its various sections, the anatomical aspects of structural framing system, principles for determining design earthquake forces, the clauses in the relevant IS and other codes, important aspects of ductility, stiffness, strength and capacity design of buildings to resist earthquakes. There is one section called 'open forum' where various questions commonly asked by both public and technical persons on various aspects of earthquake design, have been dealt with explicit answers. There is a noteworthy section which includes a practical designing example of an eleven storey residential RC building, giving step-by-step procedure with explanations, covering all aspects, which would benefit particularly the young design engineers.

The purpose of this book has been to present a logical and practical basis for the design of RC buildings against earthquake forces. The book has more than 300 pages and is enriched with more than 250 coloured figures and photographs (with high quality production and printing) to make it interesting and easily understandable.

The Awards Committee and the Governing Council of ACCE confer the **ACCE Nagadi Award 2004 for Best Publication in Civil Engineering to Design of Reinforced Concrete Structures for Earthquake Resistance by Prof. D. S. Joshi et al, on 17th June 2004.**

Chairman Awards Committee	Secretary General ACCE (I)	President ACCE (I)

STRUCTURAL ENGINEERS

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Patrons : 37 Organisation Members :22 Sponsor : 8

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TOTAL STRENGTH 1792

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- Materials Technology, Ferrocement
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- Rehabilitation of Structures
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- Geo-Tech & Foundation Engineering
- Environmental Engineering
- Non Destructive Testing
- Bridge Engineering

& Other related branches

1. To restore the desired status to the Structural Engineer in construction industry and to create awareness about the profession.
2. To define Boundaries fo 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.

Gem 17 : Prof. Z.S. Makowski-
Pioneer of Space Structures

Dr. N. Subramanian
Er. Vivek G. Abhyankar

Prof. Z.S. Makowski (April 15, 1922 – Nov. 5, 2005) – A pioneer of Space Structures (a) in Polish Armed Forces uniform (1945) and (b) At Imperial College (1951)

EARLY LIFE AND EDUCATION

Zygmunt Stanistaw Makowski was born on 15 April 1922 to Juliusz August Makowski and Helena Makowska in Warsaw, Poland. Young Makowski had his early education at the Tadeusz Czacki Grammar School, Warsaw and started his higher education at Warsaw Technological University in 1942. He continued his higher education in 1943 in the Civil Engineering Department of the Technological University of Warsaw. However, the outbreak of World War II made it impossible for him to continue his education. At the time, Poland was an occupied country. During 1942-1944 Makowski took part in the resistance movement and in the Warsaw uprising of the Polish underground army and also took lessons in secret sets organized by the Warsaw Polytechnic. After the defeat of the uprising, he was taken as a prisoner-of-war camp at Sandbostel Germany (near Frankfurt-am-Main) until six weeks before the end of the war. He was freed by the United States troops, but still eager to free his country, he went to Italy and joined the second Polish Corps under General Anders.

After the war, he started engineering studies, initially at Reale Universitá degli Studi di Roma in 1946. However, he did not complete his study there and resumed his university studies in the fall of 1946 in the Polish University College, London. He received his Dipl. Ing. with distinction on 30 January 1950. He was then offered a lecturer position at Imperial College in London, where he also worked on his doctorate. The subject of his thesis was the analysis of the existing Dome of Discovery, erected for the 1951 festival of Britain Exhibition. His thesis was entitled '*Theoretical and Experimental Stress Analysis of Braced Domes*', and supervised by Prof. A. J. S. Pippard. In the pre-computer days, such an analysis was a significant accomplishment. This research was the seed for his life-long interest in space structures. Makowski received his PhD from University of London on 20 July 1953. He also received his DIC on 14 October 1953 from Imperial College.

ACCOMPLISHED TEACHER AND RESEARCHER

Z.S. Makowski started his academic career with the Polish University College, London as an assistant lecturer (1949–1951). In 1951, Prof. Pippard invited young Makowski to join his staff at the Imperial College of Science and Technology, London. There, he joined as an assistant lecturer and one year later was promoted to the position of lecturer (1952–1958), and then became a senior lecturer (1958–1962). Figure 2 shows him in a laboratory at the Imperial College, during 1961.

Fig 2 Prof. Makowski in a laboratory at the Imperial College, London during 1961

Professor Makowski then went to Battersea College of Advanced Technology and worked as the Head of the Department of Civil Engineering during 1962-1966 and then as Head of the Department at the University of Surrey from 1966 to 1984. Under Prof. Makowski's able guidance, the Department achieved many distinctions in various areas especially in the fields of structures, geotechnics, materials, and fluid mechanics. He was elevated as the Dean of Faculty of Engineering at the University of Surrey and served in this position thrice during 1966–1968, 1976–1979 and 1984–1987.

Note: Polish University College was a predecessor of Battersea College of Advanced Technology which, in turn, was the predecessor of the University of Surrey.

Space Structures Research Centre (SSRC)

Professor Makowski established the Space Structures Research Centre (SSRC), a very unique and only one of its kind in the world, in May 1963. In addition to this center he managed the Department and Laboratory of Engineering Structures. From the beginning, he was very active in the activities of SSRC and initiated research to promote the utilization of spatial structures. Figure 3 shows him at SSRC during 1997. For his activities at SSRC, He could be called the father of Spatial Structures. The main areas of activity at SSRC consisted of conducting R & D on space structures such as domes, barrel vaults, single, double and multi-layer grids, and frame foldable plates. In addition, research was undertaken on 3-D towers and tension structures. As a result, the methods of analysis, design, and understanding the behaviour of such structures were improved (Behnejad and Parke, 2014). During his academic life, Prof Makowski supervised 17 Ph.D. students and several Master's students at the University of London, Battersea College of Advanced Technology, and the University of Surrey. In 1972, the SSRC acquired a building for its headquarters. The building was roofed by a double layered Nodus grid and was donated by the British Steel Corporation (see Fig. 4). In 2018, SSRC is still continuing the work started by Prof. Makowski in 1963 to study the behaviour of spatial structures, with additional emphasis on structural morphology, with several PhD students engaged in doing research on various aspects of space and spatial structures. The SSRC is now a part of the Department of Civil and Environmental Engineering of the University of Surrey.

Fig 3 Prof. Z.S. Makowski at the Space Structures Research Centre, 1997 (source: Archives, University of Surrey)

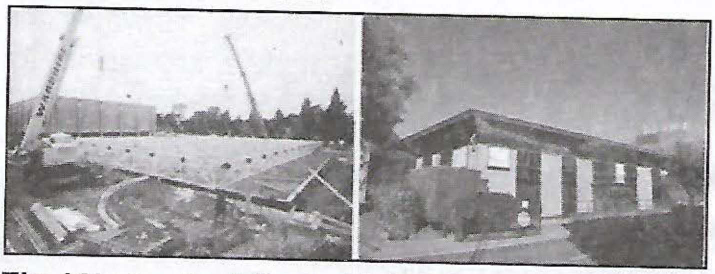


Fig 4 Nodus building which was donated by the British Steel Corporation to SSRC (a) During Construction, (b) After completion

Professor Makowski also served as the external examiner of several Ph.D., M. Phil., and D. Sc. candidates from a number of universities in the U.K., École polytechnique fédérale de Lausanne, Vrije Universiteit Brussel, Université de Montpellier, Université des Sciences et Techniques du Languedoc (Montpellier II), University of Nairobi, University of Sambalpur, IIT- Madras, University of Melbourne, and University of Sydney (He was an external examiner for the Ph.D. thesis of the senior author on Steel Frame Folded Plate Roofs).

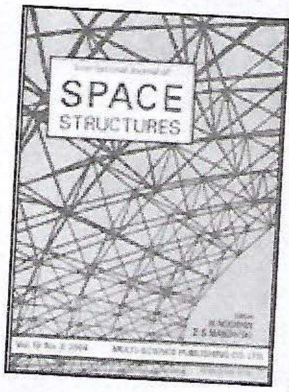
He was extremely well-known through his lectures and seminars on space structures, conducted at numerous universities all over the world. He was invited to give lectures not only at departments of civil or structural engineering, but also at schools of architecture.

INTERNATIONAL CONFERENCES ON SPACE STRUCTURES

Professor Makowski realized that it is important to disseminate the research work done on space structures in his research center and throughout the world, for the benefit of other researchers as well as practicing engineers. As there was only one journal [Quarterly *Journal of the International Association for Shell and Spatial Structures (IASS)*], he organized a series of International Conferences on Space Structures organized through SSRC. He was also a member of various organizing committees, and advisory board of liaison committees, which conducted major international symposia on space structures, as he had extensive experience in organizing conferences and residential short courses.

The first International Conference on Space Structures was conducted at SSRC in 1966 and was a grand success, as 700 participants from 44 countries attended it. The huge volume of the proceeding of this first conference was edited by R.M. Davies and published by Blackwell Scientific Publications, Oxford and Edinburgh. The second third, fourth, and the fifth conferences were held in the SSRC every nine years after the first (till the death of Prof. Makowski) i.e., during 1975, 1984, and 1993 and edited by respectively by W.J. Supple, H. Nooshin, and G.A.R. Parke and C.M. Howard. During these conferences several pioneers of space structure development were also honoured. The last conference in this series was combined with an IABSE-IASS event in London, 2011 and the proceedings were edited by G.A.R. Parke and Disney. These proceedings are recognized as major contributions to the development of space structures technology and architecture. More than 400 engineers from 51 countries attended the 2nd conference, over 500 engineers and architects from 63 countries attended the 3rd , 400 participants attended the 4th and the 5th was attended by over 400 engineers and architects (Behnejad and Parke, 2014).

INTERNATIONAL JOURNAL OF SPACE STRUCTURES



As already mentioned, at that time there was only one journal (Quarterly *Journal of IASS*) dealing with space structures. Hence, Prof. Makowski mooted the idea of bringing out another *International Journal of Space Structures*, and the first issue was published by Multi-Science Publishers in 1985. Professors Makowski and H. Nooshin were the founders and editors of the Journal for the first 21 years. In 2006, Rene Motro and John Chilton took over as editors. Later on in 2008, Prof. Motro carried on with the editorship alone. As Prof. Motro was elected as the President of IASS, Olivier Baverel and Bernard Maurin became editors. Currently the journal is being published by Sage Publishing with Prof. Sigrid Adriaenssens of the Princeton University, USA and Prof. Olivier Baverel of the Ecole des Ponts ParisTech, France as joint editors , with the senior author of this article serving in the editorial board from 2007

[journals.sagepub.com/home/sps].

PROFESSIONAL RELATIONSHIP WITH ARCHITECTS AND INDUSTRIAL DESIGNERS

Prof. Makowski had always placed special emphasis on the professional relationship between engineers, architects, and industrial designers. He had contacts with the late Dr Max Mengeringhausen, who developed the MERO jointing system for skeletal space frames and his successor Dr H. Eberlin (now the company is called MERO-TSK International GmbH & Co. KG). He had a close friendship for decades with architect Stephane du Château (1908–1999), who had designed several space frames, and discussed du Château's projects in several of his publications.

Prof. Makowski is seen with architect Stephane du Château and Dr J. S. Blair, of Stewarts and Lloyds Ltd, in Fig. 5. Moreover, in 1971, due to his recommendation, an Honorary Degree was awarded to Monsieur du Château by the Vice-Chancellor of the University of Surrey (see Fig. 6). The Italian Pritzker Prize-winning Architect Renzo Piano was also in close touch with Prof. Makowski and visited SSRC several times (Behnejad and Parke, 2014).

For a number of years Professor Makowski was a member of the Engineering and Technology Advisory Committee of the British Council, as well as a member of Council of the SEFI (European Society for Engineering Education).

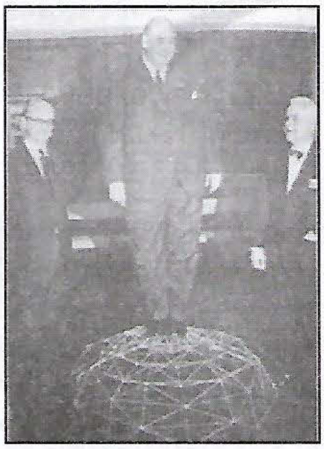


Fig 5 'Practical' test of a wire model of a Double Layer dome by Dr J. S. Blair, of Stewarts and Lloyds Ltd, with Prof. Makowski and du Château during a conference in Corby, UK, 1965

Fig 6 Stephane du Château (receiving an Honorary Degree), Prof. Z. S. Makowski and Dr. D. M. A. Leggett (Vice-Chancellor of the University of Surrey), 1971 © Archives of the University of Surrey

Awards and Honours:

Prof. Makowski received many awards and honours, some of the more important ones are listed below:

1. In 1971, he received a Special Prize from the Institution of Structural Engineers for his design of the Heathrow Jumbo Jet Hangars, and in 1972 a prize from the British Steel Corporation and the British Constructional Steelwork Association – the judges described the hangars as 'superb pieces of engineering design, integrating structure and services'.

2. In 1974, he received the Golden Wing Prize from the International Club of Plastics Experts in recognition of his contribution to research on the structural applications of plastics.

3. In 1977 he received the Queen's Jubilee Medal for his work on Space Structures.

4. In 1980 he was admitted into the Freedom of the City of London.

5. In 1982, in recognition of his contribution to the advancement of Science and Technology, he was awarded the Fellowship of the City and Guilds of London Institute.

6. In 1983, he was elected as a fellow of the Fellowship of Engineering.

7. In 1986, Professor Makowski received the Honorary Membership of the International Association for Shell and Spatial Structures (IASS) during the Association's symposium in Osaka, Japan. During the ceremony, the President of the IASS at that time, Prof. Yoshikatsu Tsuboi, said that the conferment of the IASS Honorary Membership to Prof. Makowski is a formal recognition of the leading role he played over the years in the field of Space Structures.

8. In 1989, Warsaw University of Technology (WUT) awarded an Honorary Doctorate to Professor Makowski.

Since 1989, *Z S Makowski Prize* is instituted and being awarded to the best overall performing student in the final year of the first degree in the Department of Civil and Environmental Engineering of the University of Surrey.

DESIGNER OF NOVEL STRUCTURES

Prof. Makowski used the results of his research and scientific experience while designing numerous buildings and structures. He acted as consulting engineer to several projects, carrying out structural analysis and design of domes, barrel vaults and double layer grids in steel, aluminium and glass fiber reinforced plastics. As a part of his consultancy, he checked the analysis of the Astrodome in Houston, USA, shown in Fig 7. He developed a novel form of light-weight stressed-skin aluminium pyramidal space grid system which has been used in several structures in the UK and abroad. The most notable example of this stressed-skin system is the aluminium roof covering of the headquarters building for the Congress of the International Union of Architects erected in London, 1961 (see Fig. 8). Aluminium pyramids, 2.4 m square by 2.1 m high and made of 20 gauge aluminium sheet are used in this structure. One of these stressed-skin units was tested by Makowski, with students H. Nooshin and D. Sarna under gravity load at Imperial College, London, in 1962 (Prof. Nooshin, later became a close colleague and friend of Prof. Makowski).

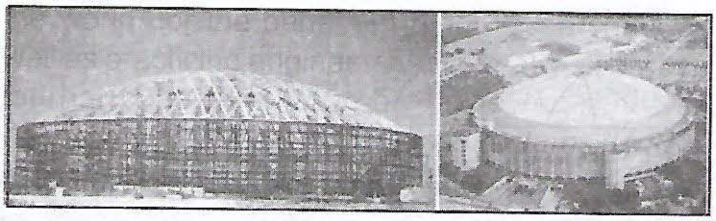


Fig 7 Astro dome, Houston USA (a) during construction, (b) After completion [www.ballparksofbaseball.com]

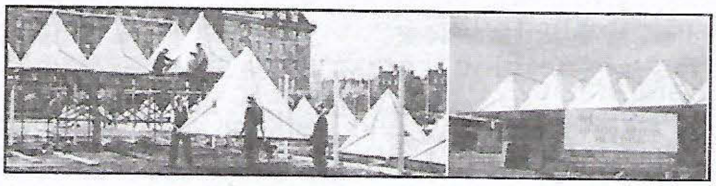


Fig 8 Headquarters building for the Congress of the International Union of Architects, London, 1961 © Architectural Press Archive / RIBA Library Photographs Collection

At SSRC, Prof. Makowski, with his colleagues, was involved in the design of several large span structures. The more notable examples include (1) The suspended prestressed cable network roof structure over the Farahabad Sports, Stadium (Takhti Stadium) in Tehran, Iran (together with H. Nooshin and J. W. Butterworth), which is shown in Fig. 9(a) [An experimental investigation was carried out to predict the behaviour of this roof at SSRC as shown in Fig. 9(b)], (2) Complex of five exhibition pavilions in Nancy, France, covered with prefabricated double layer grids, (3) The three-way grid over the King Hussein Sports Stadium in Amman, Jordan (together with H. Nooshin), (4) The space frame over the Spanish Government Hydraulic Research Laboratory in Madrid, Spain (together with H. Nooshin), (5) the Unibat double layer grid over the freight terminal for the RAF at Mildenhall, UK, (6) The huge dome covering of the indoor Indraprastha Stadium (Indira Gandhi Stadium) in New Delhi, erected for the IX Asian Games 1982 (He acted as a consultant to the Engineers India Ltd., and together with P. Mullord, was involved in the analysis, design, and construction of this structure). This steel structure has a clear span of 150 m and, at that time, was the largest of its kind in India (see Fig.10), and (7) The aircraft hangars at Heathrow Airport in London.

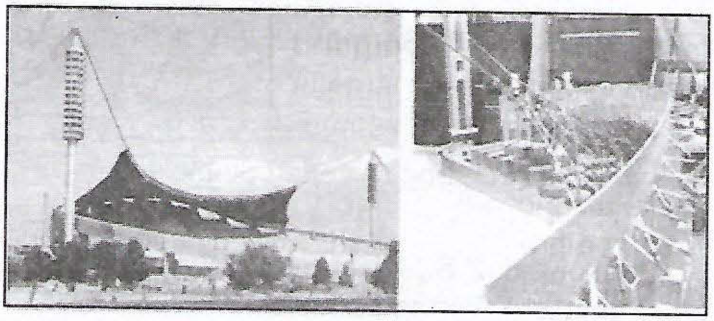


Fig 9 (a) Takhti (Farahabad) Sports Stadium in Tehran, Iran, opened in 1974 (© Omidali Samavati) and (b) Model of the prestressed cable roof of this Stadium tested at the Space Structures Research Centre

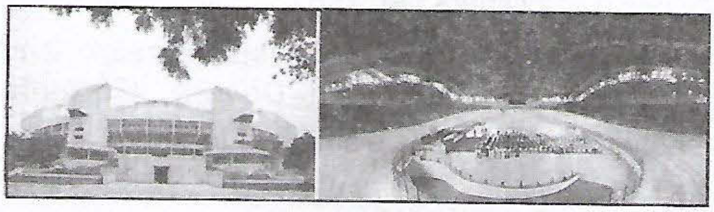


Fig 10 Interior and Exterior views of Indira Gandhi (Indraprastha) Stadium in New Delhi, India, constructed in 1981. © Sambath Kumaar

When Prof. Makowski formed his own consulting firm M/s Z.S. Makowski and Associates In 1968, the company was appointed as the structural consultant to BOAC and was responsible for the construction of two aircraft hangars at the London Airport, Heathrow, covered with prefabricated diagonal steel grids having a clear span of 153 m (See Fig. 11). Different aspects of the structural design and analysis of these hangars may be found in Crook and Makowski, 1972.

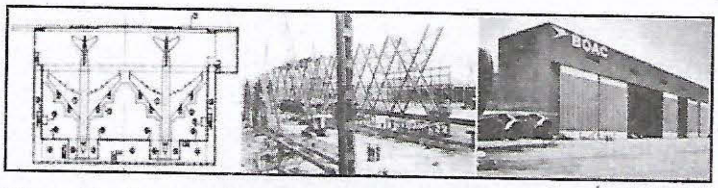


Fig 11 Aircraft hangars at London Airport, Heathrow (a) A schematic plan view, (b) Spine girder and fascia girder during the construction, and (c) View of Entrance

BOOKS AND PAPERS

Prof. Makowski, based on his research and experience, authored over 140 publications, many of them translated into many languages. Some of the notable publications are listed below:

1. Makowski, Z.S., *Räumliche Tragwerke aus Stahl* , (1st ed. in German), Verlag Stahleisen m.b.H., 1963.

2. Makowski, Z.S., *Constructions Spatiales en acier*, (1st ed. in French), Verlag Stahleisen m.b.H., 1963.

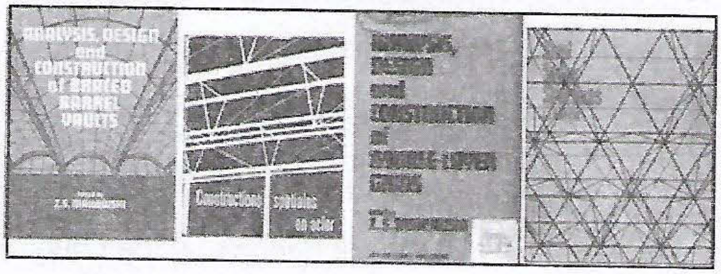
3. Makowski, Z.S., *Steel Space Structures*, (1st ed. in English), Verlag Stahleisen m.b.H., 1964.

4. Bunni, U.K., Disney, P. and Makowski, Z.S., *Multi-layer Space Frames*, Constrado, 1980.

5. Makowski, Z.S., Analysis, *Design and Construction of Double Layer Grids*, Applied Science Publishers Ltd, 1981.

6. Makowski, Z.S., Analysis, *Design and Construction of Braced Domes*, Granada Publishing, 1984.

7. Makowski, Z.S., Analysis, *Design and Construction of Braced Barrel Vaults*, Elsevier Applied Science Publishers Ltd., 1985.



His book *Steel Space Structures* was published in 5 languages and reprinted several times since its original publication in 1963. When Prof. Makowski retired in 1987, a document entitled '*Z S Makowski at Sixty Five*' was published by the University of Surrey. This document contains details of his professional activities, list of his publications, well as a collection of letters received by him. The letters proved to show the multifaceted qualities Prof. Z.S. Makowski: a man of integrity, a ki considerate friend, a leader with a mission creative technologist, and above all, a dedicat preacher of space structures.

As a mark his retirement the book entitled 'Studi in Space Structures' was released and contains number of articles contributed by several of colleagues and friends on various aspects of spa structures, providing a valuable source information to engineers and architects in the fie

EPILOGUE

Prof. Z.S. Makowski passed away peacefully Surrey, U.K. on November 5th, 2005. He leav behind his wife Cecylia Grzesik, whom he marri on 20 October 1951, his 2 sons (Jerzy Zbigne and Ryszard Jan) and 2 daughters (Anna a Barbara).

Acknowledgements

The authors wish to acknowledge the permissi granted by Prof. G.A.R. Parke, Professor Structural Engineering and the Director of Th Space Structures Research Centre, University Surrey, to use photos and material from his pape cited in the references.

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2. G M Crook and Z S Makowski -"Analysis, desig and construction of the space frame roofs for th BOAC 747 hangars at London Airport *Proceedings of the IASS Symposium on Tensic Structures and Space Frames*, Architectura Institute of Japan, Tokyo and Kyoto, 1972, pp. 487 501.

3. Mitchell, J., *The University of Surrey: A Histo of Shaping the Future*, University of Surre Guildford, 2011, 258 pp.

4. Nooshin, H., *Z. S. Makowski at Sixty Five* University of Surrey, Guildford, 1987, 165 pp.

5. Nooshin H. (ed.), *Studies in Space Structures*

Multi-Science Publishing Company Limited, United Kingdom, 1987, 347 pp.

6. Ulmer, A., *Doctors Honoris Causa of Warsaw University of Technology* , Publishing House of the Warsaw University of Technology, Warsaw, 2000, pp. 135-136.

7. Parke, G.A.R. and Behnejad, S.A., "Z.S. Makowski: A Pioneer of Space structures", *International Journal of Space Structures*, Vol. 30, No. 3 &4, 2015, pp. 191-201.

8. "Zygmunt Stanis³aw Makowski - biographical note", Information Technology Center of the Warsaw University of Technology [https://ci.pw.edu.pl/Nauka/Akcja-Biogramy-zasluzonych-profesorow/Doktorzy-Honoris-Causa/Makowski-Zygmunt-Stanislaw]

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Performance-Based Durability Design of Bridges for 120 years

Dr. Sharvil Alex Faroz

Abstract

This article introduces the methodology for durability design of concrete bridges for a working life of 120 years. A change of design approach is recommended, from the conventional "prescriptive approach" practiced by current design standards to "performance-based approach" which considers the actual ageing process through probabilistic treatment. Corrosion of steel reinforcement is considered as the most critical process and subject to performance-based design. The target design-life and durability limit states are attributed to concrete elements according to their structural importance and ease of maintenance. The structure can be designed using a full probabilistic approach for a target probability of corrosion initiation for a life of 120 years. This article is based on experience of the author in dealing with durability assessment of infrastructures.

1 Background

Significant amount of national funds are invested worldwide in the construction of infrastructure required to support society's needs. In India it was forecasted that an investment of about Rs. 65 lakh crores in infrastructure was required for the 12th five year plan (http://planningcommission.nic.in). Such high initial cost of infrastructure justifies the importance of durability for such facilities. Failure to provide an adequate durability will inevitably result in undesired early repairs or even collapses resulting in high economic and public loss. Durability also plays an important role in sustainable development. Sustained bridge will not consume resources unlike new construction and the associated pollution of environment will be avoided (production of 1 tonne of cement clinker leads to the emission of approximate 0.95 tonne of CO_2).

During the past two years we have seen bridge mishaps like the Savitri river bridge collapse, in Mahad Maharashtra (2nd Aug, 2016), bridge collapse near Araria district Bihar (18th Aug, 2017), Gokhale bridge incident in Mumbai Maharashtra (3rd July, 2018) the latest Majerhat Bridge collapse in Kolkata (4th Sep, 2018). These examples show that bridge collapses have become unpredictable due to lack of capabilities with the asset owner in predicting residual-life and degradation modelling of such assets.

New bridge constructions such as the elevated metro rail line are spread through the city and pass from busy residential and commercial centres (Fig. 1). We can not imagine the loss of lives and property caused by an unpredictable collapse of a metro bridge on a busy day. It is most important because these concrete bridges will have collateral damage in addition to the loss due to collapse of the bridge itself.

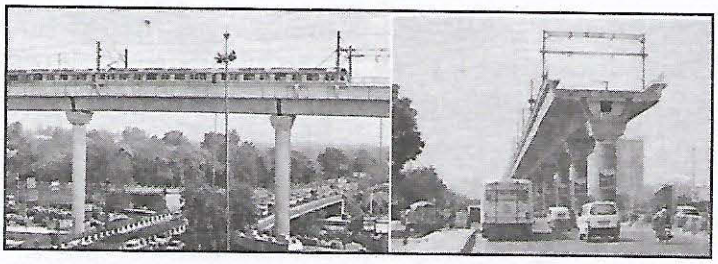

Fig 1 Typical examples of metro rail bridges in India

To avoid catastrophic failures, generally design codes specify a long design service-life for such important infrastructures. For example, IRS CBC:1997 specifies the following life (Table 1) for the concrete railway bridges.

Design life requirement as per IRS CBC:1997

Bridge location	Design life (years)
Sea	50
Coastal area	80
Rest of India	100

However, authorities and designers are ambitious to achieve a life of 120 years even for bridges in sea and coastal regions. The codal provision hardly have any rationale/guidelines for the durability design aspect for such ambitious projects.

2 Concrete Durability

Concrete durability problems arise from the aggressiveness of the environment to which the structure is exposed. The deterioration mechanisms of concrete components under environmental actions include: (1) Chloride-induced reinforcement corrosion, (2) Carbonation-induced reinforcement corrosion, and (3) Physical and Chemical attack of salts in sea water. Furthermore, the internal expansion reactions in structural concrete, e.g., alkali-aggregate reactions (AARs) and sulphate reactions, also affect the long service-life of a project.

Based on the specifications on raw materials, the risks of salt attacks and internal expansion reactions for structural concretes can be made low enough for long-term durability. Accordingly, the carbonation-induced corrosion and chloride-induced corrosion remain the most critical processes for durability design. The main reasons for such durability problems are the interconnected porous nature of the hydrated cement paste and the heterogeneity of concrete properties. Better quality control in the selection of materials and processes on site can control the problems related to the concrete system.

The construction of a structure involves several parties; the owner, design consultants (architects and engineers), concrete producer and contractor. A schematic representation of the parties and their tasks (responsibilities) is illustrated in Fig. 2. These parties are as a chain where any weak link in the chain compromises the durability of a structure.

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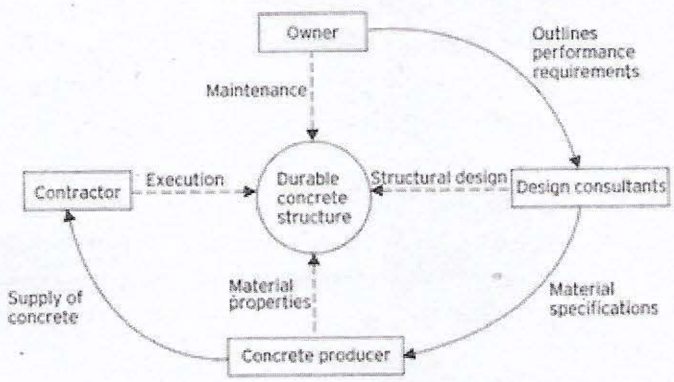

Fig 2 Schematic representation of parties involved in construction of durable concrete structures

Proper channels of communication should exist among the parties involved at all stages of construction to ensure that the main objective of construction of a durable structure is achieved.

In the construction industry, durability is expected to be achieved through strength. But there is no simple or unique relationship between strength and any of the durability parameters. The key to concrete durability is the achievement of a "designed concrete pore structure". The simple way towards this goal is by specifying the concrete accurately for its intended purpose. The specification should address different aspects such as intended service-life, quantifiable description regarding serviceability requirement and failure, acceptable level of risk and possible extent of maintenance.

3 Approaches for Durability Design

Conventionally, the durability design of a concrete structure for a target working life is achieved using a "prescriptive (deemed-to-satisfy) approach". The prescriptive approach, based on acquired experiences and empirical data, provides the requirements for material composition and structural details for given environmental actions and target working lives. This approach is followed by most design codes in use (IS 456:2000, IRC 112:2011, IRS CBC:1997). The provisions include limits of

- Minimum cement content,
- Maximum water cement (w/c) ratio
- Minimum grade of concrete
- Nominal concrete cover
- Maximum chloride and sulphate content

Following these rules the designer could assume that the structure would achieve an acceptable long, but unspecified service life. However, the fulfilment of a particular service-life, if all of these prescription are satisfied, cannot be quantified using this approach. In other word, if all requirements are satisfied, what will be the achieved service-life? To answer such a question it is necessary to incorporate more advanced concepts related to concrete durability, due the need to better foresee and prevent distresses, in particular the corrosion of the reinforcement.

The corrosion process in concrete is highly complex with various physical-chemical interactions among saline solutions, solid phases of concrete and moisture. The complicated nature of the process leads to significant uncertainty when modelling corrosion. In addition there is significant uncertainty associated with some of the parameters which dictate corrosion initiation, such as the time dependent diffusion coefficient, the critical chloride content and the provided concrete cover. The presence of such engineering uncertainty necessitates the adoption of "performance-based approach" for design. The performance-based approach involves modelling the real ageing process of structural materials under environmental actions and employs mathematical models to evaluate the required properties and structural dimensions for expected design lives through probabilistic treatments. These methods allow the uncertainty associated with all levels of corrosion process to be incorporated into the analysis, leading to a robust and informed design. The performance-based approach is an 'engineered approach' to durability design

3 Design Life and Durability Limit State

The asset owner desires a working life of 120 years for a particular metro rail bridge. Aiming for this target, the durability design should first decide the working lives for bridge elements on the basis of their structural importance and technical feasibility. The basis being that the principal elements have the same working life as the whole project (120 years), whereas the secondary or replaceable elements can be shorter. For these elements the maintenance and replacement schemes should be specified in the design phase.

Durability Limit States (DLS) are needed for quantitative durability design using the performance-based approach. These are the minimum acceptable performance levels for different durability processes. For corrosion process, two DLS can be defined: (a) corrosion initiation state, and (b) corrosion to an acceptable extent. IRS CBC:1997 specifies that under external loads the permitted design crack width can be 0.1 mm or 0.2 mm depending on exposure condition, for Reinforced Concrete (RC) elements. Whereas no cracking is permitted for Prestressed Concrete (PC) components. In the metro project, PC elements, principal RC elements, and RC elements with high maintenance difficulty should adopt DLS (a), while secondary RC elements may adopt DLS (b). The stages in life of a corroding concrete element is shown in Fig. 3 for visualising various DLS.

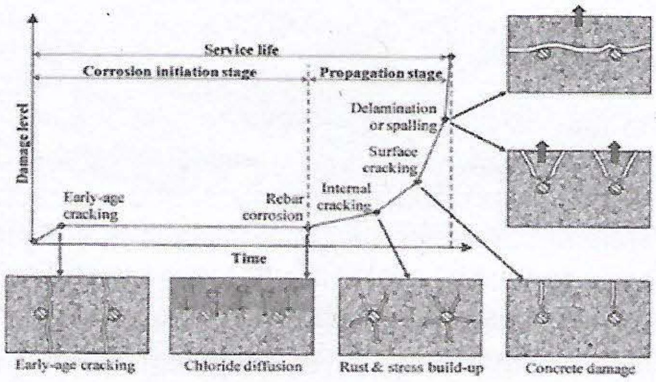

Fig 3 Schematic description of phases in corroding concrete


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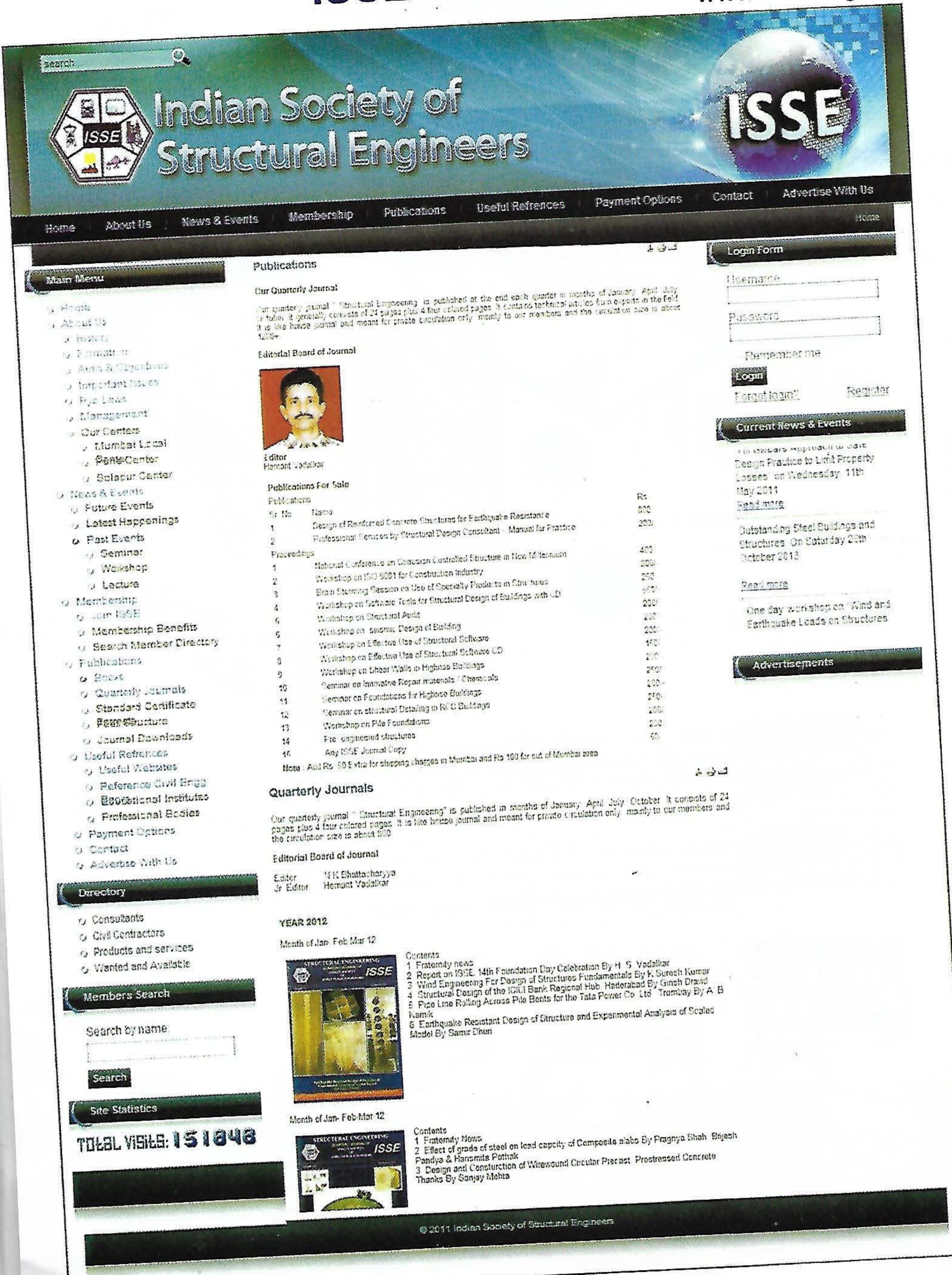

4 Design Methodology

Considering structural design of durability for corrosion processes, the design factors are listed in Table 2 as concrete cover quality (diffusivity, carbonation resistance), concrete cover thickness, and crack control criteria. The concrete cover quality and thickness are designed through the durability models with the target service lives and appropriate DLS adopted. For the performance design procedure, the concrete surface is supposed to be exposed directly to aggressive agents (Cl^-; CO_2). The model-based design follows a reliability analysis format with a target probability of failure, $P_F = 0.1$.

Table 2. Durability Requirements

Requirement	Corrosion	
	Chloride -induced	Carbonation-induced
Material		
w/c ratio	✓	✓
cement type	✓	—
cement content	✓	✓
Cl^- content	✓	—
Structural level		
cover quality	✓	✓
cover thickness	✓	✓
crack control	✓	✓

4.1 Corrosion Model

The design model for chloride-induced corrosion is adapted from the analytical model of diffusion. With the DLS specified as the corrosion initiation state (a), the design equation can be written as,

$$G = C_{th} - C(C_s, x_d, D_c, T_{SL}) = 0$$

The objective being to obtain cover thickness (x_d) and specifications of the concrete such as the chloride diffusion coefficient of concrete (D_c) for the specified design life (T_{SL}) subject to

$$P_F = P(G<0) = 0.1$$

C(.) is the chloride concentration function in concrete. This model has four governing parameters

1. Threshold chloride concentration (C_{th})
2. Concrete surface chloride concentration (C_s)
3. Chloride diffusion coefficient of concrete (D_c)
4. Concrete cover thickness (x_d)

4.2 Preliminary Design

These four parameters have significant dispersions for a given exposure condition, and their statistical nature must be taken into account to guarantee a large enough safety margin for the durability design. As mentioned earlier, the target probability of failure is fixed as $P_F = 0.1$ for the design at this stage. Towards this aim, the design is performed by a full probabilistic scheme using directly the statistical properties of the parameters.

The statistical properties of model parameters are analysed on the basis of the long-term in-place structural investigations and exposure tests in vicinity of the proposed project. It is advised to identify these parameters and must be included in the feasibility investigation for the project. The investigation comprises of environmental data extracted from site investigation for atmospheric temperature, humidity, and air-borne chloride content or CO_2 concentration and wind speed. Further, the evaluation of chloride/carbonation profiles of structures in the vicinity of the proposed project or similar projects elsewhere must be incorporated. This investigation helps to statistically characterise the diffusive property of concrete and evaluate the distribution of surface chloride concentration. Since the concrete cover is the only structural parameter in the design equation, correct specification of concrete cover thickness is the central issue for durability design. The statistical properties of concrete cover thickness are important for correct estimation of reliability with respect to the design equation, and are related closely to the construction methods and practice. Statistical analysis based on the data of achieved cover thickness of similar concrete infrastructures must be investigated.

4.3 Design assisted by testing

Based on the preliminary design, the required construction material and specifications for executing the design can be used as design basis for the project. However, every project is unique and precise characteristic of the design are quantified by testing the material used at project site.

During the construction, the properties of structural concretes should be tested in on-site laboratory.

In parallel, the constructed concrete elements, prefabricated or cast-in-place, are inspected for the achieved quality, and particularly for the thickness of concrete over. These data provide the information on the realistic construction quality of concrete elements, thus help to update the statistical properties of parameters in the durability assessment models. This updated data source, form the essential basis for predicting the durability performance of concrete elements during its service-life.

The in-situ data of chloride diffusion coefficient of structural concretes are to be collected from the on-site laboratories for different concrete elements. The chloride diffusion coefficients are measured on structural concretes under standard conditions by rapid migration method. The measurements of concrete cover thickness for the cast-in-place concrete elements are done either through, ground penetrating radar, electromagnetic test, ultrasonic and radiography. The critical threshold chloride concentration is evaluated by the accelerated chloride threshold testing.

Additions of admixtures such as ground granulated blast furnace slag, fly ash, silica fume and metakaolin alter the properties of ordinary Portland cement concrete. Testing of such concrete is crucial to study its chloride ingress property and statistically characterise it to apply in the performance-based design.

4.4 Monitoring and Maintenance Planning

The maintenance planning is to establish the techniques and intervention periods of maintenance, on the basis of the durability states of the concrete elements. The strategies of maintenance planning is to be preventive/proactive, which refers to the intervention at early stage of deterioration, normally at low maintenance costs. During the service-life, the deterioration processes will be monitored via periodical inspection and sensors. The maintenance actions are to be taken at early stage of deterioration for elements with the help of these inspections and monitoring. Since concrete elements are designed in such a way that the probability of corrosion initiation ($P_F = 0.1$) will only be exceeded after 120 years, technically all elements can be exempted from maintenance during the service-life. However, given the uncertainty associated with the concrete construction, e.g. early-age cracking, unintended lower concrete cover, accidental use of saline water for concrete mixing and/or curing and the unexpected environmental actions during service-life, e.g. the global warming and long-term change of ambience, a basic maintenance planning is necessary for concrete elements. Through monitoring of as-built concrete components it is possible to establish the service-life of the casted components in real-time.

The basic maintenance planning considers mainly two aspects: the durability performance monitoring, and maintenance of the elements. Monitoring involves: potential mapping, resistivity mapping, embedded anode sensors, cover thickness measurement, air permeability, chloride profile, etc. The maintenance scheme consists of performing the surface chloride extraction by electrochemical method or a cathodic protection system can be installed to protect the steel bars against the unexpected durability failure. It should be noted that this basic maintenance scheme is to interact with the durability inspection/monitoring data and the real-time durability assessment during the service-life. A predictive maintenance scheme is setup for the same.

4.5 Summary

In essence the performance-based approaches is fundamentally a measurement and verification design procedure. Fig. 4 provides a schematic of this performance-based approach.

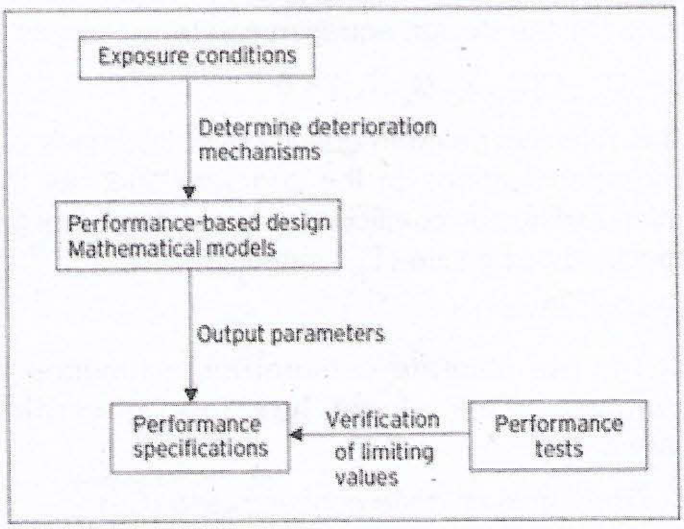

Fig 4 Schematic representation of the performance-based approach

5 Inclusion in Tender document

In the present scenario, asset owners of metro rail bridges specify the conventional prescriptive requirements as per code in the design basis report. However, they wish to know how much really is the achieved service-life. It is necessary that the change should begin at tendering stage of the project.

The tender document should mandate the requirement to study the durability factors and incorporate in the feasibility exercise, to evaluate all the environmental parameters responsible for degradation, including review of previous statistics for similar projects elsewhere. This will be the design basis for the preliminary performance-based design. The asset owner must emphasise on specific durability limit state as per their requirement and mention the expected service-life and target probability of failure. Testing of in-situ components along with the magnitude of tests to be performed are necessary part of the design basis. The asset owner must press for the requirement of probabilistic durability design in the tender document with a design report. And finally long term monitoring and updating of actual service-life must be included as part of the contract.

6 Conclusion

There is massive capital investment on infrastructure in our country. To safeguard these investments and avoid costly repairs and renovation due to premature deterioration of RC structures, it is prudent to verify the properties of concrete cover (its penetrability and thickness). This technical article recommends the change of approach to the asset owners/concrete industry from traditional prescriptive approach to the performance-based specifications. It highlights important steps involved in the durability design of concrete components. This article emphasises on testing of actual in-situ condition for giving the actual project specific performance. It encourages using monitoring of the project after construction to keep a track of effects of the ever changing ambience on the bridge to evaluate the service-life in real time. It is recommended to bring about a change of approach from the tendering stage itself.

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Review of Special confining links in column as per IS13920-2016 and ACI318-2014

By Hemant Vadalkar

From the past earthquake records and observations, it has been seen that many column failures during earthquake occurred due to buckling of main bars near beam column junction as there was no adequate lateral restrain. Even during the structural audit and repair works, it has been observed that confining reinforcement has not been provided in the beam column junction of RCC frames. Either in the design or during construction, some how, the confining reinforcement is missing in the junctions of beam and column and near the joints. For anticipated ductile behaviour of framed structures, confining reinforcement plays very important role and can't be ignored. The diameter of confinement reinforcement, its spacing and detailing of joints must be paid special attention for improving performance of structure during earthquake. We need to create more awareness amongst structural engineers and site engineers regarding this important provision in the code.

For ductile design and detailing of reinforced concrete structures, calculations of special confining reinforcement as per clause 7.6 of IS13920:2016 is required.

A) Factors affecting quantum of confinement reinforcement : There are various parameters that affect the amount of confinement reinforcement as cited in reference 3 like -

1.**Ratio of concrete strength to tie strength fck/fy** : With the increase in concrete strength, more confinement steel is required.

2. **Axial stress in concrete** : Other codes consider this parameter in calculating confining steel. IS code does not account for this effect in the equation for confining reinforcement. However, IS13920 clause 7.1 puts cap on maximum axial stress in column Pu/fck. B.d to 0.4. This may be to avoid crushing failure.

3. **Unconfined cover concrete thickness** : This is considered through Ag/Ak parameter where Ag is gross concrete area and Ak is area of confined concrete core.

4. **Longitudinal reinforcement and its spacing** – IS code does not account for this effect.

5. **Curvature ductility factor** – Amount of confining reinforcement in plastic hinge zone of columns has significant effect on curvature ductility as suggested by many researchers.

American code, New Zealand code, Canadian codes consider effect of axial stress in column and higher grade of concrete in calculation of confining reinforcement.

With some sample examples, comparison of confinement reinforcement in columns with IS13920:2016 and ACI318-2014 has been made.

Equating two equations given in IS code for confinement reinforcement Ash for rectangular links

$Ash /(Sv. h) = 0.18 * fck/fy *(Ag/Ak -1)$ ...... Eq (1)

or $Ash /(Sv. h) = 0.05 * fck/fy$ .... Eq(2) whichever is higher

equating the equations, $Ag/Ak >= 1.27$

It means second equation will govern the area of confinement links for large column sizes where ratio of $Ag/Ak<1.27$

As the ratio of Ag/Ak is more than 1.27, it indicates effective core concrete area is less and column will not be efficient to resist the load after loss of cover concrete. Thus, the area of link is more for higher ratio of Ag/Ak.

IS code defines Ash as cross section of bar forming link where as ACI code defines volumetric ratio of area of links along each face of column considering concrete core dimension and spacing of links.

ACI : $Ash / (s. Bc) = 0.3 * (f'c/fyt) (Ag/Ac -1)$ or $0.09 (f'c/fyt)$ whichever is higher.

Writing the equations in IS code on similar lines of ACI code by replacing h by Bc

IS : $Ash /(s. Bc) = 0.18 * fck/fy *(Ag/Ak -1)$ or $0.05 * fck/fy$ whichever is higher

Where f'c is cylindrical compressive strength and fck is cube compressive strength.

By substituting $f'c = 0.8 * fck$ in the expression of ACI code the factor will be $0.3*0.8 = 0.24$.

Thus , it can be seen that confinement steel as per ACI is around 33% more than that given in IS (0.24/ 0.18 = 1.33) if volumetric ratio is compared from the above equations.

ACI also considers effect of axial stress ratio or higher grade of concrete in calculation of confinement steel. If the axial stress ratio $Pu/(Ag*f'c) > 0.3$ or $f'c>70$ Mpa, area of hoops is further enhanced based on the number of longitudinal bars and concrete strength factor and given by expression $Ash /(s. Bc) = 0.2*kf*kn*Pu/(fyt*Ach)$. This is not considered in the present IS code.

B) Effect of Ag/Ak ratio for various column sizes :

It is interesting to see the effect of Ag/Ak ratio for different column shapes. Assuming clear cover to links as 40mm, it can be seen that for square columns of size more than 700mmx700mm, the ratio of Ag/Ak < 1.27, thus second equation of IS code governs the area of link bar. For column size less than 700x700mm, first equation governs. The trend of Ag/Ak for square columns can be seen from Fig 1.

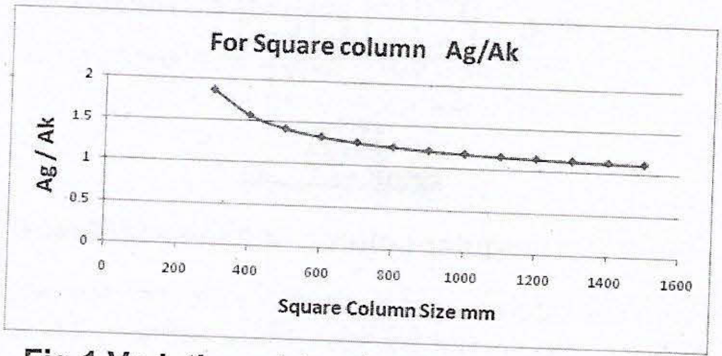

Fig 1 Variation of Ag/Ak for square columns

Considering minimum lateral dimension as 300mm as per latest code, variation of Ag/Ak can be seen for walls of 300mm thickness and 400mm thickness from Fig2. It can be seen from the figure that the ratio does not go below 1.27 though it reduces as the wall length is increased. Since the ratio of Ag/ Ak does not fall below 1.27, equation 1 as given in IS13920 will always govern for confining reinforcement for all the walls.

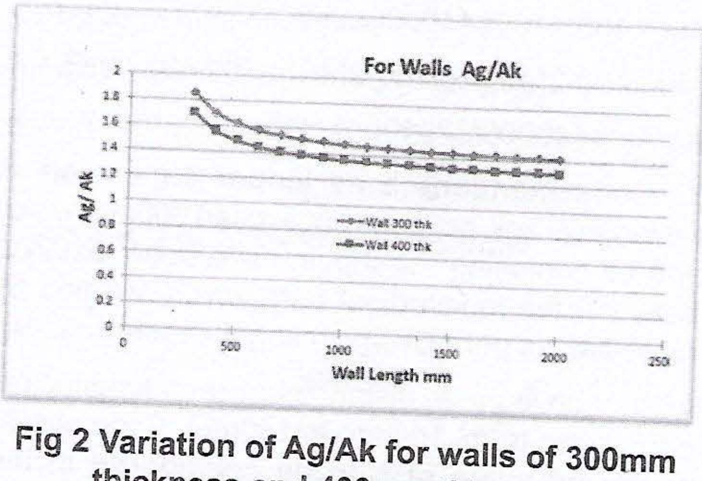

Fig 2 Variation of Ag/Ak for walls of 300mm thickness and 400mm thickness.

c) Sample calculations for confining reinforcement as per IS and ACI code for different columns

Case 1) Column of dimension 400mm x 1100mm

Consider following column details as shown in Fig3.

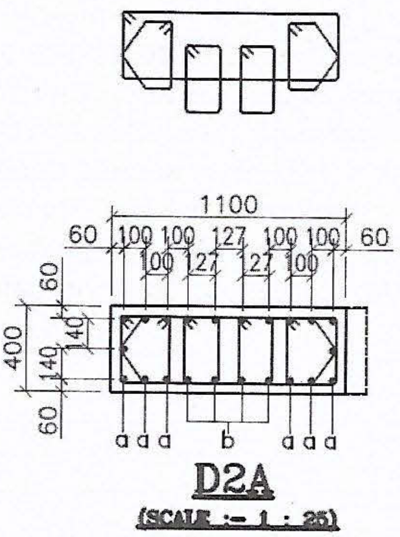

Fig 3 Rectangular column 400mm x1100mm

From consideration of ease of concreting, spacing of links should not be less than 100mm. Thus, considering spacing of links as 100mm. As per the equation given in code IS13920:2016 Clause 7.6

Ash area of bar forming link is given by higher of

Ash = 0.18 * Sv * h * fck/fy *(Ag/Ak -1)

Or = 0.05 *Sv * h * fck/fy

Considering M35 concrete, Fy=500 Mpa

Sv = 100mm minimum (considering ease of placing of concrete)

Ag = 400*1100=440000mm2

Ak= (400-2*40) * (1100-2*40) = 326400 mm2

Ag/Ak =440000/326400=1.348

IS Code mentions h as longer dimension of rectangular link but does not exceed 300mm which is a bit confusing. It should have been distance between the longitudinal bars which are tied by cross link as per ACI which sounds more rational.

In the above case, distance between longitudinal bars varies from 100mm to 140mm. Code should clarify on value of h to be considered in the calculations. If the main bar distances are variable for a given column bar arrangement, whether one should consider average of all the bar distances for calculations of h which is more logical.

h(average) = (2*140+3*127+6*100) /10 = 126mm

Value of Ash is influenced by value of h.

Ash = 0.18*100*h*35/500 * (1.34-1) = 0.43h

For different values of h, value of Ash is as below –

For 100mm Ash = 43.8mm2 (8mm link is Ok)

For 126mm Ash = 54mm2 (10mm link is required)

For 140mm, Ash = 61.3mm2 (10mm diameter of link is required)

For 200mm Ash=87.6mm2 (12mm link is required)

For 3000mm, Ash =131 mm2 (16mm diameter link is required.)

In short, we will not be able to use 8mm links any more in most of the cases.

Calculations checked with ACI318-2014 provisions –

As per Table 18.7.5.4

For rectangular hoop Rst = n * Ab /s. Bc = 0.3 (f'c/fyt) (Ag/Ac -1) or 0.09 (f'c/fyt) which ever is higher.

Where n = number of legs of hoop

Ab = area of one link bar

S = spacing of links

Bc = width of column perpendicular to link

f'c = cylindrical strength = 0.8*fck

Calculations to be performed along the width and length of column face and higher value is to be considered.

Considering calculations along the width =400 - 2*40 cover = 320mm = Bc

Number of effective legs of link in width n1 = 2 straight +0.7*2 inclined =3.4 nos

Considering link spacing of 100mm = S

Using above equation

3.4*Ab/(100*320) = 0.3* (0.8*35/500)(1.34-1)

Thus Ab = **53 mm2**, Thus 8mm link is just OK at 100mm spacing.

Considering calculations along the length Bc =1100+2*40= 1020

n2= 8 straight +2 inclined*.7=9.4

Using the same equation

9.4*Ab/(100*1020) = 0.3* (0.8*35/500)(1.34-1)

Thus Ab = **62 mm2.**

Higher value of 62 mm2 is to be considered. Thus, 10mm link is required at 100mm spacings.

Comparing volumetric ratio of links as per both codes -

Volumetric ratio as per ACI Ash/ S. Bc = 0.3 (f'c/ fyt) (Ag/Ac -1) =

= 0.3* (0.8*35/500)(1.348-1) = 0.005846

Volumetric ratio as per IS code Ash/S. Bc = 0.18 * fck/fy *(Ag/Ak -1)

= 0.18*35/500 * (1.348-1) = 0.00438

As per ACI318 , If the axial stress ratio Pu/Ag*f'c > 0.3 volumetirc ratio is governed by

Ash /(s. Bc) = 0.2*kf*kn*Pu/(fyt*Ach) which means , confining reinforcement is increasing with increasing axial stress ratio.

Plotting the values for the same column as given in Figure 4

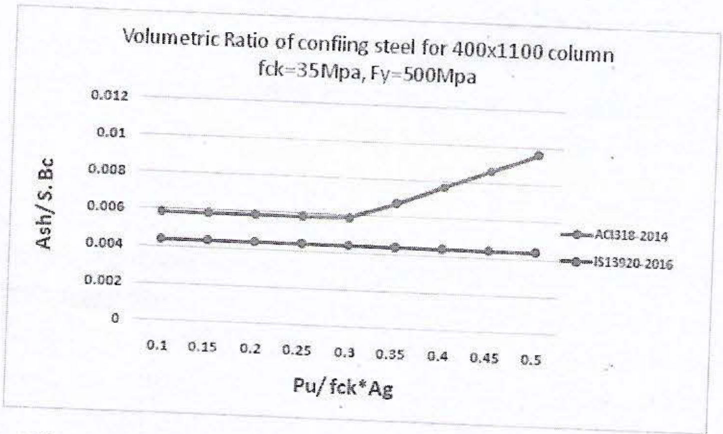

Fig 4 Volumetric ratio of confining steel v/s axial stress ratio in column

Case 2) For 600mm x 600mm column

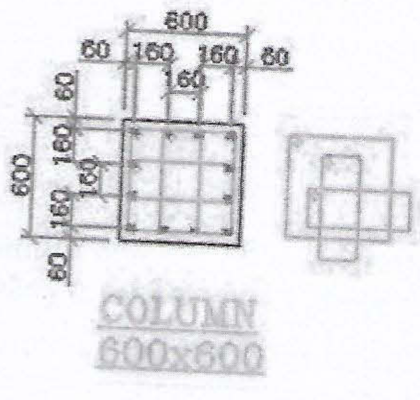

Fig 5 quare column of 600mm x600mm size

Main longitudinal bars 12 nos as shown in t arrangement. The link required can be calculat as

Ag = 600x600 = 360000 mm2

Ak = (600-80)*(600-80) =270400 mm2

Ag/Ak =360000/270400 =1.33

M35, Fe500, Sv=100mm, longer side of li h=160mm+20mm main bar +8 link*2 = 196mm

Area of link Ash = 0.18 * 100*196*35/500 (1.33- = 81.4mm2 (we need more than 10mm diamet as link)

If area of link is calculated considering volumetr ratio as per ACI Concept

h= average spacing of long bar in the width = (600 2*40) / 4 legs of link =130mm av

Area of link Ash= = 0.18 * 100*130*35/500 (1.33 1) = 55 mm2

****

Considering calculations as per ACI318-2014 fo 100mm spacing of links -

n=4

Bc = 600-2*40=520mm

Ag/Ac= 1.33

Using the equation for area of link -

n * Ab /s. Bc = 0.3 (f'c/fst) (Ag/Ac -1)

4*Ab/(100*520) = 0.3* (0.8*35/500)(1.33-1)

Thus , area of link required Ab= 72mm2

Case 2 A For 300x300mm column

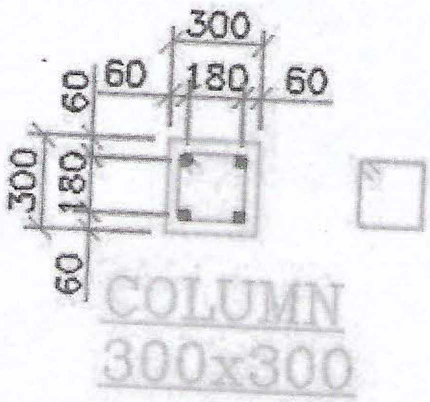

Fig 6 Square column 300mm x300mm

with 4 main bars of 25mm diameter, the link required can be calculated as

Ag = 300x300 = 90000 mm2

Ak = (300-80)*(300-80) =48400 mm2

Ag/Ak =90000/48400 =1.85

M35, Fe500, Sv=100mm, h=300-2*40cover = 220mm

Ash = 0.18 * 100*220*35/500 (1.85-1) = 235mm2 (we need more than 16mm diameter as link ... which is not feasible to provide at site)

If the same calculations are done on volumetric ratio based on ACI concept, i.e. two legs of link in the width of 220mm, effective width of each link = 220/2 = 110mm and Ash=235/2= 120mm2 which sounds more rational.

****

Considering calculations as per ACI318-2014 for 100mm spacing of links -

n=2

Bc = 300-2*40=220mm

Ag/Ac= 1.85

Using the equation for area of link -

n * Ab /s. Bc = 0.3 (f'c/fst) (Ag/Ac -1)

2*Ab/(100*220) = = 0.3* (0.8*35/500)(1.85-1)

Thus Ab= 157mm2

Case 3) For 600mm diameter circular column

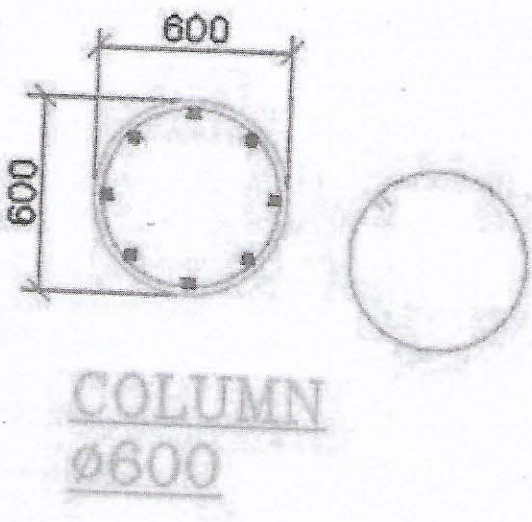

Fig 7 Circular column 600mm diameter

Fck =35 Mpa ; Fy=500 MPa ; Cover 40mm to link and pitch of spiral assumed as 100mm

Ag= 3.142/4 * 600^2 =282780mm2

Dk = 600-2*40 =520mm

Ac= 3.142/4 * (600-2*40)^2 = 212399mm2

Ag/Ac= 282780/212399 = 1.33

As per IS13920-2016

Ash= 0.09* s* Dk * fck /fy * (Ag/Ac -1)

= 0.09 * 100 * 520* 35/500 * (1.33-1) = 108mm2

As per ACI 318-2014

Rst = n*Ab/(S*Dk) = 0.45 *(f'c/fyt)(Ag/Ac -1)

n=4

S= 100mm spacing, Dk=520mm, f'c = 0.8Fck

4*Ab/(100*520) = 0.45* (0.8*35/500) (1.33-1)

Thus, area of hoop Ab =108 mm2 , same as that of IS code calculations.

D) Trends for confining reinforcement as per IS code equations

A graph for value of Ash for circular column can be plotted for various parameters like Fck =35 MPA, Fy= 500Mpa and considering pitch of spiral as 75mm. It can be seen from the graph that the direction of graph changes at 700mm diameter where Ag/Ak=1.27. Beyond 700mm diameter, Ash increases with the diameter of column. More than 10mm diameter hoop will be required.

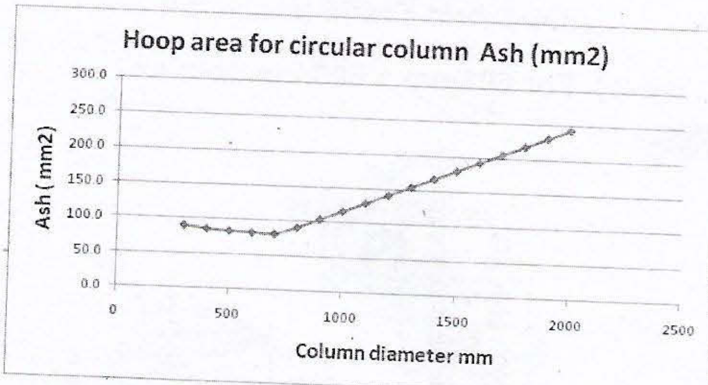

Fig 8 Hoop area for circular column with 75mm pitch, Fck= 35 MPa, Fy=500Mpa

Similarly, trend for confining reinforcement for square column can be plotted considering Fck =35 MPA, Fy= 500Mpa. Area of link Ash will be a function of h (which is the distance between the longitudinal bars but less than 300mm). In IS code it is defined as longer dimension of rectangular link but does not exceed 300mm.

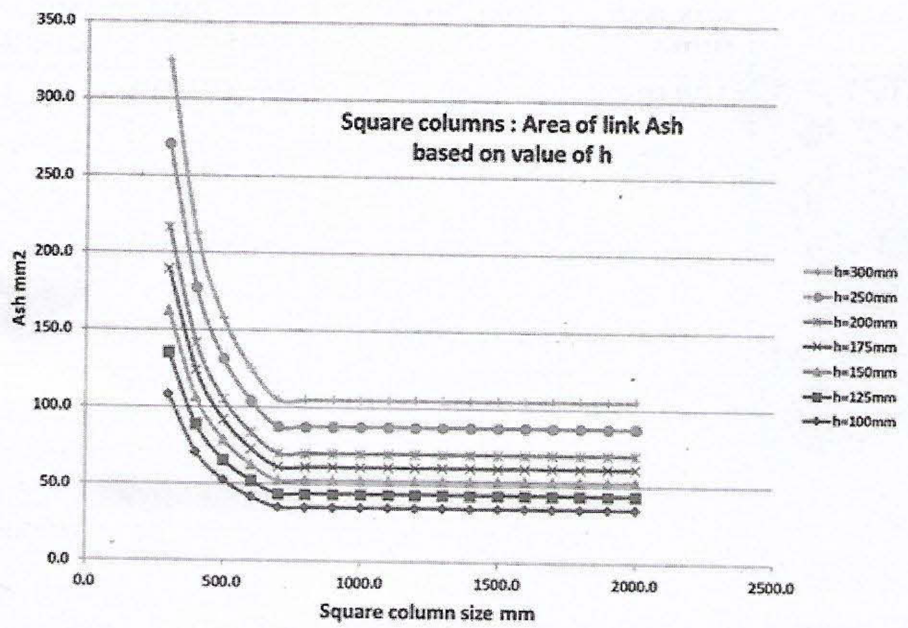

Fig 9 Area of link Ash for square columns with variable h for Fck=35Mpa , Fy=500Mpa, link spacing 100mm

Thus, from the graphs in Fig 9, it can be observed that for the main bar spacing up to 200mm, hoop of 10mm diameter will be required for column size above 600mm and 8mm hoop may be enough if the main bar spacing is less than 150mm.

Trend for confining reinforcement for columns / walls of 300mm thickness has been shown in Fig10 for various values of h (which is the distance between the longitudinal bars but less than 300mm) and for Fck=35Mpa , Fy=500Mpa

It can be seen from Fig10 that 8mm diameter link is not enough for walls. Minimum 10mm diameter link will be required if the spacing of main bars is restricted to 125mm. Higher diameter links will be required if spacing of main reinforcement is increased. Thus, to restrict the link diameter , it is necessary to keep spacing of main bars within 150mm.

For the same cross section area of column (concrete area 0.36m2) and same main steel (around 1.65%) , theoretical consumption of confining reinforcement is compared as per IS code considering various shapes of columns considering parameters like spacing of links as 100mm, cover 40mm, Fck=35Mpa, Fy=500MPa and h as centre to centre distance between main bars. The calculations are presented in Table1 . Similar comparison can be done for various column shapes and bar arrangements.

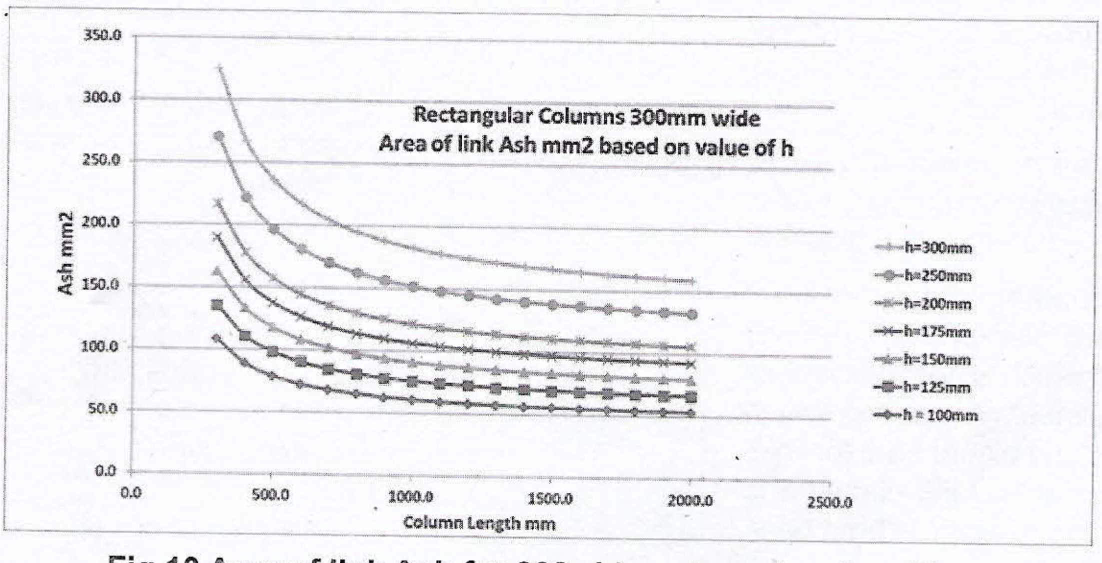

Fig 10 Area of link Ash for 300wide column / walls with varying h for Fck=35Mpa, Fy=500Mpa, spacing of links Sv=100m

Table 1 : Consumption of confining reinforcement in columns.

Sr.no	Column size	Main steel	Area of link Ash mm2	Length of links in m	Weight of links steel Kg / m3 of conc.
1	Circular 677mm diameter	12 nos Tor 25	108 mm2	1.82 m	**44 kg/m3**
2	Square column 600mmx600mm	12 nos Tor 25	67 mm2	5.33 m	**77 kg/m3**
3	Rectangular 300mm x1200mm Main bars on long side	20 nos Tor 20	70 mm2	5.67 m	**96 kg/m3**
4	Rectangular 300mm x1200mm Main bars on long side	12 nos Tor 25	156 mm2	4.77 m	**177 kg/m3**

Order of preference for column shape for having minimum weight of links will be circular, square, rectangular and walls with minimum spacing of main bars. Circular columns will require minimum weight of links and walls will have maximum.

Conclusions :

1. For rectangular and square columns, it is necessary to simplify the IS code equation of confining reinforcement on volumetric basis as given in ACI which leads to more rational calculations.
2. More clarity is required in the definition of h in IS 13920 for calculation of confining reinforcement. In practice, spacing between the main column bars is not constant. To avoid any confusion on longer side of link , h can be considered to be the average distance between the longitudinal column bars which are laterally tied.
3. For circular column, hoop diameter increases with diameter of column. Links of 10mm and higher diameters will be required.
4. In most of the building projects, 8mm diameter links are still being used for columns and walls which is not sufficient as per the code guidelines. We need to switch to 10mm diameter and higher bars for links as per the requirements. Link diameter is governed by spacing of main column bars.
5. For the same cross section of concrete and main steel, weight of links will be minimum for circular sections and will be maximum for walls.
6. It is necessary to create more awareness among structural engineers and site engineers to provide correct confinement reinforcement which will improve performance of our RCC structures.

References :

1. ACI318-2014 Building Code Requirements for Structural Concrete.
2. IS13920-2016 Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces- Code of Practice.
3. Design of confinement reinforcement for RC columns – By Dr. N Subramanian Point of view, Indian Concrete Journal 2011
4. Design of Reinforced concrete Structures – By N. Subramanian , Oxford university Press Published in 2013.

Author

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New and Events during July – Sept 2018

20th July 2018

Inaugural Function of ISSE Student Chapter was held at M H Saboo Siddik College of Engineering, Byculla Mumbai.

ISSE Trustee and editor Mr. Hemant Vadalkar , Mrs. Maanasi M. Nandgaonkar (ISSE Trustee & Treasurer), Mr. Karamchand Savla (ISSE Trustee), Mr. Shantilal H. Jain (ISSE Trustee), Mr. Madhav Chikodi, Mr. Rangnath Satam and Mr. Paresh Unnarkar (ISSE Technical Committee Member) graced the occasion.

Introduction & activities of ISSE were explained by Mr. Shantilal H. Jain.

Saboo Siddik College students were admitted as student members of ISSE. Certificates were distributed by ISSE office bearers. Mr. Hemant Vadalkar welcomed all the new student members and stressed the need of good quality construction for longer life of our structures.

A lecture on Concrete & Green Concrete was delivered by Mr. Paresh Unnarkar on how to make good and green concrete for the better life of our structures.

Prof. Zaheer Khan (Vice-Principal & HOD Civil Engg., Saboo Siddik College) Prof. Ashutosh G. Dabli , Prof. (Mrs) Grace were felicitated on the occasion. ISSE thanked faculty members for their support in forming the ISSE student chapter in their college.

Group photo of new Student Members with ISSE Trustees and Technical Committee Members & M H Saboo Siddik College Vice-Principal.

डीसी रूलमधील त्रुटी दूर करू

राज्यमंत्री पाटील यांचे आश्वासन

म. टा. प्रतिनिधी, पुणे

> राज्य सरकारने जानेवारी २०१७मध्ये विकास नियंत्रण नियमावली (डीसी रूल्स) घोषित केली. मात्र, त्यामध्ये साइट अभियंता, संरचना अभियंता, विकसक, मालक यांच्या निश्चित जबाबदाऱ्यांचा समावेश करण्यात आला नव्हता. या संदर्भात पुणे महापालिकेने सरकारकडे वेळोवेळी पाठपुरावाही केला होता. डीसी रूल्समधील या त्रुटी दूर करण्याची मागणी या बैठकीत करण्यात आली.
> – धैर्यशील खैरे-पाटील, अध्यक्ष, इंडियन सोसायटी ऑफ स्ट्रक्चरल इंजिनीअर्स संस्था, पुणे

पुणे महापालिकेच्या विकास नियंत्रण नियमावलीत (डीसी रूल्स) बांधकामांशी संबंधित घटकांच्या निश्चित जबाबदाऱ्यांचा समावेश करावा, या संरचना अभियंत्यांच्या (स्ट्रक्चरल इंजिनीअर) मागणीला नगर विकास विभागाचे राज्यमंत्री डॉ. रणजित पाटील यांनी सकारात्मक प्रतिसाद दिला आहे. विकास नियंत्रण नियमावलीतील या त्रुटी दूर करण्यास त्यांनी तयारी दाखवली आहे, तसेच नियमावलीत आवश्यक बदलांबाबतचे प्रस्ताव पुणे महापालिकेने सादर केल्यास त्यावर सकारात्मक विचार करण्याचेही आश्वासन दिले.

इमारतींचे बांधकाम सुरू असताना अपघात घडल्यानंतर पोलिसांकडून संरचना अभियंत्यांवर सरसकटपणे सदोष मनुष्यवधाचे अथवा निष्काळजीपणामुळे मृत्यूस कारणीभूत ठरल्याचे गुन्हे दाखल करण्यात आले आहेत. त्यामुळे संरचना अभियंत्यांमध्ये भीतीचे वातावरण निर्माण झाले आहे. त्याबद्दल इंडियन सोसायटी ऑफ स्ट्रक्चरल इंजिनियर्स संस्थेतर्फे (आयएसएसई) सरकारस्तरावर पाठपुरावा केला जात होता. 'महाराष्ट्र टाइम्स'ने यासंदर्भातील वृत्त प्रसिद्ध केले होते. त्या पार्श्वभूमीवर नगर विकास विभागाचे राज्यमंत्री डॉ. रणजित पाटील यांनी संरचना अभियंत्यांच्या समस्यांबाबत नुकतीच बैठक घेतली. या वेळी आमदार मेधा कुलकर्णी, पुणे महापालिकेचे शहर अभियंता प्रशांत वाघमारे, पोलिस महानिरीक्षक अमित कुमार, सहायक पोलिस आयुक्त भानुप्रताप बर्गे, सरकारचे अवर सचिव रा. म. पवार, आयएसएसईचे अध्यक्ष धैर्यशील खैरे-पाटील, सचिव किशोर जैन, वरिष्ठ संरचना अभियंता गुलाबराव भिलारे, अच्युत वाटवे आदी उपस्थित होते.

बैठकीत डीसी रूल्सच्या आधारे आर्किटेक्ट, साइट अभियंता, संरचना अभियंता, विकसक, मालक यांच्या जबाबदाऱ्यांवर सविस्तर चर्चा करण्यात आली.

20 Aug2018 : ISSE Pune followed up with Pune Municipal Corporation for modification in DC Rules and fixing role and responsibilities of various professionals involved in construction activity.

22 Sept 2018 :

Epicons Friends of concrete arranged 75th workshop on Challenges in Basement Construction along with IRClass Systems and Solutions Pvt. Ltd. at their Powai campus.

Dr. N V Nayak, eminent geotechnical consultant shared his views on why there are collapses of shoring system during basement construction. Some of the reasons cited were poor quality of geotechnical investigation and interpretation of data, inadequate design of shoring system, not considering correct water table in the design, improper excavation, client's pressure for compressed time and cost, overconfidence. He cautioned that weight of surcharge not correctly accounted in the design or terminating the shore pile in weathered rock above the depth of excavation, improper grouting of anchors may lead to failures during construction.

Mr. Mehul Parikh of Nina Waterproofing explained code provisions and waterproofing systems available for basement construction which includes fully bonded HDPE membrane system. Abhishek Jain of Sika India Pvt Ltd also explained various options in waterproofing and precautions for effective waterproofing solutions. Mr. Gilles from Dorken GMBH explained about the effective drainage system along the retaining walls and below the raft using dimpled HDPE sheets for effectively reducing the water table below the raft level.

Mr. Hiten Mahimtura, explained various shoring systems used for basement construction which includes concrete piles with a gap of 100 to 200mm, Secant piles, steel sheet piles, diaphragm walls and shown the applications in various projects.

Mr. Abhay Ghate of Optimal Consultancy and R N Joshi presented case study of basement construction which had many problems like water continuously pouring from touching piles, uplift of stitched raft and foundation occurred during construction. They have explained how the solutions were found to overcome the difficulties. Shrinkage strips were provided at 12m spacing to avoid shrinkage cracks, tow wall was constructed below the raft level to reduce the flow of water, cement grouting of soil around toe was carried out as soil improvement measure. Integrated drainage system below the raft level was provided with perforated pipe wrapped in geotextile and providing gravel around the pipe. Continuous pumping arrangement was provided from selected sump locations. It was found that drainage system below the raft was found to be economical which eliminates uplift on raft and retaining walls of the basement.

Mr. Bijay Nair of IR Class Systems explained concept of Third Party Inspection procedures and scope of work for various projects.

On behalf of Girish Dravid, a presentation explaining view point of a structural engineer was made by Mr. Surlekar. It was suggested to provide various alternatives of shoring systems to client with pros and cons and approximate costing, so that, appropriate system can be chosen for the project in consultation with various stake holders.

Mr. Deshpande presented case study on basement construction near the adjacent building by using micro-piles as shoring system

Anand Kulkarni explained how leakage in existing basement having inadequate base raft was solved by casting additional raft slab over the existing one and connecting it the retaining walls and base slab using shear connectors. Arvind Parulekar explained how rock encountered in a basement construction near Dharavi area was cut silently using diamond ropes. The function was attended by around 200 engineers.

Edited and published by Hemant Vadalkar for ISSE, C/o S. G. Dharmadhikari, 24, Pandit Niwas, 3rd floor S. K. Bole Road, Dadar(W.), Mumbai 400 028. Tel. +91-22-24314423. e-mail issemubai@gmail.com Web : www.isse.org.in for private circulation and printed by Sudarshan Arts, 10 Wadala Udyog Bhavan, Naigaon Cross Road, Wadala, Mumbai - 400 031.