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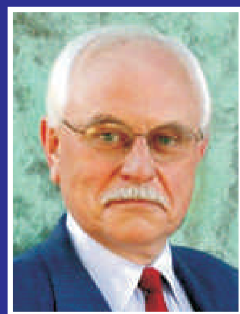
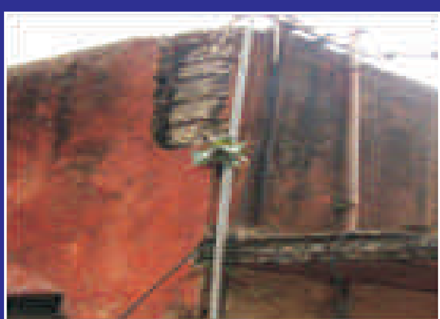
VOLUME 24 - 3

JULY - AUG - SEPT 2022



GEM 33 DR. REIDAR BJORHOVDE-

RESEARCHER, EDUCATOR, AND CONSULTANT IN STEEL STRUCTURES - see page 3



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Contents

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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WELCOME TO NEW MEMBERS

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2200 Vijay Rangrao Medsinge
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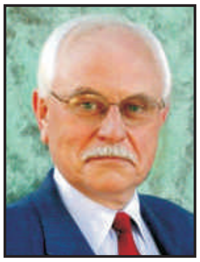
JM 67 Rishabh Bharat Bhalerao
JM 68 Sunilkumar Venkatesh Jegarkal
JM 69 Zaid Ahmed Sabeeh Ahmed Ameer

Patrons : 38
Organisation Members : 35
Sponsor : 8
Members : 2223
Junior Members : 69
IM : 04
Student Members : 259

TOTAL STRENGTH : 2,636

GEM 33 DR. REIDAR BJØRHOVDE- RESEARCHER, EDUCATOR, AND CONSULTANT IN STEEL STRUCTURES

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



Prof. Dr-Ing. Dr. Reidar Bjørhovde (6.11.1941-29.11.2021)

Prof. Dr-Ing. Dr. Reidar Bjørhovde, P.E., Ph.D., leading researcher, educator, and consultant in steel structures, died a few days ago at the age of 80. His research on the strength of steel columns was recognized as a landmark work and later formed the basis of the AISC column design provisions and other international design codes. His long career included work in industry, academia, and as a consultant, including a 22-year affiliation with Nucor Corporation. As an accomplished person, Dr. Bjørhovde has been noted for his achievements, leadership qualities, and the credentials and successes he had accrued in the area of structural engineering, especially in structural steel.

Early Life and Education

Reidar was born on November 6, 1941, in Harstad, Norway to Rebekka Josefine and Reidar Conradi Bjørhovde and had his early education in Norway.

Dr. Bjørhovde spent his college days learning about the complexities of civil engineering. He graduated from the Norwegian Institute of Technology in Trondheim (now called Norwegian University of Science and Technology) with the degree of Sivilingenior in Civil Engineering (equivalent to M.S. in USA) in 1964, and earned a Doctorate in Civil Engineering (Dr.-Ing.) in 1968. He then moved to the USA and enrolled at the Lehigh University, Bethlehem, Pennsylvania, where he earned his second doctoral degree, a Ph.D. in civil engineering in 19721. His doctoral research on the strength of steel columns earned him recognition and was later used as a basis for column design provisions in both the AISC Specification and international design codes.

Professor and Consultant

Entering the workforce in 1964 as assistant professor of steel structures and Government Scholar at the Norwegian Institute of Technology, Er Reidar worked there till 1968. Following his work at Fritz Engineering Laboratory at the Lehigh University for the Ph.D., he left academia to work in practice. Subsequently, he joined the American Institute of Steel Construction (AISC) as Regional Engineer in charge of the Boston office in Massachusetts and then as Research Engineer at AISC headquarters in New York. He returned to the classroom in 1976, first as a professor of civil engineering at the University of Alberta in Canada and worked there till 1981. From 1981-1987, he worked as a professor at the University of Arizona in Tucson. He proceeded to the University of Pittsburgh and worked as Professor and Department Chair from 1987 to 1998. Prof. Bjørhovde also acted as the Director of the Bridge and Structures Information Center (BASIC) at the University of Pittsburg, a national research center he established in 1989. Since 1998 he was the President of the Bjorhovde Group, a consulting firm and International Engineering Consortium, he owned and located in Tucson, Arizona. Prof. Bjørhovde was a registered Professional Engineer in the United States, Canada and Norway.

He has taught many undergraduate and graduate courses as well as short courses in the USA and also in many other countries on all aspects of steel, cold-formed steel, composite steel-concrete design, LRFD and limit states design, welded structures, welding technology, and construction materials.

Dr. Bjørhovde has done extensive research and involved in the design code development in the areas of stability and reliability of steel columns, structural steel connections, steel materials, and composite structures. His research on steel column stability and reliability was recognized internationally, and the column curves he generated based on the extensive experimental work done by him, is now known as SSRC Column curves (see Fig.1)1. These curves were first adopted in the Canadian Limit States Steel Design standard in 1974, and formed the basis for column design of the AISC Specification, the AISI Specification for cold-formed steel structural members, the LRFD Steel Bridge Design Specification of the American Association of State Highway and Transportation Officials (AASHTO), and the South African steel design code. The results of his work on sign structures have been adopted by AASHTO2.

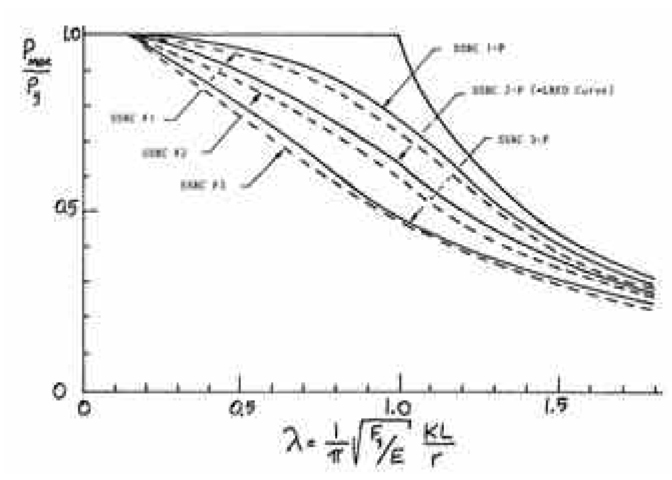


Fig. 1 SSRC Column curves

He also authored more than 300 seminal papers and articles3. He has lectured and acted as consultant to various projects in several countries, and has presented papers at various seminars/workshops and conferences more than 500 times.

Awards and Recognition

His expertise in the stability, strength, and reliability of steel buildings and bridge design and construction brought him numerous prestigious awards and opportunities to advice on research projects in the USA, Canada, and around the world.

The awards received by him include the T. R. Higgins Award of AISC in 1987, the prestigious Research Fellowship of the Japan Society for the Promotion of Science, the NATO Senior Guest Scientist Award (France), the J. James Croes Medal, the Shortridge Hardesty Award and the George Winter Award of the American Society of Civil Engineers, the Duggan Medal of the Engineering Institute of Canada, the IMCA Award of the Mexican Institute of Steel Construction, the Lynn S. Beedle Award of the Structural Stability Research Council, the AISC Lifetime Achievement Award in 2011, and the Charles Massonnet Award of the European Convention for Constructional Steelwork (ECCS) in 2012. On October 18, 2017 Marquis Who's Who, the world's premier publisher of biographical profiles, presented Dr. Bjørhovde, with the Albert Nelson Marquis Lifetime Achievement Award.

The Singapore Structural Steel Society has named him an Honorary Fellow in 2012. The Tsinghua University in Beijing, China appointed him to a rare Tsinghua Chair Professorship in 2012. He was a Fellow of the Structural Engineering Institute and the ASCE Board of Direction in 2014 elected him to the prestigious Distinguished Membership of the society (Dist. M.ASCE). He is also listed in many editions of Who's who and similar reference books.

Committee Work

Dr. Bjørhovde was a member of the AISC Committee on Specifications and the Canadian steel design code (CSA S16) committee. He served on the American Iron and Steel Institute's committee for the Specification for Cold-Formed Steel Structures for many years (1974-76 and 1981-2011), and was a member of the committee that developed the 2005 and 2011 Hong Kong steel design code. He was the Chairman of the Structural Stability Research Council (SSRC) during 1998-2002 and a member of the SSRC Executive Committee during 1987-2008. He was the member of the Technical Activities Division of the ASCE Structural Engineering Institute during 1999-2004, and served as its Chairman for 2002-2003.

He collaborated with European and Chinese researchers and served as an advisor to the European Convention for Constructional Steelwork (ECCS). He was a member of ECCS and its Editorial Board, as well as TC10, the committee on Connections. He is also an advisor to the Building and Construction Authority of Singapore. He founded the AISC-ECCS International Connections Workshops in 1987[4]; the 8th was held in Boston, Massachusetts in 2016.

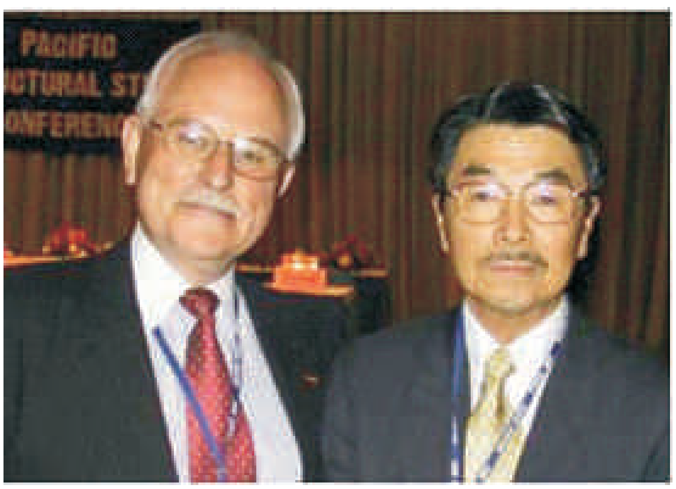


Fig. 2 Dr. Bjørhovde with Prof. Yuhshi Fukumoto of Japan

He also served as the editor for the Journal of Constructional Steel Research for nearly 20 years and was the research editor of the AISC Engineering Journal.

Encomiums5

"Reidar has been a big part of AISC standards development since I came to AISC in the mid-80s and before, and I always admired his dedication tothe industry and his enthusiasm," said Cindi Duncan, AISC's Director of Engineering.

"Reidar was a good friend to all of us in the steel fabrication industry, and he was a special person in my life," added Dr.Ted Galambos, emeritus professor of structural engineering at the University of Minnesota. "He was, in my ideal opinion, a real gentleman! I appreciated his sophisticated mind, his liberal views, his musical talent, and our ability to talk with each other in the German language. He gave a lot to the structural engineers of the world. He was well versed in structural mechanics, but in his publications he was always able to communicate to the practicing engineer. Our profession, including especially AISC, lost a very sensible guide! I will miss him very, very much."

"I've known Reidar for more than three decades as a friend, colleague, and collaborator," said Scott Melnick, senior vice president at AISC. "One of my fondest memories was traveling in China with him in 2001 as delegates to the Pacific Steel Construction Conference. His knowledge of steel and his network of international contacts were unparalleled, but even more impressive were his erudite attitude, willingness to explore new places and ideas, and eagerness to meet new people.

Hobbies

A true Renaissance man, Bjørhovde was a serious student of classical music. For nine years (1998-2007) he was a popular weekend announcer for Tucson's classical music radio station KUAT-FM. As a passionate patron of the arts, he particularly enjoyed the Tucson Symphony Orchestra and attended orchestra concerts wherever his international travels took him. He loved history and his standard poodles, and there was nothing he liked better than good wine and a lively discussion. He was also an avid photographer. He could speak in seven languages!

Fig. 3 Dr. Bjørhovde and others during the Wine testing event -ISCG Meeting, Cape Town

Eulogy

Prof. Reidar Bjørhovde died on Monday, November 29, 2021, at Tucson, Arizona from the effects of Parkinson's. He was 80.

Family

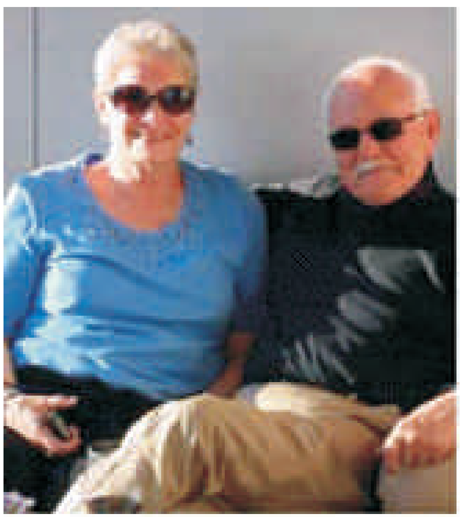


Fig. 4 Dr Reidar with his wife Pat

Reidar was married to his beloved wife, Pat, for 49 years. They celebrated their 40th wedding anniversary in October 2012. Together they loved traveling, concerts, collecting art, and spending time with family and friends. He enthusiastically supported her work for nonprofit arts organizations.

He is survived by his three sisters, Kirsten, Randi, and Gerd, son Ian (Cheryl Pendergrass) and daughter Heather, and children from his first marriage, Erik (Ee Fei Koo), Anne (Valentin Rossebø), and Knut (Armine). He has 12 grandchildren: Elisabeth, Owen, Taylor, Cory, Runar, Aksel, Jacob, Victor, Katinka, Laurits, Jamie, and Anna, and several nieces and nephews.

I have referred to his work on steel column and beam-column connection classification in my book: Design of Steel Structures, Oxford University Press, 2016, 883 pp.

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About The Author

Dr. N. Subramanian,
Ph.D., FNAE, F.ASCE, M. ACI, FIE is an award winning author, consultant, researcher, and mentor, currently based at Maryland, USA, with over 45 years of experience in Industry (including consultancy, research and teaching). He was awarded with ‘Life Time Achievement Award’ by the Indian Concrete Institute (2013), Tamil Nadu scientist award (2001), Gourav Award of the ACCE(I) (2021), and the ACCE(I)-Nagadi best book award for three of his books (2000,2011,2013). He is the author of 25 books, including the famous books on Design of Steel Structures, Design of RC Structures, Principles of Space Structures, and Building Materials, Testing and Sustainability and about technical 300 papers.

Email : drnsmani@gmail.com

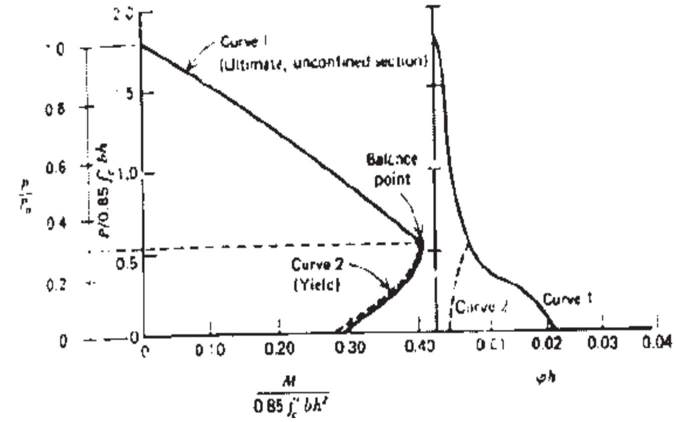
DUCTILE DESIGN OF SHEAR WALLS – BOUNDARY ELEMENTS

By Vatsal Gokani

A shear wall is a vertical element of a system that is designed to resist in-plane lateral forces, typically wind and seismic loads. Seismic loads are inertial forces. Buildings designed as per codal provisions cannot be designed to remain elastic under maximum considered earthquake forces. They are designed to withstand the earthquake by imparting enough ductility to them so that there is significant damage but there is no loss of vertical carrying load capacity.

Ideally, ductile shear walls should not undergo any compression failure i.e. they should be designed as under-reinforced sections. Below is a P-M interaction diagram showing the strain levels at various axial load levels. It is basic structural engineering knowledge that design of a compression member, which has axial loads above the balanced point, is an over-reinforced design and inherently non-ductile. This is because, above the balanced point, the edge reinforcement does not yield before concrete crushes. Below is a diagram (from the book Reinforced Concrete Structures by Park and Paulay), which shows that a compression member with axial loads above the balanced point is non-ductile. Below the balanced point, a compression member shows ductile behaviour. It is good engineering practise to keep the maximum axial stress level in shear walls to 0.4fck. This keeps the shear walls below the balanced point. This is also mandated in certain building codes such as EC-8.

Secondly, ductile design philosophy also involves designing end lengths of walls as ductile elements. These ductile end zones are known as boundary elements. Under cyclic seismic loading, the ends of shear walls come under repetitive compressive and



tensile strains. Seismic loads being much higher than what structures are designed for, the cover region in the end zone spalls. The boundary elements should retain their strength even after cover spalling under such cyclic loading. The exposed vertical bars in these end zones should not buckle. For these two reasons, the boundary elements have to be provided with ductile links similar to columns.

The ductile design of shear walls according to IS 13920 – 2016 entails providing boundary elements at the end of the walls in which the extreme fibre compressive stress, using elastic stress distribution from seismic forces, is more than 0.2fck. The boundary elements can be terminated at a height above which the stresses reduce to 0.15fck. This stress limit is obtained from ACI 318 code. In reality, this idealised elastic stress distribution is nothing but a calibration to check the level of compressive stresses at wall ends and estimate whether ductile boundary elements are required or not. It does not provide any insight into the non-linear behaviour of the wall and the actual required length of the boundary elements. To estimate the length of the boundary element, ideally, neutral axis depth 'c' should be calculated at nominal moment capacity at maximum axial load (obtained from seismic load combinations). The boundary zone element length should be the maximum of (c/2 and c-0.1Lw) where Lw is the length of the shear wall. Such a strain-based procedure is recommended in ACI 318 and is the most accurate. IS 13920 -2016 also does not explicitly require us to calculate the end zone length but states that the boundary element should have adequate axial load capacity, assuming short column action, so as to enable it carry axial compression arising from factored gravity load and lateral seismic shaking effects. The minimum reinforcement in ductile boundary elements should be 0.8% and the non-ductile mid-zone should be 0.25% as per IS 13920 -2016.

The author's interpretation of the IS 13920 boundary element provisions (based on similar codal provision wording in ACI 318-95) is that the central portion of the wall (mid-zone) should be ignored while considering the strength of the boundary elements. However, the statement in IS 13920 – 2016 does not explicitly state that the mid zone should not be considered in design and it is open to interpretation.

To better understand the various codal provisions around the world, the author has presented the end-zone length calculations and subsequent reinforcement calculations of a 40-storied building at the 1st floor level and 25th floor. The following methodologies are adopted:

1) IS 13920 – 2016 method provided in Section 10.4.2
2) Boundary Zone Length calculations using P/A +- M/Z up to 0.15fck stress level
3) UBC 97 provisions
4) ACI 318 provisions

The sizes, forces and reinforcement calculations in the shear walls are given in the table below:

Shearwall Properties, Forces and Reinforcement Required			
	Wall at 1st Floor	Wall at 25th Floor	
p,reqd	1.74	0.25	Reinf % Reqd (from ETABS)
Pu,max	32638	12730	Max Axial Load in Seismic Combinations (kN)
Mu	10925	3375	Major Moment @ Pu,max (kN)
fck	60	40	Concrete Grade (N/mm²)
fy	500	500	Reinf Grade (N/mm²)
b	350	350	Width of Shear Wall (mm)
Lw	5125	5125	Length of Shear Wall (mm)
A	1793750	1793750	Wall c/s Area (mm²)
Z	1532161458	1532161458	Wall Section Modulus (mm³)

Calculations as per IS 13920 – 2016 Provisions

The following are the boundary element and reinforcement calculations as per IS 13920-2016. The mid-zone concrete and reinforcement is ignored as per the codal provision.

Boundary Zone Length as per IS 13920 - 2016			
	Wall at 1st Floor	Wall at 25th Floor	
Z (assume)	1500	1000	Zone Length (mm) - Each Side, Assumed
Z/Lw	0.29	0.20	Zone Length Factor
Pu,zone	19333	7183	= Pu/2 + Mu/(Lw-Z)
Asz,reqd	20850	5060	Reinf Required in Boundary Zone (mm2)
pz,reqd	3.97	1.45	Reinf % Required in Boundary Zone
pm,reqd	0.25	0.25	Reinf % Required in Mid Zone
Reinf,z	26-T32	18-T20	Reinf provided in Boundary Zone
Reinf,m	18-T12	26-T12	Reinf provided in Mid Zone
pz,prov	3.98%	1.61%	Reinf % provided in Boundary Zone
pm,prov	0.27%	0.27%	Reinf % in Mid Zone
p,prov	2.44%	0.79%	Reinf % provided

Calculations using P/A +- M/Z up to 0.15fck stress level

The boundary element length is calculated using elastic stress level in shear walls and the end zone length is taken as the length from edge of shear wall to the point where the stress is 0.15fck.

Boundary Zone Length as per (P/A) ± (M/Z)			
	Wall at 1st Floor	Wall at 25th Floor	
σ max	25.3	9.3	Max Stress: P/A + M/Z (N/mm2)
σ min	11.1	4.9	Min Stress: P/A - M/Z (N/mm2)
Z/Lw	1.0	0.7	Zone Length Factor
Z	2562.5	2562.5	Zone Length (mm) - Each Side
ρz,reqd	1.74%	0.80%	Reinf % Required in Boundary Zone
ρm,reqd	NA	NA	Reinf % Required in Mid Zone
Reinf,z	23-T25 + 20-T20	27-T16 + 16-T12	Reinf provided in Boundary Zone (per Zone)
Reinf,m	NA	NA	Reinf provided in Mid Zone
ρz,prov	1.96%	0.81%	Reinf % provided in Boundary Zone
ρm,prov	NA	NA	Reinf % in Mid Zone
ρ,prov	1.96%	0.81%	Reinf % provided

According to the author, this is not the right interpretation of end zone length calculations, even though it is followed by a number of consultants. The reason is that the end zone length calculations should be based on capacity moment rather than analysis moments and secondly the stresses in a concrete cross section are non-linear (parabolic stress strain curve and zero stresses in tension). Capacity design is the mainstay of ductile design since seismic forces are actually much higher than design forces. Analysis moments have no significance in boundary zone calculations.

Calculations using UBC-97 provisions

UBC-97 provisions are fairly accurate calculations based on the axial load levels. The concept can be easily understood in the image above from Park and Paulay. The higher the axial load level, the farther is the neutral axis depth from the compression edge of the wall and the longer is the end zone length.

Boundary Zone Length as per UBC 97			
	Wall at 1st Floor	Wall at 25th Floor	
Pu	32638	12730	Max Axial Load in Seismic Combinations (kN)
Po	87714	50315	Max. Axial Capacity (kN)
Pu/Po	0.372	0.253	
Z/Lw	1.00	0.20	Zone Length Factor
Z	2562.5	1032.7	Zone Length (mm) - Each Side
ρz,reqd	1.74%	0.80%	Reinf % Required in Boundary Zone
ρm,reqd	NA	0.25%	Reinf % Required in Mid Zone
Reinf,z	23-T25 + 20-T20	18-T16	Reinf provided in Boundary Zone (per Zone)
Reinf,m	NA	24-T12	Reinf provided in Mid Zone
ρz,prov	1.96%	1.01%	Reinf % provided in Boundary Zone
ρm,prov	NA	0.25%	Reinf % in Mid Zone
ρ,prov	1.96%	0.56%	Reinf % provided

Calculations using ACI 318 -22

The most accurate calculation and the most analytically rigorous method is provided in ACI 318 and is based on the strains in the shear wall at maximum axial load and the nominal moment capacity (without material safety factors) at that axial loads.

Boundary Zone Length as per ACI 318			
	Wall at 1st Floor	Wall at 25th Floor	
c/Lw	0.54	0.36	Neutral Axis Ratio
Z/Lw	0.44	0.26	= max (c/2, c-0.1Lw)
Z	2562.5	1309	Zone Length (mm)
ρz,reqd	1.74%	0.80%	Reinf % Required in Boundary Zone
ρm,reqd	NA	0.25%	Reinf % Required in Mid Zone
Reinf,z	23-T25 + 20-T20	22-T16	Reinf provided in Boundary Zone (per Zone)
Reinf,m	NA	20-T12	Reinf provided in Mid Zone
ρz,prov	1.96%	0.97%	Reinf % provided in Boundary Zone
ρm,prov	NA	0.26%	Reinf % in Mid Zone
ρ,prov	1.96%	0.62%	Reinf % provided

Observations and Conclusions

1) The IS 13920 – 2016 provisions for boundary zone lengths are the most simplistic in terms of calculation rigour. However, these calculations result in the most uneconomical designs. They may not be accurate since the boundary length has to be assumed and is not based on strains that can be experienced by a wall at its moment capacity.

2) The boundary length calculations using (P/A +- M/Z) elastic stress values up to 0.15fck produce full length boundary elements even at the 25th floor. According to the author, as discussed above, such calculations are not accurate as they are based on linear stress assumption and not based on capacity moments but analysis moments. The (P/A +- M/Z) elastic stress value is just a good approximation to find out whether the boundary element is required or not.

3) The boundary length calculations as per UBC-97 are fairly simple and accurate. UBC-97 indirectly considers the length of boundary zone based on strain levels. On the 1st level boundary element extends over the full length of the wall whereas at the 25th level extends to approximately 20% of the wall length.

4) The boundary length calculations as per ACI 318 are complex but the most accurate. The zone length calculations are explicitly based on strain levels at nominal moment capacity. On the 1st level, boundary element extends over the full length of the wall whereas at the 25th level extends to approximately 25% of the wall length.

5) Arbitrarily selecting the boundary zone length will not yield the most accurate results. Such designs can be uneconomical yet unconservative. Simple provisions of UBC-97 predict the boundary zone length fairly accurately. A rigorous approach of ACI 318 can be followed for the most accurate boundary element length calculation. Following any one (UBC-97 or ACI 318) for the computation of boundary zone lengths will yield economical and accurate designs.

About the author :

Vatsal Gokani is a consulting engineering practicing in Mumbai in the name of Gokani Consultants. He has designed many Concrete buildings and has 17 years of experience. He can be reached at v.gokani@gokaniconsultants.com

DETERIORATION ISSUES IN OVERHEAD WATER TANK

By Er. R. D. Kalgutkar, Prajakta Bhise, Nilesh Gaikwad

Introduction :

Overhead water tank is a prime & important structural component of a building subjected to cyclic loading. Being a 'water retaining structure', Indian Standards Code has made specific provisions in designing & construction, to avoid deterioration issues in it.

While conducting structural audits of buildings, we encountered numerous deterioration issues in OHWT. These are elaborated in this topic.

We have noticed that codal provisions are not completely adhered. During construction, certain critical parameters are not given importance. During usage, the dampness issue is not addressed on priority.

Main topic :

Every building structure / part, like OHWT is constructed to serve for a particular life span. Not following codal provisions of IS:3370-2009, IS:456-2002 & subsequent non maintenance leads to deterioration of OHWT.

Structural audit is a tool to find out the factors & reasons for the deterioration of this part of structure. Audit highlights the structural as well as non-structural issues which contribute to deterioration of OHWT

Observations :

We have observed the following common issues in OHWT.

1. Dampness & salt formation at base slab & walls
2. Cracks on base slab & walls
3. Spalling of plasters / concrete from base slab & walls
4. Pushing away of walls
5. Cracks at a junction (between slab - wall, between walls)
6. Cracks in columns & beams supporting OHWT
7. Exposed & corroded reinforcement in column, base slab & walls

Reasons :

Above mentioned deterioration issues are mainly due to,

1. Design errors
2. Construction faults
3. Lack of maintenance / repairs

1. Design errors :

Refer Image nos. 8, 9

Structural cracks at the junction between slab & walls are seen when haunches are not provided for walls having single surface zone reinforcement or wall thickness is less than required.

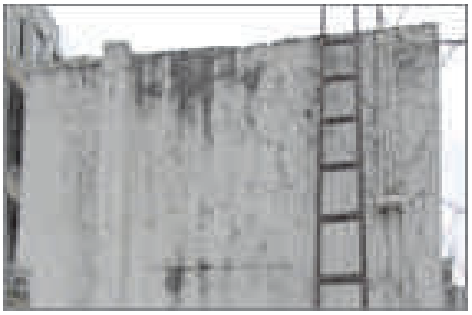


Image No. 1

Image No. 2

Image No. 3

Image No. 4

Image No. 5

Image No. 6

Image No. 7

Image No. 8

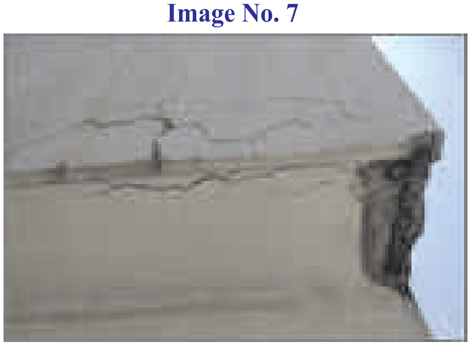


Image No. 9

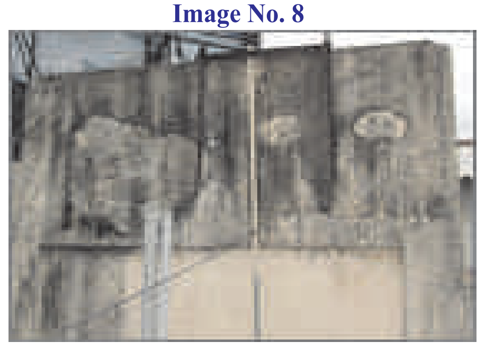


Image No. 10

Refer Image nos. 2, 3

Designed strengths at junctions in between walls should be achieved by providing proper hoop steel & development length. If not provided, structural cracks are seen.

Wall thicknesses less than required & lack of provision of haunches at junctions in between walls, add to dislocation at these junctions.

Refer Image nos. 4, 6 & 9

Structural cracks in columns/beams of OHWT, are observed when the column/beam cross-sections are smaller, and the percentage of main steel reinforcement is lesser than required & wider spacing between lateral ties/links. Moisture ingress in these components add to corrosion of steel & cracks are seen in concrete cover & eventual spalling of concrete occurs.

2. Construction faults :

Refer Image no. 1

Shrinkage cracks are due to use of excessive fine aggregate, use of cement having higher heat of hydration, lean cement mortar for plaster & insufficient curing. Crocodile cracks are due to higher silica content in fine aggregate. Insufficient curing especially during hot & dry days.

Refer Image nos. 10

Delamination of plaster is seen when the surface is not roughened before plaster or no bonding cement layer is applied or cement mortar is lean.

Refer Image nos. 2, 3, 4, 8, 9

If designed hoop reinforcement steel at the junction of walls is not provided & development length is not maintained, separation cracks develop at these junctions.

Refer Image nos. 5

Leakages & dampness in base slab & walls is due to porous gap graded aggregate, lean concrete, honeycombing & improper waterproofing.

3. Lack of Maintaince / Repairs

Refer Image nos. 4, 6 & 7

When the waterproofing layer is damaged, or weathered, corrosion in reinforcement is observed which is due to ingress of moisture into structural members - base slab, beams & columns. Cracks develop in these members & due to increased volume of corroded reinforcement bars spalling of concrete is seen & further aggravated deterioration.

Conclusions :

It is to be noted that the codal provisions should be strictly followed for design & construction of OHWT. Equally it is important to maintain this part of the structure in good condition by repairing periodically to arrest moisture movement into structural components of OHWT.

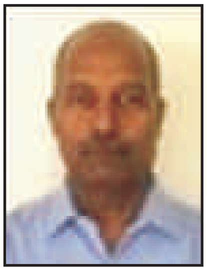
Irrespective of size & bending moments in base slab of OHWT, nominal surface reinforcement at top portion of base slab should be provided to avoid development of cracks in upper portion of slab. When such cracks develop, the waterproofing layer also gets cracked.

Extra steel reinforcement should be provided at the junction of walls with required development lengths especially when single surface zone steel is provided. Similarly, in such cases, haunch with embedded steel bars connecting base slab steel & wall steel is must, to avoid dislocations at base level. Also lateral ties/links in columns & beams of OHWT should be closely spaced.

Higher grade of dense concrete, correct placing of reinforcement bars, use of deformed bars, close spacing of bars and use of small size bars lead to diffused distribution of cracks, and hence are preferred practices as per codal provision.

To enhance the life of OHWT at par with the main structure, precautions as mentioned above should taken.

About the Author:



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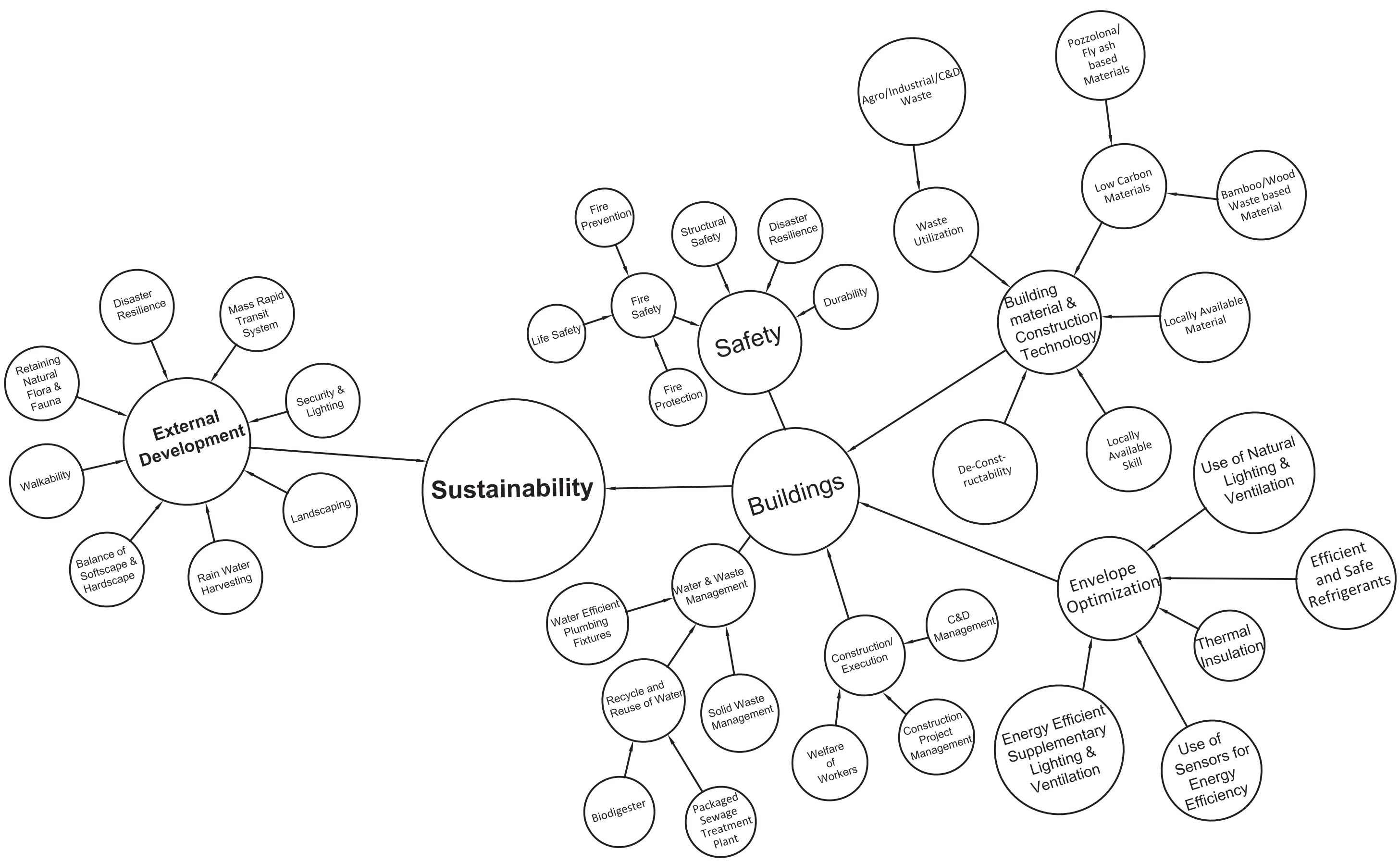
SUSTAINABILITY IN BUILDINGS AND BUILT ENVIRONMENT

(Sustainability Model : BIS/SP/2/2022)



Overall
- Accessibility
- Gender responsiveness
- Smart solutions/Digital Technologies
- Maintenance

SANJAY PANT
DY DIRECTOR GENERAL, BIS

PILE TERMINATION CRITERION - CONSULTANT'S POINT OF VIEW

By Shekhar Vaishampayan

1.0 Introduction

As design engineers we all know, for given critical structural loading combinations, shallow footings or raft may not be suitable foundation system whenever stratum at proposed founding depth does not have sufficient shear strength (Low Safe Bearing Capacity) or is likely to undergo more than permissible settlements (high compressibility). In urban setting, it is not possible to excavate to founding depth whenever high ground water level is met with in congested localities. In such cases, piles are adopted as foundation system to transfer loads to lower competent strata. Pile is thus a structural member used for transferring structural loads to lower competent soil or rock stratum.

For any and every project location, reliable geotechnical investigation is essential for determining the sub surface profile and its engineering properties. Knowing the structural loads, decision about type of foundation can be taken. Let us say it is decided to adopt piles as foundation system. Then type of pile is selected. Once type of pile is selected, pile capacities are estimated using standard methods either from IS codes or relevant technical reference literature. In some critical cases, American, British or Euro codes are also used. In all these, it is important to know that pile capacity and corresponding lengths are estimated. Actual capacity is confirmed only after pile load tests are conducted.

Piles have been used as foundation system probably since 1950s in India. Traditionally only a tripod with winch was used to install bored cast in-situ piles in varying subsoil conditions. Different pile installation techniques suitable for different soil conditions were adopted. Bailer-Chisel method with temporary casing or permanent MS liners were useful where ground water was deep and pile length was limited. Direct Mud circulation method was adopted in case of long piles and high ground water conditions causing unstable sub surface conditions. Driven piles precast and cast in-situ were also used in certain subsurface conditions where founding stratum was medium dense to dense sand.

2.0 Pile Installation Procedure

Installation of Pile foundation can be divided into three main tasks to be achieved. These are as below;

Task One : Centering the piling rig at correct location of pile and ensuring that it stays there. It is needed to make sure that centroid of column and centroid of pile cap match as closely as possible.

Task Two : Based on the geotechnical investigation data and response of piling tools to founding stratum, to reach selected founding stratum to achieve targeted pile capacity and terminate piles.

Task Three : Concrete the pile shaft in such a manner that pile integrity is achieved and it meets all durability criteria. Quality of concrete and diameter of pile shaft should be as uniform as possible for full pile length.

Pile installation being specialist job, selection of suitable method and termination of each pile were traditionally entrusted to piling engineers and rig operators. However, with more experience of piling technique coupled with pile load results, empirical methods of pile termination were evolved.

These guidelines have been used for terminating different types of piles. Task performed by piling rig is task two i.e., to reach the correct founding stratum in minimum time to achieve maximum progress. Following factors/aspects related to execution need attention. These are discussed below.

3.0 Selection of Rig

For given site & subsurface conditions, selected rig shall be capable of reaching founding stratum in reasonable time, without causing damage to machinery. Following factors contribute to selection of suitable rig.

a) Rig characteristics; type of Kelly, rated torque, Maximum pull down force (Thrust), maximum pressure that can be applied.
b) Size of plot and surrounding structures to assess maneuverability of piling rig.
c) Access Road to plot.
d) In case of soils, it is simply required to reach specified depth without collapse of any stratum above and below ground water level and keep it stable till concreting is completed.
e) However, in Mumbai and surrounding region in Maharashtra, we end up terminating piles in bed rock. Rock mass characteristics consisting of Rock core recovery values, Rock quality designation values and saturated crushing strength of rock cores need to be studied to estimate pile capacities.

Considering that for given soil/rock and ground water conditions, piling rig capable of achieving design depth is selected. On commencement of actual installation if the behaviour of subsurface layers is as predicted then no change will be needed. However, if the response of successive subsurface layers is different when compared with theorical inferences drawn from geotechnical investigation data, then revised pile termination criterion will be developed without compromising safe capacity of piles.

We can thus define 'Pile Termination Criterion' as the field method by which it is possible to ensure that pile capacity is achieved. It acts as guideline to supervising field engineers to determine founding stratum and depth at which safe load carrying capacity of pile will be achieved. In other words, if this guideline is achieved in every pile installed then each and every pile load test likely to be successful.

Following are the methods used for determining pile termination

1) Engineering Hiley's Formula for driven piles
2) Chisel Penetration Energy Method for bored cast in-situ piles (particularly in rock)
3) Pile Penetration Ratio for Hydraulic Rotary Rigs used for bored cast in-situ piles
4) Cole & Stroud Method for bored cast in-situ piles (particularly in weak rock) 1976
5) Pile Jacking Method for bored cast in-situ piles (soils)

In the present writeup, we are not covering Cole and Stroud method (1976) as it is mostly not used in and around Mumbai as it obstructs flow of work. Pile Jacking method is prevalent in countries having deep alluvium such as Bangladesh, Malaysia, and Indonesia.

4.0 Engineering Hiley's Formula

In case of driven piles, hammer of certain weight adequate to drive pile of design diameter and length implying estimated weight is used. Penetration of pile is measured after fixed number of blows. Thus, when standard amount of energy is transferred to soil layers, pile penetration is inversely proportional to shear strength of soil layers pile penetrates. Please note that each successive soil layer must fail to permit penetration of pile. For weak soil, penetration is more, and penetration will reduce with increase in shear strength of soil. So, to achieve required capacity of pile, penetration per blow (Set) can be estimated. if graph is plotted between shear strength of foundation stratum and penetration, it will be hyperbolic in nature.

Engineering Hiley Formula.

$$Qu = \frac{WH\eta\_b}{C + \frac{s}{2}}$$

Where

Qu =Ultimate Load of Pile in kN

W = Weight of hammer in kN

H = Height of drop of hammer in cm

'ηb' = Efficiency factor

's' = Set value in cm

C = Total Elastic compression

Sample Calculations:

Parameters		
Weight of hammer	W	35 kN
Height of drop of hammer	H	200 cm
Efficiency factor	η_b	0.80
Set value (Last 10 blows)	s	15 cm
Total elastic compression value	C	0.98 cm

Therefore, Ultimate load of pile in MT by Hileys formula is

$$Qu = \frac{35 * 200 * 0.80}{0.98 + \frac{15/10}{2}}$$

Qu = 3237 kN

5.0 Chisel Penetration Method

Using above principle as basis, Late Mr K R Datye proposed Chisel Penetration Energy Method in 1970. This method is simple and can be easily used by site staff. Therefore, this method was used in Mumbai extensively for many years. Even today if pile installation is to be done by conventional tripod method, then pile termination criterion can be adequately defined by chisel penetration method in conjunction with borehole data.

$$E = \frac{WHN\eta}{A_p p}$$

Where;

E = Chisel Penetration Energy in kN-m/m2/cm

W = Weight of Chisel = 2 x (Pile diameter in mm) in kN

H = Standard Drop of Chisel = 2.00 m (in case of DMC it is 0.50 m)

N = Number of blows of chisel in 30 minutes duration say 270

'η' = Efficiency factor to account for energy loss in rig and buoyancy of water in pile hole.

Ap = Pile Cross sectional Area in m2.

'p' = penetration of chisel in cm

In fixed unit time interval of 30 minutes quantum of energy transmitted will be also fixed amount. Penetration achieved by Chisel in rock stratum is inversely proportional to strength of rock. It is very similar to Engineering Hiley's formula. It has been found out by empirical relationship that for after reaching Chisel penetration energy value of 225 T-m/m2/cm, effort level and time required for chiseling increases very fast. Therefore, generally after reaching this energy level, socket length is limited to 0.50 to 1.00 times pile diameter based on site condition.

Around 1990, there was change in type of construction activity. More ambitious civil engineering structures such as fly over bridges, I T Buildings, high rise buildings at locations of very poor soil conditions or very congested localities were to be constructed in double quick time. An upgradation in pile installation technology was the need of hour.

Hydraulic rotary piling rigs have met with this demand admirably. Hydraulic rotary piling rigs as used today are very powerful, energy efficient and use effective method of cutting in to rock to form socket in reasonable time period. All types of users, clients, contractors, government agencies, architects, designers, have got used to hydraulic rotary rigs as established method of installing pile foundations. it is very useful method to avoid vibrations to adjacent structures.

Sample Calculations

Parameters		
Pile Diameter	D	600 mm
Weight of Chisel	W	12 kN
Standard Drop of Chisel	H	2 m
Number of blows of chisel	N	270
Efficiency factor	η	0.75
Pile Cross sectional	A_p	0.28 m^2
penetration of chisel	p	3.00 cm

Therefore, Chisel Penetration Energy in T-m/m2/cm by Chisel Penetration Method is

$$E = \frac{12 * 2 * 270 * 0.75}{0.28 * 3}$$

E = 5785.7 kN-m/m²/cm

6.0 Pile Resistance Ratio (PRR)

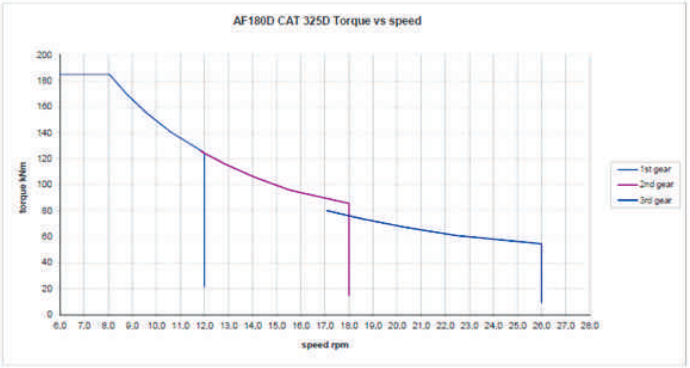
Initially, termination of piles using hydraulic rotary rigs was not clear. Late Mr V T Ganpule (1994) proposed use of Pile Resistance Ratio (PRR) for terminating piles, based on Chisel Penetration Energy Method.

P

$$PRR = \frac{2\,\pi\,(N\,T)\,t\,\eta}{Ap\;\;\mathrm{p}}$$

Where;

PRR = Pile Resistance Ratio (kN-m/m2/cm)
N = Number of Revolution per minute
't' = Time in minutes
T = Torque in kN-m
'η' = Efficiency factor for given machine
Ap = Pile Cross Section Area in m2
'p' = penetration of auger in cm



Sample Calculations

Parameters		
Pile Diameter	D	600 mm
Number of Revolution per minute	N	10
Time	t	10 min
Torque	T	150 KN-m
Efficiency factor for given machine	η	0.90
Pile Cross Section Area	A_p	0.28 m^2
penetration of auger	p	15 cm

$$\mathrm{PRR} = \frac{2 * \pi * (10 * 150) * 10 * 0.90}{0.28 * 15}$$

PRR = 20185 kN-m/m²/cm

For termination of pile, recommended minimum of PRR is 10000 KN-m/m2/cm.

In fixed unit time interval of 10 minutes, quantum of energy transmitted will also be a fixed amount. Penetration achieved by rock auger in rock stratum is inversely proportional to strength of rock. It is very similar to Engineering Hiley's formula. Every rig manufacturer provides Torque applied vs Kelly Bar RPM graph. Again, this graph is hyperbolic in nature. In other words, product of RPM and Torque is a constant. Using this as basis, following guidelines can be used for terminating piles.

7.0 Field Control Criterion for Hydraulic Rotary Rigs

Therefore, followings observations shall be recorded throughout installation of piles on reaching rock stratum. These observations can be made by any site staff easily.

a) Standard time interval 10 minutes for every penetration shall be used.
b) Penetration achieved in each time interval shall be recoded.
c) Number of revolutions per minute (RPM) of Kelly bar shall be measured physically over one full minute.
d) Start of rock socket length is defined by change of drilling tool.
e) At termination of pile, Kelly RPM shall be between 8 to 10, Torque value applied at this rpm is practical rating of the rig in use.
f) Pile shall be terminated after drilling for minimum of one pile diameter in rock with RPM 10 is reached.
g) At each penetration interval, rock cuttings shall be collected and preserved with proper labeling. This should be compared with available geotechnical investigation data and consistency with each should be established.
h) Only after confirming that it satisfies both borehole and penetration reading, decision about pile termination should be taken.
i) It is possible that one may have to drill through rock of higher strength than rating of rig. At such times rock coring shall be used.

Principally, selection of rock socket length should be/ always will be on conservative side as pile must not fail.

One of the hindrances against development of any pile termination formula or methodology is lack of data maintained by piling contractors. It is important that such data bank is created at least for their in-house use. Most contractors maintain data needed only to prepare measurement bills for quantity verification. It is not even done on most important and prestigious sites of their own.

In conventional pilling work, safe loading carrying capacity was limited to about 5000 to 6500 kN/m2 x Pile Cross sectional area. This is compatible with grade of concrete mixed at site & piling equipment.

However, there is now endeavour to match safe compressive strength of concrete used in pile and safe loading stress on pile. This has increased loads per pile, and it has also increased length of rock socket.

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.

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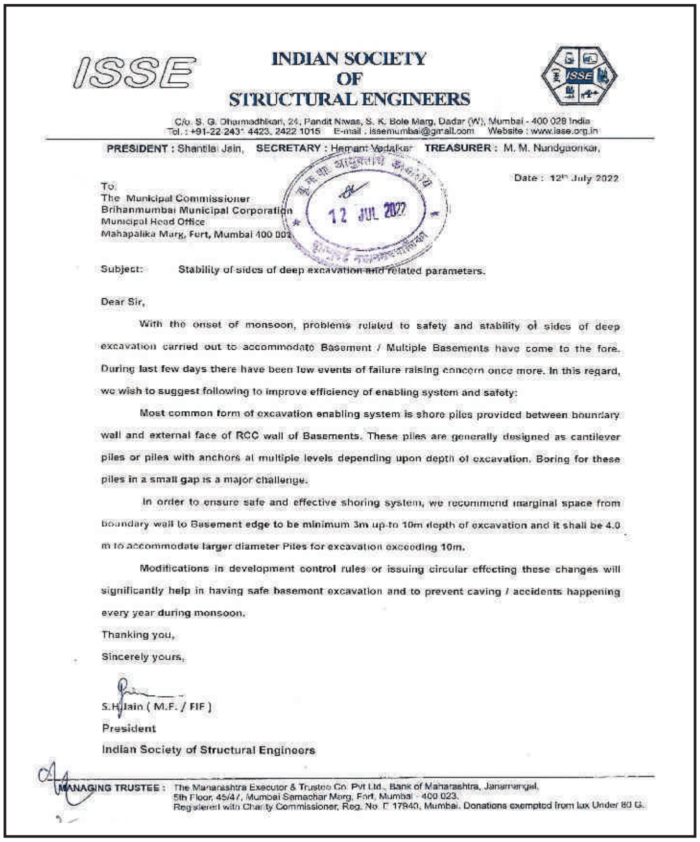

ISSE

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

C/o. S. G. Dharmadhikari, 24, Pandit Niwas, S. K. Bole Marg, Dadar (W), Mumbai - 400 028 India
Tel. : +91-22 2431 4423, 2422 1015 E-mail : issemumbai@gmail.com Website : www.isse.org.in

PRESIDENT : Shantilal Jain, SECRETARY : Hemant Vadalkar TREASURER : M. M. Nandgaonkar,

Date : 12th July 2022

To
The Municipal Commissioner
Brihanmumbai Municipal Corporation
Municipal Head Office
Mahapalika Marg, Fort, Mumbai 400 001

12 JUL 2022

Subject: Stability of sides of deep excavation and related parameters.

Dear Sir,

With the onset of monsoon, problems related to safety and stability of sides of deep excavation carried out to accommodate Basement / Multiple Basements have come to the fore. During last few days there have been few events of failure raising concern once more. In this regard, we wish to suggest following to improve efficiency of enabling system and safety:

Most common form of excavation enabling system is shore piles provided between boundary wall and external face of RCC wall of Basements. These piles are generally designed as cantilever piles or piles with anchors at multiple levels depending upon depth of excavation. Boring for these piles in a small gap is a major challenge.

In order to ensure safe and effective shoring system, we recommend marginal space from boundary wall to Basement edge to be minimum 3m up-to 10m depth of excavation and it shall be 4.0 m to accommodate larger diameter Piles for excavation exceeding 10m.

Modifications in development control rules or issuing circular effecting these changes will significantly help in having safe basement excavation and to prevent caving / accidents happening every year during monsoon.

Thanking you,

Sincerely yours,

S.H.Jain (M.F. / FIF)
President
Indian Society of Structural Engineers

MANAGING TRUSTEE : The Maharashtra Executor & Trustee Co. Pvt Ltd., Bank of Maharashtra, Janmangal, 5th Floor, 45/47, Mumbai Samachar Marg, Fort, Mumbai - 400 023.
Registered with Charity Commissioner, Reg. No. F 17840, Mumbai. Donations exempted from tax Under 80 G.

NEWS AND EVENTS DURING JULY – SEPT 2022

by Hemant Vadalkar

5 Aug 2022 : ISSE book release function on performance based design and Wind load code commentary by Dr. Suresh Kumar. ISSE had arranged a physical function and technical lectures after a gap of almost two years. ISSE President Shantilal Jain welcomed all the civil engineers for the function. Hemant Vadalkar spoke about activities of ISSE. Dr. V. N. Gupchup Ex. Principal VJTI and Guru of many structural engineers graced the occasion as a chief guest. A book written by Vatsal Gokani on performance based design was released by chief Guest Dr. V N Gupchup. Also a small booklet on IS875 Part3 Wind load written by Dr. Suresh Kumar of RWDI was also released. Technical presentations were made by both the authors. Product demonstration was given by Ultratech Cement building materials team as a live demo . Function was sponsored by Ultratech Cements Limited. It was a great event and more than 200 engineers attended the function at SASMIRA Auditorium, Worli, Mumbai.

6 Aug 2022 : SEFIarranged a webinar on first revision of IS16700 Criteria for Structural safety of tall concrete buildings. Code committee members Alpa Sheth, Prof. CVR Murthy, Anil Hira and Ranjith Chandunni made presentation and deliberated on the proposed changes in the code. This online programme was attended by more than 500 engineers across India.

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29 Aug 2022 : SEFI arranged e-conference for Discussion on New BIS document SSD-II 06 (19914) : REQUIREMENTS FOR STRUCTURAL DESIGN AND PROOF CHECKING CONSULTANCY SERVICES FOR STRUCTURES. During 29 Aug 2022 to 3 Sept 2022. Many engineers expressed their views on this document published by BIS for wide circulation.

15 Sept 2022 : On the occasion of Engineers Day, ISSE student chapter at MIT ADT, Loni campus near Pune was inaugurated. Prof. Abhijit Galatage from MIT ADT was instrumental for setting up of ISSE student chapter with the help of Hemant Vadalkar. Hemant Vadalkar visited college two years back and made presentation on Structural Software. Pune ISSE centre Chairman Er. Shivadatta Patane was the guest for the inauguration function along with MIT college students and faculty members. More than 30 students have been enrolled as student members of ISSE. Certificates to all student members and a set pf technical publications of ISSE was given to the newly formed chapter.

16 Sept 2022 : Letter to BIS - Response to Requirements from Structural Design and Proof Checking Consultancy Services for Structures Doc. No. SSD II/06(19914) July 2022

ISSE

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

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Tel. : +91-22-2431 4423, 2422 1015 E-mail : issemumbai@gmail.com Website : www.isse.org.in

Ref : ISSE/response/ proof checking code/
Date : 16 Sept 2022

To,
Umang Jaggi
Member Secretary of the Sections Committee
Bureau of Indian Standard
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG,
NEW DELHI 110002

Kind Attn : Chairman SSD II ssd2@bis.gov.in

Sub : RESPONSE TO REQUIREMENTS FOR STRUCTURAL DESIGN AND PROOF CHECKING CONSULTANCY SERVICES FOR STRUCTURES
Doc. No.: SSD II/06(19914) July 2022

Dear Sirs,

Indian Society of Structural Engineers (ISSE) is a premier association of Professional Structural Engineers with its Head Quarters at Mumbai looking after the development of the profession to the benefits of Indian society with the focus on construction industry in particular. We assist Government, Self Government, Municipal Corporations and other Corporate bodies in framing their policies for sound and stable structures in their built assets.

We publish a technical journal for free circulation amongst members and complementary to heads of industries. We have published an award winning book on "Design of reinforced concrete structures for earthquake resistance" for the guidance of Professional structural engineers. This deals with practical design aspects as per relevant codes of practice. We have more than 2200+ structural engineers as our members.
We would like to express our views on behalf of our practicing structural engineers.

We would like to address some basic queries regarding this document:

a) What is the Objective of the document?

- It appears that the operating part of the Document is Table 1 which defines the Minimum Qualification and Experience of "Team Leader" of PDC and PC. The only person of concern to the Building Authority having Jurisdiction (BAHJ) is the engineer taking on the liability and responsibility of the structural design and hence minimum qualification and experience of the Structural Engineer on Record (SER) is pertinent, not that of the Team Leader. (The Team Leader and SER are often different).

Page 1

- Without prejudice to the above, we would like to understand the underpinning rationale of Table 1. We will assume that "Team Leader" has been replaced with "SER" in the document. In Mumbai, for example, it takes more than ten years of experience with a BTech/BE degree to get a licence for designing buildings over 100m. Table 1 assumes that the same skills are required for a building of 51 m and for a building of 200 m. This does not reflect a true appreciation of design of tall structures. Further, the skill of a structural engineer for a 12 m high but very complex, structure having a floor plan exceeding 15000 sq. m. will be far more than has been prescribed in Table 1. *Table one prima facie appears too simplistic and needs far more nuanced understanding of structural design.*
- If "the objective of the document is not to regulate the profession of Structural Engineering", what is the objective?

b) Who is the Target Audience of this Document?

- This Document does not appear to be addressed to a structural engineer and is of no immediate use to a practicing engineer.
- Typically the Building Authority Having Jurisdiction (BAHJ) mandates the requirements of the Structural Designer and Peer Reviewer. *Are BAHJs the target of the document?*
- If this document is for BAHJs, ***is it the mandate of Bureau of Indian Standards to prepare model guidelines for BAHJs?*** And should not the document, even in a fully revised format be renamed "Model Guidelines...."

c) Did a Study of Existing Systems for practice of structural engineering within the country and across a few other countries inform this document?

- Many BAHJs across the country (e.g. Mumbai, Pune Thane, Nashik, Ahmedabad, Baroda, Kolkata, Odisha, to name just a few) have far more sophisticated qualifications, roles and responsibilities already called out in their building bye-laws. This document will compromise the existing robust procedures already in place. The Indian Society of Structural Engineers is the leading structural engineering body in Maharashtra. As is fairly well-known, maximum number of high rise buildings (especially those exceeding 120 m height) across the country are being designed by Mumbai structural engineers, almost all of whom are ISSE members. ISSE would have been happy to participate in the committee and share their experience to formulate the minimum qualifications and experience of structural engineers, *especially since Mumbai has a rich history of Licensing of Structural engineers going back 40 years and ISSE would have shared the journey of licensing of structural engineers and the minimum qualifications and experience requirements with the committee.*
- "The Gujarat Professional Civil Engineers (GPCE) Act, 2006" clearly defines all the requirements for a Structural Engineer, Peer Review Consultant. The Act could be reviewed for Reference. It may be noted that GPCE is a State Act and not a central Act (similar to the State RERA Acts) and was formulated by Gujarat Disaster Management Authority for the State of Gujarat and not by a central Ministry nor an organisation like Bureau of Indian Standards. It is our belief that this matter is a State subject.

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v) We do not believe it is the mandate of the Bureau of Indian Standards to come up with a document to advise BAHJs on how to administer and regulate structural design in their respective constituency. It is already being regulated more efficiently, robustly and more comprehensively in many cities across India than what the document prescribes. The big cities that do not currently have such a system may study the model of Mumbai (or other city having a good system in place).

vi) If at all the Bureau of Indian Standards would still like to continue on this task it has embarked upon, we suggest a reconstitution or at least a comprehensive expansion of the drafting/working committee to a Pan-India representation and which will reflect the wisdom of those who have been working with fine-tuning such a system over decades.

With thanks and regards.

Yours faithfully,

For Indian Society of Structural Engineers

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17 Sept 22 : BIS document SSD-II 06 (19914) : REQUIREMENTS FOR STRUCTURAL DESIGN AND PROOF CHECKING CONSULTANCY SERVICES FOR STRUCTURES. ISSE also had sent comments / letter to BIS about its views on the subject. The copy of letter is available on ISSE website.

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17 sept 22 : Epicons Friends of Concrete arranged a webinar on Soil Retaining Systems Design, Supervision & Likely Risks. Experts in geotechnical engineering Shekhar Vaishampayan- MD Sub Surface Consultants made presentation on various types of soil retaining system used for deep excavation for basements or metro projects and shared his experience. He explained about the safety issues , design errors and how to avoid accidents. Data collection during drilling operation and recording it is very essential to understand the strata and take actions as per the change observed at site during drilling operation. He stressed the need to continuously monitor the deflection of retaining system. This is a very important indicator which gives warning of any abnormal behaviour if the displacements are more than expected. Mr. Arvind Parulekar from Epicons shared his case studies and project learnings. Session was very informative for civil engineers involved in design and construction of deep basements.

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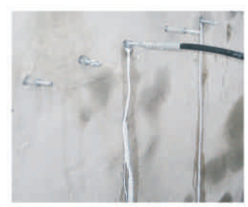

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