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HISTORY OF STRUCTURAL ENGINEERS WORLD CONGRESS Inc. (SEWC Inc.)

Here are a few important points taken from the paper on " Structural Engineers World Congress Idea to Reality" by Roland L. Sharpe, Founding President SEWC Inc.

Informal discussions in the mid 1980s between U.S. and Japanese structural engineers and researchers raised questions about the role of the structural engineer, required capabilities, and how the SE public image could be improved. Further informal meetings and mail correspondence over several years examined in some detail how this interaction could best be accomplished. These interactions led to small meetings at American Society of Civil Engineers (ASCE) Congresses in the U.S. which culminated in 1994 when six professional organizations agreed to form a coalition to sponsor and organize the first Structural Engineers World Congress (July 18 to 23, 1998) in San Francisco.

Recognition of the need for a worldwide SE Congress, need for interaction between SEs worldwide, scope and impact of SE services and effects on society, the need to improve the image and credibility of the SE grew over the next few years.

Range of SE Activities

Although several international organizations exist that serve structural engineers needs in specific areas such as bridges and buildings (IABCE), tall buildings, earthquake engineering, they do not serve many aspects of SE concerns. A consensus developed in the Task Committee that the WSC should include the full range of SE issues - technical, professional, ethics, education, legal, construction, products, and other related issues. There should be exhibits along with sessions on these topics. It also became apparent that the WSC should be "people" oriented with the theme of getting to know each other better. It was estimated that there are about 50,000 structural engineers in the U.S. and perhaps 200,000 or more in the world.

The question of who is a structural engineer was examined. As noted previously, in Japan civil engineers design bridges and other structures associated with infrastructure and mostly work for the government. This appears to be true for some other countries. It was recommended that all engineers designing and constructing structures are considered structural engineers and should be included in WSC. The Committee felt strongly that the WSC should not become a membership organization and should not compete with existing international SE organizations.

In the 1998 congress in San Francisco, there were about 1800 participants from 49 different countries. A large number of exhibitors presented their products. Subsequently in 2002, it was held in Yokohama, Japan and in 2007 in Bangalore, India. Each of the congresses in Japan and India attracted more than 1300 delegates.

The Structural Engineering World Congress (SEWC) is dedicated to the Art, Science and Practice of Structural Engineering. SEWC Congress brings all the structural engineers on a common platform at-least once in 4 years

SEWC presents excellent opportunities for Structural Engineering professionals to interact with each other and to learn more about what is happening in the World of Structural Engineering now and about the trends for the future.

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Guidelines to Authors

Contributions resulting from original research in the area of structural Engineering, analysis, design, structural materials and other related topics in the form of technical papers to be published in the International Journal of Structural Engineers World Congress (SEWC) are welcome.

Prospective authors are free to prepare the manuscripts in their own convenient format and submit in MS Word file. The publisher will modify the format according to the standard format of the journal before printing.

The authors are requested to particularly not to miss mentioning the page number of the paper / book in the list of reference.

The manuscript submitted will be peer reviewed and the comment will be made known to the author.

President's Message

Sundaram R
President Structural Engineers World Congress, Worldwide
Member, Advisory Board, IASS

We are overwhelmed by the tremendous response and encouragement received for the last three SEWC Journals from all over the world. The fourth edition of SEWC Journal is being released now.

We are sure that our continued efforts will definitely fetch good results and we will do our best to maintain good quality of the publication in all aspects.

We will have articles covering wide range of subjects and will strive hard to make the journals accessible to more people as our aim is to help disseminate knowledge and information among the structural engineering fraternity.

SEWC India is organising an International Colloquium in New Delhi on the subject on 'Architecture-Structure' Interaction during November 18th - 20th 2013th in New Delhi and many top-notch Architects and Structural Engineers from different parts of the world are delivering Keynote speeches to share their knowledge.

Congratulations to Dr. B.K. Raghu Prasad, Editor-in-Chief, Mr. K.P. Pradeep, Editor of the Journal of SEWC and all others who have contributed for bringing out this latest Journal.

Editorial

Prof. Raghu Prasad B.K
Editor-in-Chief

Concrete structures are increasingly important in all developing countries because of rapid increase in urbanization. Concrete structures have become more popular because of new materials like self - consolidating concrete of even very high strengths like 80 MPa are possible.

Structures of very large depths are quite common particularly in tall buildings which consist of transfer girders of depths of the order of a meter and above. The present day flyovers and spans which are very large for the city metro require girders of very large depths over a meter high. The theories of analysis and design of concrete structures both with working stress and limit states do not mention about disadvantages of such large depths. The general feeling is that larger the depth, greater is the safety. However, the so called 3rd evolution in the analysis of concrete structures which is fracture mechanics does mention about size effect. Larger the size, lower is the tensile strength of concrete which is classified as quasi-brittle. Tensile strength is directly related to shear strength. Unfortunately even the codes of practice are yet to completely make recommendations about the same. In the past, there have been examples of failures of large sized structures due to absence of consideration of size effect. Sizes which are very large more than a critical value, not only have reduced tensile strengths but also fail in a brittle manner. (Compared to those obtained in a standard laboratory tests done on obviously small sized specimens)

Structural Design of a Foldable Tensegrity Footbridge

Julien Averseng[1], Jérôme Quirant[2], Jean-François Dubé[3]

Abstract

Tensegrity systems, built from struts and cables in a state of self stress, are rarely thought as realistic solutions in structural practice. Although they inspire lightweight, visually transparent and deployable solutions, these systems mainly suffer from an intrinsic low stiffness, which excludes them from many applications. They are also dependent on a self stress state, for which the design and control in situ can be complex. In this paper, these aspects are addressed in the case of a tensegrity linear structure made of 4-struts modules destined to form the framework of a footbridge. Analysis procedures are presented with the purpose of approaching the common challenges and presenting a complete design process. The global behavior of the structure is investigated, showing the potential benefits in terms of static behavior. For several geometric propositions, design conditions are raised on the selfstress state and resistance aspects are taken into account to determine realistic cross sections. Finally, the setting procedure is presented and simulated, demonstrating the feasibility of this kind of structure.

1 Introduction

Tensegrity systems are a particular class of reticulate structural systems. They appeared as potential solutions in structures during the fifties with the work of Snelson [1], Fuller [2] and Emmerich [3]. These systems are composed of a set of compressed struts inside a continuum of tense cables, the whole system being in a stable self stressed state [4]. In consequence, tense elements can be reduced to cables, bringing lightness and transparency to these structures. In addition, some configurations [5][6] accept folding mechanisms, extending the area to spatial applications [7]. In spite of these benefits, these systems are little developed in the structural field mainly because of a lack of accessible analysis and setting methods. In this paper, we present an application of these methods for the design of a deployable rectilinear modular structure serving as a support for a 12 m long footbridge (Figure 1).

2 Modular tensegrity beam

The structure being studied is generated from an elementary pattern that is a 4-struts tensegrity module comprising 12 cables distributed in two horizontal layers and diagonal elements between lower and upper nodes. Large linear or plane structures can be built [8] by juxtaposition of these modules (Figure 2).

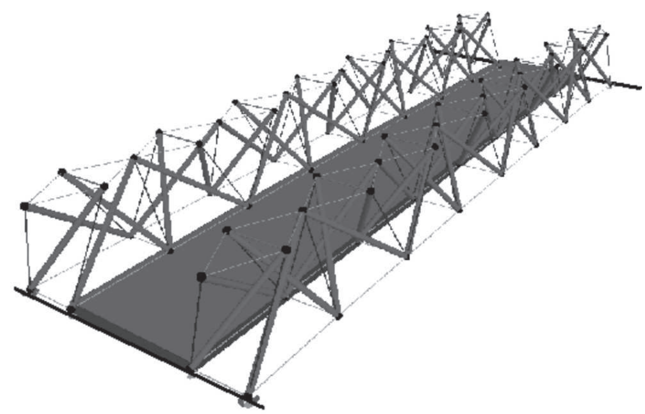


Figure 1. A tensegrity footbridge

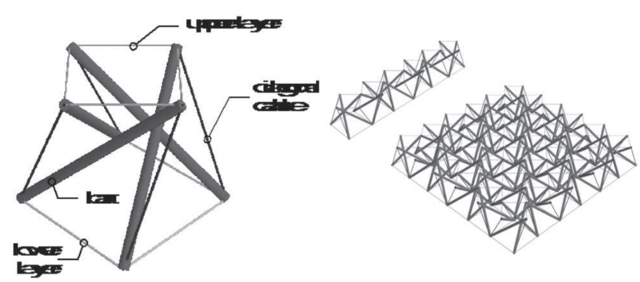


Figure 2. Elementary quadruplex module and examples of linear and plane structures

2.1 Selfstress

This organization has also interesting consequences on the selfstress state, which is defined from the static equilibrium of every node in the structure (Equ 1).

$$Aq = f \quad \text{With} \quad q_i = \frac{N_i}{l_i} \qquad (1)$$

In this equation, q is the vector of the force densities (normal force divided by length) of elements, f is the vector of external forces to the nodes and A is the equilibrium matrix [9]. A selfstress state q0 verifies by definition Equation 1 with f=0. Therefore, it is included in the kernel of matrix A, a vector space from which a structured basis can be established [8].

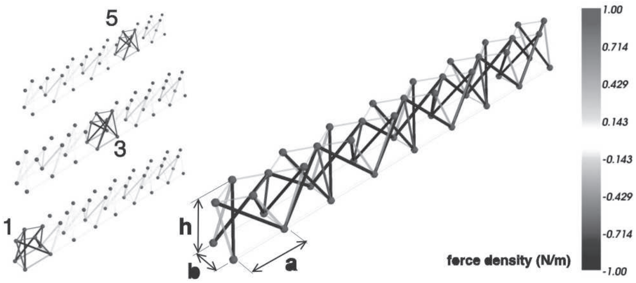


Figure 3. Local selfstress states nb. 1, 3 and 5 and uniform reference state in a tensegrity beam

In the case of modular structures, this base comprises local states geometrically bounded to each module (Figure 3). The initial selfstress being generated by the selfstress base, this organization allows setting the initial stresses module by module. Besides, this localization means that the failure of an element may have an impact only limited to the concerned module. The structure presented here is a linear system formed by juxtaposition of n modules, geometrically defined by their length a, width b and height h, alternatively oriented (Figure 3). The selfstress state is chosen so as to have a uniform level among modules.

2.2 Structural behavior

The reference configuration is a beam with 12 modules such as a=b=c=1 m. The characteristics of the elements are given in Table 1.

struts	cables	nodes
fy = 360 MPa, E = 210 GPa	fy = 1000 MPa, E = 160 Gpa	m = 1 kg
tube D = 60 mm, t = 4 mm	D = 12 mm (layer) and 16 mm (diagonal)	

Table 1. Characteristics of the elements in the reference structure

The structural behavior is analyzed using an explicit dynamic computer code [10] implementing kinetic relaxation, which allows quasi-static analyses while considering various nonlinear effects. Under a uniform vertical loading flin, we observe a response that is typical of tensegrity systems while being scarcely intuitive (Figure 4). In this figure, we define the selfstress level as the highest normal force value before loading.

The normal forces in all elements increase in absolute value (Figure 5) whereas a conventional flexible structure sees compression and traction in its upper and lower parts.

This result can actually be explained by a significant geometric rearrangement of the elements, a mobilization of finite mechanisms that come at the price of an important deflection. In consequence, we observe that high levels of selfstress significantly influence the force-displacement behavior. Anyhow, the vertical displacement remains too large. Adjusting cross sectional areas of elements could

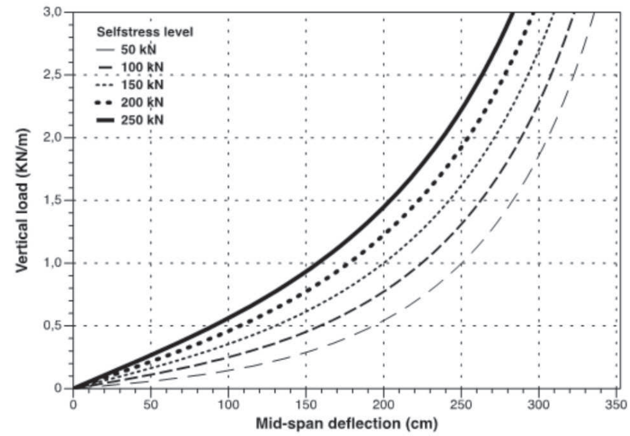


Figure 4. Load - deflection behavior for different selfstress levels

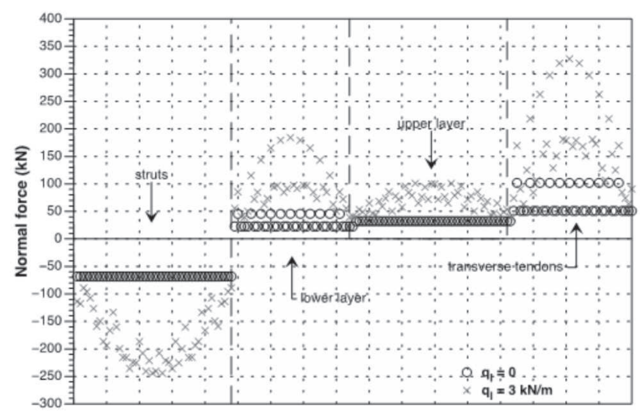


Figure 5. Evolution of internal forces from a selfstress level of 100 kN

be another solution to improve the rigidity, but it affects directly the weight. A major problem for this structure is that it definitively has an intrinsic lack of rigidity under vertical loading.

2.3 Stiffness improvement

Actually, when submitted to bending, this particular structure exhibits an important geometric distortion in its upper layer. This suggests that the flexural rigidity may be improved by blocking theses deformations, which can be done by introducing transverse cables between upper nodes (Figure 6).

The impact of this modification on the flexural behavior is

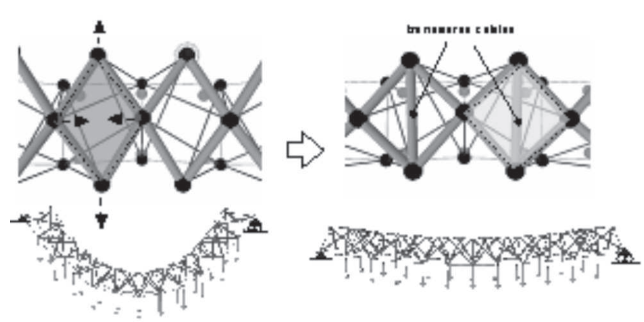


Figure 6. Rigidification by restricting distortion mechanisms in the upper layer (views from above)

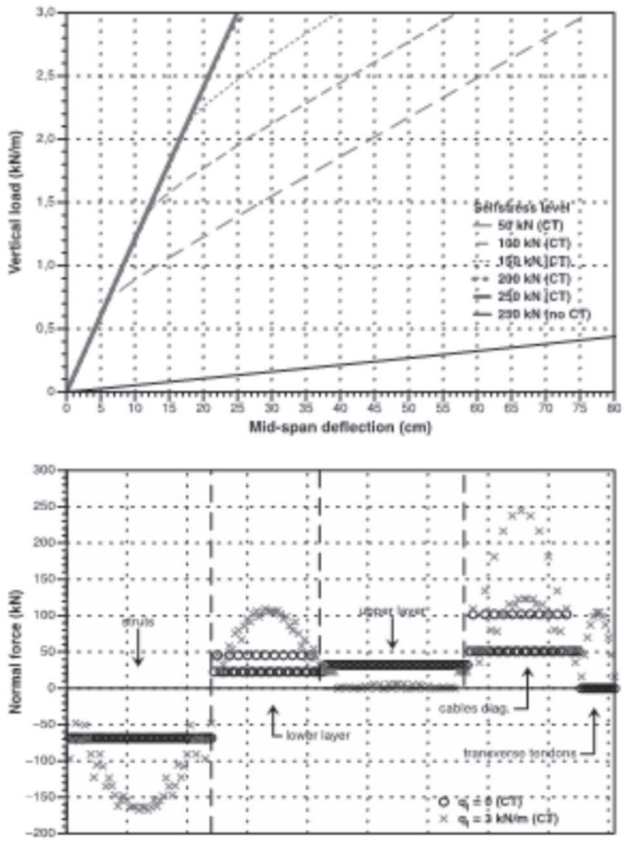


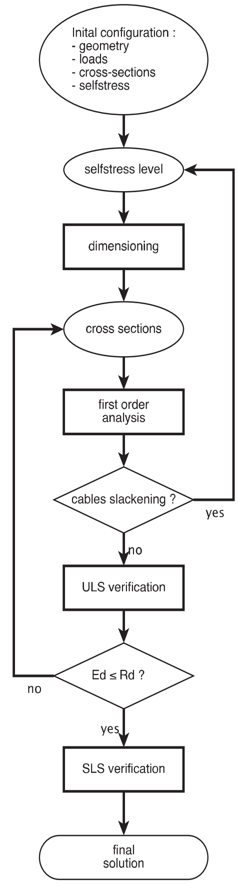
Figure 7. Load - deflection behavior for different selfstress levels (up) and evolution of normal forces for a selfstress level of 100 kN (down) - results given for 12 m beam made from square modules (a = b = h = 1 m) with transverse cables (CT)

consequential. Indeed, the force-displacement curve is significantly stiffer and adopts now a more conventional bilinear form. The initial phase corresponds then to the first order elastic linear behavior of the structure. The following evolution is softer and occurs more or less early with the level of selfstress, revealing the slackening of cables in the upper layer (Figure 7).

Because the mechanisms are blocked, the behavior is now that of a conventional 3D reticulate truss with tense and compressed elements. The difference in our case is that the layers are made of cables maintained in tension only by the selfstress state. In order to keep these elements active, the selfstress level must be adapted to avoid slackening. Under this condition, the behavior is regular and can be characterized using a simple first order analysis.

3 Optimization of the structure

In every structure, cross sections have to be properly determined to verify limit states, which is to sustain the internal forces and to bring sufficient rigidity. In the European design codes [11] taken as reference in this study, Ultimate Limit States (ULS) refer to rare conditions in which the resistance of sections and elements must be ensured. On the other side, Serviceability Limit States (SLS) designate



the nominal conditions up to which comfort, aspect and rigidity are to be guaranteed.

Obviously, cross sectional characteristics, which can be determined from ULS criteria, as well as the geometric configuration have a direct influence on the self weight and the structural behavior. In consequence, an optimal realistic solution that verifies ULS and performs well under SLS conditions can only emerge from an iterative process. We propose for this reason to carry out the design of the modular tensegrity beam using an exploratory study combining an optimum design procedure (Figure 8). For a given geometric configuration and a given external load, its purpose is to determine the minimum selfstress level required and cross sections for every element. The principle is, at first, to control the selfstress level so as to avoid cable slackening in the loaded state, after a first order analysis. Cross sections are also adjusted when necessary until the ULS resistance criteria are verified, on sections and elements, using the buckling curves defined in the EN1993-1-1. Finally, the structural performances such as the total weight and the flexural rigidity are evaluated. This ULS design is done for a set of feasible geometric configurations among which an optimal solution is investigated.

This exploratory study is conducted by varying three geometric parameters. In this case, the number of modules n and the vertical slenderness L/h vary from 5 to 15 while the lateral slenderness L/b from 10 to 40. The span L is fixed to 12 m and the uniform load is taken equal to 4 kN/m, corresponding to the ULS load on one beam that is supporting half of the 2 meters wide footbridge, comprising the dead load of a wood deck. The characteristics of materials are those of the reference structure (Table 1). To evaluate the quality of each designed solution, the total mass mtotal and the first-order mid-span deflection vmax are calculated and combined in a global efficiency indicator Eff (Eq. 2).

$$E_{ff} = \frac{1000}{m_{total} v_{\max}} \tag{2}$$

The values of this indicator versus the geometric parameters are represented in Figure 9.

As can be observed, the number of modules has to be limited because of its direct impact on the structural efficiency. Figure 9.b shows that the width is less influent. Indeed, high

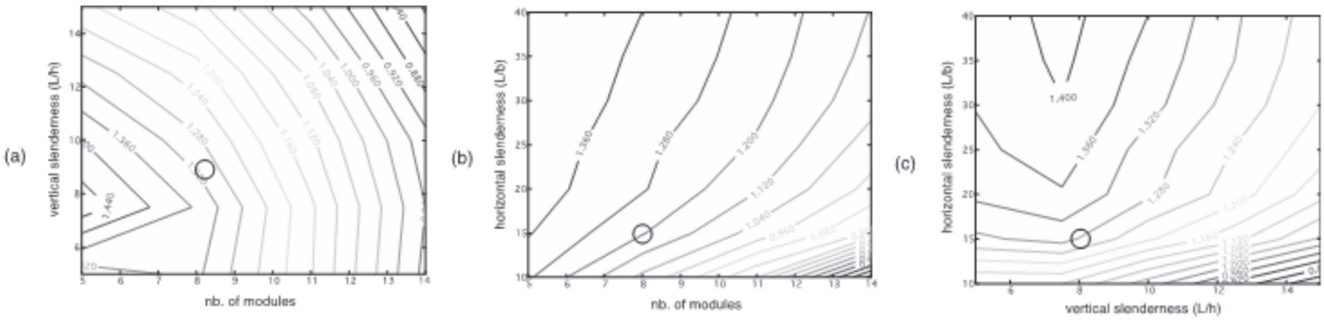


Figure 9. Base 10 logarithm values of the structural efficiency factor for (a) L/b = 20, (b) L/h = 10 and (c) 8 modules

values of L/b result in shorter transverse cables and struts, which helps in reducing the total weight while the flexural rigidity, more influenced by the height, is not affected. Optimum solutions can be found in Figure 9.c among cases comprising 8 modules, L/h = 8 and high values of L/b.

The proposed solution is encircled in Figure 9 and its characteristics are presented in Table 2. We can note that the width had to be increased because a non-linear analysis of the theoretical solution demonstrated that for lower width values, the structure is not stable. Actually, the minimal width for a stable solution with h = 1.3 m (L/h = 9) is approximately 80 cm (L/b = 15), which remains near the optimal zone. This demonstrates that a stability check is necessary consecutively to the exploratory design process.

The SLS performances of the structure are finally evaluated through a non-linear quasi static and a modal analysis. The flexural deflection observed under SLS load is 88 mm (L/130). The first vibration mode of the single beam is a lateral bending oscillating at 2.6 Hz while the others modes have frequencies beyond 5 Hz. When coupled to a second beam and the deck in between so as to model the complete footbridge, the first mode is shifted to 5.7 Hz, which is above the critical range of natural frequencies defined in major design codes (Figure 10).

4 Setting principle

The final setting of the tensegrity beam is simulated using an explicit dynamic computer code that computes the time evolution of a structural system while allowing interacting with it [10]. By taking into account interactions such as contact between elements, this tool allows simulating folding

geometry	h = 1,30 m, b = 80 cm, L = 12 m a = 1,5 m (8 modules)
struts	A = 7 cm^2 (D/t = 30), E = 210 GPa f_y = 360 MPa
cables	E = 160 GPa, f_y = 1000 MPa
	upper layer : A = 0,61 cm^2 (D = 9 mm) diagonal cables : A = 2,28 (D = 17 mm)
	lower layer : A = 1,18 (D = 12 mm) transverse cables : A =1 (D = 10 mm)
selfstress level (maximal normal force): 190 kN m_{node} = 1 kg, r = 7850 kg/m3, mtotal = 438 kg	

Table 2. Raw characteristics of the optimal solution

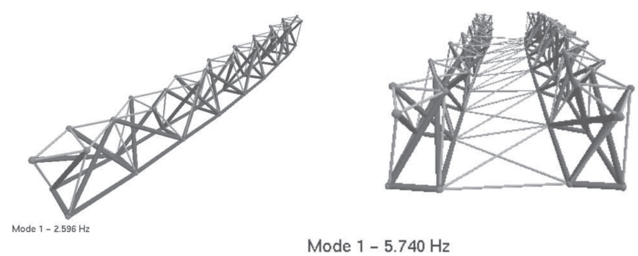


Figure 10. First vibration modes: single beam and footbridge

and deployment processes while controlling the mechanical state of the system and its stability. In this case, the selfstress base is structured as local states, module by module. In consequence, a minimum of 7 diagonal active cables, some of them being shared between two adjacent modules, is sufficient to set up a regular selfstress state in this structure made of 8 modules. The setting of the structure comprises two phases (Figure 11).

The first phase consists in the deployment of the beam without active cables, thus under zero selfstress, from a compact packed state to the final length by imposing a displacement to the end nodes. In this phase, it is assumed that all elements are connected to the nodes using perfect

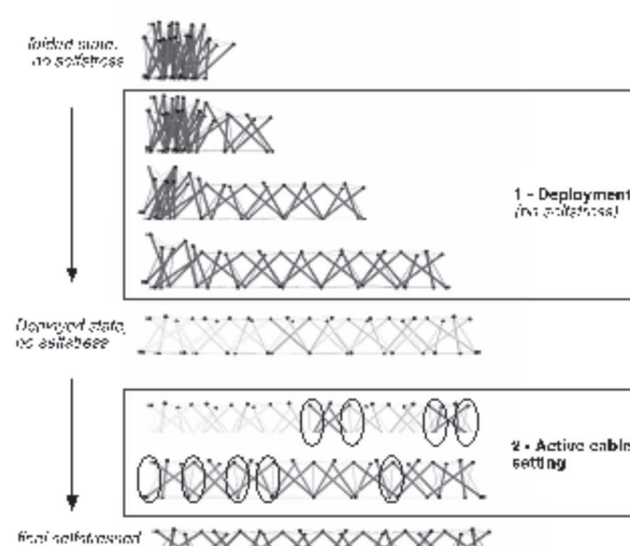


Figure 11. Setting procedure: deployment and active cable setting

hinges. In particular, the joints between struts need to be specifically designed to allow this mobility while ensuring continuity between struts in this temporary state, like on the model of the foldable tensegrity ring [12] built at the LMGC (Figure 12).

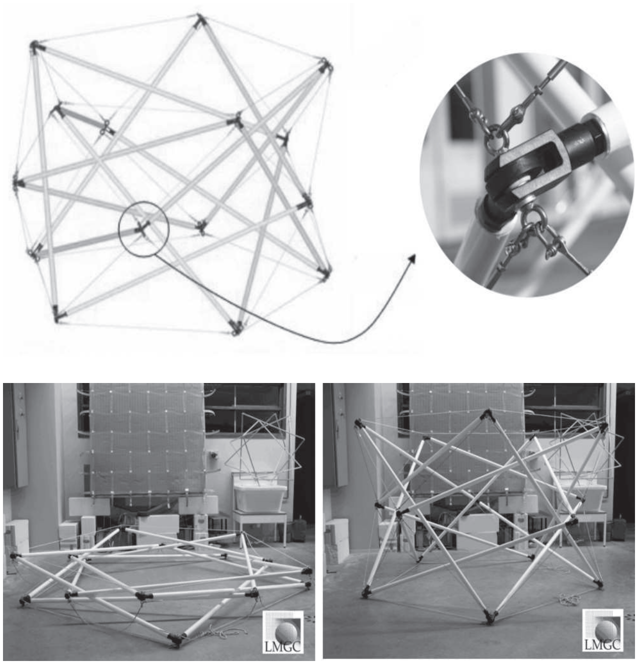


Figure 12. Foldable tensegrity ring [12]

The second phase is the setting of selfstress by adding the active cables and adjusting their length properly [13]. The principle is to introduce first an indeterminate selfstress state, to identify it with direct or indirect force measurements, and to apply corrections in order to make it correspond to the desired target selfstress state. In our case, supposing a perfect initial geometry, a 26 mm shortening of the active cables was necessary to produce a regular selfstress state with a level of 190 kN. The next steps in building a complete footbridge are to deploy two tensegrity beams of this kind and to joint them with the deck.

Conclusion

Tensegrity systems have todays little practical applications in structures. In this paper, we demonstrate that they can be considered as an attractive alternative to conventional solutions. The main steps of a realistic design under ULS and SLS loads are exposed, taking as example a beam supporting a 12 m long footbridge. The proposed solution, found through an optimization exploratory study verify the ULS and comfort criteria under SLS. However, some aspects such as the design of nodes that must allow rotations between struts or the connection of cables need to be developed more deeply in an upcoming study.

Although the proposed solution may not appear as lightweight as a conventional one, improvements are possible in the choice of the materials, especially for struts. This kind of structure has also the distinct capacity of being carried in a folded configuration and deployed quickly and easily. Although there still remains the necessity to set selfstress on site, this solution appears compelling, all the more so as the methodology presented should be applicable to a wider range of configurations.

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Studies on Impact and Permeability Properties of Mixed Fiber Reinforced Concrete

Giuliani Gian Carlo[1], Giuliani Mauro Eugenio[2]

Abstract

The present experimental investigation is conducted by judiciously combining the two types of fibers of glass and steel to make a concrete called Mixed Fiber Reinforced Concrete through which the advantages of these both fibers can be achieved for an efficient concrete. Glass fiber and steel fiber were mixed by volume percentage in different proportions of 0-100, 25-75, 50-50, 75-25 and 100-0 in each of the total fiber percentages of 0.5, 0.75, 1.0 and 1.5. By doing this it is aimed to generate higher Impact strength and better permeability properties of MFRC. Based on the test results, it has been found that mixed fiber combination results in substantial increase in the Impact strength, and the durability properties are improved. The Impact strength of MFRC standard cubes with mixed fibers of 100% steel fiber in 1.0% total fiber content is highest and is 129.42% more compared to reference plain concrete. The failure of MFRC cubes is gradual compared to brittle failure of plain concrete. The MFRC specimens with a total fiber percent of 1.5 are found to have highest resistance to penetrability of chloride ions. With 100 percent glass fiber in the total fiber percent of 1.5, the permeability of the specimens is found to be negligible and is 05 coulombs. There is a decrease in penetrability by 99.91 percent when compared to the base reference specimens. On the whole MFRC gives composites which are efficient with higher Impact strengths, better crack behaviour and better permeability.

Index Terms - FRC, Impact Strength, Chloride Permeability

Introduction

The use of concrete as a structural material is limited to certain extent by deficiencies like brittleness, poor tensile strength and poor resistance to impact strength, fatigue, low ductility and low durability. The role of fibers are essentially to arrest any advancing cracks by applying punching forces at the rack tips thus delaying their propagation across the matrix. The ultimate cracking strain of the composite is thus increased to many times greater than that of unreinforced matrix.

The use of steel fibers in concrete dates from 1910 to improve the properties of concrete. The main interest in fibrous reinforcement of concrete originates from the work carried out in the early sixties by various researchers [1, 2, 3]. Hannant [8] studied the effect of post cracking ductility on the cement mortar and concrete specimens reinforced with short steel fibers. The flexural strength test resulted in enhanced ductility compared to specimens without fibers. Noaman and Shah [16] studied the bond properties of randomly oriented and aligned steel fibers using steel fiber and concluded the fibers have good bonding characteristics and hence improvement in the properties of concrete. Gopalratnam et. al. [7] studied the properties of SFRC subjected to impact loading. The results indicated a higher energy absorption of the specimen compared to specimens without fiber. Majumdar [15] carried out investigations to study the influence of glass fiber reinforcement in cement mortar and concluded that the glass fiber enhances properties of cement mortar. Shah et. al. [18] investigated the toughness durability characteristics of glass fiber reinforced concrete and concluded that glass fibers exhibited higher energy absorption leading to a more durable concrete.

Combinations of two dissimilar fibers are increasingly being used. These combinations are known as hybrid fibers. The resulting hybrid fiber reinforced concrete performance exceeds the sum of the individual fiber performance due to positive interaction between the fibers. The production and properties of these new materials has been the subject to considerable research and development effort in the last decade. N. Banthia et. al. [17] studied the resistance offered by the hybrid fibers in concrete using steel fiber and polypropylene fiber in the cement matrix. After conducting crack test they concluded that polypropylene fiber enhances the efficiency of steel fiber.

The study includes the introduction of glass fiber into concrete at various percentages, combining glass fiber with steel fibers to enhance impact strength and permeability properties.

Experimental Investigation

The following materials are used for the casting of specimens.

Cement

Ordinary Portland cement of 53 Grade from Ultra Tech conforming to I.S: 12269 [14] is used.

Fine Aggregate

River sand locally available is used as fine aggregate conforming to I.S: 2386 and I.S: 383 [11, 9].

Coarse Aggregate

Machine crushed well graded angular granite aggregate of nominal size from local source is used.

Water

Potable water locally available is used for mixing and curing the concrete.

Glass Fiber and Steel Fiber

Fibers of alkali resistant glass with an aspect ratio of 857:1 and steel fiber with an aspect ratio of 55 are used conforming to ASTM C 1666M [6] and ASTM A 820M [4]. The details are given in table 1.

Steel	AR-Glass	Fiber
Steel wire	Cem- FIL ARC 14 306 HD	Type
7850	2600	Density kg/m^3
210	73	Elastic modulus GPA
250	1700	Tensile strength MPA
1mm	14 micron	Dia.
55	12	Length mm
Mono filament	212 million /kg	No. of fiber

Table 1 Properties of Fibers (Glass and Steel)

M 25	Grade
425	Cement (kg)
682	Fine aggregate (kg)
1277.76	Coarse aggregate (kg)
0.5	Water cement ratio

Table 2 Materials Required for 1 Cubic Meter of Concrete

Concrete Mix Details

The details of the M25 Concrete mix used are given in table 2 is arrived at as per I.S: 10262 [10, 12, 13].

Preparation of the Salt Solutions

Sodium Chloride solution is made with 3 % by mass in distilled water, abt. 99.5 % LR and molecular weight 58.44. The molarity is 0.51 M. Sodium Hydroxide solution is made with 0.3 M in distilled water, purity 97 % LR and molecular weight 40.

Sealant

Ana bond and Fixit Silicone sealant are used for sealing the specimens in the cells.

Preparation and Testing of the Test Specimens for Impact by Drop Weight Impact Apparatus

M25 concrete mix with various percentages of mixed fiber was considered in the present investigation. The mix was cast in disc moulds of size 150 mm diameter x 63.5 mm thick. For each varying proportion of replacement of glass fiber by 0%, 25%, 50%, 75% and 100% in the mixed proportions, five discs for each were casted with 0%, 0.50%, 1.00% and 1.50% of total fiber for testing at the age of 28 days. The concrete is poured into moulds in 1 layer and the surface was smooth finished.

Demoulding of the specimens was done after 24 hours and specimens were kept immersed in a curing tank for curing. After curing the specimens in water, the specimens of MFRC are removed at the age of 28 days and were tested as per ACI 544.2R procedure. The number of blows for ultimate failure is noted. The results are shown in table 3 and figure 1.

Preparation and Testing of the Test Specimens for Rapid Chloride Permeability Test.

The concrete mix with various percentages of mixed fiber with and without microsilica was cast in cylindrical moulds of size 50 mm diameter x 200 mm height for each varying proportion of replacement of glass fiber by 0 %, 25 %, 50 %, 75 % and 100 % in the mixed proportions were casted with 0 %, 0.50 %, 1.00 % and 1.50 % of total fiber for testing at the age of 28 days.

Demoulding of the specimens was done after 24 hours and specimens were kept immersed in a curing tank for curing. After curing the specimens in water, the specimens of MFRC are removed at the age of 28 days and were tested as per ASTM C 1202 (5) for evaluating the electrical conductance of concrete samples to chloride ion penetration. The results are shown in tables 4, 5 and figure 2.

Results and Discussions

Impact Strength of MFRC

In base reference specimens (control), the impact strength is found to be minimum. There were no visible signs of first crack. The failure is brittle. The number of blows resisted by the control specimens at ultimate fracture is found to be 136.3 (Referring to table 3).

The specimens with fibers are observed to have more resistance to impact failure when compared to control specimens. With increase in total fiber percentages upto 1.0, the impact strength of the MFRC specimens is found to increase. The MFRC specimens with a total fiber percent of 1.5 have shown lesser resistance and drop in number of blows when compared to the MFRC specimens with 1.0 percent total fiber. This may be due to some balling effect of the fibers. But the impact strengths of these specimens are found to be more than the impact strengths of the specimens with other total fiber percentages.

In the various percentages of mixed fibers in different total fiber percentages of 0.5, 0.75, 1.0 and 1.5, the specimens with 100 percent steel fibers are found to have highest resistance against failure by drop weight impact. The impact strength is observed to be decreasing with increase in percentage of glass fiber. The minimum numbers of blows, the MFRC specimens have resisted are found in specimens with 100 percent glass fiber but the impact strengths of these specimens are more when compared with control specimens.

The MFRC specimens with a total fiber percent of 1.0 are found to have highest energy absorption in terms of resistance to number of blows dropped on them and hence highest increase in impact strength is observed in specimens with 1.0 percent total fiber. In the same 1.0 percent total fiber and 100 percent steel fiber, the number of blows resisted by the specimens at first crack and at ultimate failure are found to be 288.0 and 312.7. There is an increase in ultimate blows of these specimens by 8.58 percent over the first crack. With 100 percent glass fiber in the total fiber percent of 1.0, the numbers of blows resisted by the specimens at first crack and at ultimate failure are found to be 188.3 and 196.3. There is an increase in ultimate blows of these specimens by 4.25 percent over the first crack. In the same total fiber percent of 1.0 and with mixed fibers of 25 percent glass fiber and 75 percent steel fiber, the numbers of blows resisted by the specimens at first crack and at ultimate failure are found to be 246.7 and 267.0. There is an increase in ultimate blows of these specimens by 8.85 percent over the first crack. It is observed that the specimens with mixed fibers of 25 percent glass fiber and 75 percent steel fiber are found to have highest resistance over the first crack to ultimate failure. This is true in the specimens with other total fiber percentages.

Permeability Properties of MFRC without Microsilica

In base reference specimens (control), the chloride permeability is found to be maximum. The permeability in the control specimens is found to be high and it is 5411 coulombs. The specimens with fibers are observed to have more resistance to chloride permeability when compared to control specimens. With increase in total fiber percentages upto 1.5, the chloride permeability of the MFRC specimens is found to decrease. The MFRC specimens with a total fiber percent of 1.5 have shown higher resistance to chloride penetrability when compared to the MFRC specimens with other different total fiber percentages.

In the various percentages of mixed fibers in different total fiber percentages of 0.5, 0.75, 1.0 and 1.5, the specimens with 100 percent steel fibers are found to have more permeability. The chloride penetrability is observed to be decreasing with increase in percentage of glass fiber. The minimum permeability is found in specimens with 100 percent glass fiber.

The MFRC specimens with a total fiber percent of 1.5 are found to have highest resistance to penetrability of chloride ions. In the same 1.5 percent total fiber and 100 percent steel fiber, the chloride penetrability is found to be very low and is 106 coulombs. There is a decrease in penetrability by 98.04 percent when compared to the base reference specimens (control). With 100 percent glass fiber in the total fiber percent of 1.5, the permeability of the specimens is found to be negligible and is 05 coulombs. There is a decrease in penetrability by 99.91 percent when compared to the base reference specimens. In the same total fiber percent of 1.5 and with mixed fibers of 25 percent glass fiber and 75 percent steel fiber, the penetrability is found to be negligible and is 48 coulombs. There is a decrease in penetrability by 99.11 percent when compared to the base reference specimens.

Conclusions

Impact Strength of Mixed Fiber Reinforced Concrete (MFRC) The following conclusions are drawn.

1.There is an increase in number of blows resisted by the

S.No.	Total Fiber (%)	Mixed Fiber (%)		Average Number of Blows		Increase in Ultimate Blows (%)	
		Glass	Steel	First Crack	Ultimate Failure	over the First Crack	over the Base Reference Specimens
1	0	0	0	--	136.3	--	--
2	0.50	0	100	244.3	257.0	5.19	88.55
3	0.50	25	75	223.7	236.3	5.63	73.15
4	0.50	50	50	205.7	216.3	5.15	58.69
5	0.50	75	25	183.7	191.0	3.97	40.13
6	0.50	100	0	156.0	161.7	3.65	18.63
7	0.75	0	100	274.0	291.7	6.45	114.01
8	0.75	25	75	246.7	267.7	8.51	96.40
9	0.75	50	50	225.3	237.7	5.19	73.88
10	0.75	75	25	197.7	206.7	4.55	51.65
11	0.75	100	0	160.0	166.7	4.19	22.30
12	1.0	0	100	288.0	312.7	8.58	129.42
13	1.0	25	75	277.7	302.3	8.85	121.79
14	1.0	50	50	260.0	276.0	6.15	102.49
15	1.0	75	25	237.7	251.0	5.59	84.15
16	1.0	100	0	188.3	196.3	4.25	44.02
17	1.5	0	100	219.3	233.0	6.24	70.95
18	1.5	25	75	203.3	220.0	8.21	61.41
19	1.5	50	50	190.0	201.0	5.78	47.47
20	1.5	75	25	181.3	189.7	4.63	39.18
21	1.5	100	0	162.3	169.0	4.13	23.99

(Base Reference Specimen = 0% Fibers) Table 3 Impact Values of MFRC Disc Specimens

Specimen No.	Total Fiber %	Mixed Fiber (%)		Average Charge Passed (Coulombs)	Chloride ion Permeability	Charged Coulombs as per ASTM C 1202
		Glass	Steel			
1	0	0	0	5411	High	> 4000
2	0.50	0	100	2127	Moderate	2000 -4000
3	0.50	25	75	1225	Low	1000 -2000
4	0.50	50	50	727	Very low	
5	0.50	75	25	524	Very low	100 -1000
6	0.50	100	0	208	Very low	
7	0.75	0	100	1741	Low	1000 -2000
8	0.75	25	75	884	Very Low	
9	0.75	50	50	571	Very low	
10	0.75	75	25	324	Very low	
11	0.75	100	0	103	Very low	
12	1.0	0	100	413	Very low	100 -1000
13	1.0	25	75	378	Very low	
14	1.0	50	50	148	Very low	
15	1.0	75	25	105	Very low	
16	1.0	100	0	76	Negligible	< 100
17	1.5	0	100	106	Very low	100 -1000
18	1.5	25	75	48	Negligible	
19	1.5	50	50	24	Negligible	< 100
20	1.5	75	25	23	Negligible	
21	1.5	100	0	05	Negligible	

Table 4 Chloride Ion Permeability of Mixed Fiber Reinforced Concrete Specimens

Total Fiber %	Mixed Fiber (%)		Percent Decrease in Permeability of Specimens over the Base Reference
	Glass	Steel	
0	0	0	----
0.50	0	100	60.69
0.50	25	75	77.36
0.50	50	50	86.56
0.50	75	25	90.32
0.50	100	0	96.15
0.75	0	100	67.82
0.75	25	75	83.66
0.75	50	50	89.45
0.75	75	25	94.01
0.75	100	0	98.09
1.0	0	100	92.36
1.0	25	75	93.01
1.0	50	50	97.26
1.0	75	25	98.05
1.0	100	0	98.59
1.5	0	100	98.04
1.5	25	75	99.11
1.5	50	50	99.56
1.5	75	25	99.57
1.5	100	0	99.91

Table 5 Permeability of Chloride ion in Mixed Fiber Reinforced Concrete Specimens with Various Percentages of Microsilica

fiber reinforced specimens when compared to specimens with no fibers showing better impact strength. The MFRC specimens contributed better to impact resistance beyond the first crack occurrence to ultimate fracture of the specimen

2. The specimens with 100% steel fibers have exhibited higher resistance to impact failure in all the total fiber percentages and the maximum was observed in total fiber percent of 1.0.
3. The mixed fibers of 25% glass and 75% steel showed uniform fracture.
4. Beyond the optimum total fiber percent of 1.0 the specimens with higher percentage of total fiber of 1.5 showed

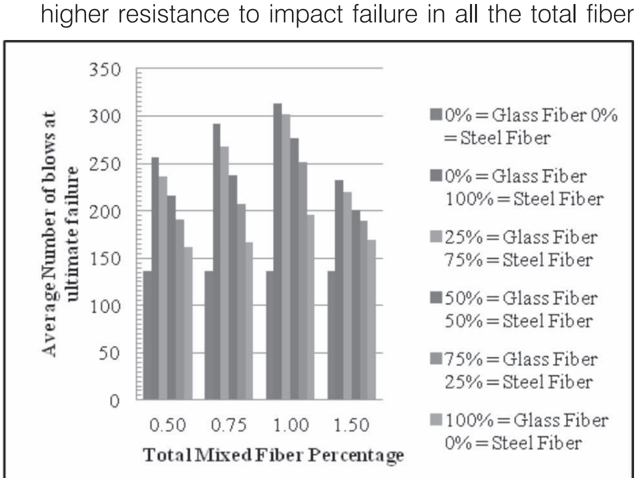

Figure 1 Comparison of Ultimate Impact Blows of MFRC Specimens with Various Percentages of Mixed Fibers in Varying Total Fiber Percentages

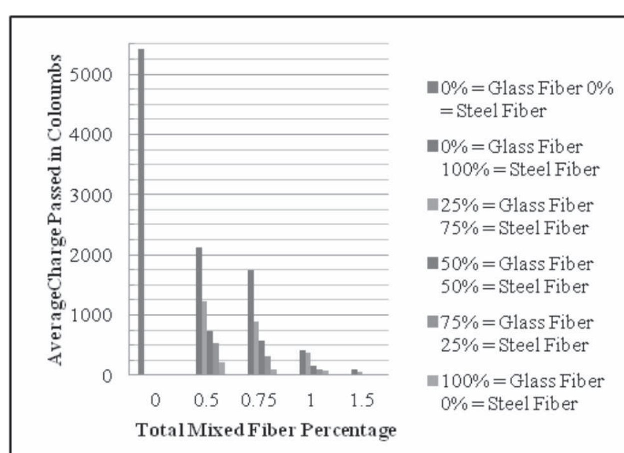

Figure 2 Comparison of Average charge Passed in Coulombs of MFRC Specimens with Various Percentages of Mixed Fibers in Varying Total Fiber Percentages

decrease in the resistance to impact fracture and fractured at lesser number of blows when compared to specimens with 0.75% and 1.0% total fiber. This may be concluded as occurring due to the balling effect of fibers in the higher percentages.

Permeability Properties of MFRC

The following conclusions are drawn.

1. With increase in total fiber percentages the penetrability of the MFRC specimens was found to decrease.
2. The maximum resistance to charge coulombs passing was observed in specimens with total fiber percent of 1.5.
3. The specimens with 100% glass fiber showed more resistance to penetrability of chloride ions.

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The Life and Times of Pier Luigi Nervi - The "Architect" and The "Builder"

Bhavani Balakrishna

Engineer-architect Pier Luigi Nervi (1891-1979), also known as the "Master Builder", is one of most versa tile and innovative structural engineers of the 20th century. Each of his creations, spread across Italy, Europe, American and Australia, is a synthesis of engineering and architecture, aimed at the "simultaneous satisfaction" of "functional, economical and aesthetic" requirements. Architectural historian, Sir Nikolaus Pevsner described him as "the most brilliant artist in reinforced concrete of our time". Nervi was said to have the rare combination of competence and knowledge of an engineer, the imagination of an architect and the practical wisdom of a businessman. He could switch to designing, drawing, calculating, modeling, writing or teaching effortlessly.

While his contributions to the industry in reinforced concrete and prefabrication are certainly noteworthy, what set him apart was his fundamental "truthful" and "correct" approach to engineering and architecture that still holds true even in this era. During the Age of Reason in the nineteenth century, while engineers dealt with practicality of construction, architects had sadly resorted to decorating in styles of ages past. It was against this backdrop that Nervi that focused on the relationship between structure and form and the science and art of building and, more generally, between engineering and architecture. Nervi always remained a designer and builder throughout his life, non-conforming to the practice of isolation of architecture from engineering. This only shows that Nervi was truly an independent thinker and innovator who did not let traditions or prejudices obstruct his vision of finding beauty in structural coherence.

Pier Nervi was born in Sondrio, Italy on June 21, 1891. Nervi, the son of a postmaster, graduated from the School of Civil Engineering in Bologna in 1913. He gained practical training with building firms specializing in concrete construction, in Bologna before World War I and in Florence afterward. He joined the army engineering corps when Italy became involved in World War I. When the war was over, he joined a group called "The Society for Concrete Construction" but his innovative approach to architecture received critical attention only in 1932 when he had already left the group.

When Nervi graduated from Bologna in 1913, engineers in Italy were beginning to realize the significance of reinforced concrete. Working for a firm of cement contractors, Nervi

Photo 1: Pier Luigi Nervi under the Viaduct of Corso Francia, Rome, ca. 1960, Photo Oscar Savio

learned the great number of uses of concrete. In 1923 Nervi established his own firm, Nervi and Nebbiosi Engineering Co., in Rome choosing reinforced concrete as the basis of his construction technique. His first all-concrete building was a small cinema in Naples, built in 1927. It was around this time that Nervi and others recognized that the material could accommodate an experimental approach using continuous curving systems. Nervi's mastery over reinforced concrete is sufficient expression of his love for its adaptability. "Concrete is a living creature which can adapt itself to any form, any need, any stress," he once said.

In 1929, he built the Municipal Stadium in Florence which went on to establish his reputation. The stadium, made of reinforced concrete, has a grandstand roof cantilevering some 55 feet and exterior stairs of cantilevered spirals.

In 1932, Nervi formed a new firm, Nervi and Bartoli Engineering Co, and that company in 1936 developed a pair of airplane hangars for the Italian Airforce in Orvieti which featured endless diamond patterned ribbing. These hangers

Fig 1: ACI Honorary Member Pier Luigi Nervi, 1891-1979

had a reinforced concrete roof made up of a lattice of diagonal bow beams, 6 inches (15 centimeters) thick and 3.7 feet (1.1 meters) deep, intersecting at about 17-foot (5-meter) centers. They supported a deck of reinforced, hollow terra-cotta blocks covered with corrugated asbestos-cement. The single-span roof measured 133 by 333 feet (40 by 100 meters), and its weight was carried to the ground through concrete equivalents of medieval flying buttresses. The 30-foot-high (9-meter) doors that accounted for half of one of the long sides of the hangar were carried on a continuous reinforced concrete frame.

For the Orbetello, and Torre del Lago airplane hangars that were built in 1939-42, Nervi used the same structural plan he had employed in Orvieto in 1936, but instead of using complex and expensive wooden moulds that he had needed to erect the great ribbed vault in Orvieti, Nervi divided the ribs into as few small and identical elements as possible. These individual elements were prepared in series in moulds that could be reused dozens of times. The elements were then lifted into place by using a light scaffolding system and joined with concrete pour. Once the work was completed, there was no trace of this fragmentation and the resulting structure proved to be statically monolithic.

Photo 2: Palazzo del Lavoro in Turin, Italy

Paul Goldberger of the New York Times described them as "graceful, flying forms of concrete." Unfortunately, all eight hangars were destroyed by dynamites of the retreating German forces in 1944. The hangars were also the first structure for which Nervi used reduced scale models made at Milan Polytechnic by Guido Oberti apart from static calculations to check the validity of his own original structural conception in the final phases of the project. He later went on to use this procedure for most of his later works.

Around this time, Italy being engaged in the Great War, steel was primarily reserved for military purposes and rationed for any other uses. Reinforced concrete, which requires steel for the reinforcement, was no longer viable. Hoping for better times, Nervi started experimenting alternative solutions. During several such experimentations, Nervi discovered "ferrocemento". Nervi superimposed various layers of metal netting formed by small diameter (1 mm or less) steel wire. He then pressed mortar made of cement and sand (but no gravel) into the metallic mesh using a trowel or float .This operation was conducted on one side of the mesh until the cement emerged from the other side after having saturated it. Naturally, the operation also depended on the plasticity of the mortar, which had to be accurately prepared. Both horizontal and vertical slabs could be made in this manner and they did not require wooden moulds to shape the mortar until it hardened. The extreme subdivision and the uniform distribution of the reinforcement in the concrete created a material that was different from ordinary reinforced concrete- a homogeneous, isotropic, elastic material that resists both traction and compression - ferrocemento. Thus, by the end of the war, Nervi possessed a new material, "ferrocemento", and a new building technique, structural prefabrication.

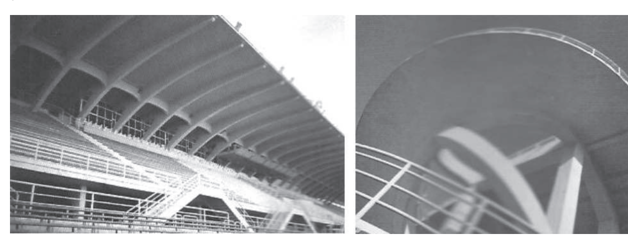


Fig 2: (left and right): Nervi’s first great work: the stadium in Florence, 1930

Post war, Nervi's first signature achievement was the design and building of an exhibition hall, the Salone Agnelli, in Turin (1947-1949) using "ferrocemento" and the prefabrication techniques he had developed for the hangars. The hall is rectangular and covers an area of 240 feet x 309 feet. On one of the two shorter sides is a semi-circular apse. Windows are arranged in the corrugation of the prefabricated roof elements. A semicircular apse 132 feet in diameter adjoins the main hall which is 240 feet long. Its roof consists of corrugated pre-cast units. The half-dome roof of the apse is also constructed with prefabricated elements. The vaulted construction of the hall consists of prefabricated elements which spring from in situ concrete abutments. The prefabricated units are of "ferro-cemento" and

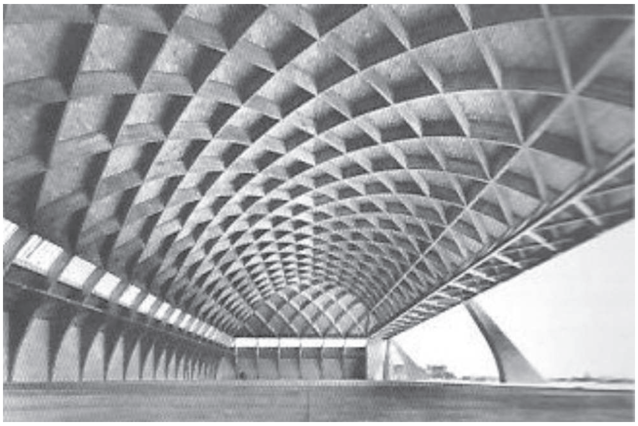


Fig 3: Hangar at Orvieto, 1935

have a length of approximately 15 feet and a width of 8 feet 3 inches. The thickness of the curved pre-cast parts is less than 2 inches. This small thickness is achieved only by the increased rigidity through the corrugation and the transverse webs at either end. The individual units are joined by in situ concrete. Originally the design envisaged a curved sheet of glass as a division between the hall and the adjoining apse. The lower edge of the curved glass was to continue the line of the dome down to the floor. The V-frames were to carry the half-dome of the apse taking the thrust in the inclined supports. In the final proposal, the elegance of the curved sheet of glass as a demarcation line to the enclosed space is disrupted by openings and the placing of supports. The half-dome of the apse rests abruptly on the inner supports of the gallery. The load-bearing construction of the half-dome consists of prefabricated units, which had been cast in concrete moulds. In situ concrete is cast between these prefabricated units, the underside of which forms part of the ceiling. The Exhibition Hall C built in 1950 consists of a low rectangular building with a shallow central dome resting on four supports. The dome and the roof of the surrounding arcade are of prefabricated units. The units employed for the dome are of the same type as those of the half-dome. For the roofing of the arcade corrugated units of "ferro-cemento" " were used, The arches spanning between the four supports of the dome are inclined so as to correspond approximately to the direction of forces from the dome.

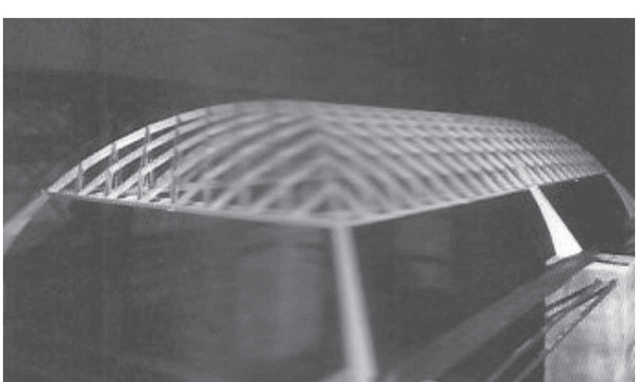


Fig 4

Nervi's first significant construction outside Italy was the UNESCO Headquarters in Paris (1953-58) on which he worked with M. Breuer and B.H. Zehrfuss. Its most remarkable feature is the fascinating folded structure in exposed concrete of the walls and roof.

Pier Luigi Nervi designed and constructed four structures for the 1960 Rome Olympic games: three sports facilities and a viaduct serving the Athlete's residential quarter, the Villaggio Olimpico. These are considered some of his most famous international works. The Palazzetto dello Sport (Small Sports Palace; 1956-57; with Annibale Vitellozzi) in the Flaminio area of Rome is characterized by its sixty-meter diameter dome, which is held up -or better, held down- by 48 radial, pre-fabricated, Y-shaped struts whose divergent upper arms develop the rim "decoration". The externally smooth dome only reveals its large rhomboidal ribbing internally. The Palazzo dello Sport (Sports Palace; 1956-59; with Marcello Piacentini) in the EUR area is also dome-shaped, but with a hundred-meter diameter. The internal surface is characterized by minute pleated V-shaped

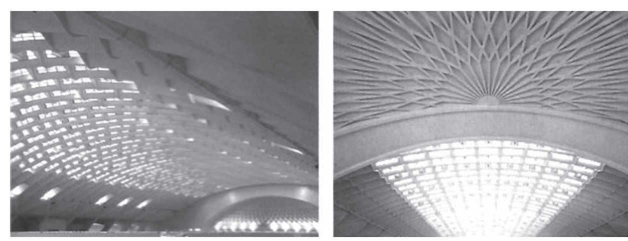


Fig 5: (left and right) Central hall (Hall B) of Turin Exhibition Complex, Turin, Italy, 1948

"waves". Externally, the dome is concealed by a high glass cylinder, which only partially reveals the structure of the perimetral stands. The Stadio Flaminio (Flaminio Stadium; 1956-59; with Antonio Nervi) is characterized by the numerous moulded frames, upholding the slim ridged canopy covering the grandstand, which vary in width along the oversail and completely flatten out along the rim. The Corso Francia Viaduct (1958-60), also in the Flaminio area, is characterized by varying width pillars -the sections change from rectangular to cross-shaped and the transitional surface from one to the other in a hypar surface, formed by straight

Fig 6 a: Pier Luigi Nervi in front of one of his ferrocement boats

Fig 6 b: Nervi reintroduced the use of ferrocement: experimental ware-house in ferrocement, La Magaliana, Rome, Italy, 1945

lines- and by the V-shaped beams, precast and prestressed. The works erected for the 1960 Olympic Games represented a crucial milestone in Pier Luigi Nervi's career.

For the Palace of Labor in Turin (1960-1961) Nervi combined reinforced concrete with steel (designed by G. Covre) to create a large rectangular hall filled with a forest of tree-like structures forming ceiling and support. At the Burgo Paper Mill outside Mantua (1961-1962) he used steel cables (also by Covre) suspended between concrete piers to create a clear span of 525 feet.

Fig 7: Sports Palace in Roma, Italy, 1958-60, P.L Nervi with Marcello Piacentini

The Bus Station at the George Washington Bridge in New York City, famous for the butterfly-like wings of exposed concrete that make up its roof, and the Field House for Dartmouth College in New Hampshire (both 1961-1962) were erected at the same time that he was delivering the Charles Eliot Norton Lectures at Harvard University. Nervi's bus station design helped him win fame in the United States, but its design was altered later because open spaces between the concrete wings, left to allow bus fumes to escape, let cold winds off the Hudson River intrude.

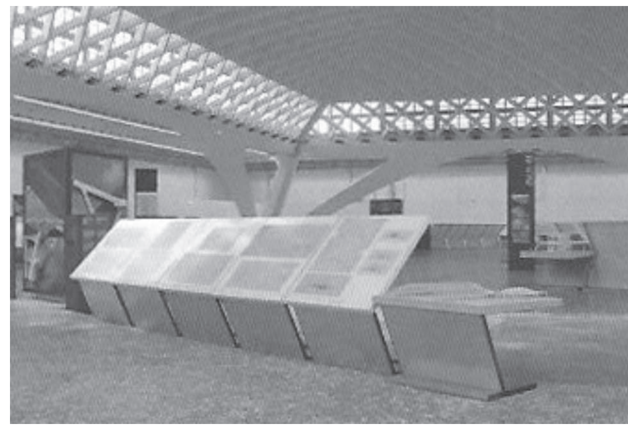


Fig 8: Hall C of Turin Exhibition Complex Turin, Italy, 1950, whiite hosting the International Nervi Exhibition, 2011

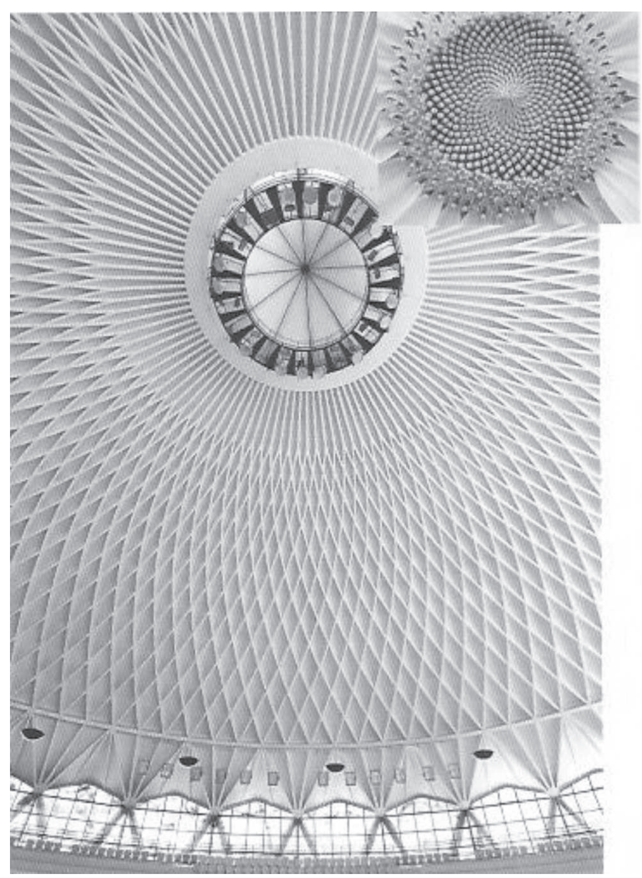


Fig 9: The small Sports Palace in Rome, Italy, 1957, P.L. Nervi with Antonio Vitellozzi. The elegant of the concrete ribs seems to be inspired by the geometrical network of a sunflower core

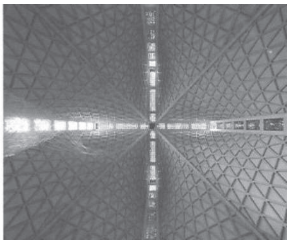


Fig 10: (left and right) St. Mary's Cathedral, San Francisco, CA, 1963-71, Pier Luigi Nervi with Pietro Belluschi, Mc Sweeney, Ryan & Leew and Leonard F. Robinson (Structural engineer)

In the years 1946-61, Nervi was also a Professor of engineering at Rome University. From 1960-78 he again started his own office in collaboration with his sons Antonio (Architect), Mario (Engineer) and Vittorio (Architect). Nervi died in Rome in 1979.

Some of his other notable works include Kursaal (Ostia, 1950), the Sala delle Feste (Chianciano, 1952), Field House at Dartmouth College (USA, 1961), Skyscraper in Montreal's Victoria Square (1962-65, with L. Moretti), the Australia Square skyscraper in Sydney (1962-67, with H. Seidler), the Ponte Risorgimento in Verona (1963-68)the Norfolk Scope arena (USA, 1968, with Williams & Tazewell and W. Blum), the Papal Audience Hall in the Vatican (1966-71)the St Mary's Cathedral in San Francisco (1971, with P. Belluschi) and the Italian Embassy in Brasilia (1979, with his son Antonio).

Through his designs, Nervi successfully reinforced that re-

Fig 11: Victoria Square Tower, Montreal, QC, Canada, 1961-66, Pier Luigi Nervi with Luigi Moretti

inforced concrete was not only a significant structural material but also an artistic design medium. Nervi was nominated Honorary Member of important institutions like the American Institute of Architects, the American Academy of Arts and Letters, the Royal Academy in London, the International Association for Shell Structures (IASS), the American Concrete Institute (ACI), and elected member of the Académie des Beaux Arts of the Institut de France in Paris. Designated Charles Eliot Norton Professor of Poetry at Harvard in 1961-62, he was nominated Doctor honoris causa of the universities of Edinburgh, Munich, Harvard, Varsavia, Budapest, etc., and was bestowed many prizes and medals, like the Royal Gold Medal for Architecture in 1960, the Emil Mörsch Medal in 1963 and the Feltrinelli prize of the National Academy of Lincei in Rome in 1968. In 1964 he received the Gold Medal of the American Institute of Architects, the highest honor in American architecture.

In his writings, Nervi constantly reminded readers that 90 percent of his contracts were awarded in competitions where the governing factors were economy and speed of construction. With reference to the concept of "correctness", he writes in the book "Aesthetics and Technology in Build-

Spring Convention Sessions on Nervi and the Art of Building in Concrete

Pier Luigi Nervi is regarded as one of the most inventive structural engineers of the twentieth century. Named an ACI Honorary Member in 1969, Nervi worked at the intersection of the art and science of construction. He has been described as "the most brilliant artist in reinforced concrete of our time."

In 2009, on the 30th anniversary of Nervi's death, a broad research and educational program was promoted with the intent of disseminating Nervi's cultural legacy and exploring the complexity of his extraordinary stature as a structural artist. The program culminated in the international traveling exhibition "Pier Luigi nervi - Architecture as Challenge," highlighting some of his most celebrated works. The exhibition is cosponsored by ACI in recognition of Nervi's ACI Honorary Membership and is expected to tour in North America in 2013.

On March 18, Mario A. Chiorino, FACI, Emeritus Professor of Structural Analysis, School of Architecture, Politecnico di Torino, Turin, Italy, will celebrate Nervi as the grand master of concrete structures and will present a preview of the international exhibition at the International Lunch of the ACI Spring 2012 Convention in Dallas, TX. In two technical sessions on the theme "Structural Concrete: An Art Form," additional speakers will review the work of other eminent pioneers and discuss recent trends in the merging of architecture and structural engineering.

Go to www.concrete.org/Convention/Spring-Convention/Front.asp for more event details.

Fig 12: Pirelli Tower, Milan, Italy, 1955-58, Pier Luigi Nervi with Arturo Danusso and Gio Ponti

ing" that minimum that should be achieved in building is stability, durability, use of materials in accordance with their natural properties, and functional and economic efficiency. He said, "To search for an economic solution in the structural field means to find the most natural and spontaneous solution … to find the method of bringing dead and live loads down to foundations in the most direct way and with minimum use of materials."

Elaborating on the conflicts of architects and engineers, he says, " The loftiest and most difficult problems arise in architecture from the need to realize a synthesis between opposing sets of factors: harmony of form and the requirements of technology, heat of inspiration and the coolness of scientific reason, freedom of imagination and the iron laws of economy". The aim is thus the "simultaneous satisfaction" of "functional, economical and aesthetic" requirements.

Fig 13: (left and right) Patazzo del Lavoro in Turin, Italy, 1959-61, Pier Luigi Nervi with Antonio Nervi and Gino Covre

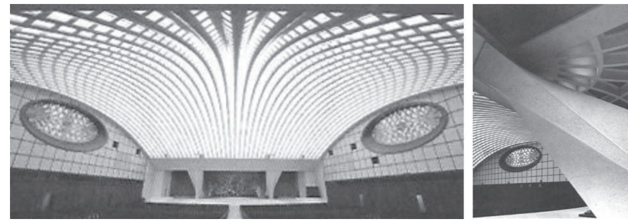


Fig 14: (upper and lower) Papal Audience Hall, Vatican City, Rome, Italy, 1963-71, Pier Luigi Nervi with Antonio Nervi

Nervi was a keen experimenter - he relentlessly searched for new construction methods and new materials. Hence, it should come as no surprise that he had several invention patents in the period 1950-60. He strongly believed that, "if a structural project is approached without a close study of possible methods of execution, and based only on conventional technique, it is in danger either of being a dead letter or of having to be much modified during execution."

While he himself was a "creative engineer", he felt that the "masterpieces of the past" were a result of "structural architecture, led by a synthesis of static-aesthetic sensitivity, technical knowledge and mastery of execution" which could be "achieved through combining the ability and specialized knowledge in a single individual or through a genuine co-operation between different specialists".

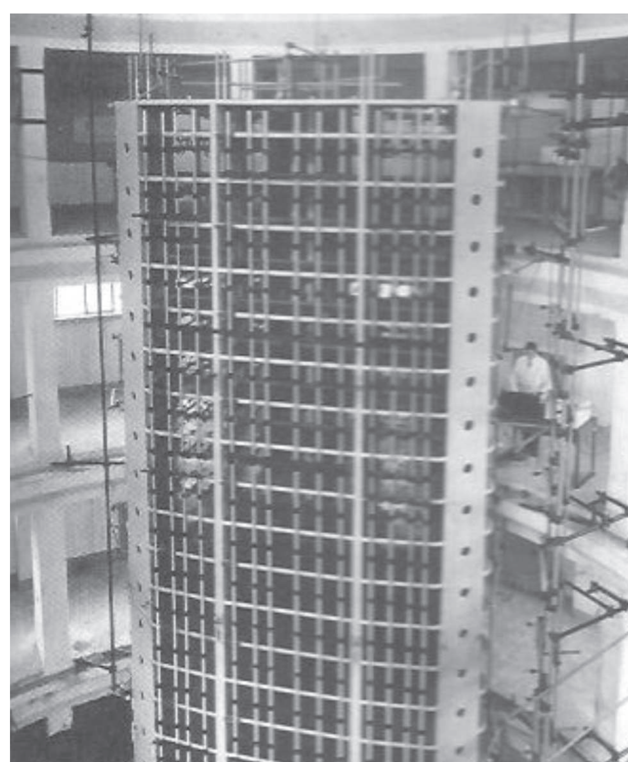


Fig 15: Microconcrete large-scale model (1:15) used for tests to failure for the Pirelli Tower, Milan, Italy

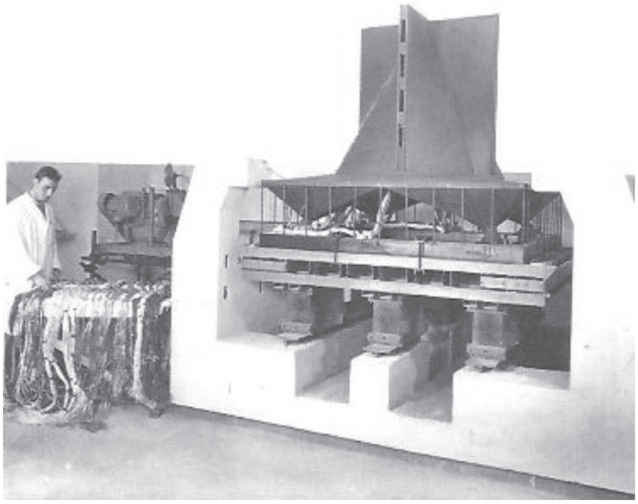
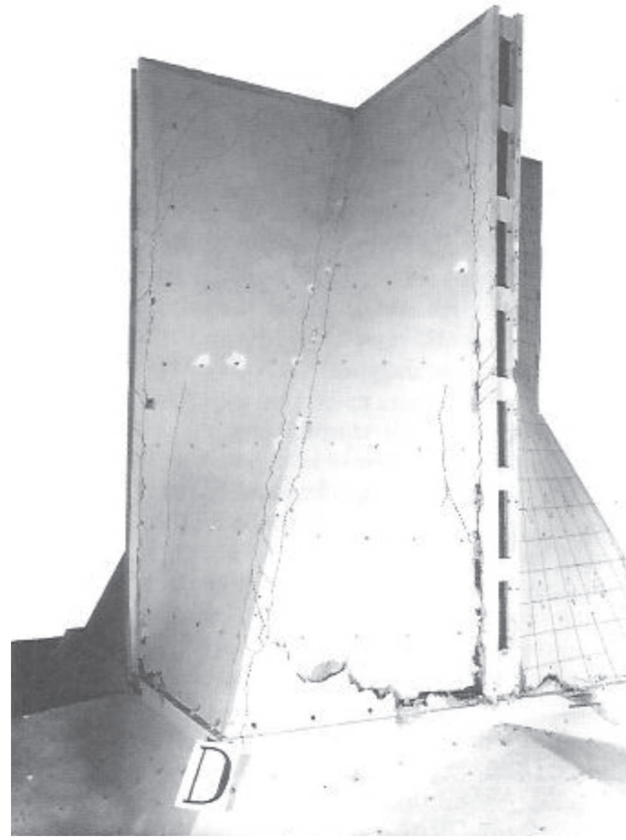


Fig 16: St. Mary's Cathedral, San Francisco, CA: (above) elastic model (1:37) under dynamic test; and (below Microconcrete model (1:15) show following the ultimate limit state tests

This engineer-architect-builder genius who has several patents and masterpieces to his credit had only to say this about his approach - " If I ask myself what were the principles by which I approached my problems I see clearly that a single aim and a single method have always governed my work as a creative engineer. The first step in any design was the search for the most suitable structural solution technically and economically; then followed patient and passionate work: the detailing and calculation of the various structural elements so as to refine the form and thus meet the static and structural requirements. In examining and deciding upon the most suitable structural scheme, the most varied solutions gradually emerged. In so doing, I was always anxious to arrive at an independent decision free from existing aesthetic theories as well as from solutions I or others had already discovered."

Nervi's persistent desire to find "correctness" and "truth" in all aspects of architecture and building science has inspired architects and builders of later generations, to create structures that utilize new forms of material and new forms of construction while balancing function, economics and aesthetics.



Scale (1:100) model was used for the wind tunnel tests, two medium-scale (1:40 and 1:37) resin models were used for static and dynamic tests in the elastic field (with special attention to seismic tests due to the building's location)

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Pier Nervi Projects

Australian Embassy Paris

The Australian Embassy in Paris is located 400 metres southwest of the Eiffel Tower, on Rue Jean Rey in the 15th arrondissement of Paris, near the Bir-Hakeim bridge on the Seine. The embassy is situated on a triangular shaped block, and comprises a pair of nine-storey buildings. The Chancellery Building houses Australia's missions to France, to UNESCO and to the OECD, and the apartment of the ambassador to France; the other building contains 34 staff apartments, all with views of the Seine and the Eiffel Tower. It is regarded in Paris as being a building of architectural importance.

The embassy, and several pieces of its original furniture, were designed in a modernist style by Australian architect Harry Seidler, with Marcel Breuer and Pier Luigi Nervi as consulting designers.

The Embassy complex comprises two buildings, one of seven floors housing the Chancery and the residence of the Australian Ambassador to France, the other of nine floors comprising 29 apartments and the residence of the Australian Ambassador to the OECD. The buildings are linked at ground floor level and share a common basement incorporating car park, staff amenities, and building services and plant. The 29 apartments are a mix of two, three and four bedroom configurations ranging in size from 120m^2 to 250m^2. The building includes lifts dedicated to residential use, and staff amenities include a small indoor heated swimming pool, squash court, and common-use recreation lounge.





The Embassy was built from precast modularized concrete, with a quartz and granite faced exterior and prestressed precast floors. Its two buildings are curved to form two quarter circles, the two arcs of an "S"-shaped complex, with the radii of the circles lined up to match the axes of the Eiffel Tower and the Champ de Mars.

The land for the embassy, formerly housing a disused Paris Métro station, was purchased by the McMahon government of Australia in 1972. Construction started on the Embassy in 1975, and it was completed in 1977. The Australian Embassy in Paris was formally opened in 1978.

The building was designed and constructed to comply with the requirements of both the local and Australian building codes of the period, although it no longer fully meets current standards. The remaining useful life of the complex, subject to completion of mid-life refurbishment works and ongoing maintenance programs, is at least 25 years.

The true art of Luigi Nervi is the ability to close the gap between art and technology to create spaces that border on poetry without renouncing, in the conversion of the inspiration into a design and of the design into a construction, the modus operandi of engineers, but rather emphasizing engineering procedures with original and innovative contributions.

Cathedral of Saint Mary of the Assumption

The Cathedral of Saint Mary of the Assumption, also known locally as Saint Mary's Cathedral, is the principal church of the Roman Catholic Archdiocese of San Francisco in San Francisco, California. The first original cathedral was built in 1854 and still stands today and is now known as Old Saint Mary's Church. In 1891, a second cathedral was constructed but was destroyed by arson in 1962. The present-day cathedral was commissioned just as Vatican II was convening in Rome. Prescriptions of the historic church council allowed the Archdiocese of San Francisco to plan boldly in the building of its new cathedral. That resulted in the modern design of the present structure. The cornerstone for construction was laid on December 13, 1967.

Completed in 1971, this Roman Catholic cathedral - the

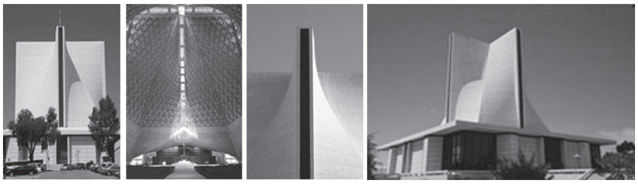


Figure: Various Views of the Cathedral (Source: Internet)

Mother Church of the Archdiocese of San Francisco - soars 190 feet (58m) into the air and is topped with a 55-foot-tall (17m) golden cross. It is the third St. Mary's to serve the people of San Francisco. The oldest - dubbed Old St. Mary's - still sits at California and Grant Street, at the border of Chinatown. The other was destroyed in a fire.

The architectural style of this cathedral is usually described as Expressionist Modern. Designed by Pier Luigi Nervi and Pietro Belluschi, the cathedral flows upward in graceful lines from each of its four corners, meeting in the middle to form a cross.

Its saddle roof is composed of segments of hyperbolic paraboloids in a manner reminiscent of St. Mary's Cathedral in Tokyo, which was built earlier in the decade. Due to its resemblance to a large washing machine agitator, the cathedral has been nicknamed "Our Lady of Maytag" or "McGucken's Maytag".

The reinforced concrete roof is covered with white Italian marble. The four corner pylons support the cupola, which rises to 19 stories. The pylons extend down 90 feet (27m) into the bedrock in order to provide more stable support. According to architect's records, the inner surface of the cupola is made up of 1,680 pre-cast triangular coffers of 128 different sizes, designed to distribute the weight of the cupola.

The windows are huge and provide wonderful views of the city of San Francisco, and the red brick floor - which may seem out of place to some - is meant to reflect California's Spanish Mission heritage.

A kinetic sculpture by Richard Lippold sits above the altar. Fifteen stories high and weighing one ton, this modern piece consists of 14 tiers of triangular aluminum rods.

The church has a very unusual and modern architecture, a curve, square based pyramid rising gracefully to a cruciform point. The motif of the cross is continued both inside and outside the Cathedral, with coloured tiles ascending the sides of the building. The layout inside is also very modern.

The building was selected in 2007 by the local chapter of the American Institute of Architects for a list of San Francisco's top 25 buildings.

Exhibition Building, Turin, Italy

Designed and constructed immediately after the War, the Hall B at the Turin Exhibition allowed Nervi to explore the principle of structural prefabrication with considerable use of prefabricated elements and use of ferroconcrete (steel mesh and small diameter steel rods cast in a thin concrete pour). While the Società del Palazzo delle Esposizioni had commissioned the engineer Roberto Biscaretti di Ruffia to construct a new exhibition hall, to be used as a showcase for Turin's automobile industry, atop the remains of the Palazzo della Moda, designed in 1936 by Ettore Sottsass and bombed during the War, it was Nervi & Bartoli who were awarded the project.

The complex was built in two stages, the first structure being completed in 1949 and the second a year later. The initial building, Salone B (Agnelli), was spatially, materially, and structurally the more extravagant of the two. The hall is rectangular and covers an area of 240 feet x 309 feet. On one of the two shorter sides is a semi-circular apse. Windows are arranged in the corrugation of the prefabricated roof elements. A semi-circular apse 132 feet in diameter adjoins the main hall which is 240 feet long. Its roof consists of corrugated pre-cast units. The half-dome roof of the apse is also constructed with prefabricated elements. The vaulted construction of the hall consists of prefabricated elements which spring from in situ concrete abutments. The units are of "ferro-cement" and have a length of approximately 15 feet and a width of 8 feet 3 inches. The thickness of the curved precast parts is less than 2 inches. This small thickness is achieved only by the increased ri-

gidity through the corrugation and the transverse webs at either end. The individual units are joined by in situ concrete.

The second building, Salon C, consists of a low rectangular building with a shallow central dome resting on four supports. The dome and the roof of the surrounding arcade are of prefabricated units. The units employed for the dome are of the same type as those of the half-dome. For the roofing of the arcade corrugated units of "ferro-cement" were used, The arches spanning between the four supports of the dome are inclined so as to correspond approximately to the direction of forces from the dome.

To overcome the severe winter months and reduce the construction time, Nervi came up with a three-part process wherein the first two parts could proceed simultaneously. On site, the main structural supports of Salone B were built using poured-in-place reinforced concrete, whereas off-site, the prefabricated sections of the vaulted roof were cast independently using ferro-cement. Once the main structural piers and floors were in place, the installation of the roofing members proceeded without the need for additional formwork. The individual roof sections, measuring 8 by 13 feet and folded in the profile of a sinusoidal wave, were aligned and arched into the desired profile of the vault and bound to each other using poured-in-place concrete, the whole made possible by the use of movable metal scaffolding. The final phase of construction required bonding the roof shells to the poured-in-place piers. This was achieved via the on-site pouring of reinforced-concrete rib beams. They spanned from pier to pier and were poured within the upper and lower extremities of the roof section. It was this system of in situ concrete beams that ensured that the prefabricated roof shells and the structural piers worked together monolithically.

Field House at Dartmouth College 1963

The Leverone Field House at Dartmouth College in the United States of America is among the early projects that Pier Luigi Nervi completed in the North American continent. The legend goes that Dartmouth College's Business Manager, Richard W.Olmstead was a big fan of Nervi and approached him to design the sports facility.

The connection between Nervi and uniquely designed stadiums had been firmly established in the aftermath of the 1960 Rome Olympics. The Leverone Field House completed in 1963 is yet another example of the engineer-architect's penchant for large span roofs. After all, Nervi is credited with the design and execution of the radically airy concrete hangars that he built for the Italian Air Force. This is apart from the several sports arenas that he built as part of his work for the 1960 Rome Olympics.

The Leverone Field House which houses 91,800 sq ft of indoor track facilities, a weight room, practice areas for rugby, softball, golf, lacrosse and football, stands testimonial to the world famous designer's to think out of the box and come out with truly incredible structural and architectural ideas. It is not surprising that the Leverone Field House is counted among the global masterpieces of reinforced concrete thin shell sports facilities.

The most striking aspect of the sports facility is the roof that spans an area, covering more than two acres, which is roughly equivalent to two football fields. The sweeping expanse of the barrel-vaulted roof is the most striking aspect of the design. The roof is 63 ft high at the center, its tallest point. Nervi was always known for his penchant for geometrical shapes and it is no different in the case of the Leverone Field House. A series of diamonds and triangles adorn the roof inside the stadium.

Curtain walls of glass and steel have been used to seal up the buildings ends, perfectly complimenting the unique design of the roof. The sports facility has since undergone a makeover and now features an advanced sound system, apart from renovation for better lighting. The floor has also been resurfaced in recent times. Certain structural changes have also been made to the roof, apart from the removal of a few doors in order to facilitate optimum utilization of natural light.

George Washington Bridge Bus Station, New York City (1963)

The George Washington Bridge Bus Station in the Washington Heights area of Manhattan in New York City is among the few buildings that Pier Luigi Nervi did outside of Italy. The world-renowned engineer-architect built this landmark

bus station which is owned and operated by the Port Authority of New York and New Jersey. According to estimates the bus station serves about 17,000 passengers and 950 buses on a typical weekday.

The bus station has three levels and lies above a subway train station at the 175th street in Manhattan. The bus station has been constructed over the busy Trans-Manhattan Expressway, a twelve -lane highway. The bus station was constructed at a cost of approximately US $ 16, 00,000. The construction of the bus station involved creation of 26 triangular sections made up of poured concrete and steel. Out of the 26 rectangular sections, 14 slope upwards forming a butterfly effect. Each one of the triangular sections measure 92 by 66 feet. Another interesting feature is the exposed concrete. The overall architectural design of the building has been conceived to complement that of the George Washington Bridge.

The bus station's architecture and engineering was appreciated, with it receiving the 1963 Concrete Industry Board Award as the "structure that represented the best in conception, originality, and applicability in concrete for both design and construction."

The bus station was originally conceived as a replacement for a series of sidewalk bus loading areas that existed in the particular area of Manhattan. Another interesting feature of the bus station is the murals of Othmar Amman, the civil engineer who designed the George Washington Bridge and that of George Washington. The upper tier of the bus station has ample space containing the waiting area for passengers and shops.

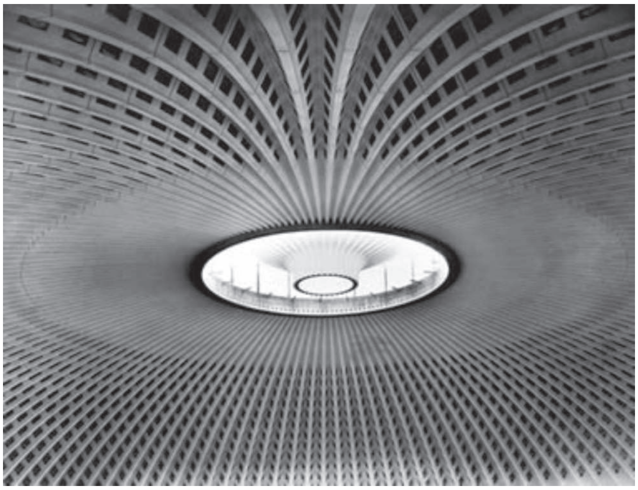
The George Washington Bus Station is yet another representation of Nervi's design prowess. The intricately crafted reinforce concrete forms that form the unique butterfly formation of the bridge has made it one of the iconic structures in Manhattan, an area that is already filled choc-a-block with landmark buildings and sculptures. The seamless integration of the bus station with the adjoining surroundings is another highlight of the project, which was one of the few that Nervi did outside Italy.

Palazzo dello sport EUR (now PalaLottomatica)

For the 1960 Olympic Games, Nervi designed two domed sport arenas, the Palazzetto dello Sport (small sport palace) and the larger Palazzo dello Sport. Both arenas have a double curved shell roof construction which is built using prefabricated aced shaped ferrocement elements filled with poured-in-place concrete.

The Palazzo dello Sport (Sports Palace; 1956-59; with Marcello Piacentini) in the EUR area is dome-shaped, but with a 100 meter diameter. The internal surface is characterized by minute pleated V-shaped "waves". Externally, the dome is concealed by a high glass cylinder, which only partially reveals the structure of the perimetral stands. The Palazzo dello Sport seats 16000 people under the webbed ceiling which is stabilised by a tension ring.

The "wave elements" used in the dome of the Palazzo dello Sport are made up of V-shaped elements that vary between 4 and 5 meters and have large side apertures with perforated diaphragms to allow the passage of the lighting system and other installations.

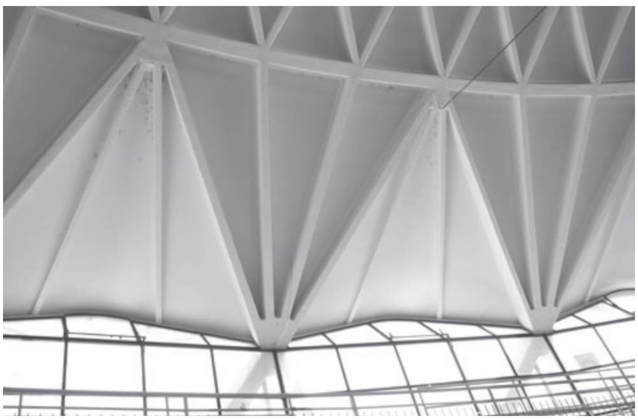


Pallazetto dello Sport(small sport palace)

The Palazzetto dello Sport (Small Sports Palace; 1956-57; with Annibale Vitellozzi) in the Flaminio area of Rome is characterized by its 60 meter diameter dome, which is held up -or better, held down- by 36 radial Y-shaped struts whose divergent upper arms develop the rim The externally smooth dome only reveals its large rhomboidal ribbing internally.

At the top, daylight enters via a compression ring and a cupola. The base of the shell is supported by exterior Y-shaped buttresses which rest on a pre-stressed reinforced concrete ring. The ribbed sections are all prefabricated, joined by in-situ poured concrete at their tops and bottom and gathered at the ends so that the loads are transmitted through prefabricated triangular sections into 48 joints that rest on the ring formed by the upper section of seating. This in turn is supported by in-situ columns with angular, warped surfaces. The flexibility of ferro-cemento and precast elements had allowed Nervi to create lightweight, curved forms that would have been near impossible with traditional poured concrete which settles with gravity.

Precast "rhomboidal elements" were employed to create the entire dome of the Palazzetto dello Sport.The 12 types of elements all have the same thickness (2,5 cm). They were prefabricated in series by using a mould created with a model. Once these elements were mounted on the scaffolding, only resting on two points - the reinforcement was placed in the channels and the cast was executed. The reinforcement also extends to the extrados of the dome where a 3 cm thick layer of concrete was cast.



Paper Mill, Mantua, Italy (1962)

The Paper Mill in Mantua, Italy is considered one of the most challenging projects that was done by Pier Luigi Nervi. The building completed in 1962 spans over a sprawling 86,000 sq ft of space that was designed for containing modern machinery for paper manufacturing.

The roof of the building is what distinguishes it from other similar industrial designs of those times. Nervi chose the roof design since there was a need for a clear span of 525 ft for allowing future expansion of the plant.

The overall length needed for placing the machine that would transform wood pulp into paper was 100m. The building had to be conceived as one that would be a big box that would hold the machine.

The designer had to take into consideration the future expansion factor. The design had to be such that it could be flexible enough to allow for the expansion of the space in such a manner that an identical machine could be accommodated parallel to the existing machine. Moreover, provision also had to be made to ensure that there was enough space between the two machines in order to allow the staff in the factory to work freely without any space constraints.

Another challenge confronting the designer was that for at least 150 meter the entire area had to be free from vertical structural elements. This was after considering all the working areas that would make up the industrial complex. Nervi had to come out with a solution for the roofing element of the building without having columns inside the building.

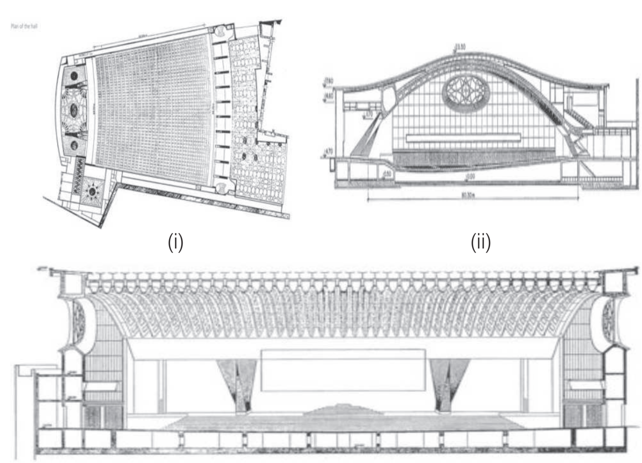
The architectural and structural engineering genius came up with yet another innovative idea in the form of a suspended roof that would not be supported by columns inside the building. Nervi came out with the solution that has been used in the construction of suspension bridges. The roof of the factory complex consists of a steel deck that is suspended using four steel cables. While the central span is of 535 feet, with two side cantilevers of 140 ft each. The other highlight of the building is the 164 ft high reinforced concrete supports that hold the roofing element. The building is considered one of the most unique and innovative designed for industrial premises and counts among Nervi's great works.

Paul VI Audience Hall

The Paul VI Audience Hall is a building in Italy, mostly in Rome but partially in Vatican City, but the Italian part of the building is an exterritorial area of the Holy See used by the Pope as an alternative to Saint Peter's Square for conducting his Wednesday morning General Audience. The building, with a seating capacity of 6,300, was designed in reinforced concrete by the Italian architect Pier Luigi Nervi and completed in 1971. One of the more arresting features of the hall is the twenty-meter-wide brass and bronze sculpture La Resurrezione ("The Resurrection") by Pericle Fazzini.

The Audience Hall, like all of Nervi's projects, attains its form from the demands of the program and the constraints of its material, Ferro cement. This perfection of this particular material is one of Nervi's masterpieces, the culmination of decades of research and testing

In plan, the main space of the Audience Hall takes on a trapezoidal shape, tapering at the stage (see Figure 1(i)). It is a subtle gesture but one that achieves one of the primary goals of the Hall-to focus the audience on the Pope. This is reinforced in section-all of the structural lines converge at the front of the hall and are anchored by large columns that flank the stage (see Figure 1 (ii) and (iii)). In the same way that the spiritual intensity of a papal audience is focused on the stage, the dynamism of Nervi's project is focused on a single space.



(i) (ii) (iii)

Figure 1: (i) Reflected Roof Plan, (ii) W-E Section through the Main Space, (iii) N-S Section through Main Space

What truly distinguishes the Audience Hall from Nervi's body of work is the treatment of natural light. The ribbed blocks of Ferro cement that make up the spanning elements are perforated; the ceiling of the Hall is essentially comprised of scores of these small apertures. In the Audience Hall, the ceiling takes on an ethereal quality; it is constantly changing in accordance with the quality of natural light.

A prime example of this challenging or rethinking can be found in the material choices for the Audience Hall-

"Every element in the structure is in fact made of white cement, blended with special inert matter containing fragments of Apuan marble. All the surfaces are left exposed and no finishing material is used, ensuring that cement, traditionally considered a poor, sad material, is here given the same worth as the precious stones employed in the nearby basilica."

The choice of marble from the Apuan Alps is an important one. In its materiality, the building is rooted in its surroundings (the Apuan Alps are a mountain range in Tuscany). Nervi finds a certain degree of dignity in using local materials. Nervi's decision to leave the Ferro cement unfinished has roots in modernist theory. The materiality of the Paul VI Audience Hall is also a challenge to conventional religious thought, particularly pertaining to the design of sacred spaces. Nervi's use of concrete, a "poor, sad material," is no accident. It is a commentary on the futility of relying on precious materials to create a sense of the divine-in his Audience Hall.

Nervi demonstrates that "Space achieves sanctity through structure and thoughtfully designed spaces, not expensive skins or claddings."

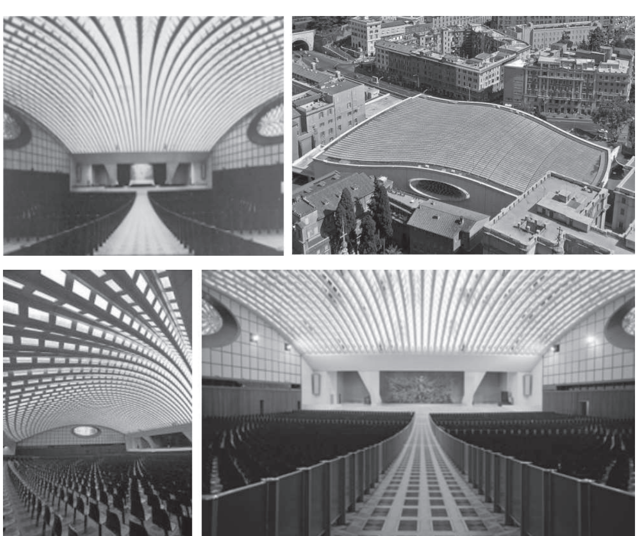


Figure 2: Various views of Paul VI Audience Hall (Source: Internet)

Sacro Cuore (Bell Tower), Florence (1962)

No mention of Pier Luigi Nervi could be complete without discussing on the Chisea del Sacro Cuore, which when translated means 'The Church of the Sacred Heart', located in Florence. The renowned architect was involved with the design of the imposing Bell Tower in the iconic church complex.

The construction of the church was carried out in different phases and makes for some interesting reading. It was first, Ludovico da Casoria, a Franciscan priest and the founder of the Congregation of Frati Bigi, who had the church built between 1874 and 1877. The church's original design has been modeled on San Salvatore al Monte.

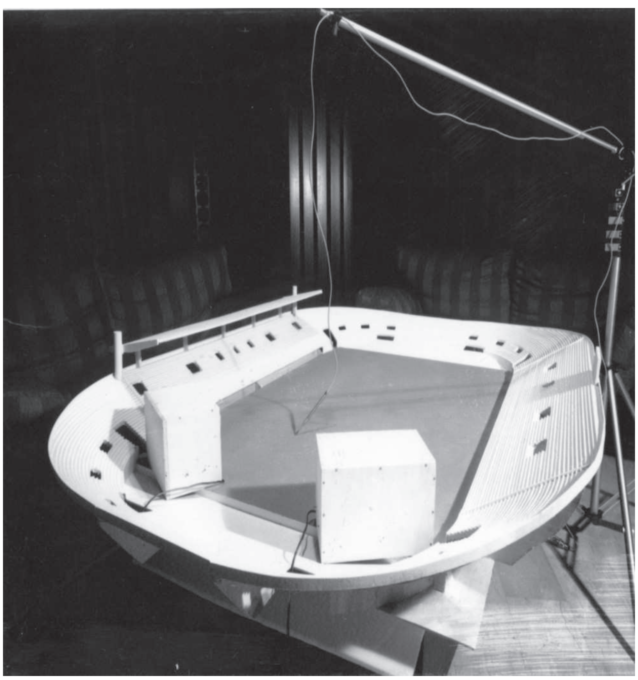
It was in 1941 that it was decided to give a facelift to the church. A complete restructure was ordered and this is when architect Lando Bartoli was heading the project. The restructuring took place between 1956 and 1962. A modern bell tower was conceived as part of the restructuring project. This is when Bartoli sought the assistance of Nervi. The imposing bell tower is to date considered an architectural marvel.

The design of the church incorporates innovative engineering ideas. Eight pilasters made of reinforced concrete are what hold the bell tower on the top. While the bottom of the pilasters forms the portal of the church the top sustain the bell. Adding to the architectural aesthetics is the delicate ornamental work that can be found throughout the structure.

The church could be said to be a work where several well-known personalities worked together, along with Nervi. For example, bronze art work on the front doors has been done by well known sculptor Angelo Biancini. A stained glass window in the church featuring the 'Resurrection' is credited to Marcello Avelani. Other prominent works of art that adorn the church include a 'Last Supper' by Giovvani Stradano, 'Stations of the Cross' by Giovanni Haynal, and the 'Apparitions of the Sacred Heart' by Antonio Ciseri.

The architecture of the building is such that every component perfectly complements the others. For example the interiors of the church feature mahogany beams, which perfectly complements that light-toned marble with exquisite design patterns that makes up for the flooring. The project is counted among the top projects with which the world-class architect was involved with during his lifetime.

Stadio Flaminio, Rome (1957)



The Stadio Flaminio is one of the outstanding works of Pier Luigi Nervi. Located along the Via Flaminia, three kilometers northwest of Rome's city centre, the stadium has hosted several sports events. The interior spaces of the arena include a covered swimming pool, rooms for fencing, amateur wrestling, weightlifting, boxing, and gymnastics.

When it was built in July 1957, it was conceived as a venue for football matches. The Stadio Flaminio had in fact served as the venue for the football final of the 1960 Rome Olympics. The stadium which can seat over 30,000 spectators has a main grandstand that shows the renowned architects master touch. The main grandstand with a cantilevered roof is one of the highlights of the architecture of the sports arena. The roof's long span with its folded precast plates is another distinguishing feature of the stadium.

One of the important features of this engineering and architectural marvel is the use of various different shapes that has been incorporated in the structural design, while at the same time taking care of the specific functional needs of a sports facility. While several other modifications have taken place since the stadium was first used, the basic structural elements remain the same. The sweeping expanse of the stadium makes it ideally suited for a variety of sports, whether it is rugby, football, or even musical concerts.

The attention to detail which is a hallmark of all the works of Pier Luigi Nervi is also evident in this particular work of the great architect and engineer. The design of the stadium and the seating arrangement in particular is praiseworthy. Every spectator in the stadium is assured of a clear view of the goings on in the ground in the centre. In fact, this is one of the reasons behind the popularity of the stadium over the years.

The stadium has been actually built on a preexisting stadium which was demolished for the purpose in 1957. The original design of the stadium was intended to seat more than 40,000 people in the stadium. This was later on reduced for adaptation to safety standards. The addition of temporary stands made up of tubular material has been tried out in recent years in order to increase seating capacity, particularly during the rugby season. There have also been proposals put for permanently increasing the seating capacity in recent times.

The Pirelli Tower, Milan

In 1950 Alberto Pirelli, the president of the Pirelli Company, required that a skyscraper be built in the original area where the first factory was constructed in the 19th century. The project was developed by architect Gio Ponti, with the assistance of Pier Luigi Nervi and Arturo Danusso. At 127.1 meters, it was the tallest building in the city and was built of concrete approximating 60,000 tons. Construction of the tower began in 1956 in a time that Italy was experiencing an economic boom. The tower was to be surrounded by low lying buildings on a hectagonal plot of land. Upon its completion in 1958, it became one of the symbols of Milan and of the national economic recovery.

Characterized by a bold, structural skeleton, smooth refined curtain wall façades, and tapered sides like the bow of a ship, it was among the first skyscrapers to abandon the customary block form. Its facade was clad with thermopane windows, held in place by anodized aluminum mullions and positioned outside the floor slabs to achieve the effect of an unbroken curtain wall.

The roof, supported by a separate structure, seemed to float over the building. The main problem which the erection of a tall slender structure poses is the provision of wind bracings - two rectangular supports which branch out in the upper floors serve as cross bracings. The width of a support in the basement is 6 feet 7 inches tapering to 12 inches at the top of the building. The triangular end walls form lateral wind bracings to some extent but they are mainly for longitudinal stiffening. A curtain wall of 108,000 square feet is attached to the structure. The floors span right across without intermediate supports, approximately 79 feet. This unusual span is achieved by rows of pre-stressed reinforced concrete beams at 5 feet centres, which have a depth of 2 feet 6 inches.

Thompson Arena at Dartmouth College

Dartmouth's Business manager Richard W. Olmstead was a fan of Luigi Nervi and visited him in Rome, admiring the engineer's repetition of cool, crystalline geometric units to create refined and poetic concrete buildings. Thompson Arena at Dartmouth College in New Hampshire USA was built in 1975, is a 3,500-seat hockey arena in Hanover, New Hampshire. It is home to the Dartmouth College Big Green men's and women's ice hockey teams. The building is named for trustee Rhode Island banker and conglomerateur Rupert C. Thompson, Jr.('28). The building replaced Davis Rink, the original "indoor" home of Dartmouth hockey from 1929 to 1975.

Pier Luigi Nervi had designed the Thompson Arena. The Arena has 3,500 individually-backed seats plus room for 1,000 standees, all encircling an ice sheet 200 feet long by 85 feet wide. There are several unique features to Thompson Arena, which was dug into the ground and involved the use of 9,500 yards of pre-cast and cast-in-place concrete and 600 tons of reinforced steel. The vaulted ceiling includes 1,024 triangular sections, each weighing one ton. Buttresses extending over the bleachers hold up the press box on the south side of the ice, while additional buttresses suspend the venue itself on the outside. At the east end of Thompson Arena, various championship banners fly, symbolizing titles won by both the men's and women's programs. The Arena also includes five spacious, carpeted dressing rooms, plus two complete training rooms. There is also a storage room and skate-sharpening area, as well as the William Smoyer '67 Lounge, where postgame receptions are held following all men's contests. In Smoyer Lounge, one can find numerous individual and team photos displaying the hallowed history of both the Dartmouth men's and women's hockey programs. Thompson Arena is constructed in a way that it recessed into the ground and hides intriguingly behind a row of early twentieth century houses. As the largest hall in town, Thompson Arena represents a radical change from the predecessor ice surface of Davis Rink. Thompsons's sixty four feet vault allows for more than twice as many spectators and a lavishingly larger sheet of ice.

The arena was originally lit with Holophane Prismpack luminaries using 1,000-watt metal halide lamps. The Prismpack fixtures were replaced as part of Dartmouth's ongoing quest to reduce its energy footprint with Holophane Prismpack V. Dartmouth College relies on Holophane Prismpack V to light hockey games at legendary Thompson Arena Fixtures reduce energy, create shadow-free environment.

The Thompson Arena constructed at a cost of $4.4 Million and the Construction spanned from 1973-1975. The structural design followed is Reinforced Concrete Thin Shell method.

Like many of his early period projects in Thompson Arena also Nervi has used ferrocement technology. Ferrocement can be used to mold prefabricated elements of any geometric shape, and the elements can then be connected by cast-in-place concrete. This characterizes the structural fabric of the vaults and domes of Thompson Arena building.

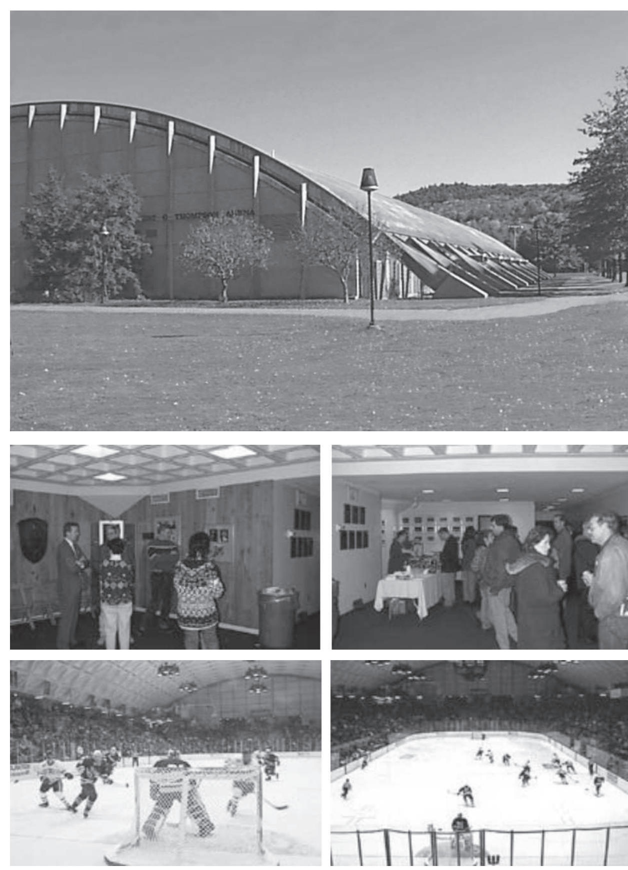


Figure: Various Views of Thompson Arena (Source: Internet)

Tour de la Bourse, Montreal (1964) (collaborating with Luigi Moretti)

Another landmark building that Pier Luigi Nervi was involved with is the Tour de la Bourse, the stock exchange tower in English, a 48 storey skyscraper located in Montreal, Canada. Pier Luigi Nervi collaborated with Luigi Moretti for the project which was completed in 1964.

The building is among the four skyscrapers that were de-

signed by the renowned engineer-architect. The other skyscraper projects with which he was involved include the Australia Square in Sydney (1963-65), the Pirelli Tower in Milan (1955-58) and, again in Sydney the MLC Centre (1971-77).

It was during the time of its completion the tallest building in Canada, a title that was usurped by the Toronto-Dominion Centre in 1967. It is still a landmark skyscraper in the country and is presently the 13th tallest building in Canada. The original project conceived actually had plans for the construction three identical towers, a move that was later scaled down due to financial constraints. The project was in fact considered one of the most ambitious planned at that point in time around the world.

The stock exchange building was conceived during the time when Montreal was in the midst of an economic boom and the need for a grand building was felt. The final project that was given the go ahead consisted of actually two identical towers, out of which only one would be built.

This project once again showcases the brilliance of Nervi, when it comes to combining functional design with aesthetics to showcase the importance of the building. The building has since its inception become some sort of role model for other skyscraper projects globally.

The building is considered to be a trendsetter in the 'International' style of skyscraper architecture. At 190 m the building was the tallest reinforced concrete tower until the completion of Lake Point Tower in Chicago in 1968. The bronze-tinted, anodized aluminium curtail wall of the building with the contrasting and slightly slanted pre-cast concrete columns at the four corners of the building give it a distinct look. The building is divided into three blocks and considered a trendsetter when it comes to skyscraper design and execution. The Tour de la Bourse, Montreal stands tall as a testimonial to the genius of one of the greatest designers the world has seen.

UNESCO Headquarters, Paris

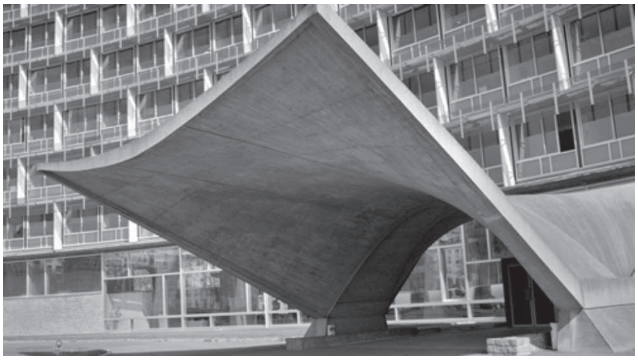
In 1952, Marcel Breuer, Bernard Zehrfuss and Pier Luigi Nervi, under the supervision of an international group of five architects (Lucio Costa, Walter Gropius, Le Corbusier, Ernesto Rogers and Sven Markelius) were commissioned with the construction of the UNESCO Headquarters in Paris, on Place de Fontenoy, in front of the École Militaire and the Eiffel Tower.

The complex is composed of three buildings across an area of 7,722 square meters. The curved north side of the Y-shaped secretariat forms part of the group of buildings around the Place Fontenoy. The south side opens towards a new square which is bordered by the projecting Conference Building. The outline of the main building forms an extremely clear-cut shape. Lifts, staircases and vertical services are in the core of this Y-shaped block. The vestibule space diverges into corridors which lead to offices on both sides. Further lifts and secondary stairs are at the ends of each wing. The conference block is linked by a clip to the office building. This block contains the architecturally interesting Conference Room and several session rooms.

The Secretariat is a reinforced concrete frame structure in which the main supports are inset from the face of the building. The floors are carried by a series of main and secondary beams. All services are in ducts above the central corridors. The beams taper off towards the outside where they carry the brise-soleil of reinforced concrete. In the ground floor, the upper structure is supported on a rigid portal frame at ground level in which the columns are raked to give greater stability. The shape of the supports is dictated by the geometric problem of transforming the elliptic section at the base to the rectangular junction with the ceiling. The plastic form thus evolved is the characteristic element of a purely architectural treatment of reinforced concrete.

The canopies at the entrances to the Secretariat are of unusual design. The canopy of the north side shown above still adheres to the conventional solution of a horizontal roof on supports; but it has an interesting pattern on the underside. The whole is designed for rough concrete work leaving shuttering marks visible. The down pipes are cast into the columns. The canopy of the south-west side shows an unprecedented solution - an asymmetrically placed arch forms the support for a three-dimensionally curved slab cantilevering on both sides.

The roof of the Conference Building is a folded slab construction in reinforced concrete, which is stiffened by a central up-stand beam carried by six supports. In the Conference Room an extremely spacious solution is achieved by raising the slab between the folds. The roofing slab Is being utilized to full advantage by following the direction of the compression forces. The continuation of the folded slab structure around the gable walls provides the necessary lateral stiffeners for wind moments.



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Impact of a Fire on Structural Concrete

Frank Clement

Summary

Designers are specifying more and more durable concrete in order to meet the required design life, clients are expecting for their structures (Shuttleworth,[1]). The modern, reliability-based service life design is implemented in most new designs and in re -design of existing structures and has been adopted by national authorities and individual clients in countries all over the world. Currently there are no real standards on how to design concrete for a specific design life. The current concrete codes are recommending various mix designs and reinforcement cover for design life of approx. 50 years

Especially tunnels are now usually designed for a service life of 100, 120 or even 200 years. Th is by far surpasses the assumed design life according to most codes and standards. All uncertainties regarding, the designer has to take into account environmental exposure, material properties and deterioration modelling in order to meet the required desi gn life. Thus service life design, based on functional requirements, can be carried out by sticking to the same mechanical concept as the one that is used for structural design. Other issues that need to be considered in relation to the required design life are water tightness and its behaviour under a fire. Both will influence the design life of a permanent concrete structure. By designing concrete with high durability the designer should also consider the behaviour of the concrete during a fire and in some cases this behaviour is negatively influenced by the concrete mix properties and the used materials. Strength loss and spalling of concrete during exposure to a fire could decrease considerable the design life of a structure. The correct choose of the concrete mix design and the appropriate protection in case of a fire can maintain the required service life of the structure. Clients and designers have to choose their optimum solution for the given circumstances.

Keywords

Structural deterioration, concrete spalling, fire protection options, passive fire protection.

Theme

Fire protection -Materials - Concrete

1. Concrete behavior under a fire load

Clients and designers are increasing the required design life by using high quality, dense concrete. Due to there low permeability these type of concretes resists to severe environmental exposure. On the other hand during a fire these concretes can be subject to spalling, resulting in a reduction of the durability and service life of the structure.

Despite the benefits of its non-combustibility and low thermal diffusivity, low permeable concretes will have the tendency to spall at lower temperatures. Concrete suffers from two problems during a fire:

- Deterioration in mechanical properties (particularly above 30 0°C)
- (Explosive) spalling

The spalling of concrete is unpredictable and a n umber of factors are influencing this phenomenon. [Breunese,2]

1.1 Fire load

It doesn't need any explanation that the fire load will influence the deterioration and spalling phenomenon of concrete First we have to consider the influence of:

- Heating rate: a fast temperature increase is resulting stresses and tensions in the concrete.
- Maximum temperature: in a tunnel fire typical temperatures of 1100°C to 1400°C can be reached
- Duration of the fire

1.2 Concrete Composition

If we consider the materials the following properties have an influence

- Permeability: The permeability is influenced by the cement type, water cement ratio and the use of fillers like silica fume. With the new types of superplasticizers lower water cement ratio, even below 0,35, are used resulting in concrete with a low permeability.
- Aggregate type and size: Different types of aggregates have different thermal expansion coefficients which lead

to stresses in the concrete while it is heated. Also the maximum size will influence the spalling behaviour.

- Moisture content: Concrete has always some water inside which is physical bonded in the pores. Depending on the environment the amount of pore water is fluctuating. If this pore water is heated the pressure can easily reach 100MPa. Modelling i s showing that there is a big influence on the pressure between "dry" and "wet" concrete.

With the close pore model (figure 1) estimations of the pressure can be calculated assuming an equilibrium between liquid and gas phase of the water in the pores.

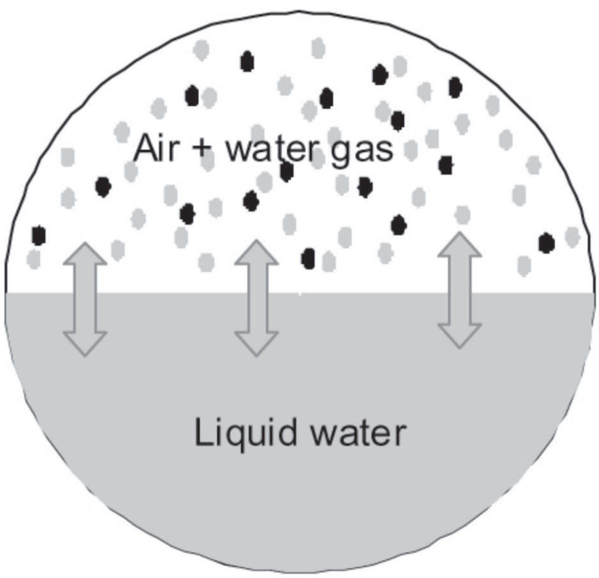


Figure 1: closed pore model

With increase temperature the pressure in the pores will increase depending on the filling degree with water (figure 2).

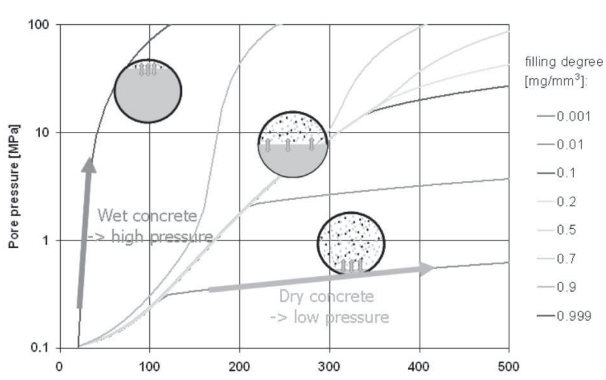


Figure 2: influence of temperature on pore pressure

1.3 Structural properties

It does not to be highlighted that the geometry, compressive loading, supports and restrained expansion will have an influence on the spalling behaviour since they will induce additional stresses in the concrete

1.4 Application

We should not forget the way of casting which can influence the spalling behaviour as experienced in a test done by Efectis the Netherlands. This test (figure 3) is showing the difference in spalling behaviour of the same concrete during the same fire test. The panel s were casted horizontal and one plate was tested with casting side down and the other one with mould side down. The one with the mould side down was spalling more severe. This test is illustrating that although concrete has the same composition, spalling can be influenced by the way of casting. Influences like moulds, vibration and stability of the mix should not be underestimated.

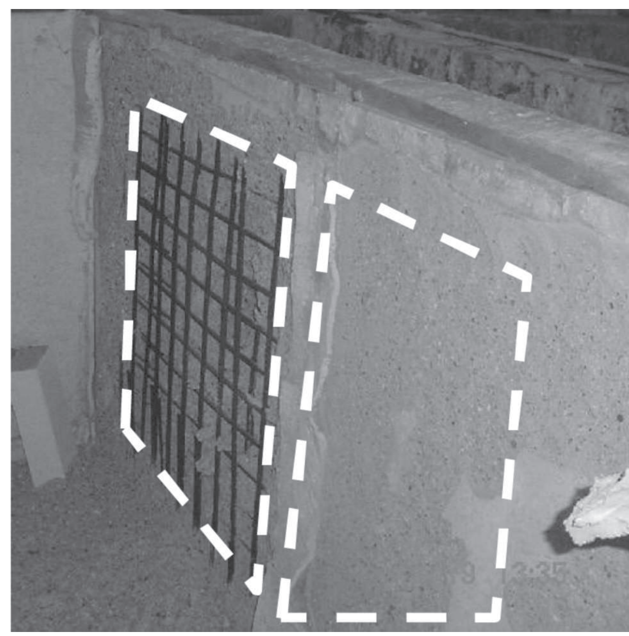


Figure 3: influence of casting on spalling behaviour

2. Fire protection

To date, the criteria for passive fire protection has been in practice to determine a thickness of thermal insulation to limit the interface temperature below a "critical" level. This "interface critical temperature limit criterion" is insufficient to protect the concrete against spalling because the heating rate (i.e. rate of temperature increase) has a greater influence upon the occurrence of spalling than the temperature level itself. So another criterion should be considered, in replacement of, or in addition to the critical temperature criterion, namely the critical heating rate criterion (ITA [3], Khoury [4]).

2.1 Structural Issues

As mentioned above, the better the quality of concrete, the worse it performs under fire. Designers are asking more and more for high durable concrete in order to have a structure with a life time of more then 100 years. In order to achieve this high durability requirement, concretes are designed to have a low permeability. But this high durable concrete with a low permeability will have a higher risk of spalling. This was dramatically evident with the Channel Tunnel fire in November 1996, with almost complete loss in

concrete lining section from a train fire. When concrete tunnel linings are exposed to fire there are structural issues to be considered: (Khoury[5],Khoury[6]]:

- The concrete typically undergoes explosive spalling, and will continue to do so until there is no concrete left, or the fire diminishes
- The concrete is heated to high temperatures and loses structural strength
- If the structure contains active steel reinforcement, then loss in tensile strength occurs at high temperatures
- Due to the temperature gradient and different expansion rate of the constituents of the concrete, deformation cracks and fissure will appear in the concrete.

To demonstrate the structural loss in strength of concrete and reinforcement steel (figure 4). Clearly, the role of a passive fire protection system is to ultimately protect the concrete from all the issues described above. As can be seen from figure, maintaining the structural concrete below 300°C in the event of hydrocarbon or cellulose fires prevents all negative structural issues form occurring. Finally, to conclude this brief insight, the rate of heating is also crucial, and has a dramatic effect on the spalling mechanism. Thermal shock can cause quite spectacular explosive events, as the water vapour generation and thermal expansion of aggregates in the exposed surface of the concrete can be rapid.

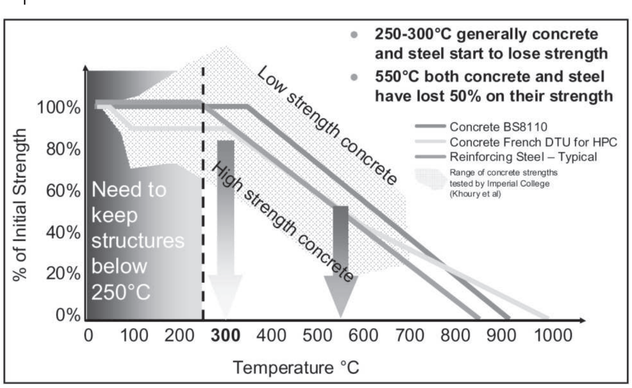


Figure 4: strength reduction due to temperature

3 Fire protection of structural concrete

Passive fire protection is designed to be installed as a shield to protect the structure from fire at any time. Passive systems do not put the fire out; but are the last li ne of defence and maintain the stability of the concrete structure and also protect against catastrophic damage to third party property and life by preventing structures like tunnels from collapsing.

There are essentially three main types of passive fire protection: spray applied mortars, prefabricated boards and PP fibre modified concrete.

3.1 Sprayed mortars

These historically have been vermiculite-cement based products applied by hand spraying with the technology being transferred from the petrochemical industry. Vermiculite based systems are relatively weak products (2.5MPa compressive strength) and may not offer adequate mechanical properties in light of increasing client demands for more durable solutions where cyclic loading resistance is required. Vermiculite systems need to be mechanically bonded to the structure with (stainless) steel mesh.

It is vital for sprayed systems to have adequate durability to resist both physical and chemical attack during the normal service life. The new developments in fire protection products are combining high durability with excellent fire protection. These products are typically based on light weight concrete technology giving a compressive strength of 15 MPa minimum.

These products are designed for application wit h the well know shotcrete technology and the modern methods of robotic spray application, allowing application rates of between 150 and 250m2/hr depending on the protection thickness required. The tolerance of applications is normally +/ - 4mm, which cannot be achieved by hand application methods at these rates. The thickness of spray applied thermal mortars is determined by the size and duration of the anticipated fire.

3.2 Pre-fabricated boards

Pre-fabricated fire protection boards offer a clear a dvantage where there are no curved walls or complex geometries e.g. cut and cover and immersed tube tunnels as shown. Furthermore, the surface finish of the board systems is appealing to clients. However, they are not well suited to curved profile tunnels and are generally more expensive than sprayed systems, which can prove cost prohibitive. By using mechanical fixations, boards are fixed to the concrete structure. Inspection of the Main strsucture is difficult, costly and time consuming since boards have to be removed for this.

3.3 Polypropylene Fibre Modified Concrete

In recent years, fibre manufacturers have promoted multi - and monofilament polypropylene fibres (32 to 18 micron diameter fibres) to contractors and design teams, detailing that the addition of 1 to 3kg of fibres added to the concrete mix gives an extremely economical solution to concrete "fire protection".

From testing [Adfil, 7], fibre modified concrete will exhibit less spalling, and in some cases no spalling whatsoever. One theory is that the melting of fibres at approximately 160°C produces channels for escape of the steam that allows water vapour inherent in the concrete matrix to escape without generating internal pressure, thus inducing high permeability at the critical time required and there by preventing explosive spalling. Another theory claims that micro -cracking around the fibres contributes to steam reduction. For specific design fires, the quantity of fibres required will alter accordingly - then larger the design fire, then greater the quantity of fibres required. As an example, for an ISO834 cellulose design fire, approximately 1kg/m3 of fibres are required, whereas for RWS hydrocarbon design fires, the quantity may increase to approximately 3kg/m3 as indicated. Concrete mixes with high fibre contents tend to be difficult to pump and place, and careful mix designs using admixture technology to overcome these problems is required.

Although the fibres offer an anti-spalling system, they do not protect the structural concrete from the detrimental effects of high temperature nor do they protect any structural reinforcement at the heat exposed concrete tunnel lining. Consequently, the use of fibre modified concrete should be considered carefully for use in structurally reinforced concret e tunnel linings.

4. Behaviour of box shaped structures during a fire

Recent fires in tunnels have shown that the repair of an unprotected tunnel after a fire can be costly. Not only repair costs have to be considered but also the economical impact has a n impact on the total costs (Munich, [8]). E.g. the cost of the Chanel tunnel fire in 1996 was estimated on M € 255. The repair costs was around M€ 50 and the economic losses were estimated on M € 205.

Around the globe a lot of immersed tunnels are used in order to connect opposite banks of rivers or lakes. In case of a fire these structures it would be difficult to repair as investigated (Nieman [9]). The concrete will not only loose its bearing capacity but will crack severely which makes it difficult to repair. One of the causes of these cracks is the thermal gradient between the heated inside (700 upto 1300°C) and the cold unheated side (20°C).

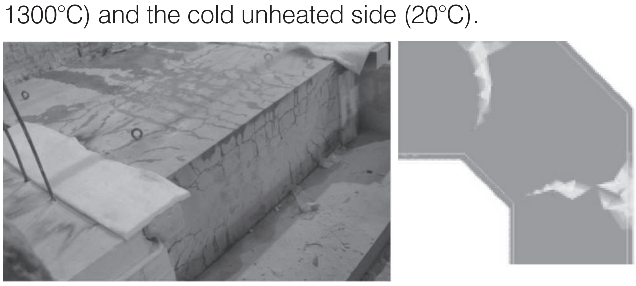


Figure 5: test and simulation box shaped structure during a fire

This thermal gradient over the thickness causes a large difference in thermal expansion, which can only be accommodated by the bending of the walls and roof. The corners of the structure cannot accommodate the inward bending of both walls and roof and as a consequence cracks will develop here, from the unheated side. The structural integrity of the tunnel is not directly threatened by the appearance of the crack, leaving the long-term risk of corrosion of the reinforcement the main danger.

5. Structures with integrated fire protection

As a consequence of several notable fires, including the latest ones in the Frejus road tunnel, South of France and the fires in the Channel tunnel, the European understanding of the problems associated with the safety of tunnels in fires has improved dramatically. A number of research programs have been started soon after the year 2000 [Ingason,10], and are finalized their findings through the EU funded SafeT project, which are translated into mandatory national and regional requirements under the EU Directive platform that will be of considerable benefit to the tunnel operators and travelling public. Currently in Europe national requirements are adopted. Furthermore since 2005, recent refurbishment projects of major road tunnels in Europe have required wholesale upgrade of the safety features in the tunnel (emergency lighting, escape routes, warning systems, active fire protection systems etc). This is clearly set to continue this year, and the coming years (Verani [11]). Also designers are anticipating on this and are coming up with innovative designs.

5.1 Suspended emergency escape route

The formulation of safety designs for many new or exi sting tunnels has led to the development of a large number of innovative solutions (Focaracci, [12]). Attention has been focused on measures to facilitate evacuation and on the systems installed. One of the new innovative solutions is the suspended escape route which can be used in new and existing tunnels (Figure 6).

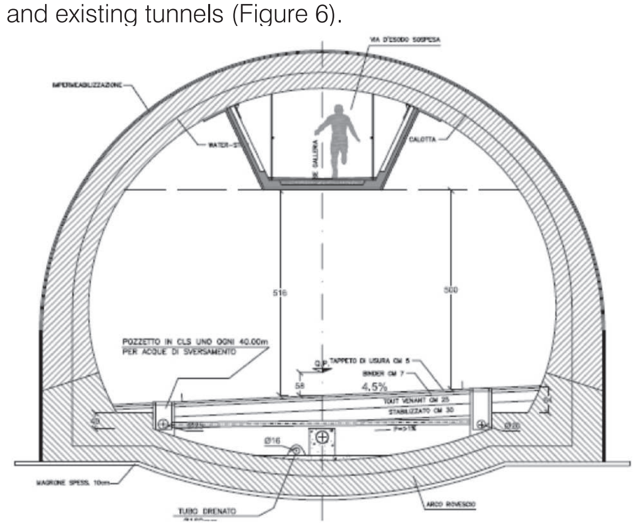


Figure 6 : innovative design of an escape route

The design consists of an enclosed walkway suspended from the crown of a tunnel, providing an emergency exit large enough to provide an easy escape route. Access to it is along connecting stairways sited inside side chambers or at parking areas. The walkway is trapezoid or rectangular in shape and it is fixed to threaded bars or bolted plates. The structure is in concrete or steel and it is protected with materials designed to resist high temperatures.

The first application of this innovative design has been done in Carrara , Italy. This structure provides the following advantages over conventional solutions:

- industrialised construction: the prefabricated components are produced under controlled conditions in a workshop and are subsequently assembled and fitted on site;
- low production costs: the suspended enclosed walkway is an alternative to the costly excavation operations required for other solutions. Excavation is only required for the side chambers which give access to the walkways;
- rapid installation: anchor bolts are fitted in advance in the crown of the tunnel, while the structure is already pre-assembled on the ground with threaded bars and the steel

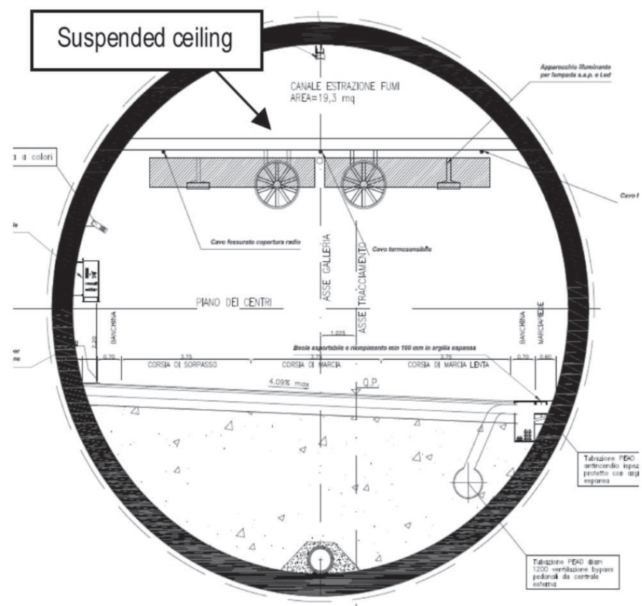
reinforcement grid for the MEYCO Fireshield 1350 mortar. The next stage consists of raising the structure and temporarily fixing the units together and to the crown of the tunnel. The last stage consists of placing the layer of MEYCO Fireshield 1350 mortar which provides protection against fire.

A full scale test was therefore performed inside the S. Croce Tunnel at Carrara and performed by the Energy Department of the Polytechnic of Turin. Numerical simulations were perform ed beforehand designed to assess the environmental conditions during the test, to establish the levels of ventilation and to conduct finite element analysis of the prefabricated segments. The fire resistance and insulation test was performed by heating the outer surface of the escape route operating two oil burners for 120 minutes and measuring the following:

- The temperature of the outer surface of the escape route at the nozzles of the burners;
- The vertical distribution of the temperature inside the escape route;
- The distribution of the temperature inside the different layers of the floor and the walls of the escape route;
- The temperature of the internal walls of the escape route;
- Temperature of the threaded bars;
- The rate of flow of air inside the escape route;
- The concentration of carbon monoxide inside the escape route;
- The opacity of the air inside the escape route;
- The increase in the length of the threaded bars;
- Displacement of the joints between the prefabricated segments.

5.2 Upgrade of tunnel ventilation

In case of a fire tunnel operators have to provide a safe



environment for the tunnel users guiding them to the nearest emergency exit. Controlling the smoke and the fire spread is mandatory in this case. A well designed ventilation system can provide the tunnel operators with the necessary tools. In existing tunnels these ventilation systems often are not designed according to the latest requirements and an upgrade is necessary. Designers are anticipating on this and are providing solutions using suspended ceilings in combination with vans and dampers. These constructions have because of the limit space, slim in design. Pre-stressed precast concrete elements would be in that case an option. Dampers and vans are installed in order to extract smoke and supply fresh air.

In case of a fire the ventilation system has to be operated immediately and should maintain the efficiency for the duration of the fire. This means that the concrete has to be protected against temperatures up to 1400°C during a fire. In this cause a monolithic structure of 3 cm fire protection mortar and 12cm of concrete could be the solution. The 3 cm of fire protection mortar would protect the concrete and the pre-stressed reinforcement so it could maintain the structural integrity and its function.

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Dreams, Genius Loci and Structures

Philippe Samyn[1]

Summary

Structures and services are profoundly characteristic of the designs of Samyn and Partners in a quest for constructions that tally with the client's dreams and wishes and are in harmony with the genius loci (the special atmosphere of a particular place). The development of my theory on volume and displacement indicators permanently fuels our structural design work.

Projects that did not come to fruition also foster research and are therefore just as important as those that were actually built. The approach will thus be illustrated by projects as well as completed constructions.

Key words

Morphology, genius loci, programme of the client, volume indicators, efficiency and sustainability, constructive details

Themes

Geometric design of the structures - programme of the client - genius loci - art of building

1. The Theoretical Basis

The quest for the most suitable way of both meeting the client's wishes (i.e. dreams) and ensuring a harmony with the genius loci, as well as the search for lighter structures, constitute the leitmotif running through my work, be it in respect of building structures or civil engineering works.

The designer of these structures has the freedom to choose the forms and proportions, which are decisive factors when it comes to material savings. Indeed, the efficient or inefficient character of the structure is practically acquired the moment the first drawings are made. This quest for optimisation is therefore very different to that aimed at lightening a pre-imposed form as much as possible, which is the customary approach for mechanical engineers.

It is in order to guide this initial act that I have developed the concept of volume indicator (W), an adimensional number which in a way is the "structural gene" of a bidimensional form all parts of which are equally stressed, under given support conditions and load cases. (Samyn[1]).

For a bidimensional form under given support conditions and load cases, W only depends on the proportions of the structure, i.e. the relation between its length (L) and its height (H). Figure 1 gives the value of W in function of L/H for different forms, for an isostatic span under uniformly distributed loads.

This indicator can be used to highlight, from the initial thinking, the penalising impacts of issues relating to mechanical and dynamic instability and those relating to deformations and connections, and enables these to be taken into account. It therefore calls for a certain discipline under pain of seeing these issues become predominant and leading to a configuration far removed from the optimum.

Bearing in mind the potentialities and constraints of the site, in the preliminary calculation I therefore focus on seeking a reasoned geometry for the entire structure and its component parts, with special attention devoted to their connections and the construction systems.

For the building, issues relating to physics, such as lighting, thermal qualities and acoustics, and those relating to the structure's cohabitation with other building components, such as the shell, lead to a reflection in terms of a system in which the optimums of each component temper each other in the search for the overall optimum.

So it is, for example, that the optimum geometric slenderness ratio of an inhabited bridge is higher than that leading to the optimum of the structure alone further to the need for an additional amount of material to support the front wall (Figure2).

Efforts aimed at striving for lower materials consumption were for a long time limited by economic realities, customs or even a certain expression of power, which is outmoded today.

Scientific and technological progress linked to "exact" sciences, be it in respect of software packages for the calculation and driving of manufacturing robots, or in the physics of the environment and materials, now makes this approach increasingly credible.

The awareness of the "finite" nature of our planet, with the gradual disappearance of the feeling that there would always be "some other place" to which we could consign our waste, contributes towards the current craze for sustainable development and therefore also to an enhanced receptivity to my approach. On the other hand, progress in natural and human sciences enables a better understanding of the physical, human and natural phenomena, of which constructions are the seat in harmony with the site they occupy. The fact that account is taken of them thus feeds the conceptual thinking very productively.

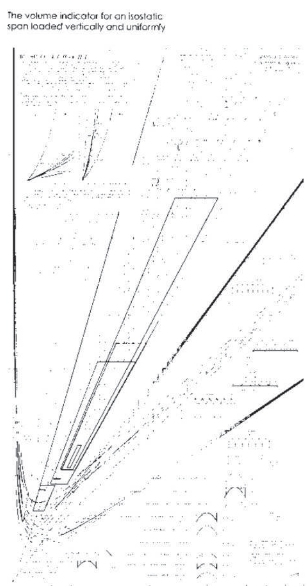


Figure 1

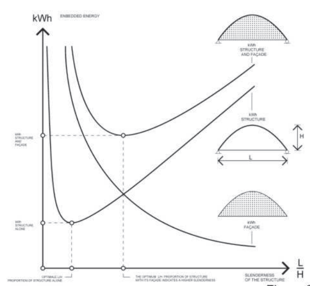


Figure 2

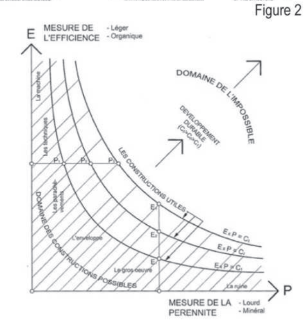


Figure 3

My work always begins with hesitation, verging on melancholy. This stage is necessary to rid oneself of all preconceived ideas and to avoid copying oneself or, unconsciously, copying others, in a quest reduced to aestheticism.

The search for the most dispassionate design possible goes hand in hand with my detachment vis-à-vis fashionable effects which, with the profusion of advertising and easy credit facilities, have for over a century contributed to the (often cynically programmed) obsolescence of industrial production, be it the useful life of an incandescent light bulb, acidic paper or constructions.

Apart from the harmful effects on our environment, this attitude focused on the "trend" of the moment has too often in our line of business led to an accumulation of ill-considered proposals, the declared qualities of which, for that matter, people have only rarely taken the trouble to check.

This required obsolescence is not to be confused with the necessarily limited life of certain highly efficient constructions, either on account of their extremely specialised nature (the operating ward of a hospital) or their extreme lightness (the sounding balloon used by meteorologists).

Without elaborating on the subject here (Samyn [2],[3]), the art of building design therefore aims to optimize the product of the efficiency (E) and permanence (P) of the construction, with the progress made by mankind making it possible to increase the factor C(t, l) = E . P (Figure 3).

This factor C(t, l), depending on the given time (t) and place (l), is therefore the measure of the art of building. Since any construction of which the product of E . P were lower than C (t, l) is impossible, the hyperbole C(t, l) = E . P thus characterises a "virtuous" construction (which we would nowadays refer to as "sustainable").

Finally it should be recalled that we work for motionless constructions (which are not driven, do not sail and do not fly) at the scale of the meter or the foot and inch, in a system of chiefly parallel forces, be they static or dynamic, and not for mobile objects, at the scale of the micron or the light year or in a system of chiefly radial forces.

We also work for human beings whose physiology and soul are invariants over several millennia. The theoretical geometry-related tools developed specifically since the dawn of time by and for construction thus retain all their relevance. Hence both the text written by Vitruve [4] and the more current one by Christopher Alexander et Al [5] are still applicable.

The same goes for "Het plastisch getal" ("the plastic number") by Dom van der Laan[6], the laborious learning of which remains a prerequisite for any builder seeking the right design. Whilst he shows that the number=3 +1 may guide the design, he also reminds us that it is subordinate to the order of magnitude and thus to the rhythms accentuating our ideas.

Here, therefore, in concisely summarised form, is the intellectual basis that I hold dear.

The following description of some of my projects, which is necessarily brief, explains my approach. The website www.samynandpartners.be and a number of listed reference works provide more in-depth information on each project. The book by Puttemans and Spehl [7] offers a more overall view of my works up to 2008.

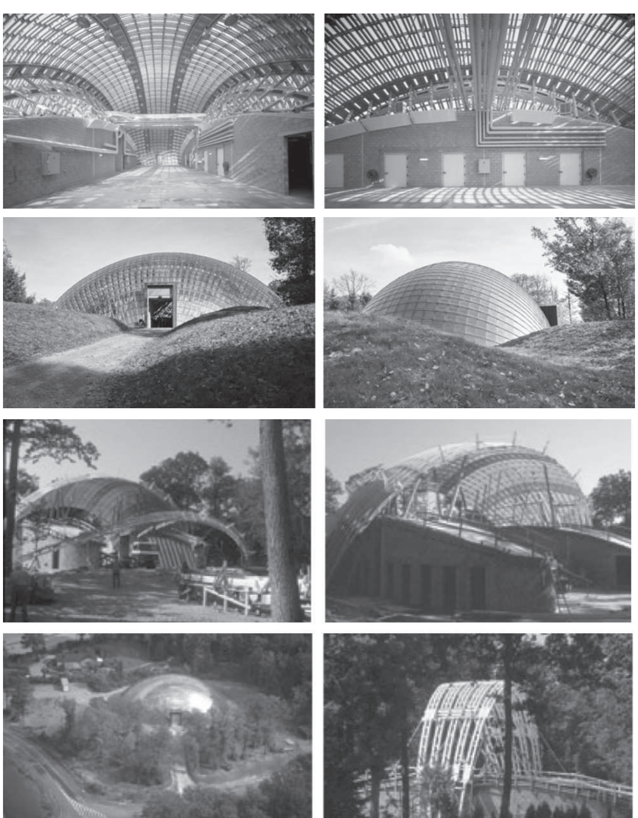
2. A selection of planned and completed projects

2.1 The forestry department branch building, Marche-en Famenne, Belgium

The forestry department branch building in the Belgian Ardennes, which was designed in 1992, owes its structural form so much to the very nature of the programme (a place where tree seeds are collected, processed and stored) that the first time I visited the site accompanied by fellow agricultural engineers in charge of running the national forests, one of them absent-mindedly lifted up one of the large spruce canes stored on the plot, and it brought me to mind the structures made with bent branches that were so typically used by the earliest societies. However, I had to remind myself that the module of elasticity of wood is roughly proportional to the square root of its axial mechanical resistance, and thus realised that, contrary to a well-established but false idea, dry wood actually bends at a smaller radius of curvature without breaking than green wood, although with a greater stress. I also had to remind myself that unless pitch pine is used without knots, a bent cane will always snap at the knot. This means that there's nothing better than to use planks and to ensure a reasonable and uniform distribution of the knots. I was also imagining being able to count on the creep of wood to see the stress induced by the bending of the canes disappear through relaxation. I was then happy to see this hypothesis confirmed a lot more quickly than I had imagined. After having ten 6-m-long canes bent by a cable tied to their ends and fitted with a butcher's spring balance and a length adjustment device, I asked the forest warden to stretch the cable every day so as to maintain the imposed form despite the spring balance's shortening further to the reduction of the stress which cancelled itself out in less than 10 days.

However, this did not provide an answer to the weakening on account of the knots and still less to the way in which the pieces could be jointed to absorb the transverse contraction resulting from the drying of the wood. The issue of the insecticide and fungicide treatment that was required also remained unresolved. These are all questions that did not bother the first peoples, since the pieces were bound together with ropes and, in the case of permanent structures, tightened until the wood dried completely.

I therefore opted for the oven bending and colourless fungicide-insecticide treatment of planks of 6.14 to 6.21 m in length and of sections of 5 x 10 cm (5/10), 6 x 12 cm (6/12), 7 x 14 cm (7/14) and finally 8 x 16 cm (8/16) bent according to the respective radii of curvature of 10 m, 12 m, 14 m and 16 m.



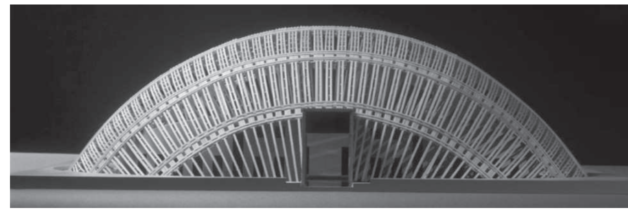


Figure 4

The ovoid form, 43 m long, 27 m wide and 12.5 m high, was therefore built by a succession of arches in two identical layers recalling Kabyles necklaces, arranged in radial fashion and made up of these arched planks (Figure 4).

Each layer is made up, starting from its top, of four 5/10 planks on which three 6/12 planks then two 7/14 planks and finally one 8/16 plank successively overlap. Two transverse planks secure the planks of each section and of each layer to each other, plumb with their overlaps. They also serve to secure the arches to each other.

The radii of curvature plumb with the successive overlaps between the 5/6, 6/7 and 7/8 planks are respectively 21.50 m, 24.50 m and 27.30 m (Figure 5).

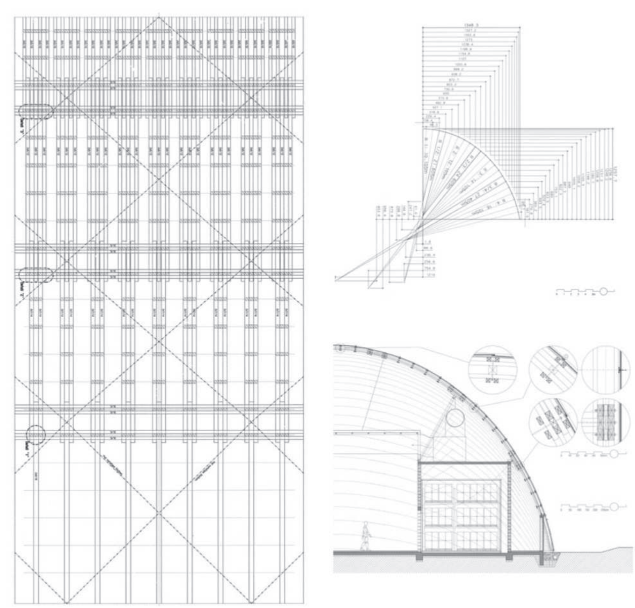


Figure 5

This system enabled a very simple assembly of the pairs of arches secured on both sides with a central arch installed in the first place progressively constituting the ovoidal calotte (Samyn[8]).

A glass roof formed with glass panes, on small aluminium T-shaped sections held in their bottom corners by cast-aluminium claws in the shape of whale tails, guarantees the water and airtightness of the enclosed volume. Inside, secondary structures in insulated brickwork house laboratories and offices at a constant temperature as well as coldrooms, some of which are kept at -12°C.

The enclosed space, which can be accessed by the lorries bringing cold and damp seeds in autumn, can be amply ventilated mechanically and heated by radiant thermal panels. The comfortable atmosphere in summer never ceases to amaze visitors. In fact the brickwork is cooled at night, creating a layer of cool air at floor level whilst, under the sun, the hot air produced by the greenhouse effect remains concentrated in the upper part of the calotte.

This building was completed 17 years ago and is still in pristine condition. It was designed when I was still looking for the way of objectifying my structural choices and did not have the volume indicator at my disposal, which I was only to discover some years later.

Figure 6

The design was therefore guided by reflection and geometric discipline, as had already been the case for the M&C Ricerche research centre at Venafro - Isernia, which was designed in 1989 (Figure 6).

Whilst the latter's structure is composed of Moorish arches in steel lattice work, also arranged in radiant fashion to bear the textile roof, the organisation of the building is amazingly similar to that of the forestry department branch building, albeit with a very different programme and genius loci. On the other hand, the other structures presented below in chronological order owe their morphology to the rigorous use of the volume indicators.

2.2 The roofing of Leuven railway station, Belgium

On the edge of the mediaeval city of Leuven, I sought to reconcile the scale of the small neo-classical railway station building with that of the "river" of railway tracks (De Coninck[9]).

Passenger safety and comfort and data legibility were also determining factors, be it as regards the reduction of obstacles on the platforms, wide enough pedestrian areas, fire safety, information signs facilitated by the general design, and comfort as regards humidity and warmth, lighting (by day and by night), acoustics, touchscreen facilities, etc.

The catenaries' self-stable support posts are the only points of support, and serve as a collector for rainwater collected by the roof.

The distance between them is 52, 39, 39 and 52 m (L) on each of the axes corresponding to the five lines of platforms which are the same distance apart (14.50 m).

Ten external tripodal supports and fifteen intermediary quadripodal supports bear pairs of arches that are 6.5 m high (H). The central arches therefore have a slenderness ratio of L/H = 6 (39 m / 6.5 m) whilst the end arches have a slenderness ratio of L/H = 8 (52 m / 6.5 m) (Figure 7).

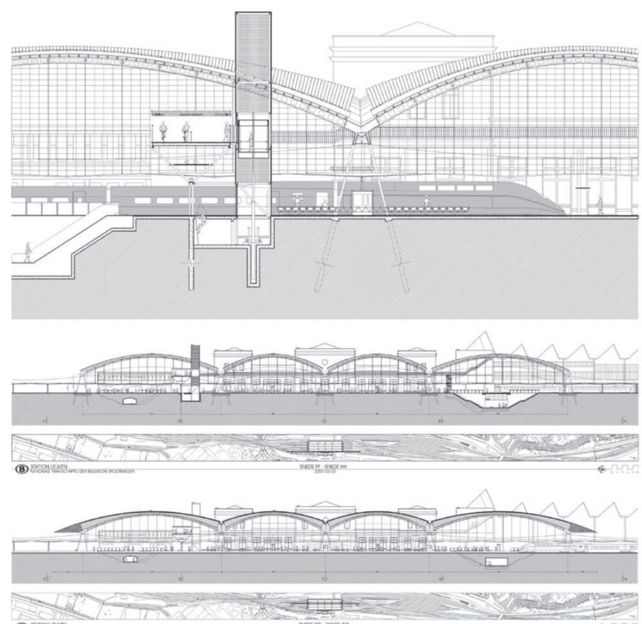


Figure 7

Two large glass façades parallel to the external platforms complete the structure and substantially impact on its efficiency. So it is that the slenderness ratios of the arches, which are higher than the optimum necessary to absorb the vertical loads, is fully justified both by the influence of the horizontal stresses caused by the wind and by the influences of the façades, and thus means that a very light structure is made possible. The crescent-shaped arrangement of each pair of arches guarantees not only the transverse stability of the structure but also, with its glass roof, the lighting and natural ventilation of the covered area (Figure 8).

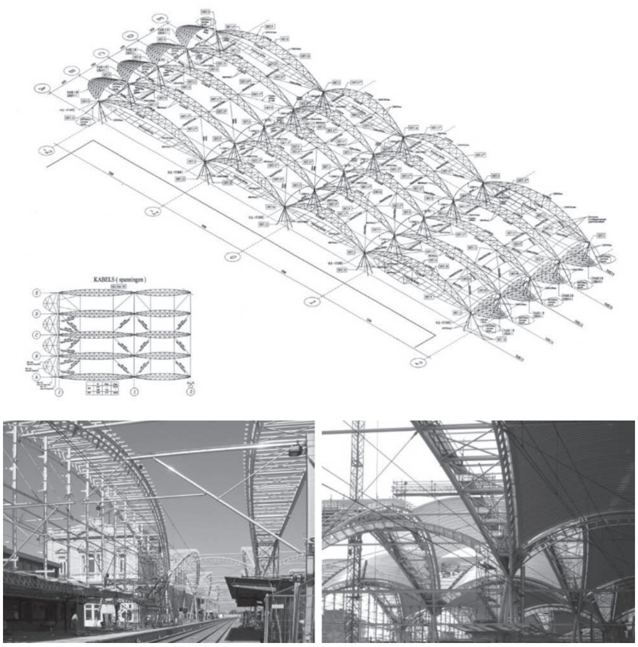


Figure 8

Freestanding vaults in arched profiled steel sheets (perforated for sound absorption) which are insulated and clad with metallic sealing, bear from arch to arch and diffuse both the daylight and the light given out at night by the single projectors secured to the tripods and quadripods.

The glass roofs between the arches and the vault supports, the arches in reduced reconstituted sections and the fixings of the vaults, at a distance from the arches, contribute to the overall impression of lightness (Figure 9). The work was done in phases so as to cause as little disruption as possible to the station's operations. Work started in 2002 and was completed at the end of 2010.

2.3 Wind turbine masts

There is currently a certain paradox in the desire to produce "green" energy by using wind turbines, given the large amount of materials used in the construction of their masts

and foundations. The heavy equipment needed to lift the turbines to the top of the mast also calls for suitable roads, which encourages the installation of wind turbines in busy areas or the laying of new roads.

Figure 9

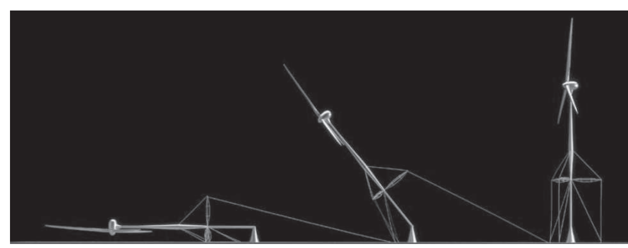


Figure 10

The masts and foundations of traditional wind turbines are typically made up of a heavy broad tubular steel structure, resting on a thick slab of concrete, itself perhaps supported by a large number of piles. The soil contamination resulting from this large amount of concrete cast in situ in turn constitutes an environmental problem. Finally, the imposing presence of the mast is unsuited to certain sites. With this in mind, from the year 2000 onwards I set about developing a mast that avoided the above-mentioned problems for the company "La construction soudée" of the Fabricom-Suez group. The design was the object of a European patent issued in 2004 (EU patent [10]). Use of the volume indicator enabled quite a simple comparison of countless alternatives and was decisive in the search for the optimal morphology. The final configuration of the mast is illustrated in Figure 10. Thanks to the studded stays, it is possible to raise the mast - fitted on the ground with its blades and turbine - without heavy lifting equipment, and therefore to install it in remote locations, as illustrated in Figure 11 for the consultation made by Enelerga in the Apennines. Both the mast and its foundations are very light and the whole unit can in the long run be disassembled easily, steel and geotextile infrastructure included.

However, whilst the questions relating to connections and elastic instability can easily be taken into account independently of the dynamic characteristics of the unit constituted by the blades and the turbine, this is not the case for the study of dynamic behaviour and fatigue. Taking these characteristics into account, as Aerodyn was able to do with a Jeumont Framatome 750 kW turbine since they were in possession of its characteristics (Aerodyn [11]), nonetheless did not have any effect on the proposed morphology and a certification was obtained in 2004 (Germanischer Lloyd[12]).

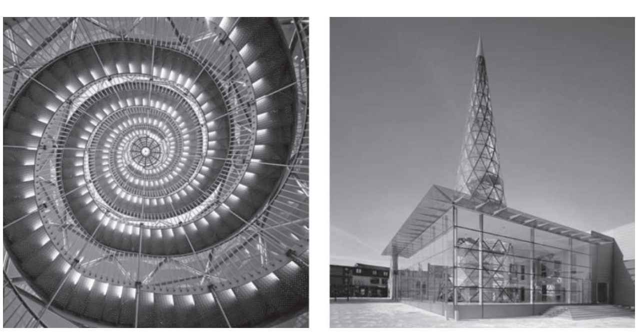
Figure 11

2.4 The house of glass, Lommel, Belgium

Lommel is well known in the glass industry for the quality of its sand, and on this account has long attracted glass artists from all over Europe. So it was quite natural that the town council should have wanted a house of glass (glazen huis).

The building, erected at the back of a small mediaeval house on the town's main square, consists of a parallelepiped volume and a conical tower 33 m high, both with large expanses of glass.

Two helicoidal staircases enable visitors to climb to the top of the tower, where works of art in glass are displayed, and to come back down (Figure 12).



For the tower, as will be quite understandable, I wanted as slender a structure as possible so that it would not be visible behind the joints of the triangular double glazing panels. In the end I opted for a remarkable variant proposed by the engineer Aurelio Gangi whose company had in the meantime been selected.

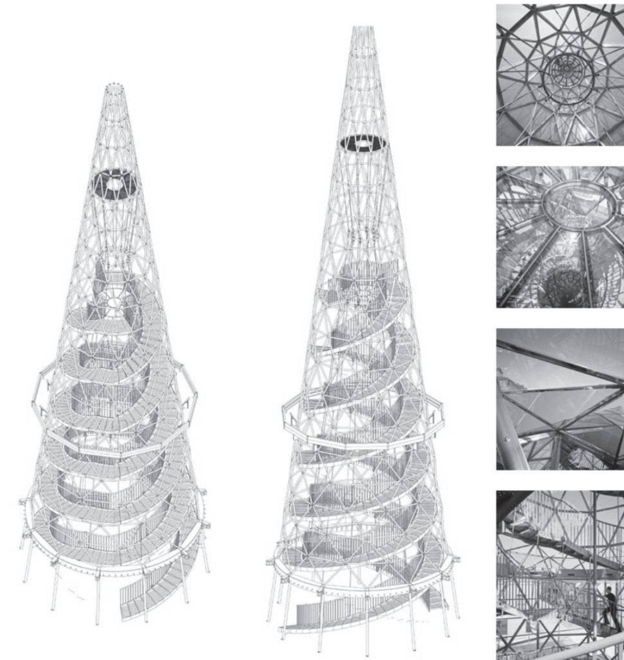


Figure 12

Figure 12

The structure is composed of the triangular frames in bent stainless steel sheeting housing the glazing and steel tube segments 60 mm in diameter connecting the sides of the triangles with each other, but without touching each other at the theoretical axis of the joints (Figure 13).

This is therefore a triangulated bar structure but with hollow nodes and not a dual structure, in the meaning of Ture Wester (Wester [13]), which would be hypostatic. Real-scale tests were carried out to ensure the feasibility of the proposal. The bearing capacity is all the more impressive in that the twin-staircase system is suspended from the top of the structure after the fashion of the gondola of a hot-air balloon.

2.5 Façade of the atrium of the headquarters of the Council of the European Union in Brussels, Belgium

A fascinating series of design questions was put to me as "head and design partner" of the temporary joint venture "Philippe Samyn and Partners, architects and engineers / Studio Valle Progettazioni, architects / Buro Happold, engineers", which was formed for the new headquarters of the Council of the European Union. I will only deal here with the atrium's large façade-bridge, and in particular its structure

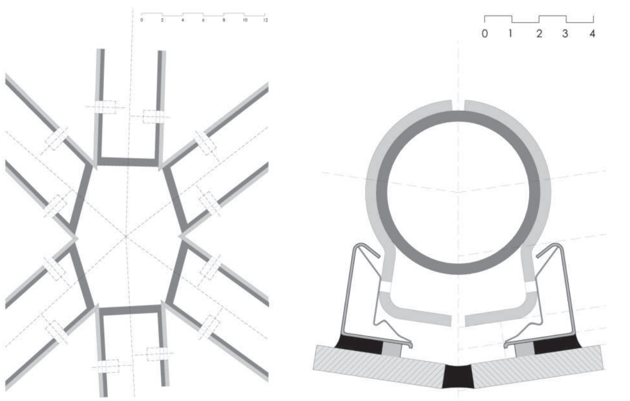


Figure 13

(Samyn[14]). Synthetically, the ensemble is made up of a building of 13 floors above grade in the shape of an L comprising a restored and renovated old historic part, basically housing offices, supplemented by a glass-sided atrium roughly cubic in shape, in which a large lantern-shaped structure contains the main conference rooms (Figures 14 and 15).

A new rail link runs diagonally through the building's subgrades from one side to the other, preventing any vertical support of the North façade along the main street, Rue de la Loi.

The façade, which guarantees a temperate climate in the atrium and also acts as a ballistic shield, is made up of two chiefly glass walls 2.7 m apart. The inner wall contains the load-bearing structure and a façade in laminated double glazing, whilst the outer wall is made up of jointing in patchwork of old oak casements and laminated single glazing, recycled in the 27 countries currently making up the European Union, and borne by frames in HE140B, 3.54 m high and 5.4 m wide.

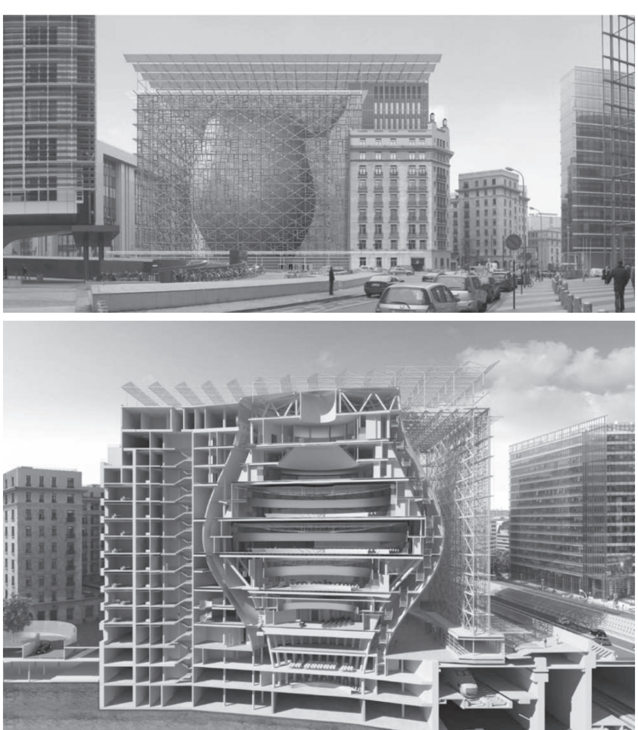


Figure 14

Horizontal lattice beams connecting the two vertical walls guarantee the absorption of wind-related loads.

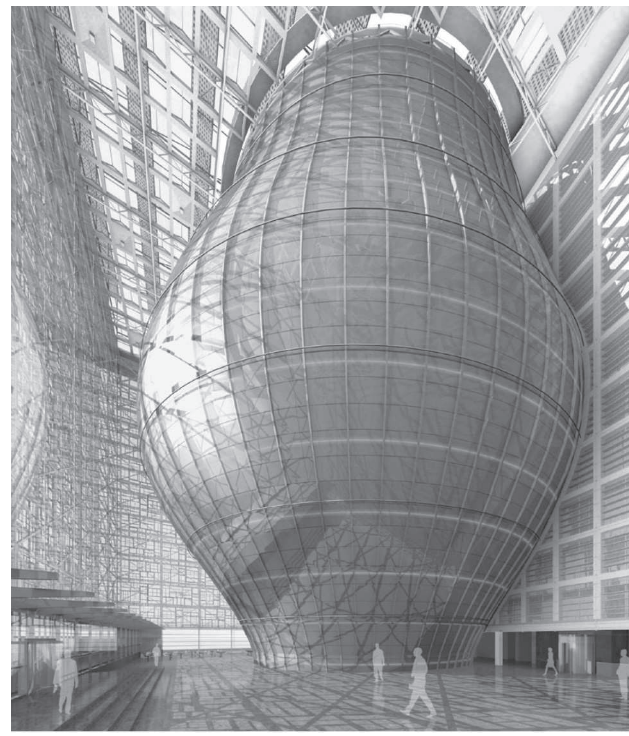


Figure 15

The logic of the complex immediately prompted me to design a load-bearing structure in a cross-mesh grid with bracing, 5.4 m wide and 3.5 m high, and with a highly statically indeterminate character (Figure 16).

I was surprised when, from the moment the sketch was made for the preliminary design, the geometric optimization of the sections combined with the minimum and technologically necessary sections, led to a structure implying a much lower material consumption than that which would have resulted from a parabolic-arch structure supplemented with the necessary vertical uprights on the glass façades.

This optimisation of W is done very simply by starting, for each bar, from a unitary section and then adapting it in iterative fashion in function of the stress of which it was the

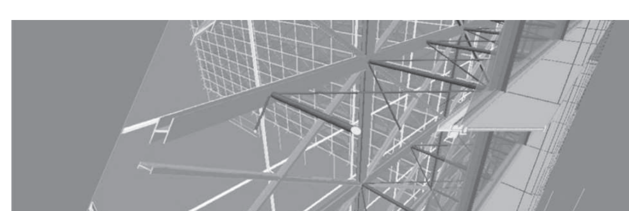


Figure 16

centre. The reality of the calculations was in fact slightly more complex due to certain specific security requirements, but did not in any way change the building principle and details. The steel structure is in built-up sections with a constant depth of 300 mm, with flanges varying from 90 x 15 mm to 450 x 26 mm, and with webs of a thickness ranging from 10 to 26 mm (Figure 17).

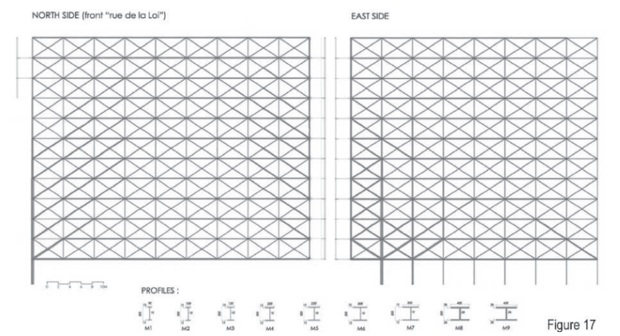


Figure 17

The asbestos removal, partial demolition and infrastructural work began in 2008 and will be completed this summer (2011), with final project completion and hand-over scheduled for 2014.

2.6 The Belgian Antarctic base "Princess Elisabeth"

This is an example of construction in an extreme location, with very low temperatures and very violent winds that moreover carry solid particles. Matters relating to building physics are vital here, and in particular the thermal insulation, the perfect continuity of the vapour barrier, and resistance to the impacts of the solid particles borne by the wind. This is why, in the context of my responsibility for the shell and the structure, I designed the latter like a large wooden box,

Figure 18

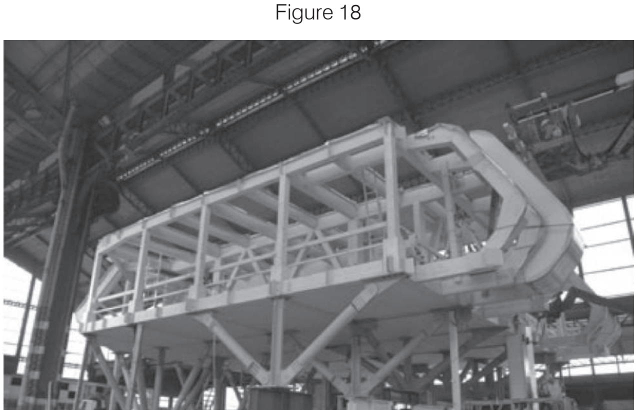


Figure 19

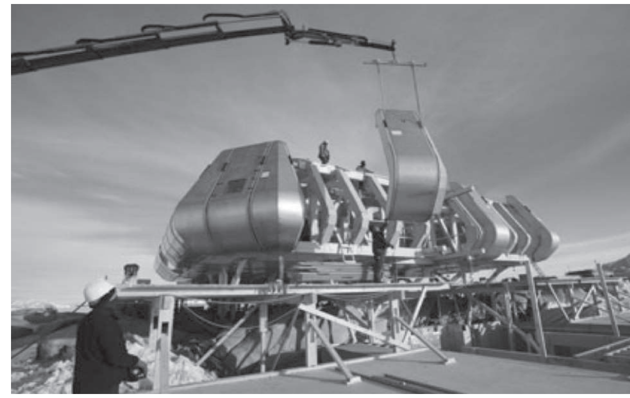


Figure 20

Figure 21

the external and internal walls of which are separated by 40 cm of insulating material, avoiding heat bridges as much as possible, as the picture of the prototype (Figure 18) illustrates. A very high-performance aluminium vapour barrier covers the entire inner side, its absolute continuity being guaranteed plumb with the load-bearing portal frames by continuous flat steel plates. A shield of 1.50-mm thick stainless steel sheeting on a 4-mm layer of closed-cell polyethylene foam surrounds everything. The building rests on four steel trusses, each one independent of the others, so as to limit to the very minimum the constraints caused by the very considerable differences in temperature. This project, which was carried out on behalf of the Polar Foundation, was studied and completed between February and July 2007, set up "with blanks" in Brussels to verify the complex in August 2007 (Figure 19), and erected in situ, under the direction of Alain Hubert, in January and February 2008 (Brown J.L. [15]) (Figures 20 and 21).

2.7 The Vesuvio-East railway station in Striano

At the heart of the market-gardening region to the south-east of Mt Vesuvius, Rete Ferroviaria Italiana is planning a railway station at the junction of the high-speed train line crossing Italy from north to south and a regional train line, the Circumvesuviana, both lines being in existence. This station will accommodate a large volume of travellers, both residents of the region and visitors to nearby Pompeii. It is to house a large shopping centre and will be supplemented by buildings offering parking for 1,600 cars.

On the south side of the embankment of the high-speed line, I therefore designed a large parabolic-section cylindrical roof covering all the station's functions and connecting the natural grade to that of the platforms, 7.20 m higher.

The large reception hall, which also houses the vertical traffic, is bordered on the side of the existing embankment by two floors comprising the premises offering services to visitors and the offices, as well as a large indoor waiting area (Figures 22 and 23).

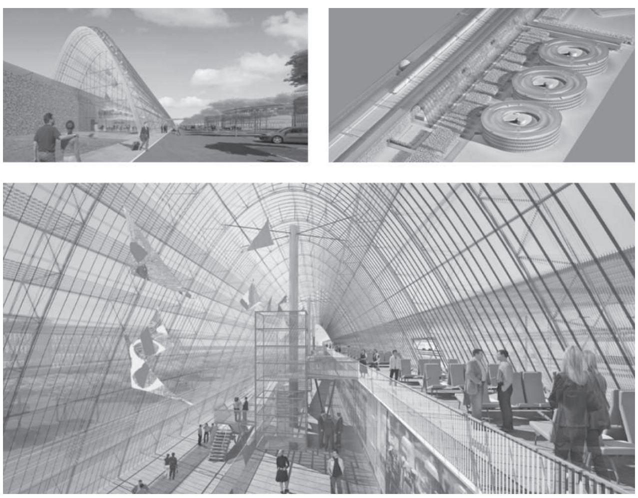


Figure 22

Figure 23

The parabolic vault is made up of metal arches in latticework spaced 5.40 m apart and covered with a very light transparent covering in synthetic material. Studied with Mauro Giuliani of Redesco in Milan, the arches are so light that the wind stresses are greater than the seismic stresses, which are nonetheless significant here.

The study of the hygrothermic comfort carried out with Alessio Gatteschi of the STI design office in Florence, followed the same logic as that applied to the forestry department branch office.

2.8 Cultural centre in Ngozi, Burundi

In this poor but enchanting country, no metal cans or plastic packaging pollute the landscape. However, the appetite for eucalyptus wood for the brick ovens dotted more or less all over the countryside and the continuous deforestation that this results in are damaging this veritable Garden of Eden.

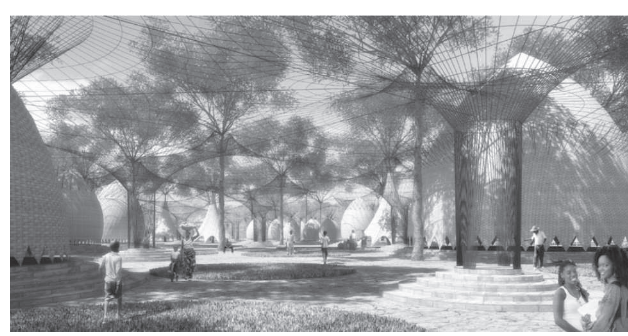
This craze for constructions in (often poorly fired) earth bricks, is oddly completely alien to the culture of the country, which is characterised by an especially fertile craftsmanship and expertise in the art of weaving, sewing and basketwork.

This is why, for this small cultural centre, I endeavoured to look for a building method in harmony with the place (a relatively open eucalyptus plantation) and the traditional expertise, by limiting the materials originating from industrial technologies to a strict minimum.

The project under study thus consists in a large wide-mesh sewn net, in para-aramid thread rope, fastened to the trees. Huts in positive (ovoid) or negative (hyperboloid of revolution) Gaussian curvature wickerwork are suspended from them (Figure 24).

Seeing as the foundations have been laid, so to speak, in the ceiling, the descending compression of the structures' loads turns into ascending traction. Here, in contrast to the usual construction formed by a continent in compression in which a few lakes of traction are arranged, the huts and the net form an ocean of traction anchored to a few islands of compression, whether it be the trees or the compressed hoops for the positive Gaussian curvature huts.

The trunks are connected to each other by ropes plumb with the net's hangers, and are thus stayed. This system uses the trunks' natural resistance to compression and frees up the maximum size of the wicker structures since they



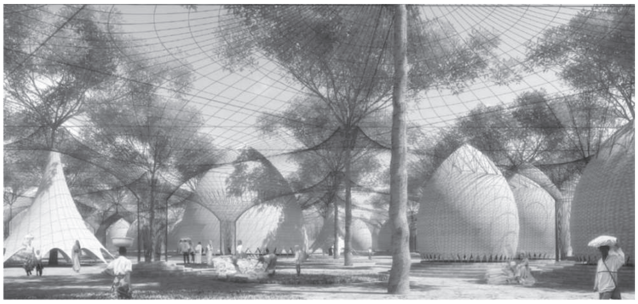


Figure 24

are no longer subjected to elastic instability. The ensemble is entirely woven, sewn and laced up in local materials, with the exception of the rope bobbin and the sewing thread. It is flexible to the wind and adapts to the growth of the trees, since the wicker huts can get bigger perfectly at their base.

2.9 An ecumenical chapel at the Verbeke Art Foundation, Kemseke-Steneke, Belgium

The specific morphological and building characteristics of this very small, totally abstract and scale-less project, intended for meditation and set in a lake in the centre of the foundation's park, make it very unusual.

This is an apparently dissymmetric structure, while in fact it presents a plan with two axes of symmetry, in "transparent" steel. The apparent dissymmetry refers us to the importance of human scale and perspective. The surface is made up of superposed elliptical conical surfaces moulds, all the main ellipse axes of which are parallel, but the large and small axes of which alternate in direction. The figure can thus only be perfectly symmetrical if designed in elevation or viewed from infinity, but we rarely see a construction from so far away and its view from closer up affords a surprisingly lop-sided image.

The structure of the pavilion is made up of facets of trapezoidal sheets attached to each other at their edges folded at 90°. It owes its stability to the transfer of the shear forces from side to side after the fashion of the dual polyhedron so dear to Ture Wesler. The sheet is 50% perforated, which lends it considerable transparency and increases the visual ambiguity (Figure 25).

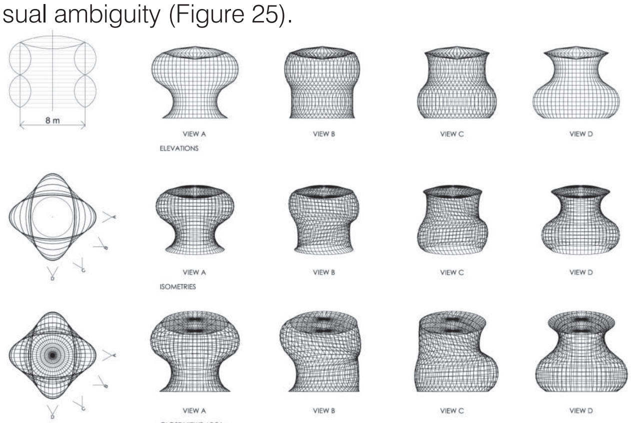


Figure 25

Conclusions

Design and reflection regarding building methods and details are very richly fuelled by the constant progress being made in human, natural and exact sciences. But above all, they must always be a response to the unchanging senses and soul of the human being, the grand design and dreams of the sleeping partner and the society to which he belongs and, finally, the genius loci.

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Study on the Way of Building Construction in Sustainable Society

Tatsuo Inada[1]

Summary

There are measures to restrict the carbon dioxide discharge like, "Thorough recycle and reuse", "The development of measures to reduce the carbon dioxide discharge amount in the steel material manufacturing stage" (Direct Reduction method), but the most effective way is to extend the life span of buildings.

In this paper I focused on the issue of the "Structural Component", which has the largest effect on carbon dioxide discharge in the construction stage of buildings, and examined the factors that have influence on the amount of the carbon dioxide discharge at the stage of structural component manufacturing, and the legal and social systems that have influence on the life span of building structures, and finally I would like to present the scenario to surely reduce the carbon dioxide discharge in the construction stage of buildings.

Keywords

global warming issue , reduction of carbon discharge , building material , Life of building

Theme

buildings - design - others / sustainablity

1. Introduction

1.1 The latest state of the global warming issue

Various arguments concerning the global warming issue, especially the presentation of middle and long term scenarios and visions including the meteorological change countermeasures after 2013, are getting into stride, such as the start of the second year of the "first promised period" of the "Kyoto Protocol" and "COP15" to be held in December this year. The argument has been developed for and against, concerning the relation between the rise of atmospheric concentration of greenhouse-effect gases and global warming. But by the exhibition of the "IPCC Fourth Report", the existence of causal sequence between the rise of atmospheric concentration of greenhouse-effect gases and global warming is internationally acknowledged with no doubt.

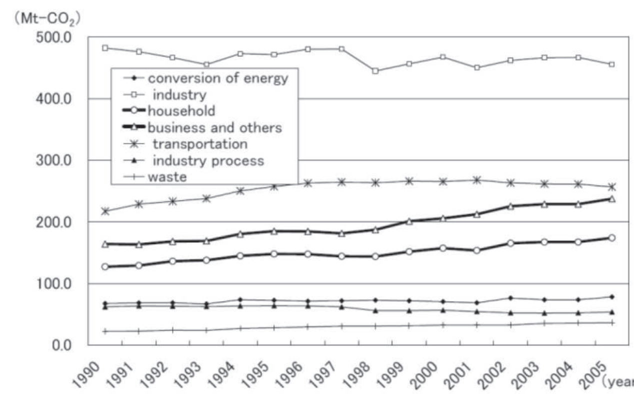

Figure-1: Transition of carbon dioxide discharge by group (Japan) (The discharge volume by the electric power industry is distributed to each user group) Based on the Statistics of Ministry of the Environment

The volume of artificially discharged greenhouse-effect gases and the natural absorption volume by forest and ocean must balance to maintain the healthiness of sustainable social and natural environment.

Definitely speaking, at least until 2050, global annual carbon dioxide discharge volume must be reduced to 50%, compared to that of present. July 2008, The Japanese Cabinet Council has decided the "Plan to create Low Carbon Society". The plan shows the determination to reduce the national annual carbon dioxide discharge volume 60 to 80%, compared to that of present, as a long term goal. To actualize such global warming prevention measures, I firmly recognize the necessity of continuing investigation and its practice, too in the construction field.

1.2 The aim of this paper

Fig.1 shows the graph of the transition of Japan's annual carbon dioxide discharge of each group after 1990. The graph shows 5.6% decrease in the industrial group (manufacturing industry and others), and big increase in the business group (+41.7%) and household group (+30.4%) , compared to the standard year (1990). The value of the business and household group shows the carbon dioxide discharge of the practical use stage of buildings.

This graph clearly shows the urgent need to reduce carbon dioxide discharge in the construction field, and by the National Institute for Environmental studies, various quantitative investigations to reduce carbon dioxide discharge in the practical use stage of buildings has been conducted, and as results of it, definite scenarios showing the possibility to actualize large volume reduction in 2050 is presented. The carbon dioxide discharge of the building stage of buildings is said to be half of that of the practical use stage, and concerning this subject, quantitative investigation and the presentation of definite scenarios is yet to be done.

Taking into account such circumstances, in this paper I would like to study the measures to reduce carbon dioxide discharge in the building stage of buildings, present the concrete scenario to realize low carbon society and clarify the problems to be subjugated.

	Structural element			**Non-structural elelment**		
	Concrete	Reinforcing rod	Steel frame	Other materials	Construction work	Total
S factory	18.8%	4.0%	18.6%	29.8%	28.8%	100.0%
S office	15.1%	3.3%	14.7%	37.8%	29.1%	100.0%
RC house	25.3%	8.9%	2.2%	33.4%	30.3%	100.0%
SRC house	22.8%	8.2%	7.5%	31.7%	29.7%	100.0%
RC shool	22.1%	8.4%	3.5%	36.6%	29.5%	100.0%
RC office	19.0%	7.0%	4.1%	39.7%	30.2%	100.0%
SRC office	16.5%	5.4%	11.1%	37.7%	29.3%	100.0%
Totalization	19.9%	6.5%	8.8%	35.2%	29.6%	100.0%
	35.2%					
	Strucrural Materials			other	Construction work	Total
	Construction Materials					

Table-1: Carbon Dioxide Discharge of the Structural and Non-Structural Element in its Production Stage

	Carbon dioxide discharge (t- CO2)
Electricity	0.116
Air Conditionin	0.130
Sanitary	0.071
Elevator	0.011
Total	0.328

Table-2: carbon dioxide discharge of the equipment apparatus in its production stage

2. The volume of carbon dioxide discharge in the building stage of buildings

The "LCA guideline" edited by the Architectural Institute of Japan shows the carbon dioxide discharge per unit floor area by each input element (1995 value). From the guideline data, I extracted 7 data (without the data of wooden buildings), split it to structural and non-structural element and showed the percentage of carbon dioxide discharge in table-1. Similarly the "LCA guideline" shows the carbon dioxide discharge of the equipment apparatus of its production stage in standard office buildings. It is shown in table-2.

Further, to grasp the carbon dioxide discharge of the structural materials in their production stage, I surveyed the volume of the structural materials of the recent 77 buildings designed by the design office that I belong, and calculated the carbon dioxide discharge volume of its production stage. I adopted the following value, based on the "LCA guideline", as the standard unit of each element. Structural steel and reinforcing rod : 1.14ton-co2/ton Mixed concrete : 0.49ton-co2/ton Table-3 shows the carbon dioxide discharge by the structural materials by classification of ground, under-ground, structural steel, reinforcing rod and mixed concrete.

		Carbon dioxide discharged		
		Per unit floor area (t-CO2/m2)	**Rate (%)**	
Ground part	Steel	0.144	29.3%	60.6%
	Concrete	0.155	31.4%	
Under ground part	Steel	0.050	10.2%	39.4%
	Concrete	0.144	29.2%	
Total		0.493	100.0%	100.0%

Table-3: carbon dioxide discharged from structural materials

Table-4, based on the results of table-1,2 and 3, shows the details of carbon dioxide discharge in the building stage of buildings by kind of work and construction material classification.

"Non structural element" in table-4 means the volume of carbon dioxide discharged at the production stage of interior and exterior finish work materials, ceiling and floor work materials and others. "Others" in table-4 means the volume of carbon dioxide discharged at the work site, including the discharge volume by operation of heavy construction machinery, transportation of materials and lighting and air-conditioning.

By the facts stated above, the carbon dioxide discharge volume in the building stage of buildings, classified by kind of work, are expressed as follows.

1) The carbon dioxide discharge volume in the building stage of buildings by the production of

construction materials is approximately 1ton-co2/m2

2) The carbon dioxide discharge volume in the building stage of buildings by the production of

structural materials is approximately 50% of 1). And of it as detail, 60% by the ground part and 40% by the under-ground part of the building.

3) The carbon dioxide discharge volume by the production of equipment apparatus is 35% of 1)

4) The carbon dioxide discharge by the production of non-structural element (interior, exterior, ceiling and floor materials and others is 15% of 1)

Work classification	**Details**		**Carbon dioxide discharged**				**Remarks**
			Rate	**Per unit floor area**	**Rate**		
Construction work	Ground part	Steel	35.0%	0.144	10.2%	21.2%	Structural element
		concrete					
	Under ground part	Steel		0.155	11.0%		
		concrete		0.050	3.6%	13.8%	
	Non-structural element			0.144	10.2%		
Equipment work	Electricity		35.0%	0.165	11.7%		Non-structural element
	Air conditioning			0.116	8.2%	23.3%	
	Sanitary			0.130	9.2%		
	Elevator			0.071	5.0%		
				0.011	0.8%		
Others			30.0%	0.423	30.0%		
Total			100.0%	1.409	100.0%		

Table-4: Carbon dioxide Discharge Volume in the Building Stage of Buildings by Kind of Work and Construction Material

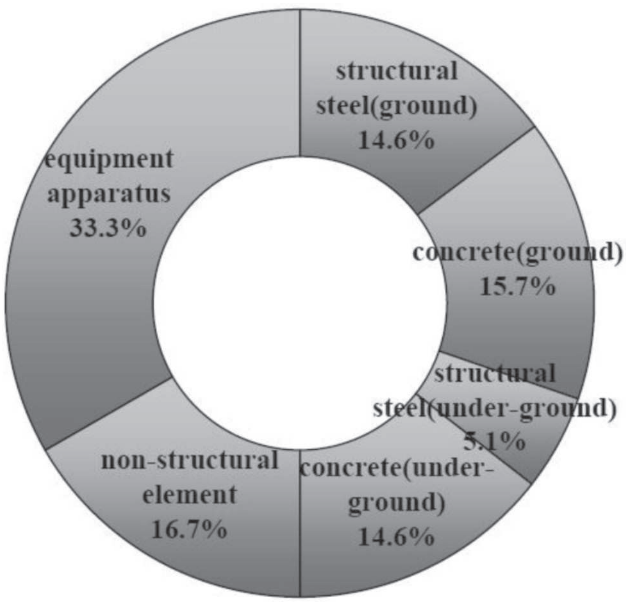

Figure-2: carbon dioxide discharge volume in the building stage of buildings by kind of construction materials

Figure-2 shows the graph of the carbon dioxide discharge volume classified by kind of construction materials.

3. The scenario to reduce carbon dioxide discharge in the construction field

3.1 Supposition terms

In chapter-1, I showed that global warming has come to its limit and that more than 50% reduction of the global carbon dioxide discharge must be attained and as for Japan, 60 to 80% reduction. In chapter-2, based on the studies of the past and analysis of my own, I showed the distribution of the carbon dioxide discharge by kind of work and construction material classification, in the building stage of buildings.

In this chapter, based on the above stated analysis, I would like to show the measures to reduce the carbon dioxide discharge in the building stage of buildings and in the construction field as a whole, and also present the definite scenario to attain the 80% reduction goal imposed to our country.

And as the first step, based on the studies of chapter-2, table-5 shows the needed supposition terms to investigate the measures and scenarios to reduce the carbon dioxide discharge in the construction field.(including both, building and practical use stage of buildings)

3.2 The scenario to reduce carbon dioxide discharge in the building stage of buildings

(1) Measures to reduce carbon dioxide discharge in the building stage of buildings

The measures are as follows

1) Recycle:

From the viewpoint of resource circulation, steel recycling has made much progress. The steel scrap is almost 100% recycled and put into practical use. On the other hand, concerning the concrete aggregate, researches to recycle are done actively, but is not yet in its practical use stage. Although the "Eco-cement" (produced by injecting wastes of other industries, like burst furnace slag and others, into cement) is gradually being in its practical use.

2) Reuse:

The trial of construction material reuse is actively done. (both structural element and non-structural element) Because the reuse of under-ground part of buildings has a big effect to carbon dioxide discharge reduction (became clear by the analysis of chapter-2), I would like to add it to my subject of investigation.

No	Supposition items	Supposition terms
1	Carbon dioxide discharge volume by the construction field	One third of all Japan
2	Carbon dioxide discharged in the building stage	One third of all the construction field
3	Carbon dioxide discharged by the structural materials in its production stage	70% of all the building stage of buildings
4	Carbon dioxide discharged by the structural materials in its production stage	35% of all the building stage of buildings (50% of all the construction materials)
5	Carbon dioxide discharged by the equipment apparatus in its production stage	23.5% of all the building stage of buildings. (one third of all the construction materials)
6	Carbon dioxide discharged by the non-structural element in its production stage	11.7% of all the building stage of buildings. (1/6 of all the construction materials)
7	Carbon dioxide discharged by construction work	30% of all the building stage of buildings
8	Carbon dioxide discharged ground part construction work of buildings	Steel material:15% concrete: 15% (to all construction material)
9	Carbon dioxide discharged by under ground part construction work of buildings	Steel material:5%, concrete:15% (to all construction material)

Table-5: supposition terms to investigate carbon dioxide discharge reduction

3) Lengthen the life time of buildings:

The most effective measure to reduce the carbon dioxide discharge in the building stage of buildings is to lengthen the life time of buildings. The concept "Life time of buildings" have many correlating factors. For instance, it is ordinary that the "put in use period" of structural element, non-structural element and equipment apparatus differ, and to determine the distribution of "put in use period" of each part of buildings is very important.

4) New Technology Development:

The carbon dioxide discharge reduction aim shown in chapter-1 is expected to be attained in approximately in 2050, and before that, various new technology developments can be expected. For instance, the development of "Reduction method" in steel production industry, which will reduce carbon dioxide discharge greatly. Also one can look forward to the increase of efficiency concerning atomic, wind power and geothermal electricity generation.

Now, concerning the above stated four items, based on the results of chapter-2, table-6 shows the carbon dioxide discharge reduction effect of each element. It shows that, to lengthen the life time of buildings, reuse of under-ground part of buildings and switching the energy source to "non-carbon system" is especially effective.

(2) The scenario to reduce carbon dioxide discharge in the building stage of buildings

Based on the analysis in (1), Figure-3 shows the results of the scenario investigation to attain the 60 to 80% carbon dioxide discharge reduction goal. (in the building stage) It is clear that to attain the 60 to 80% reduction goal, present technology along with the technology to be developed in the future is not enough, and that extension of the life time of buildings is indispensable.

3.3 The scenario to reduce carbon dioxide discharge in the construction field as a whole (1) Measures to reduce carbon dioxide discharge in the practical use stage of buildings

In the previous paragraph, paying attention especially to the carbon dioxide discharge of construction materials at its production stage, I showed the discharge reduction scenario. In this paragraph, I will argue the scenario to reduce carbon dioxide discharge in the construction field as a whole, including the practical use stage of buildings. Table-7 shows the results of a adventurous analysis conducted by the National Institute for Environmental studies, about the carbon dioxide discharge reduction in the practical use stage of buildings. It shows the possibility of more than 90% carbon dioxide discharge reduction in both household and industry fields without big market fluctuations. (in practical use stage of buildings) Hereafter I would like to put this analysis as my argument basis.

No.	method	improve items	the effect of carbon dioxide discharge reduction (assumption)	Discharge reduction effect per unit area (to all construction resources)
1	Recycle	utilization of electro-steel	I assumed the carbon dioxide discharge as one third of that of blast furnace iron. Considering that 50% of construction materials are already electro steel, and the supply capacity limitation, I supposed the application rate approximately as 20%	ground: 2% under-ground: 0.66%
2		utilization of eco-cement	I assumd that by increasing the injection volume of waste to cement, the carbon dioxide discharge will be reduced to approximately 50%. Considering that some are already used, and the supply capacity limitation, I assumed the application rate as 20%	ground: 1.5% under-ground: 1.5%
3	Reuse	reuse of construction materials	I assumed that approximately 20% is possible to be applied of the non-structural elements as of now	3.3%
4		reuse of under-ground part of buildings	By table-4, 20% carbon dioxide discharge reduction is possible for all construction materials	20.0%
5	Lengthen the life time of buildings	extension of "put to use period" of structural body of buildings	I assumed to extend n1 times (35 years, as of present)	(50-50/n1)%
6		extension of "put to use period" of equipment apparatus	I assumed to extend n2 times (20 years, as of present)	(33.3-33.3/n2)%
7		extension of "put to use period" of non-structural elements	I assumed to extend n3 times (15 years, as of present)	(16.7-16.7/n3)%
8	New technology development	switching to "direct method" in steel production	I assumed the carbon dioxide discharge as one third of that of blast furnace iron. Considering that 50% of construction materials are already electro steel, and the difficulty of preparation of production line, I supposed the application rate as 20%	ground: 2% under-ground: 0.66%
9		development of "non-carbon" fuel energy source	I assumed that at most, 80% switching is possible(to atomic and solar electricity generation)	–
10		switching of energy source to "non-carbon" type	I assumed that electro steel and steel manufactured by reduction method is possible to switch	ground: 3.6% under-ground: 1.2%

Table-6: assumption of carbon dioxide discharge possibility (do not include the practical use stage of buildings)

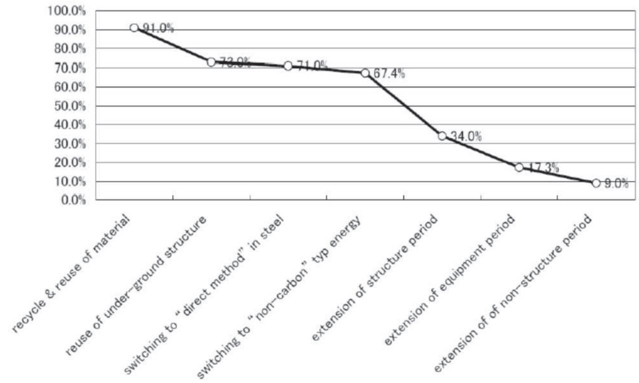

Figure-3: transition of carbon dioxide discharge decrease rate by stage (do not include the practical use stage of buildings)

(2) The carbon dioxide discharge reduction scenario including the practical use stage of buildings

In Figure-4, based on the analysis of paragraph-3.2 and (1), I showed the scenario to reduce carbon dioxide discharge in the whole construction field, including the practical use stage. From the figure, one can see that, by the efforts in the practical use stage of buildings and the trials concerning the construction materials, including technology innovation, will attain only 70% reduction and only by extending the life time of buildings, 80% reduction will be realized.

No.	method	improve items	the effect of carbon dioxide discharge reduction (assumption)	Discharge reduction effect per unit area (to all construction resources)
11	New technology development	by energy saving	reduce 50%	50%
12		Development of non-carbon energy source	I assumed that approximately 80% switching is possible	reduce 90% along with the above item

Table-7: assumption of carbon dioxide discharge possibility (in the practical use stage of buildings)

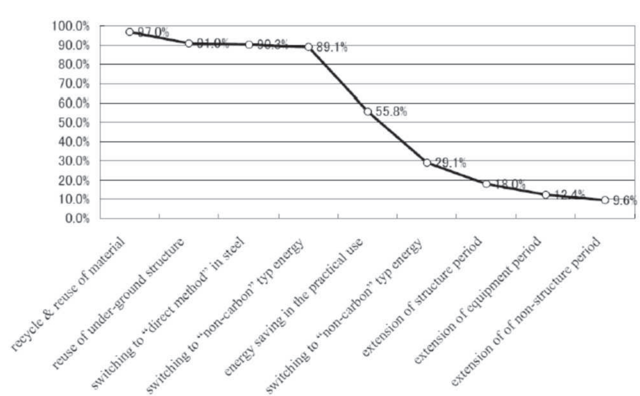

Figure-4: transition of carbon dioxide discharge decrease rate by stage (include the practical use stage of buildings)

Conclusions

In the opening of this paper, I stated that to cope with the serious effects that global warming will bring about, more than 50% global carbon dioxide discharge reduction (compared to present), is indispensable. And as the responsibility of an advanced nation, Japan too must attain 60 to 80% reduction.

The construction field also, must naturally attain this important goal. I investigated the measures to do so, by the following process.

I added the data of the recent 77 buildings designed by the design office that I belong, to the already determined data of the "LCA guideline" and calculated the volume of the carbon dioxide discharge in the building stage of buildings, by kind of materials, kind of work and part of building.

And based on it, the following became clear.

1) As the construction field as a whole, thorough energy saving in the practical use stage of buildings will bring about big discharge reduction.
2) Switching the electricity generation to "non-carbon" system, such as atomic or wind power, and switching the energy source of equipment apparatus (hot-water supply, cooking apparatus and others) to electric type, will also reduce carbon dioxide discharge greatly.
3) But, 1) and 2) will be insufficient to attain the final goal of 60 to 80% reduction, therefore also reduction measures in the building stage of buildings must be determined.
4) Thorough reuse and recycle of construction materials in its production stage is very effective to reduce carbon dioxide discharge in the building stage.
5) Thorough reuse of under-ground part of buildings alone is not enough to attain the 60 to 80% discharge reduction and to lengthen the life time of buildings is indispensable.

Thus the 50% global discharge reduction and the 60 to 80% reduction imposed to our country is not easy to attain, and continuing effort by the whole construction field is needed. And also in the "Low carbon Era", it is essential to lengthen the life time of buildings, and to do so, the following are the themes to investigate.

1) Of the carbon dioxide discharge in the building stage of buildings, discharge volume becomes especially large at the under-ground part building stage. Therefore the study of under-ground part structure from the view point of "skeleton infill" is needed.
2) To lengthen the life time of buildings mean the reduction of domestic market. So to restrain bad influence to the economy, transition of "new construction market" to the "renewal market" should be examined.
3) Along with the demand to lengthen the life time of buildings, the innovation of the "existent unqualified buildings" is pointed out as an urgent issue. To subjugate these contradictory problems is also important.
4) The effects to the carbon dioxide discharge volume by the short and long term social, economic and cultural tendency such as the population decrease, the increase and decrease of newly built building area by the business fluctuations predicted in the near future, are very important subjects that can not be ignored. Hereafter, it is essential to develop studies from such view points.

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Heavy Duty Large Span Precast Floors

Giuliani Gian Carlo[1], Giuliani Mauro Eugenio[2]

Summary

The use of precast elements for the load-bearing structures is today commonplace and it is applied to floors by using mass produced girders or panels; an alternative and more efficient solution is given by the on site casting and lifting to the design level of units capable of bearing on the corner columns only.

In this case all the secondary and the main bearing elements are integrated in the unit which can be either a ribbed slab or a waffle plate; the stresses resulting from the pretensioning or the postensioning are combined with the effects of the total dead load of the unit and not with the weight of each of the separated members and therefore the above said stresses can be higher and the structural efficiency is improved.

In general moving and hoisting a large unit is less expensive than performing the same jobs for separated members; in addition, in the case of the prefabrication of a whole unity, there is no need for connection between the elements and for on site castings.

Some examples of the prefabrication of heavy duty, large span floor units are given.

Keywords

Concrete structures - Design - Construction - Erection

Theme

Special Concrete Structures

1. In general

1.1 Planar precasting

Because of their size, two dimensional units are typically prefabricated on site; we can dub it planar precasting. Prestressed ribbed precast slabs have been successfully used in several projects to create floors with a great load capacity and so avoiding the use of main beams and secondary elements.

The advantages of using slabs come from the structural behavior (loads are transferred directly to the columns) and from the monolithic nature of the element. Not to mention the considerably reduced number of production, storage, transportation, erection and in situ assembling operations

The disadvantages arise from the need to prepare a precasting plant on site with forms, reaction beams, an accelerated curing unit and so on.

The need of suitable equipment for handling and erecting large elements, with unit weights greater than those of normal precast elements, has to be taken into account also.

The costs of setting up the above said equipments are fully compensated by the lower cost of producing similar elements.

According to our experience for a total surface area to be constructed between 8,000 and 10,000 m2, the solution with in site precast planar units becomes a competitive alternative to precasting and assembling of separate elements.

1.2 Space precasting

Another structural system can be developed by using a three dimensional load bearing scheme; we can dub it space precasting. complicated three dimensional joints which are subjected to groups of in plane and out of plane actions concentrated in a reduced area which is located outside the solid material of the member.

In general the structure can be subdivided in sub-elements with linear, planar or space forms to obtain the final space configuration; in any case, the design and the construction of these members, follow criteria which are different from the ones used in the specific above said categories.

Figure 1 the slabs in the final position

Figure 2 lifting of the plate

In general large span roofs are not easily solved with the use of the precast elements currently produced. Shells capable of covering a span of 20 meters are available, but the main beams for similar distances between the columns are too thick and appear unsuitable from the start due to a high roofing load/own weight ratio. The use of secondary and main floor precast members for large spans and the heavy superimposed loads, which are typical in these building layouts, is impossible also. New-concepts in the load-bearing system are needed to optimize the construction system.

2. Examples of planar precasting

2.2 Poli Laboratories building in Rozzano

The prefabricated prestressed ribbed slabs for the four story Poli Laboratories building in Rozzano were designed for a superimposed live load of 12 kN/m2 on a 7.20x8.40 m grid line pattern, while keeping the floor depth at 0.60 m only.

Figures 1 and 2 show the plates in the final position and the lifting of the plates

Figure 3 stacking the plates over the launching cart

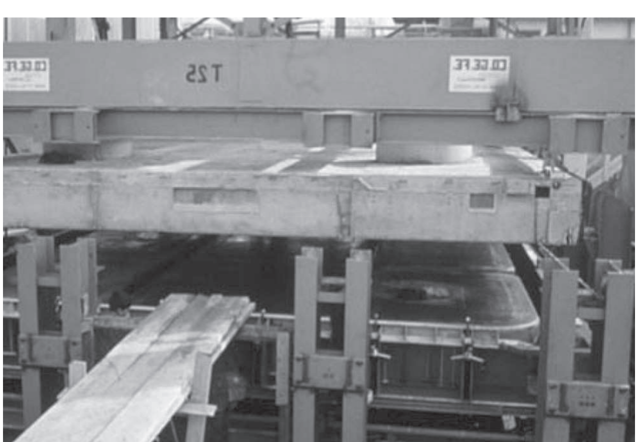

Figure 4 demoulding from the self stressing form

Figure 3 and 4 show provisional stacking the plates which were launched on rails at the ground level and one unit demoulding from the self stressing form

2.3 Roof units for the Milano City Fair exhibition center

The roof units for the new Milano Fair City exhibition center span 20 by 20 meters and are designed for the live load capacity of 6.0 kN/m2; the whole structure was precast on site.

Here monolithic plates are being used with single direction ribs and edge beams (figure 5); the ribs are prestressed with bonded strands, while post-tensioning cables are used for the main beams.

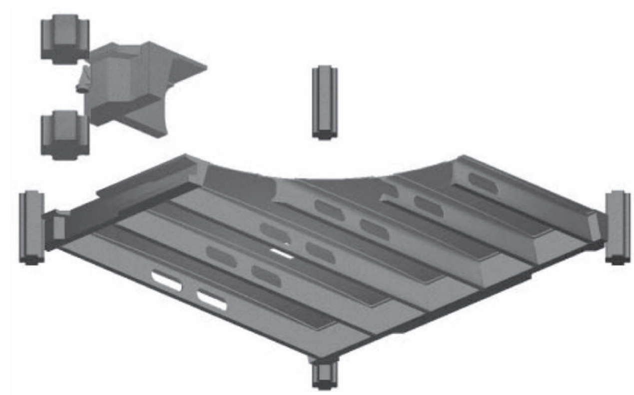

Figure 5 the structural scheme of the floor unit

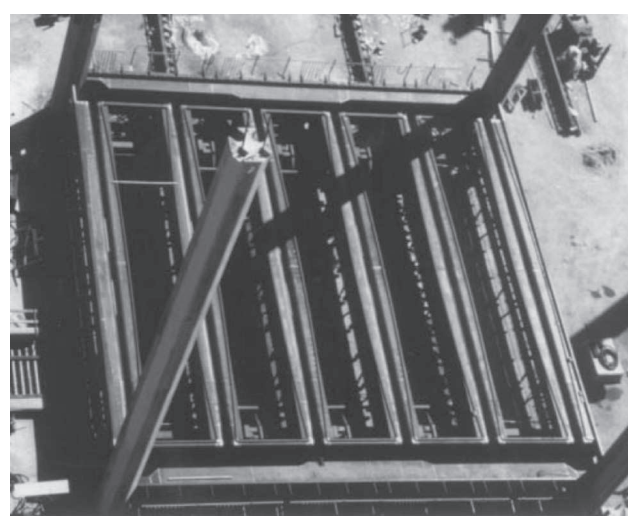

Figure 6 the self stressing form

The construction procedure was decided in the design phase and was planned according to the following steps:

- o Positioning of a self-reacting form (figure 6) inside the inner edges of the corner columns.
- o Casting of the whole unit was performed and steam curing was applied
- o Releasing of the pretension strands and stressing of the cables

- o Injection of water between the cast and the form to ease the demoulding
- o Connecting of the strands to be used for the lifting
- o Lifting of the elements into position by means of four groups of hydraulic jacks with strand recovery anchor to the tops of the columns
- o Moving on rails of the form to every new position; the form edge panel was tilted during this launching to allow passing between the columns (figure 7)
- o Starting of the construction of the precast floor units below the roof unit

The average production rate was one plate every four days but at the end of the works it was reduced to three days; the water injection was no more used because the self demoulding resulted from the pre and post tensioning. The dead load of an unit was 2600 kN.

Figure 7 moving on rails of the form

Figure 8 roof units in position over the floors

3. Examples of space precasting

3.1 Floor units for the Milano City Fair exhibition center

The first floor of this buildings had to be based on 20 by 20 meters bays for the superimposed live load capacity of 15 kN/m^2 and for housing, inside the depth of the structure, the air intake and exhaust ducts for the ground floor areas as well as piping and wiring for the at level located stands.

An innovative solution was worked out with the concept of a composite plate featuring concrete top and bottom slabs, connected by means of a shear layer composed of steel pipe struts arranged in a 3D truss pattern. In this structural configuration, the connection between the flanges (conventionally created by webs) has been replaced by struts resisting the axial loads created by the three components of the shear action (figure 9).

A B C

Figure 9 the resisting scheme of the sandwich plate
A - upper and lower ribbed concrete slabs resisting the in plane actions
B – steel struts and ties for transmitting shear actions between the slabs and prestressing for resisting the tensile stresses of the lower slab
C – the assembly of all the components

The space between the two slabs can be accessed for inspection and plant maintenance.

The construction procedure was decided during the design phase and can be resumed in the following steps:

- o The bottom slab is ribbed and prestressed with bonded strands which cross cast iron nodes embedded in the concrete and the pin connected to the steel pipes of the shear layer (figures 10 and 13).
- o The form was designed as self reacting for the provisional anchoring of the prestressed strands (figure 14)
- o Precast glass fiber reinforced concrete elements placed on the cast iron nodes, located where the pipes converge close to the upper surface, constitute the form for casting the upper slab (figure 11).
- o All the ducts, pipes and other equipment were positioned inside the plate (figure 12)
- o The upper slab was cast over the fiber reinforced concrete forms
- o The pretensioning of the lower slab was released
- o Four groups of hydraulic jacks lifted the completed plate (weighing a total of 4800 kN, including the already installed ducts) into position
- o The connection with the column was effected
- o The floor plates lay below the roof units (figures 8 and 15)
- o The self stressing form was moved on rails to every new bay position

It is worth to mention that, although this sandwich plate is quite innovative, it was designed and could be constructed using materials and components which belong to well known and available technologies:

o concrete o strands o steel pipes o cast iron

The columns are an integral part of the structural concept:

o Pre-cast, with an octagonal cross-section. 25 meters high and weighing 600 kN; the outer columns are is sight

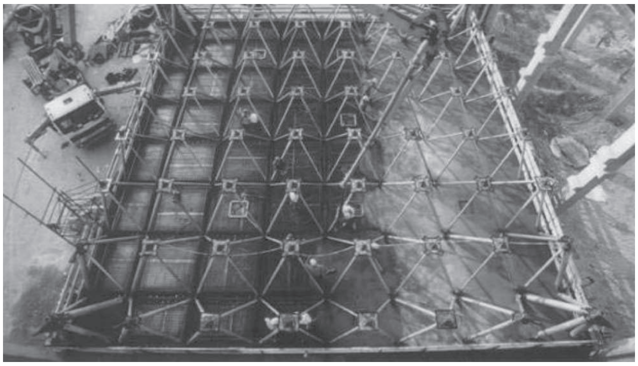

Figure 10 casting of the lower slab

Figure 11 placing of the precast concrete forms over joints

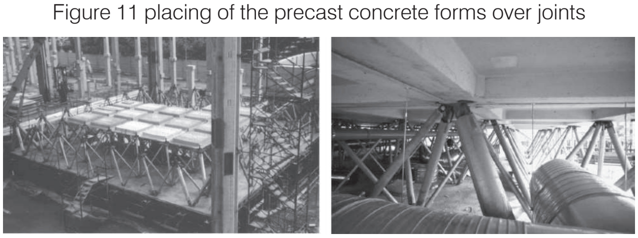

Figure 12 placing of ducts and wirings inside the plate

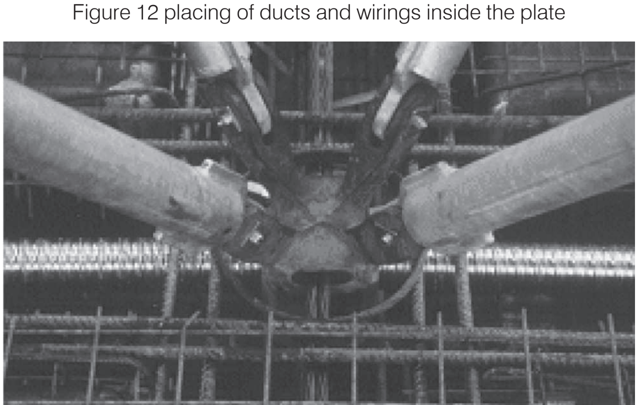

Figure 13 a cast iron joint inside the lower slab

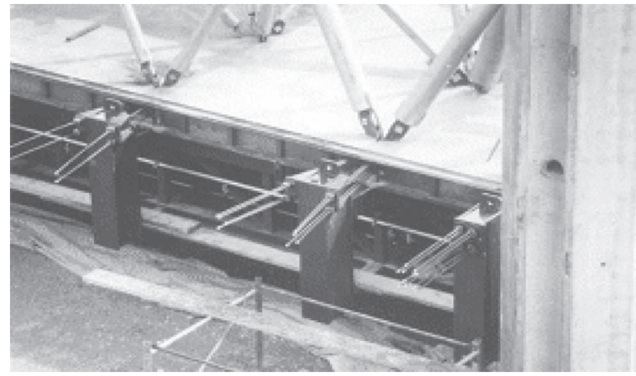

Figure 14 provisional anchoring of pretensioned strands

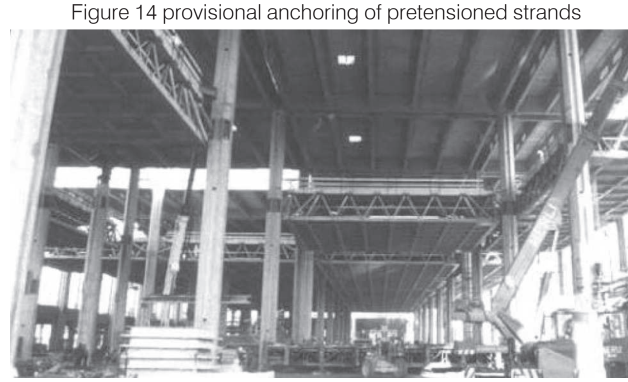

Figure 15 construction of the plates below the roof units

Figure 15 construction of the plates below the roof units

o The support of the composite slab was created by the introduction of steel-concrete elements in the column with the necessary recesses to house the bearings, thus allowing for lifting the plates without any overhanging elements and for a reduction of the column bending due to the location within the cross-section of the actions transmitted by the bearings

4. Conclusions

According to the design and construction experience earned with the illustrated examples, precast solutions are very often highly cost effective if the constructor has the suitable level of expertise required.

In many cases, while it may be far more exacting in terms of engineering, precasting is the only available choice because it is far more competitive than on site conventional construction with regard to price, the quality of the work and construction time.

A Linear Model for Gusset Plate Connections

Chiara Crosti[1], Dat Duthinh[2]

Summary

The National Transportation Safety Board (NTSB) investigation [1] of the 2007 collapse of the I-35 W Bridge in Minnesota, United States, used very detailed nonlinear finite-element (FE) analysis, which is too difficult for routine design. On the other hand, simple guidelines amenable to hand calculations were provided by the Federal Highway Administration (FHWA) for the load rating of gusset plates, but they are unable to account for the actual behavior of the connections in the global bridge analysis. An intermediate approach is presented, whereby a simplified connection in the form of linear spring models is proposed to improve on current design analysis, which typically is linear and assumes rigid connections. The model is derived from the FE analysis of a typical joint and can be placed at many locations, with modifications required by geometry. The model is verified against a two-dimensional global FE analysis of a steel truss bridge that uses detailed connection models.

Keywords: Bridges, connections, finite-element analysis, gusset plate, linear springs, steel truss

Theme: Structural design and analysis - steel bridges - joints

Acknowledgment: The authors are grateful to NTSB and FHWA for providing access to the detailed FE model used in the investigation of the collapse of the I-35 W Bridge.

Introduction

The 2007 catastrophic collapse of the I-35 W Bridge in Minnesota, United States, under ordinary traffic and construction loads, was triggered by the buckling of an undersized gusset plate [1]. Gusset plates are complicated structural components used to connect linear structural members such as beams and columns. Their use in buildings and bridges goes back many decades, if not centuries, and certainly predates the use of computers in structural analysis and design. Practical design methods ensure safety by providing a load path that satisfies equilibrium, boundary conditions and does not exceed material yield limits. The resulting stress field is by definition a statically possible yield state of stress. Safety against plastic failure is assured because, according to the Lower Bound Theorem of the Limiting Load [2], a statically possible yield state of stress is less than or equal to the limiting load, which is characterized by unrestricted plastic flow. However, there is no guarantee that the design load path is the actual one, and thus the design methods provide no information on the load-displacement behavior or stiffness in the elastic range.

Current procedures [3] for the design and load rating of multi-member gusset plates consist in checking axial, bending and shear stresses along various sections deemed critical, using elastic beam theory. These procedures are intended to ensure a safe and conservative design, but produce results that can be quite different from more realistic finiteelement (FE) results, and cannot predict either stiffness or actual behavior. To do so would require highly sophisticated and detailed FE models, such as the ones used in the National Transportation Safety Board (NTSB) investigation of the I-35 W collapse [1]. An intermediate approach is presented here, whereby a simplified connection in the form of linear spring models is proposed to improve on current design analysis, which typically is linear and assumes rigid connections. The paper starts with a literature survey that focuses on approximate design and analysis methods, and follows with a presentation of the spring model and its verification.

2. Review of the literature on approximate methods for gusset plates

Extensive reviews of the literature were performed by Chambers and Ernst [4] and Astaneh-Asl [5]. In keeping with the theme of this paper, we focus here on approximate methods of analysis and design.

Whitmore [6] tested 1:4 scale specimens of gusset plates for Warren trusses made of aluminum, masonite and bakelite using wire-bonded strain gages (a novelty at the time), brittle lacquer and photoelastic techniques. Based on these experiments, he developed the effective width that now bears his name, "by constructing lines making 30 degrees with the axis of the member which originate at the outside rivets in the first row and continue until they intersect a line perpendicular to the member through the bottom row of rivets." The maximum tensile and compressive stresses may be approximated by assuming the force in each diagonal to be uniformly distributed over this width.

Astaneh [7] proposed modeling gusset plates as wedges under a point load (figure 1). In the elastic range, closed-form solutions exist for infinite wedges, but cannot account for the actual boundary conditions. Furthermore, actual loads are transferred to gusset plates by rows of rivets or bolts, rather than at a single point. The author also indicates that wedge models cannot predict buckling, for which he proposes a fin truss model (figure 2), where the cross section of each bar is the average of the cross section of the triangle it bisects.

Astaneh [7] suggests an effective length factor of 0.7 for the struts to account for the restraint provided by the transverse direction, which applies to the buckling of a two-dimensional plate, but not that of a one-dimensional column. The use of multiple fin trusses to model gusset plates that connect multiple members rapidly becomes cumbersome, and makes the FE method very attractive in comparison for its accuracy and automation.

An elegant, statically possible load path with no moment (i.e., concentric) in the gusset-to-beam and gusset-to-column connections was developed and called the Uniform Force Method by Thornton [8], who obtained the following connection forces (symbols are defined in figure 3):

$$H_B = \frac{\alpha}{r} p, V_B = \frac{e_B}{r} P, V_C = \frac{\beta}{r} P, H_c = \frac{e_c}{r} P, r = \sqrt{(\alpha + e_c)^2 + (\beta + e_B)^2} \quad (1)$$

Dowswell and Barber [9] provided guidance on the required plate thickness t? to prevent the buckling of gusset plates and a quantitative definition of compact gussets, where c = the shorter of the distances from the corner bolt or rivet to the adjacent beam or column, E = modulus of elasticity, σy = yield stress, and l_1 = equivalent column length from the middle of the Whitmore width:

$$t_\beta = 1.5\sqrt{\frac{\sigma_y C^3}{E l_1}} \quad (2)$$

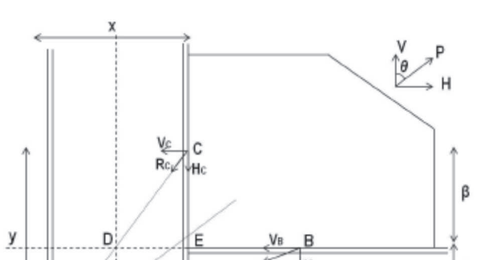

Figure 1: Wedge model [7]

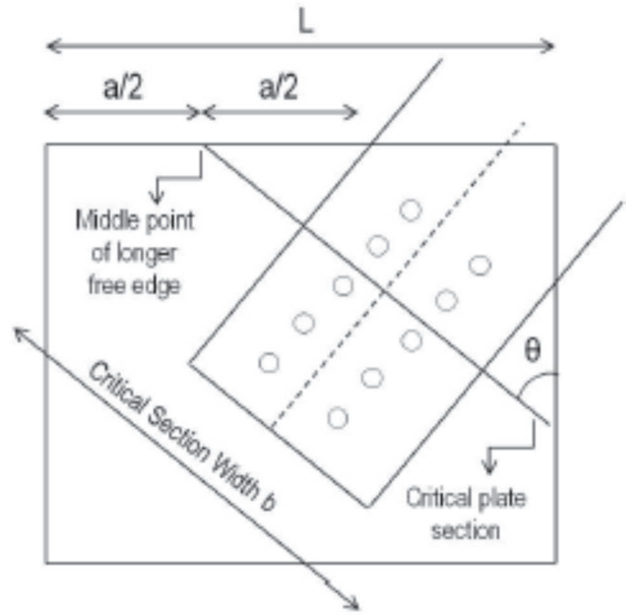

Figure 2: Fin truss model [7]

The gusset plate is compact if its thickness t ? t? and non-compact if t < t?. Dowswell and Barber [9] compared their theoretical buckling capacities Pth with experimental and FE calculations in the literature Plit. They used the average of the lengths from the middle and the ends of the Whitmore width for the equivalent column length. For compact gusset plates, using an effective length factor of 0.5, they found the ratio Plit / Pth = 1.47 and for non-compact gusset plates, using an effective length factor of 1.0, Plit / Pth = 3.08. Thus, the separation of compact from non-compact gusset plates and the subsequent different effective length factors resulted in inconsistent and problematic factors of safety for design against buckling.

Brown [10, 11] developed analytical expressions for the edge buckling of gusset plates, based on the elastic buckling stress _ ! of a plate supported on its loaded edges, but otherwise unrestrained:

$$\sigma_{cr} = \frac{\pi^2 E}{12\left(1 - v^2\right)\left(Ka/t\right)^2} \quad (3)$$

where with $ = Poisson ratio and K = 1.2 reflects the end conditions of the strip along edge a (one end fixed, the other end restrained against rotation but free to translate at the brace, figure 4). This stress can also be expressed in terms of elastic column buckling, with ? = radius of gyration and l = column length:

$$\sigma_{cr} = \frac{\pi^2 E}{\left(Kl\rho\right)^2} \quad (4)$$

Setting Eq.3 = Eq.4 with v =0.3 produces

$$\frac{Kl}{\rho} = \frac{\sqrt{12\left(1 - v^2\right)}Ka}{t} = 4.0a/t \quad (5)$$

Setting Eq.3 = Eq.4 with v =0.3 produces

For edge buckling, the critical section bisects the long free edge and is perpendicular to the brace (figure 4). Its width b is different from the Whitmore width (symbols are defined in figure 4):

$$b = \frac{L - a/2}{\sin\theta} \tag{6}$$

Only a fraction f of the total brace load contributes to the edge buckling of the gusset, the rest being transferred directly to the steel frame. Assuming the bolts or rivets carry uniform loads (figure 4):

$$f = \frac{(a/2)\cos\theta + p - e}{np} \tag{7}$$

where n = number of bolt rows in load direction, p = distance between consecutive bolt rows, and e = edge distance. From Eqs. 4 to 7, the edge buckling load is therefore:

$$P_b = \frac{\sigma_{cr} bt}{f} = \left(\frac{\pi t}{4a}\right)^2 \frac{Etnp(2L - a)}{\sin\theta\,(a\cos\theta + 2p - 2e)} \tag{8}$$

Specimen	P_y/P_d	P_y^u/P_d	P_{cr}/P_d	P_u/P_d	P_u/P_{cr}	Location
1	1.4	2.0	2.1	3.1	1.4	A
2	1.8	---	2.5	3.8	1.5	A,B
3	1.8	2.7	2.9	3.6	1.2	A,B
4	1.4	1.8	1.8	2.5	1.4	A,B,C
5	1.7	2.6	1.7	2.6	1.5	A,B,C
6	1.4	1.7	1.7	2.9	1.7	A,B
7	1.5	1.8	2.1	2.9	1.3	A
8	1.8	2.4	2.6	3.2	1.2	B

Table 1: Safety factors for various characteristic loads [12].
Pd = design load; Py
u = load at which plastic zone penetrates web of chord;
Py = initial yield load; Pcr = initial buckling load; Pu = ultimate strength.

Local yielding and local buckling preceded global buckling of the gusset plates. The load at which local buckling started depended on the extent of yielding, which covered the inner portions of the gusset plate, whose in-plane stiffness was constrained by the surrounding elastic region. The authors performed a FE analysis of an idealized triangular plate which represented region A (figure 5) under various boundary conditions with the load assumed uniformly applied along the chord (figure 6). The elastic buckling stress ?cr for a plate has the following form, with K, k = constants dependent on edge conditions (table 2), and b = loaded width:

$$\sigma_{cr} = \frac{K\pi^2 E}{12(1 - v^2)\left(\frac{b}{t}\right)^2} \tag{9}$$

$$\frac{\sigma_{cr}}{\sigma_y} = \frac{K\pi^2}{12(1 - v^2)\left(\frac{b}{t}\sqrt{\frac{\sigma_y}{E}}\right)^2} = \frac{k}{\left(\frac{b}{t}\sqrt{\frac{\sigma_y}{E}}\right)^2} \tag{10}$$

Table 2: Buckling constant from FE analysis of idealized triangular plate [12]

Case	Diagonal edge	Horizontal edge	*k*
1	Simple support with sway	Simple support	0.276
2	Simple support	Simple support	0.918
3	Fixed support with sway	Simple support	1.850
4	Fixed support	Simple support	2.156

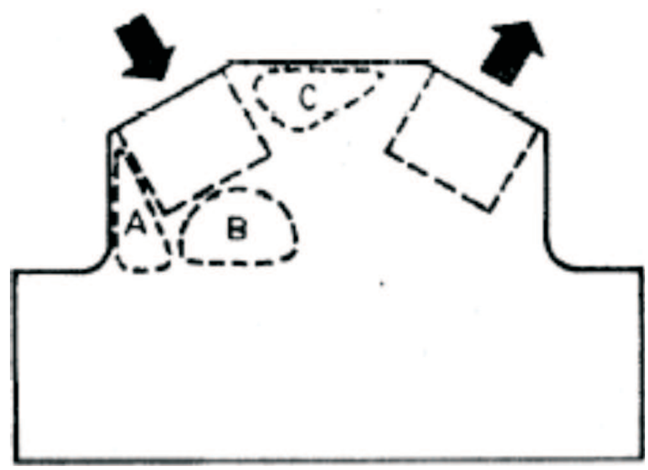

Figure 5: Regions of yielding or local buckling [12]

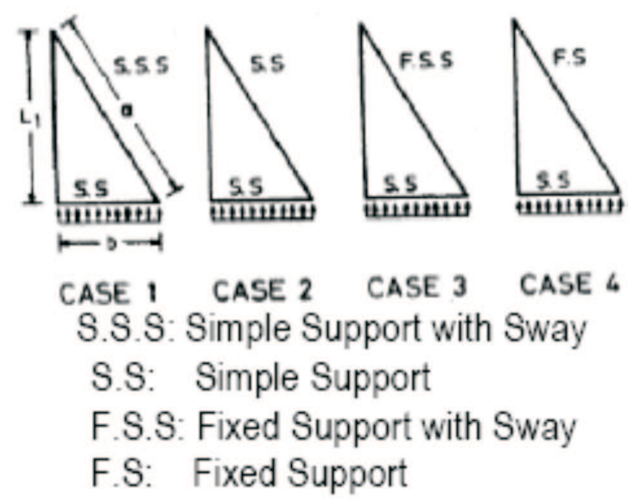

Figure 6: FE analysis of idealized plate region A [12]

stress in region A reached the allowable stress in the material, ?a = 0.58 ?y , and with L1 = length of the vertical free edge, the authors proposed the following design thickness for local buckling:

$$t_{cr} = 1.10 L_1 \sqrt{\frac{\sigma_a}{E}} \tag{11}$$

If ?cr = ?a and k = 0.276, Eqs. 10 and 11 are equivalent for L1 = b?3. Test specimens 4 and 5 (table 1) used a plate thickness t < tcr and resulted in the lowest factor of safety Pu / Pd. For all the other test specimens, this factor equaled or exceeded 2.9. Specimen 5 did not benefit from the stiffening effect of a chord flange, thus causing Pcr / Pd to be much less than Pyu/ Pd. The authors further noted that Pu / Pcr was independent of structural details and gusset type.

Subsequent to the collapse of the I-35 W Bridge, the Federal Highway Administration produced a guidance document for the load rating of gusset plates [3], which recommended that bridge owners evaluate the connecting plates and the fasteners of truss bridges. The gusset plates are rated for their resistance to tension, compression and shear. In tension, the modes of failure that need to be evaluated are yielding of gross section, fracturing of net section, and rupture by block shear. For shear resistance, several sections must be investigated to find the governing one. Finally, for compression, the buckling of an equivalent column based on the Whitmore effective width is assessed. Simple, hand-calculable formulas are presented for this load rating.

3. Simplified Model using Finite Elements

From the literature review, it is seen that there are simple design methods based on equilibrium and elastic behavior and proven safe by experiments. There is, however, no simple way of calculating the actual behavior of a gusset plate, even in the elastic range. Designers ensure that the connections are stronger than the members, then proceed with a structural analysis that assumes rigid connections. Such a structural analysis is incapable of predicting connection failure, or account for the flexibility of the connection in the global behavior of the structure.

On the other hand, there exist detailed models such as the one analyzed by the National Transportation Safety Board (NTSB) as a result of the collapse of the I-35 W Bridge [1]. Forensic investigation had already pinpointed and preliminary analysis confirmed that the trigger of the collapse was the buckling of the undersized joint U10. So there was justification in performing a detailed FE analysis of joint U10 to replicate the collapse. Each gusset plate of the detailed model was composed of 289 000 elements, whose in-plane size was 5 mm (0.2 in) in highly stressed regions and less than 15 mm (0.6 in) elsewhere [13]. Such a detailed analysis is clearly beyond routine design and requires advanced skills and powerful computers.

In the present work, we take advantage of the NTSB detailed FE model (formulated in software ABAQUS [14]) of gusset plate U10 to establish the equivalent stiffness of springs that completely model the elastic behavior of the connection. The FE model has 5 stub members attached to a pair of gusset plates (figure 7), and that model is connected to the appropriate members in the global model. For the simplified connection model, the stub members and gusset plates are replaced by 5 user-defined structural elements, called springs for short, that can each have up to a full 6 x 6 stiffness matrix for all 6 degrees of freedom (DoFs). To establish the flexibility of the equivalent spring for member 1 for example, we fix the ends of members 2 to 5 and apply a unit force and obtain the displacements and rotations at the end of member 1. We repeat the process by applying unit forces and moments corresponding to all 6 DoFs, and thus obtain the 6 x 6 flexibility matrix for member 1 in global coordinates. This flexibility matrix is inverted to obtain the stiffness matrix, which is then transformed to local coordinates and applied to the simplified spring model (figure 8). We must exercise care in defining the stiffness of the user-defined element in software STRAND/STRAUSS [15], for example, the local axis 3 is always in the longitudi-

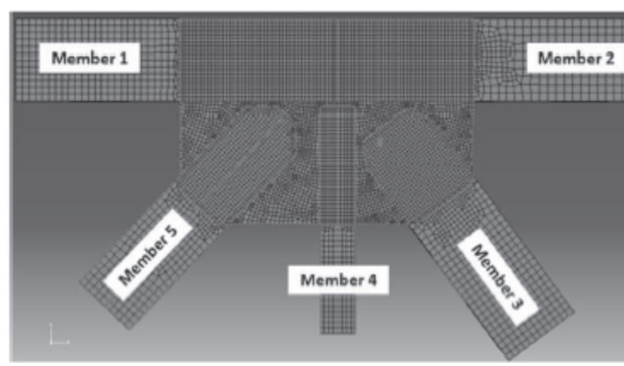

Figure 7: Detailed FE model of gusset plate

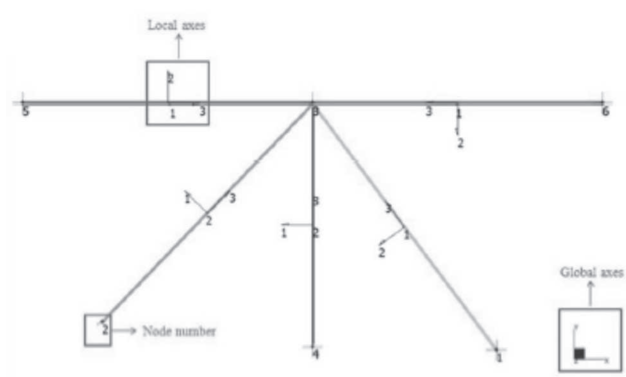

Figure 8: Equivalent springs

Node 5	U_x (mm)	U_y (mm)	U_z (mm)	R_x (mrad)	R_y (mrad)	R_z (mrad)
F_x = 444.8 (kN)	**1.560E-01**	8.190E-02	-7.380E-04	6.720E-06	7.720E-05	-2.050E-02
F_y = 444.8 (kN)	8.190E-02	**3.500E+00**	-3.420E-04	4.510E-05	-7.660E-05	-1.960E+00
F_z = 444.8 (kN)	-1.880E-05	-3.420E-04	**2.100E+01**	1.510E+00	5.080E+00	1.370E-04
M_x = 228.8 (kN·m)	3.490E-06	9.960E-06	7.770E-01	**4.300E+00**	7.500E-02	-7.600E-06
M_y = -113.0 (kN·m)	7.830E-07	1.940E-05	-1.290E+00	-3.710E-02	**-7.040E-01**	-8.010E-06
M_z = -161.0 (kN·m)	7.430E-03	7.090E-01	-5.100E-05	4.410E-06	-1.110E-05	**-6.160E-01**

Table 3: Member 1, node 5, displacements and rotations obtained in ABAQUS for forces and moments applied one at a time (detailed model)

Node 5	U_x (mm)	U_y (mm)	U_z (mm)	R_x (mrad)	R_y (mrad)	R_z (mrad)
F_x = 444.8 (kN)	**1.560E-01**	8.190E-02	1.220E-03	9.500E-05	3.780E-04	-2.050E-02
F_y = 444.8 (kN)	8.190E-02	**3.500E+00**	-3.390E-04	-4.490E-05	-7.510E-05	-1.960E+00
F_z = 444.8 (kN)	1.220E-03	-3.390E-04	**2.100E+01**	1.510E+00	5.080E+00	6.330E-05
M_x = 228.8 (kN·m)	4.890E-05	-2.310E-05	7.770E-01	**4.300E+00**	7.520E-02	-1.650E-04
M_y = -113.0 (kN·m)	-9.590E-05	1.910E-05	-1.290E+00	-3.710E-02	**-7.040E-01**	-5.550E-06
M_z = -161.0 (kN·m)	7.410E-03	7.080E-01	-2.290E-05	1.160E-04	-7.900E-06	**-6.160E-01**

Table 4: Member 1, node 5, displacements and rotations obtained in STRAND/STRAUSS with user-defined elements for forces and moments applied one at a time (simplified model)

	STRAND/STRAUSS beams with rigid joint	ABAQUS FE model with thickness				
		2.54 mm	6.35 mm	12.7 mm	19.0 mm	25.4 mm
U_x (mm)	1.290E-01	2.900E-01	2.200E-01	1.560E-01	1.680E-01	1.580E-01
U_y (mm)	5.690E+00	6.390E+00	4.410E+00	3.500E+00	3.110E+00	2.860E+00
U_z (mm)	7.460E+00	1.420E+02	3.530E+01	2.100E+01	1.690E+01	1.480E+01
R_x (mrad)	2.250E+00	1.030E+01	6.450E+00	4.300E+00	3.670E+00	3.360E+00
R_y (mrad)	-6.770E-01	-1.140E+00	-8.640E-01	-7.040E-01	-6.300E-01	-5.860E-01
R_z (mrad)	-7.360E-01	-7.940E-01	-6.750E-01	-6.160E-01	-5.870E-01	-5.680E-01

Table 5: Comparison of displacements and rotations of STRAND/STRAUSS model with rigidly connected beams and ABAQUS FE model with various gusset plate thicknesses

nal direction and pointing away from the end of the stub.

After the equivalent springs are assembled, the response of the simplified connection is verified against the detailed FE analysis, for the same set of applied forces and moments. As done previously, forces and moments are applied one at a time, for example at end node 5 of member 1, while the other end nodes are fixed (figure 8). Agreement in the displacements and rotations calculated is good, especially for the diagonal terms, with acceptable errors in the offdiagonal terms, which are several orders of magnitude smaller (tables 3 and 4). Simpler springs, with diagonal stiffness matrices (6 terms, instead of 6 x 6), did not give acceptable results.

We now focus on the diagonal terms of the above matrices and compare the displacements and rotations predicted by a STRAND/STRAUSS model with rigidly connected beams and the detailed ABAQUS model with various gusset plate thicknesses (table 5). As expected, the rigid joint produces an approximation that is not as good as the more complicated spring model, equivalent to a gusset plate of thickness t = 12.7 mm, but is reasonable for gusset plates of realistic thickness, except for the out-of-plane deformation Uz under a force Fz , an unrealistic load for an essentially planar model.

4. Global analysis

The various connection models were placed in a two-dimensional model of the I-35 W Bridge (figure 9), at a location corresponding to the U10 gusset plate that triggered the bridge collapse. The model was subjected to its own dead load, a uniform deck load of 74.92 kN/m (427.8 lbf/in), and a concentrated construction load near U10 of 115.7 kN (26 kips). Four cases were run: 1) all joints rigid; 2) U10 modeled with springs, all other joints rigid; 3) U10 modeled with detailed FE, all other joints rigid; and 4) all 5 member joints hinged, all other joints rigid. Liao et al. [16] showed by influence lines that the temporary construction loads placed near U10 significantly affected the forces imposed on it and may have triggered the collapse. Results (figure 10, table 6) show that modifying the stiffness of one connection within the elastic range produces no noticeable effect on the maximum vertical deflection of the bridge (at midspan).

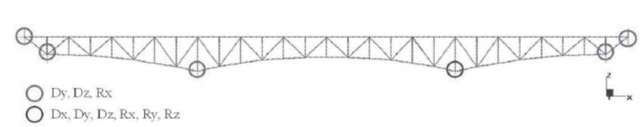

Figure 9: Two-dimensional model of I-35 W showing support conditions (restrained DoFs)

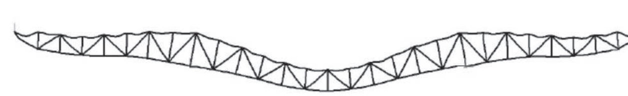

Figure 10: Typical vertical displacements

5. Conclusion and future work

The equivalent spring model produces a good approximation of the behavior of a gusset plate connection in the elastic range. A major difficulty is that it is based on a prior detailed FE model. Future work includes adapting this detailed FE model to various connections of similar geometry, simplifying the FE model for a given level of accuracy, modeling all the gusset connections of a bridge with realistic stiffness, and most interestingly, extending the model to the nonlinear range, up to connection failure.

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High Rise Buildings Construction Stage Design: A case study in Milan, the urban redevelopment of Porta Nuova Varesine

de'Angelis Paolo

Summary

Nowadays, once tender issue for complex design has been completed, because of increasingly ambitious aims made by developers who aspire to compress the time needed for design, drawings, specifications and verifications are often not adequate for construction.

This scenario creates the need of a construction design stage in order to produce a project good for construction for the General Contractor.

This design stage is generally as delicate as the previous ones, engineers indeed are called to adjust the contractor needs (construction methods, availability of equipment, time of erection, respect of time schedule) with the preceding design concepts and choices that not always took into account the necessities of a construction company.

In this article some paradigmatic examples occurred during the construction design stage of Porta Nuova Varesine Area in Milan will be presented. A former funfair area (about five football pitches) becoming a urban area densely built including two high rise buildings and other 10 buildings either residential or commercial.

Keyword

High-rise buildings, construction design stage, general contractor, post-tensioned slabs

Theme

Construction Stage Design and Variances during construction.

In this article will be presented some issues concerning the Construction Design Stage that strongly interfere with the original structural project. The aim is to show how construction methods, if not properly taken into account during design process, can affect the project even in its key concepts.

Clarifying the design flow that is usually developed for such important tender we introduced the following simplified chart.

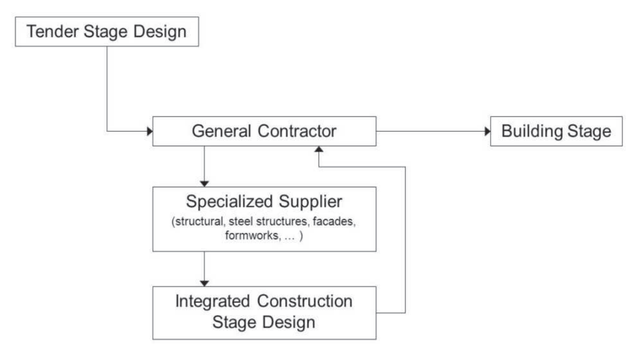

Fig. 1 - Design simplified flowchart

Construction Site Data.

The construction is located on a former recreational area in the urban area of the city of Milan (Italy), the site features a rectangular shape (in plan) with 300m by 100m edges.

Fig. 2 - Milano Porta Nuova new area: The new headquarters of the Regione Lombardia (a), the Garibaldi Repubblica towers (b), the Diamond tower (c), the Solaria tower (d) and the Bosco Verticale e (on the left))Site c and d are part of the Varesine Area (on the right)

A deep excavation was made to accommodate a 4 levels basement (parking and service areas), 12 buildings and a cultural centre elevate over street level.

The above ground built area is marked by three high rise residential (buildings 10, 11 and 12) buildings and one commercial tower (building 3).

In this article we will focus on the two highest buildings of the yard: building 3 ("Diamante" Tower) and building 12 ("Solaria" Tower)

	Diamante Tower (bld 3)	Solaria Tower (bld 12)
Height:	140 m (30 livelli)	3 pods – 35 floors(142, 112, 82 m)
Clumns / Core:	Steel / Concrete *	Concrete / Concrete
Floors:	Concrete on corrugated steel sheet	Post tensioned concrete slabs
Foundation:	Direct foundation	Direct Foundation (soil stifness improvement by diaphragms)
Intended use:	offices	residential
Architectural Project:	KPF	Arquitectonica / Caputo Partnership
Structural Project:	Ove Arup	Ove Arup
Client:	Varesine Srl (Hines)	Varesine Srl (Hines)
General Contractor:	CO.VAR. (COSTRUZIONE VARESINE)	CO.VAR. (COSTRUZIONE VARESINE)
Construction Structural Project:	Redesco Progetti srl (Milan)	Redesco Progetti srl and FV Progetti srl (Milan)
Construction years:	2009-2012	2010-2013
	*actually highest steel building in Italy	

Buildings essential features

This wide real estate operation imposed to the General Contractor a great effort for shortening the time of construction.

Very efficient production systems were adopted for the erection of the structure; as consequence, an extremely important role in the construction process was given to the formworks supplier.

The development of the Construction Project was therefore carried on taking into account all the requirements due to the adoption of the formwork climbing system.

Fig. 3 - Construction Site: Bld 3 (left) and Bld 12 (right)

Formworks.

A singular aspect that was faced is the subdivision of the core, due to the choice of climbing formwork made by the GC. (there wasn't any indication in the Tender Stage Design)

The appropriate selection of the constructive system was ruled by the most time saving system according to the cranes carrying capacity.

For building 3, Diamante Tower, a ACS (Auto Climbing System) and RCS-P (Rail Climbing System) systems were used to elevate the core floor every 5 days.

For building 12 similar systems (ACS and RCS-C) were used in order to reach similar performances.

Fig. 4 - Climbing Formwork System

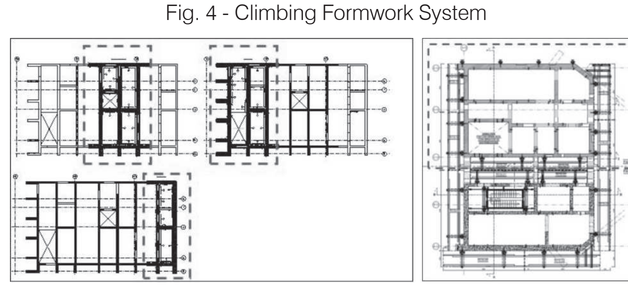

Fig. 5 - Core splicing for Bld.3 (left) Bld 12 (right)

The necessary splicing of the core into different parts (3 for bld 3 and 2 for bld 12) was in conflict with the realization of continue coupling beams as primary stability system, furthermore for bld. 3, due to a change of direction in the vertical load path (columns deviation) a belt system was introduced in the Tender Design Stage, at the 9th level of the tower.

Finally it is significant to point out that the thickness of the main beams was not the same as the thickness of walls but exceeded it by 5 cm i.e. 2.5 cm each side (internal and external).

It is clear that all these details, designed by the engineer

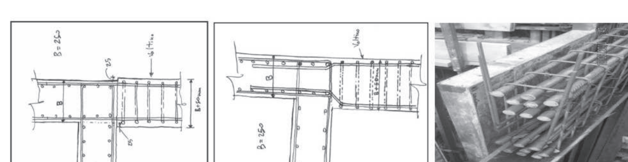

Fig. 6 - Coupling beams from the left: tender design layout, construction design layout, on site.

during the tender stage and conceived for a traditional construction system, conflicted with a time performing climbing formwork.

Thus during the Construction Design Stage it was necessary to partially modify these elements by intervening on reinforcement and positioning the coupling beams.

Concrete Casting

Another main aspect that is often demanded to a Construction Stage Design deals with the compatibility of concrete age and construction system loads acting on very young structures. in order to save time, the GC, plans to move formworks around 2/3 days after concrete pouring., hence resistance of structures with a so young concrete have naturally to be checked for the construction design load.

For example, in Fig. 7 the displacement due to concrete pumping are shown.

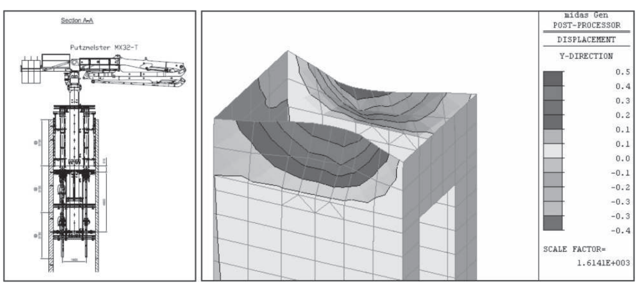

Fig. 7 - Core Check due to concrete pump

Deformations

A second construction aspect that can affect the design, concerns the differential shortening compensation methods.

In case this aspect is neglected or differently conceived by the tender designer, putting it into practice requires a thorough analysis to identify the proper corrective action required in order to obtain the desired slab horizontality at the end of construction and at long term .

For both the buildings, during construction stage design, these kinds of analysis were carried out.

Calculations results were processed in order to integrate the requirements of original design but introducing corrective measures and engineered surveying systems.

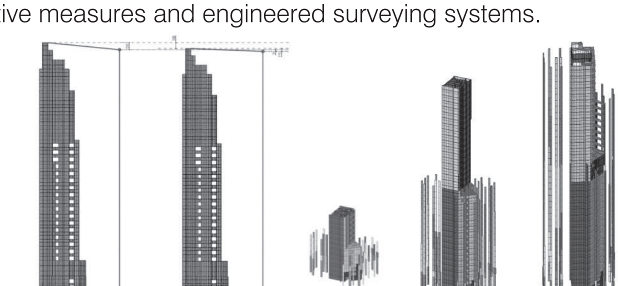

Fig. 8 - Differential Axial Shortening FEM Modeling

Project modifications

At the end we'd like to point out a further aspect that makes the construction stage design somewhat critical.

This happens every time an impacting variation of the original project is necessary.

In the case study we presented, the residential tower underwent an important modification about its slabs concept.

In the Solaria tower the tender stage design conceived the floor slabs (last span) as if were supported at the end by a steel structure called Megaframe that strongly characterized the building.

During construction stage design, the Client together with the Architect, removed this element from the project (and the budget), forcing floor balconies to stand as cantilevers.

A thorough analysis was performed to find the best solution to save time and to allow for the construction of such important cantilevered structures.

What we looked for was a solution that was compatible with already built basement levels and was suitable for the very strict comfort criteria imposed by the client.

The introduction of post-tensioning multi strand cables into the concrete slab made it possible.

Reduced thickness due to the respect of self weight on foundation, drove the choice towards an unbonded system.

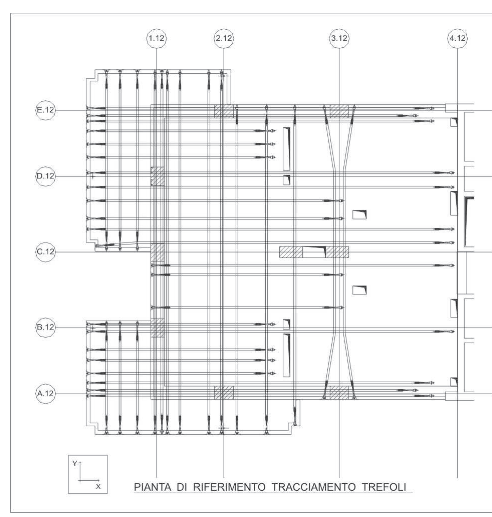

Fig. 9 - Dimensions of floor slab (left) post-tension system into the slab (right)

Conclusions

The systems for the construction of tall buildings are becoming more efficient and performing.

The time of execution of works of extraordinary importance has decreased dramatically due to the use of advanced technologies that optimize the work on site, ensuring safety and productivity.

These processes necessarily require a phase of refinement of the final design as the design choices made upstream of the construction must be reconciled with the strategic choices of the Contractor.

This activity 'has an intensity' and an extension variable according to the degree of definition of constrution issues contained on the final construction project.

The challenge of building design is nowadays put into practice by removing critical issues thath occur when the specificity and uniqueness of works of considerable importance are forced into standardized production systems that can ensure production complying with the timing proposed by investors / clients

Often, the imposition of ambitious schedules force the GC to look for constructive solutions that require high productivity insights and high-level analysis, as well as the exact definition of every detail.

Furthermore from the point of view of the responsibility of the designer, the distinction between Tender Stage Design and Constrution Stage Ddesign is not always obvious.

We believe that distinction must be clear and clear in all cases and that, in order to avoid to transfer resposnibility to the General Contractor, a detailed tender design stage should be carried out considering details and construction methods

Author Affiliation

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Recent Tower Buildings in Milan - Structural Design

Fabio Capsoni

Summary

This paper describes the salient features of the engineering approach adopted for the design of City Life Hadid and Libeskind towers.

The purpose is to show, with two concrete examples, the importance of the engineer's critical view of the structure not only to make a modern architecture feasible, but also to understand and to improve it, before proceeding with a detailed analysis.

Keywords

High-rise buildings - form architecture - design

Theme

Modern high-rise buildings structural design

Introduction

In the near future is planed the realization of the three towers in the new district of City Life, located in the north zone, this building were designed by architects Arata Isozaky (H = 220m) Zaha Hadid (H = 185m) and Daniel Libeskind (H = 170m) (see Fig. 3.1), while Redesco Progetti srl (a structural design and consulting firm based in Milano) has played and is playing activities of structural design, in different fields.

The above said building, although with no size of particular importance, especially when compared to the Burj Kaliffa of Dubai (828m) or the Freedom Tower in New York City (540m), still represent an attractive challenge for structural engineering, because the modern architecture which is very close to Sculpture.

The form in "twist" that characterizes the Hadid tower and the "Ball" of the Libeskind tower are on one hand, unique, attractive and of strong emotional impact, but on the other one set questions of phenomenological realization of which (Designers, Builders etc.) have to solve in an operational way.

Tower	Redeso Progetti Srl activities	Client
Isozaky (TcA)	Assistance to contract phase	CMB
Hadid (TcB)	Design for contract	City Life
Libeskind (TcC)	Preliminary design and Design for contract	City Life

Faced with this challenge, often the attitude of a structural engineer, supported by the availability of advanced procedures and means of simulation and calculation, the state of knowledge, state of the art of building and, not least, the economic pressure, tends to be devalued by the simple statement: "We make it stand on anyway."

Therefore the role of the structural engineer must be understood in its deepest sense "of practical and creative contribution, within the complex dynamics of the relationship between contemporary architecture and Structural Phenomenon, subject to predefined constraints."

This role is expressed in the critical view of the structure as a whole, the result of the sensitivity of the designer, leading to the identification of the resistant main systems, the bound-

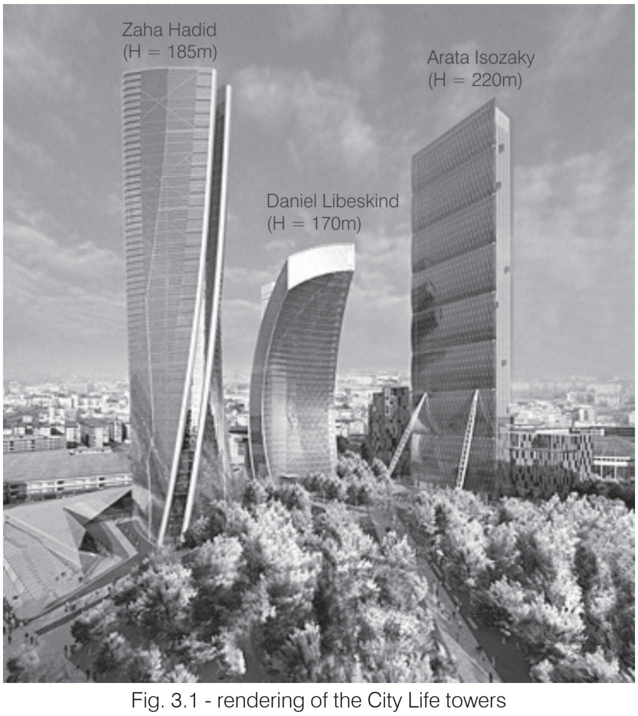


Fig. 3.1 - rendering of the City Life towers

ary conditions, the areas of singularity / problems and to assess the possible types of structural improvements.

The development of the structural projects of the Hadid and Libeskind towers had to deal with the following matters :

- identify the main structure and the key parameters that govern the behavior,
- select of strategy design and analysis to be adopted,
- check the results with a simple handmade calculation.

1.Hadid Tower

Client : City Life SpA
Architect : Zaha Hadid Architects London
Structural engineer : Redesco Progetti Srl - Milano (consultant MC2 - Madrid)
Start of construction : 2012 (expected)
General Data
height above ground: 170.0 m
height above foundation: 185.8 m
N° floors above ground : 43
N° basements : 4
Overall average floor area: 46 x 48 m
Overall average core area : 24 x 19m

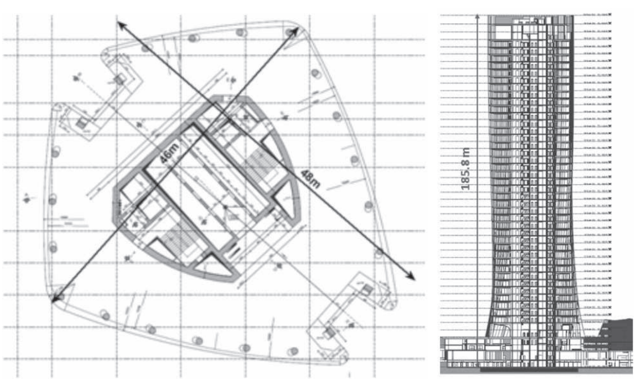


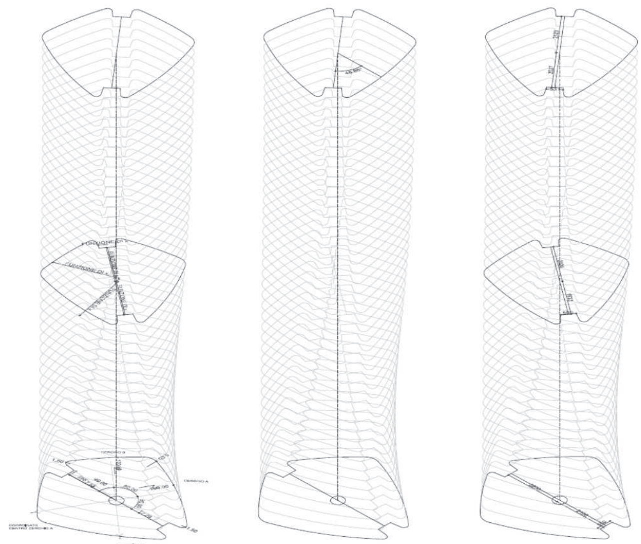
Fig. 4.1 - Hadid tower general dat

In the case of Hadid Tower early on it became clear that the main peculiarity of the structure was the effect of the torque due to the arrangement of the columns.

As a matter of fact, the columns feature a 3D angular variations on each floor (decreasing from the base toward the top) according to a rule (scaling of the dimensions of the floors and rotation around the center, according to polynomial equations defined by Architecture Fig. 4.2).

Because the columns are all inclined in the same direction, for the effect of only gravity loads, the structure is subjected to an internal self balanced torsion. Starting from the top, the columns transmit a torque to the core in correspondence of each floor up to the level 0, under which the pattern is vertical and the core is no longer subject to torsion.

This behavior has an impact not only on the dimensioning of the vertical elements (core and columns), but also on the horizontal ones (decks), because the transfer of torque from the external circuit (columns) to the core occurs through the diffusion of membrane actions in the floor slabs.



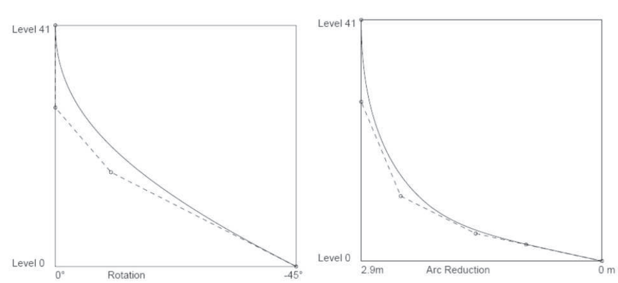


Fig. 4.2 - Architectural rule for the variations on each floor

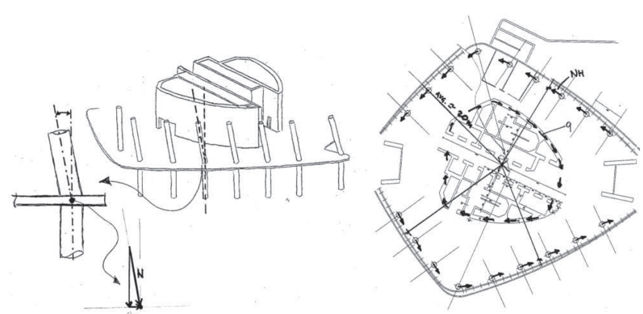


Fig. 4.3 - Sketch of load path

These considerations are illustrated in the scheme shown in Fig. 4.3, built at the beginning of our design, which not only made possible to estimate, with accuracy, the extent of the phenomenon in advance of the finite element modeling, has also led the design choices and listed in the following.

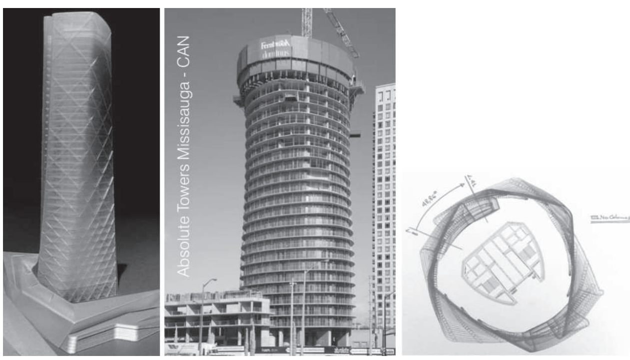
After having detected the singularity of the structures, first of all it was investigated whether it was possible to carry out a structural plan that could eliminate this phenomenon as:

1. The realization of a system of diagrid facade
2. The realization of vertical columns,

Unfortunately, both options were not feasible.

- In the first case because the cost, the integration with the facade, the shape of the plant with the presence of the "cut" at the ends, the architecture and the fact that there was still need to position of the columns within the building,

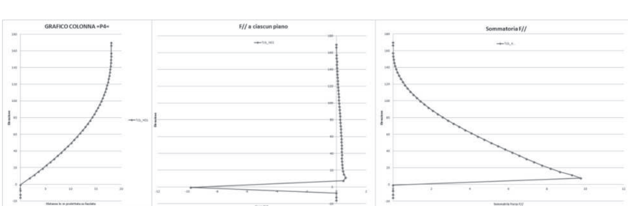
- In the second one because the columns could be positioned only within an area common to all levels that it was incompatible with the office layout of the floors and for the slab cantilever around to 7 m.



In conclusion, to meet the architectural requirements and the use of space, columns must necessarily be placed on the perimeter of the floor and then follow the rotation set by the architectural design, the torque had to be insisted.

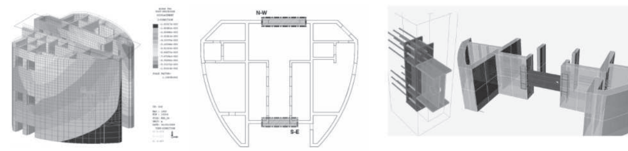
The torque has some structural implications which lead to structural :

- the limitation of the inclination of the columns, specially in correspondence with level 0, under which they assume a vertical position, because this parameter governs the value of the maximum torque in the core, independently on the pattern of columns in the floors above.

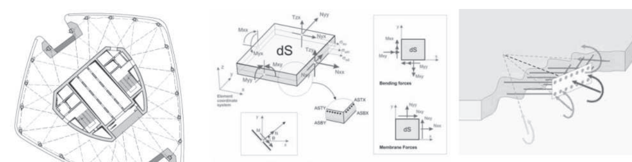


- the distortion to which are subject the connecting elements of the two core portions, for the warping effect. In fact the two halves of the core, only for the vertical loads, tend to deflect in opposite directions, this activating on a Vierendeel effect the coupling beam

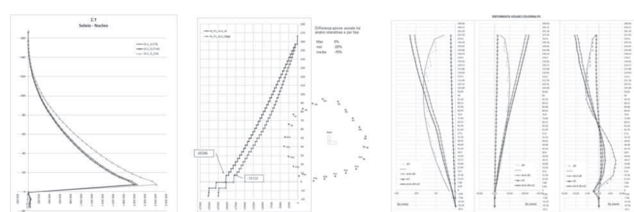
- the membrane behavior of floor slab, necessary for the transmission of torsion from the external circuit (constituted by the columns) to the core, is activated by the vertical loads. This aspect, combined with the peculiar floor plant, and operational considerations, led our firm to the use of massive (32cm th.) slab in r.c. The element design was implemented a special algorithm which, by using the results of finite element analysis, obtained the design of the reinforcement and verification taking into account the combined membrane and bending force.



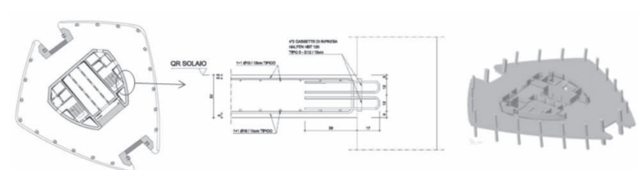
- The effect due to the phases of construction and to the rheological phenomena for the redistribution of actions and the displacement evolution, which in this case is not limited to the vertical component, but also involves the horizontal one.



- The choice of the type of connection between the different structural types (Core, Lintels, Slabs, Columns, Walls). In order to direct the load path through a resistant, well-defined and predetermined, system, which has to be as much as possible independent from the hyperstatic type of construction, was considered, and the following decisions were taken

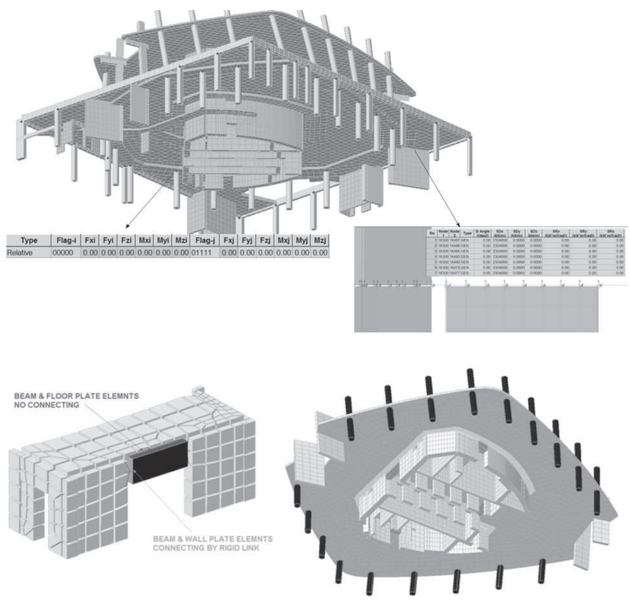


- to introduce a constraint between the floor and the core that transmits only membrane actions (hinge), so as to reduce the redistribution of internal actions of the floors depending from the effective stiffness of this detail (partialization of r.c. section) and from the column-shortening (typical of tall buildings)



- to design the core without the resistance effect against the rotation offered by the vertical elements of the underground

- to verify the lintels without the resistance effect due to the floor slab



At this point it is evident that the particular geometry of the tower involves a structural system in which the interaction between the main elements, such as core, columns, slabs and foundation, is fundamental.

For example it is not possible to extrapolate an item as the floor and analyze its functionality against vertical actions apart from the effects of the remaining part of the structure.

So it is necessary to develop a "complete immersion model", in which all structural parts are discretized with the same degree of precision, which allows for the study of all its components.

In addition a correct evaluation of the actions has to take into account the phases of construction, the rheological behavior of concrete and different kinematic conditions.

In order to "quickly" investigate the behavior changes for different configurations and provide information in support of the Architectural variants (consider that in the final phase the Architects produced 9 different versions of the tower) to overcome the computational burden due to combination of complete immersion model with the evolving and dynamic analysis (with analysis times of the order of 10/15 hours), different calculation models, in parametric form, relevant to the subject of investigations and check that is intended to be carried out, have been developed (Fig. 4.4).

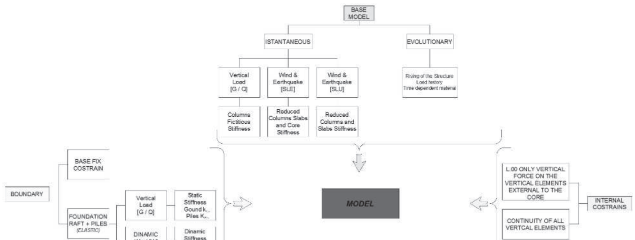


Fig. 4.4 - Tower model flow chart

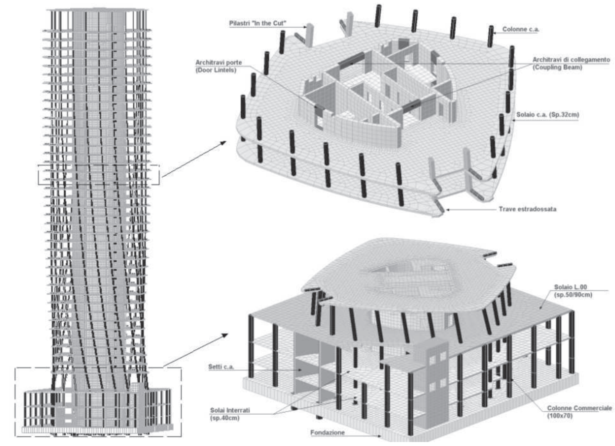


Fig. 4.5 - Complete immersion model

The common characteristics of each model are

Model	Without foundation	With foundation
Node	127 606	130 701
Beam	3 734	3 881
Plate	130 502	133 858

Instantaneous model(I)

Models with fictitious stiffness for resistance checks in which the axial stiffness of the columns was amplified to take into account, in a simplified manner, the execution phases.

A.Model with fictitious stiffness for resistance tests in which the axial stiffness of the columns was amplified to account for the execution phases

This approximation was introduced at a preliminary stage by the adopting the criterion for which the shortening of the core and of the columns in correspondence of 40th level was similar for obtaining a first estimate of the amplification coefficient of the axial stiffness of the columns (?).

The value obtained was taken on the basis of a preliminary instantaneous model (with the area of the columns amplified A'C = ? Ac) and the relevant results, in terms of axial force in the columns and torsion in the core, were compared with the analogous model associated to an evolutionary analysis.

In this way a refined estimation of the coefficient ? with a finally value of ? = 6, was obtained.

Such modeling is conservative for the columns and for the torsional effect induced by gravitational loads on the core

B.Model used for the ULS verification against wind and earthquake, in with the area of the columns not amplified, it reduces the bending and shear stiffness of floor beams and columns by 30% and is not changed that of the core. This scheme is conservative for the verification of the core that is the primary element for resisting the horizontal actions

C.Model used for the verification of deformations due to horizontal loads of wind and earthquake in SLS, which are not influenced by the load history; it is different from model B because, conservatively, also the rigidity of the core was reduced by 30%.

Two types of these models were developed

I. This model is necessary for the verification of the Core, in which the vertical elements, external to the core below the level 0 (Q. +129 m), transfer the vertical load only. This ensures that the main torque and the horizontal actions are resisted by the core without considering the contribution of secondary elements.

II. In this second case we consider the continuity of all the vertical elements in correspondence with level 0. This case is used for the verification of columns, floors, foundation and for the evaluation of the deformations.

Finally the effect of the foundation was introduced and additional 6 models, similar to the previous one, were created with the base nodes connected to the foundation raft, the relevant nodes are restrained, in the vertical direction, by elastic springs with stiffness depending on the ground characteristics and on the presence of the piles. This stiffness is differentiated in the static analysis (gravitational) and in the dynamic (seismic and wind).

Evolutionary model (E)

A. An evolutionary model in which the construction phases are simulated in steps of 4 floors at a time, including the evolution of the loads, the rising of the structure and the characteristics change over time of the materials. The results are used

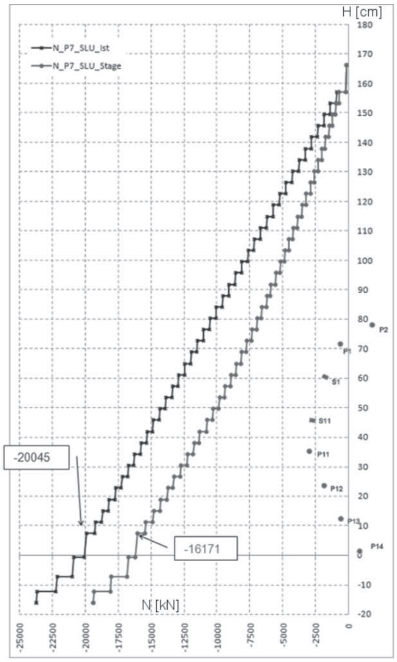


Fig. 4.6 - Column axial force due to gravity loads comparison of instantaneous and evolutionary model

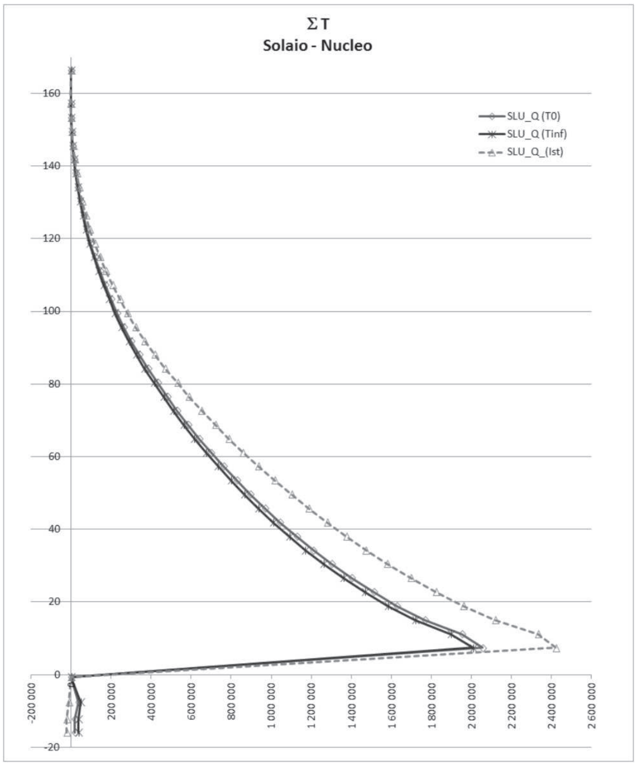


Fig. 4.7 - Core torque due to gravity loads comparison of instantaneous and evolutionary model

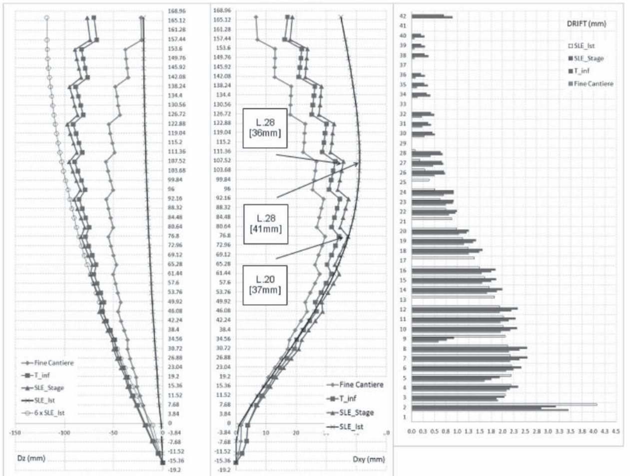


Fig. 4.8 - Deformation due to gravity loads comparison between the instantaneous and the evolutionary model

a. to compare the results (quality and quantity) with the ones obtained from the instantaneous model, and so confirm the assumptions made (Fig. 4.6 Fig. 4.7)

b. for the evaluation of the deformation induced by gravitational loads (Fig. 4.8)

Comments

In the structural design of the Hadid Tower, with its twisted shape, the overall context of the structural behavior should not be lost

- The tower is characterized by a core, with a pipe shape, of large overall dimensions imposed by the needs of vertical communication;
- This core was deliberately developed with large structural sections in order to get a very high intrinsic stiffness for the bending, shear and torsional actions so the stress states results limited;
- As a consequence, all the parameters of deformation and dynamic response are, not only below the limits of the recommendations, but in general are very limited if compared to any building of similar proportions;
- The deformations, both instantaneous and at long-term, are NOT a problem.

Libeskind Tower

Horizontal deformation	d_{xy} mm	H/d_{xy}	d_i ‰	h_i/d_i
Vertical Laod (T_0) "End of construction "	56.3	*Level 22 (H' = 100m)* **H'/1776**	**0.91**	1 097
Vertical Laod (T_{50}) "from end of construction to 50 years"	113.2	*Level 41 (H = 173m)* **H/1529**	**0.86**	1 164
Wind (Tr 50 years)	154.8	*Level 41 (H = 173m)* **H/1118**	**1.04**	960
Earthquake (SLD)	79.6	*Level 41 (H = 173m)* **H/2174**	**0.57**	1 745

Horizontal deformation

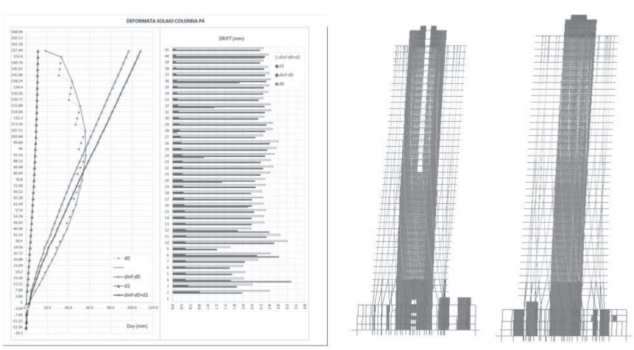


Fig. 4.9 - Horizontal deformation due to vertical and horizontal load

Mode	T (sec)	Type
1	5.074	Bend
2	4.800	Bend
3	1.410	Tor

Dynamic response

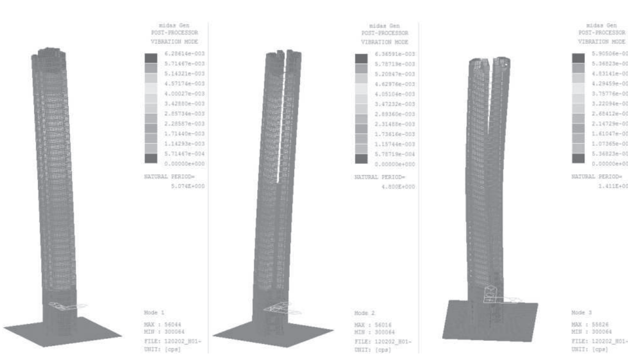


Fig. 4.10 - Modal analysis results

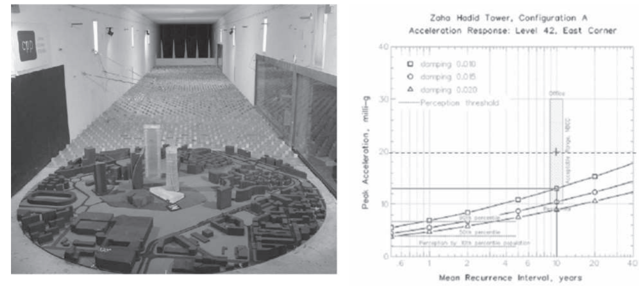
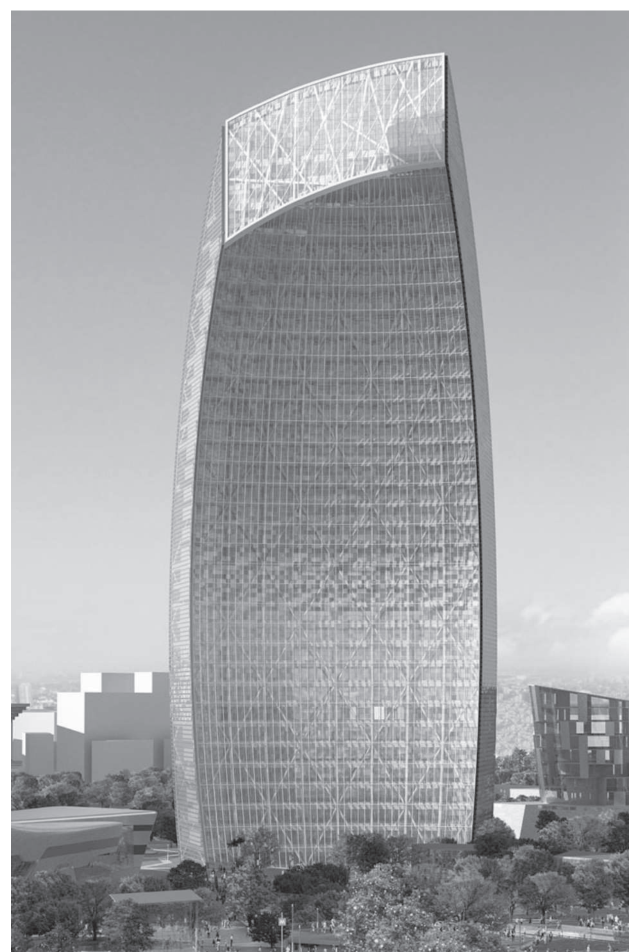


Fig. 4.11 - Wind tunnel test results

Client : City Life SpA
Architect : Daniel Libeskind New York
Structural eng. : Redesco Progetti Srl - Milano
Project status : Preliminary
General Data
height above ground: 154.5 m
height above foundation: 170.5 m
N° floors above ground : 32
N° basements : 3



A second example of "Critical analysis of the structures" is the preliminary study of the tower Libeskind.

Since Redesco Projects Srl is currently developing the project, in the following are illustrated some of the preliminary evaluations, in a schematic form.

The shape of the tower, obtained by the intersection of two spheres with 1 horizontal and 3 vertical plane (according to the diagram shown in the Fig. 5.1), is completed by a r.c. core in an eccentric back position.

This architectural choice yields in a spatial behavior of the resistant structure form; the effect of the vertical load only activates the membrane behavior of the floors (Fig. 5.2) and the rotation around the vertical axis(Fig. 5.3).

For this reason a bracing structure located in correspondence of the short facades is necessary to reduce the torques effects created in the core (Fig. 5.4).

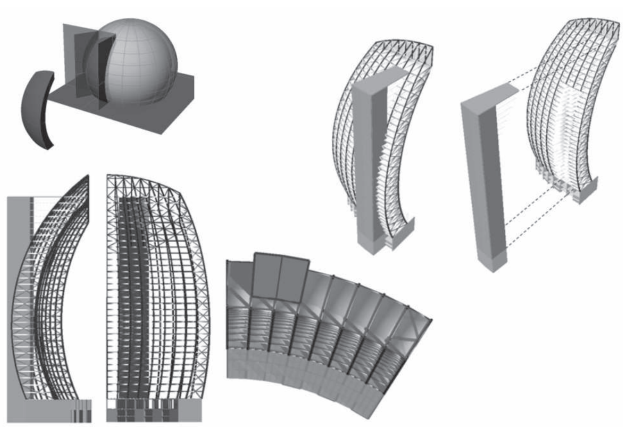


Fig. 5.1 - Structural scheme and shape generation of tower Libeskind.

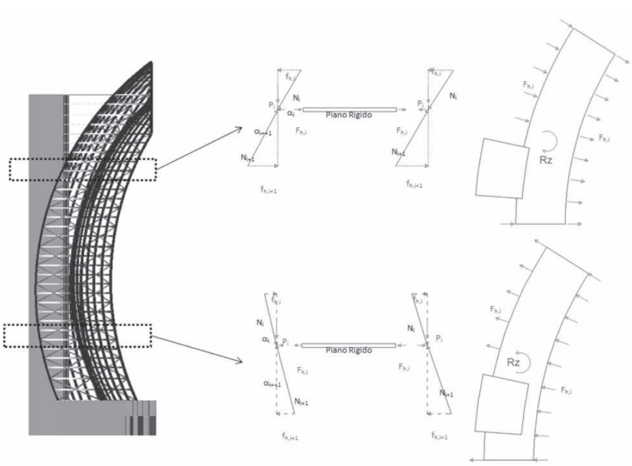


Fig. 5.2 - Sketch of horizontal actions due to vertical loads

As a conclusion, the spatial behavior of the tower, due to its geometry, required the incremental study of structure even in the early stages of design.

If we consider the typical behavior of a regular tower, the horizontal deformation obtained with a elastic instant model is conservative compared to the results get by an evolutionary model (the upper levels are not affected by what happened in advance of their construction).

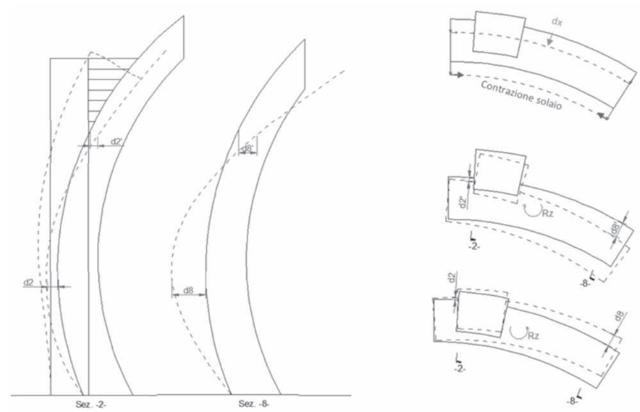


Fig. 5.3 - Sketch of horizontal deformations due to vertical loads

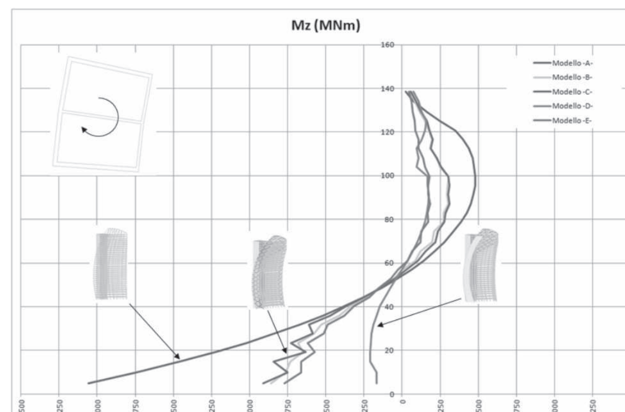
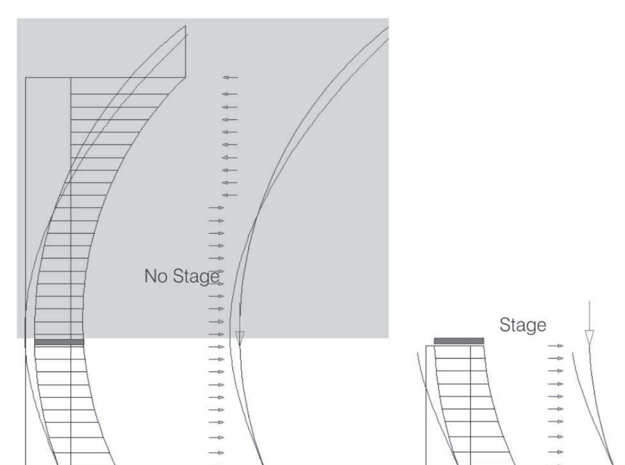


Fig. 5.4 - Core torsion due to vertical loads

This does not happen in the case of the tower Libeskind, since, generating the entire structure at the same instant of time, the effect of the vertical load of each level is redistributed by exploiting the rigidity of the structure above, which in reality has not been constructed yet (Fig. 5.5).

Once the main behavior of the structure has been identified, a number of parametric models is developed in order to identify the most efficient structural configuration and a sensitivity analysis is performed.



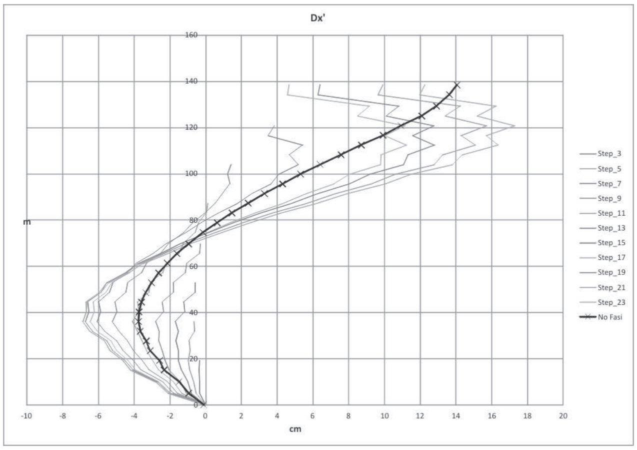
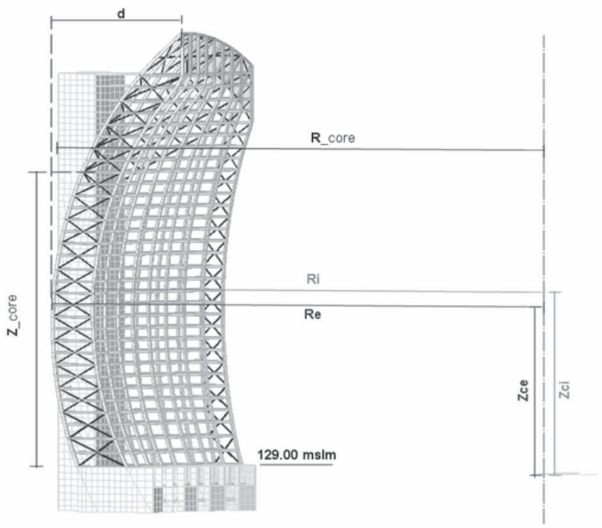


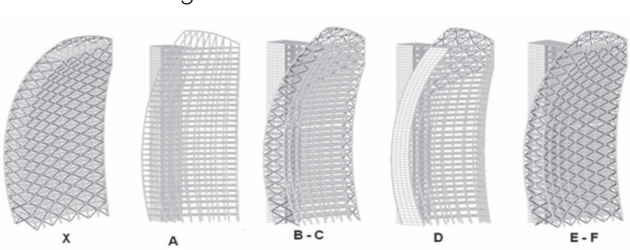
Fig. 5.5 - Comparison of the horizontal deformation with and without incremental analysis



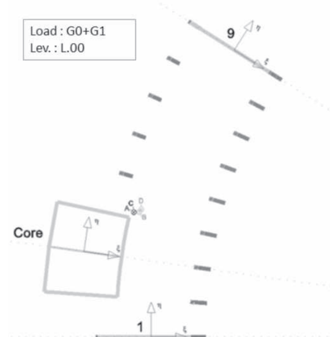
The parameters that govern the geometry of the tower are shown schematically in the following graph.

Structural configurations that were investigated are:

X. Only metal structure with bracing on all sides, but no core
A. Tower with core without bracing facade
B. Tower with core and metal braces only on the short fronts and on the top of the outer sphere
C. The same of B, but with braced profile optimized
D. Lateral bracing made with r.c. walls th.75cm
E. The same of B, but with also bracing on long facades with not connected intersection
F. The same of E, but with long facades bracing connected node crossing



Core and bracing reactions on Lev.00 for the main model type are shown in Fig. 5.6



Modello

F_ξ [kN]	A	B	C	D	E
1	-8 937	-10 581	-9 033	-1 854	-8 791
9	-9 203	10 477	10 077	23 790	7 483
Core	87 594	66 907	65 754	41 400	65 761

Modello

F_η [kN]	A	B	C	D	E
1	0	0	0	-219	63
9	2	5	4	-38	-1 525
Core	-16 124	-6 632	-7 031	-980	-1 767

Modello

Fz [kN]	A	B	C	D	E
1	19 557	19 625	18 745	73 148	15 563
9	20 333	25 428	23 868	74 911	30 945
Core	240 868	251 734	251 183	239 304	266 579

Modello

CORE	A	B	C	D	E
T [MNm]	-2 058	-859	-909	-168	-770
M_η [MNm]	-1 878	-1 683	-1 687	-1 551	-1 701
M_ξ [MNm]	325	412	428	289	543

Fig. 5.6 - Core and bracing reactions on Lev.00

The first two configurations (=X= and =A=) have been developed as border cases respectively of deformation and torsion in the core (Fig. 5.7)

The comparison between models =E= and =B= demonstrate that the bracing on the long sides were not necessary (Fig. 5.8 and Fig. 5.9),this introducing a substantial structural simplification, with consequent savings in cost and time of realization.

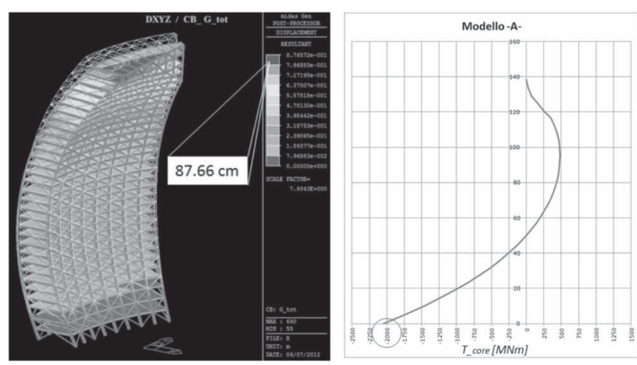


Fig. 5.7 - a) Global displacements for the model type =X= / b) Core torsion for the model type =A=

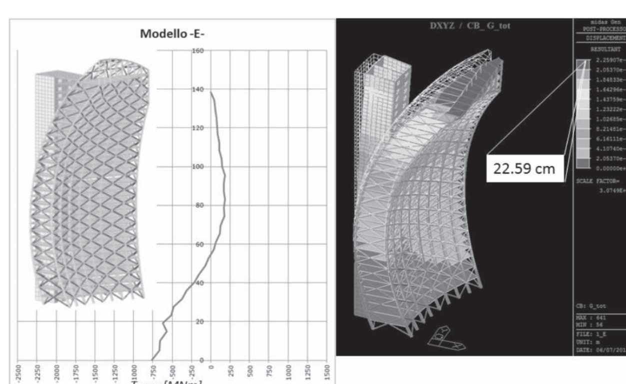


Fig. 5.8 - Model =E= core torsion and global displacements

Fig. 5.9 - Model =B= core torsion and global displacements

The studies carried out highlighted the main features that

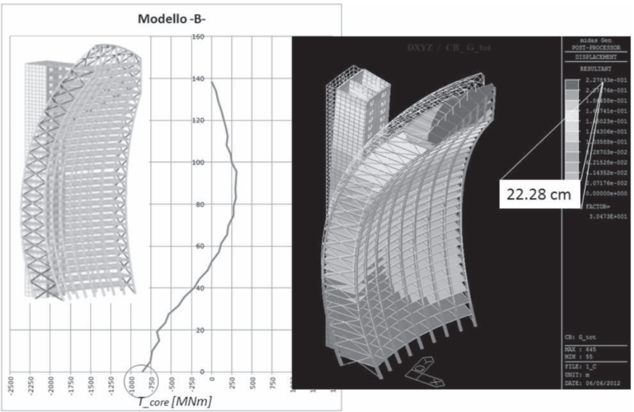


Fig. 5.9 - Model =B= core torsion and global displacements

govern the structural behavior of the tower, with its criticality.

This information is a sort of "guide" for the architects who had to make a number of changes, for aesthetic and commercial reason, so they can adopt the most efficient solutions also in reference to the structural aspects.

Conclusion

The experience exposed aims to show how the structural engineer should perform the up to date design, using available technologies, but without losing its essential role that is to make his contribution in the development of an idea to make it feasible.

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New High Rise Buildings in Milano

Gian Carlo Giuliani

A number of high rise buildings are being erected or designed in Italy and were illustrated during the first congress of the SEWC - Italian Group which was held in Milano on July 3rd, 2012.

Although no one of these constructions breaks any height record, interesting structural problems were to be solved because of some peculiar features either generated by the architectural shape (Hadid and Liebeskind towers) and by large floor overhanging (Solaria tower) and core subdivision (Diamante tower), all located in the city of Milano.

The towers have a concrete core and external columns in concrete or steel; the differential support shortening effect had to be and the study of a compensation of the deflections during the construction had to be implemented.

The construction procedure selected by the contractors called for several special incremental structural analyses also.

In the following papers, Fabio Capsoni and Paolo de Angelis, clearly illustrate the above said problems and the strategy used for finding the proper solution; all the works were carried out within our structural consulting firm,

Hadid tower

Solaria tower

Diamante Tower

Redesco Progetti srl, under the supervision of its General Manager, Mauro Eugenio Giuliani.

Author Affiliation

Gian Carlo Giuliani

President SEWC - Italian Group

MIT Architect Plans to Print Her Own Buildings

3-D Printing is probably one of the most exciting technologies emerging right now. As it becomes more accessible, it will truly revolutionize manufacturing and materials science. Neri Oxman, an architect and professor at the MIT Media Lab, is working on one such set of research using 3-D printing technology for buildings.

Existing 3-D printers, also called rapid prototyping machines, build structures layer by layer. So far these machines have been used mainly to make detailed plastic models based on computer designs. But as such printers improve and become capable of using more durable materials, including metals, they've become a potentially interesting way to make working products. Oxman is working to extend the capabilities of these machines making it possible to change the elasticity of a polymer or the porosity of concrete as it's printed, for example and mounting print heads on flexible robot arms that have greater freedom of movement than current printers.

She's also drawing inspiration from nature to develop new design strategies that take advantage of these capabilities. For example, the density of wood in a palm tree trunk varies, depending on the load it must support. The densest wood is on the outside, where bending stress is the greatest, while the center is porous and weighs less. Oxman estimates that making concrete columns this waywith low-density porous concrete in the centercould reduce the amount of concrete needed by more than 10 percent, a significant savings on the scale of a construction project.

International Code Council, Delmar Announce Series of Books to Help Learn the 2012 International Codes

INTERNATIONAL CODE COUNCIL®

People Helping People Build a Safer World™

The International Code Council (ICC), publisher of the family of International Codes, and Delmar, part of Cengage Learning and a leading provider of learning solutions for ongoing career development and education, announce the Code Basics series, a new series of books that break down the language within the 2012 I-Codes and makes the content more user friendly for building professionals.

The series consists of four titles focusing on the new commercial, green, residential and energy codes recently released by the ICC. With two currently available, and two to be released by the end of 2012, these new books will feature illustrations highlighting key updates and changes in the 2012 codes, practical on-the-job scenarios for real-world applications and straight-forward explanations. These features help to facilitate the learning of technical terms and clarify the meaning behind important vocabulary.

For more information about the Code Basics series or any other building construction products, please visit www.icc-safe.org or www.informationdestination.co

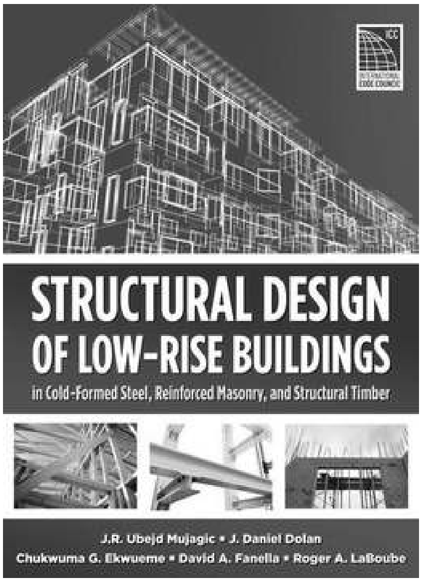
A House Set Within a Concrete "Flower"

Set in the Tokyo suburb of Setagaya, 'Breeze' is a series of terrace-style apartments set within enormous poured-concrete petals. These petals act to protect the units from northern views and light, while creating a buffer zone that incorporates second-floor access. The units open up on the southern façade with a terraced garden and city views, while bedrooms within are graced by light courts that provide privacy and a quiet environment.

Tokyo based firm Artechnic has built quite a name for itself with daring uses of concrete in residential projects, which Breeze quite obviously reinforces. Though the plasticity of form is remarkable in its own right, the architects seem quite gifted in the use of light as a material, molding it into forms as substantial as the building itself.

Courtesy: Architizer / Images by : Nacasa & Partners

Textbook for Innovative Low-rise Building Design



A new textbook that includes cold-formed steel as one of three innovative design techniques for low-rise buildings will bridge an information gap that currently exists in many undergraduate engineering curriculums. The publication, "Structural Design of Low-Rise Buildings in Cold-Formed Steel, Reinforced Masonry, and Structural Timber," was written for civil and structural engineering undergraduate students, practicing engineers, and candidates for structural engineering licensing examinations. It provides state-of-the-art information on these three innovative design techniques, which are often downplayed or ignored in college curriculums. The textbook was published by McGraw-Hill.

Site-Work on Miami's Tallest Tower has Begun

Site-work has officially begun for the stalled 74-story 1101 Brickell residential tower in Downtown Miami. Upon completion, the project will become Miami's tallest building, and will reportedly cost approx $500 to $600 million.

The building initially proposed as an 80-story structure has been a non-starter for years now due to the recession. Recently, its developer Tibor Hollo announced that 1101 Brickell would resume planning and begin construction.

Building and Construction Industry Leaders Announce Formation of the American High-performance Buildings Coalition

More than 27 leading associations representing a wide range of interests in the building and construction industry announced the formation of the American High-Performance Buildings Coalition (AHPBC) (www.betterbuildingstandards.com). These organizations have come together to promote and support the development of sustainable building standards, which are based on consensus and scientific performance data.

Innovative Engineering Overcomes Seismic, Structural Challenges of BIG's Vancouver Tower

Multi-disciplinary engineering firm Buro Happold announced that it is providing structural engineering services for the Beach and Howe mixed-use tower in Vancouver; Buro Happold is design engineer, working in collaboration with local engineer of record, Glotman Simpson. The structure meets the challenge of stabilizing a tall building whose mass is at its top and making it safe in a high seismic zone. The 49-story building, designed by BIG-Bjarke Ingels Group, combines 653,890-sq.-ft. of residential, retail, and commercial space in an urban complex at the entrance to the Granville Street Bridge.

Typically, the mass of a building is at its base. In response to the constricted urban site, the mass of the Beach and Howe tower is inverted. The tower's small triangular base curves away from the bridge to allow light and air to enter lower apartments. As it rises, the building's shape transforms into larger, rectangular floorplates that culminate in a square top. Buro Happold designed a concrete core with post-tensioned walls, which can protect against damage in case of an earthquake and also improve performance.

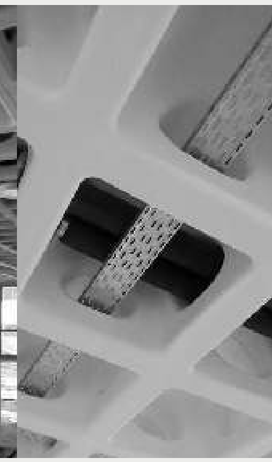
This creative solution meets the tower's structural and seismic requirements.

Concrete Waffle Slab Reduces Costs

This new structure type, Holedeck, allows for a cheaper and more sustainable way to build. It saves about 40cm in height every floor and as a result it also makes savings on the quantities needed for facades and partition walls. It is especially suitable for sustainable facilities such as solar energy.

Alarcon+Asociados, a Spanish architecture office, have developed and patented this new structure internationally. It eliminates the need for ceilings as it is not necessary to hang air conditioning, lighting, electricity or water pipes.

For more information consult the website: www.holedeck.com.



New York Plans Its Densest Development Yet

New York-based Related Companies has hatched a redevelopment plan for Hudson Yards, the city's largest piece of undeveloped land. The 26-acre site will receive an $800 million concrete roof, and include 12-million-square-feet of offices, shops, movie theatres, gyms, hotel rooms, museum galleries and 5,000 apartment units.

In the first $6 billion phase scheduled for completion by late 2017the tallest tower will top the Empire State Building, and even the shortest will have a penthouse on the 75th floor.

Designers enlisted in the project include William Pedersen of, Kohn Pedersen Fox Associates; David Childs, Skidmore Owings and Merrill; Elizabeth Diller, Diller Scofidio + Renfro; David Rockwell, Rockwell Group; Howard Elkus, Elkus Manfredi; and Thomas Woltz of Nelson Byrd Woltz.

Courtesy: New York Times

Students Create a Pre-Fabricated Skyscraper for Beijing

Mock firm 'ANDO | Andalucia Office' explores the concept of "mutant vertical cities" in Beijing with its NODO project. With this prototypical tower, the students studied the possibility of locating 'mini-cities' within massive prefabricated skyscrapers.

The design contains a singular prefabricated unit, repeated for 550 meters (1,804 feet), resulting in 150 floors containing infrastructure, housing, offices and commercial and public spaces. Some elements of the building, such as the vertical communication and facilities core, would be permanent but others would be mobile and could be located where necessary. Prefab modules for each of these elements would center on an interior hollow reinforced concrete core.China Broad Group is also exploring the idea of prefabricating skyscrapers in Asia, and plan to employ their systems in the world's future tallest building, 838 meters (2,749 feet) tall Sky City.

Courtesy: Ctbuh.org

Saudi BAUER Foundation Signs Deal to Enable the One-Kilometer-Tall Tower in Jeddah

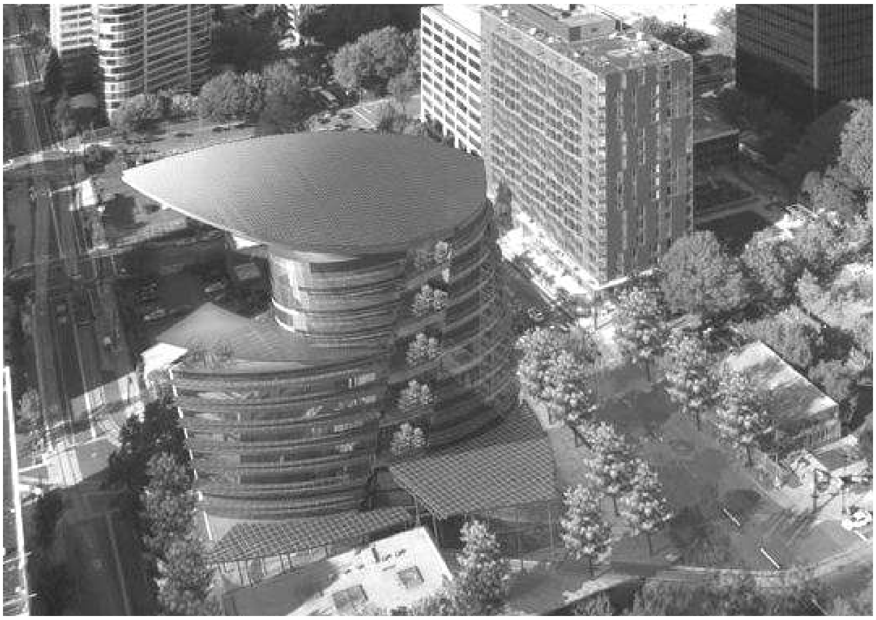
Although there is still no official word on when construction will start on Kingdom Tower, there are signs of progress. Saudi BAUER Foundation Contractors Ltd. this week announced it had signed a deal for the enabling works for Kingdom, the one-kilometer-tall tower planned for Jeddah, Saudi Arabia.

"The enabling works include 270 bored piles of 1.5 and 1.8 meter (5 and 6 feet) diameters, which have to be installed into the difficult ground conditions down to depths of 110 meters (361 feet) in order to provide the building's foundations," the company said in a press release.

Bauer also did the piling work for Burj Khalifa, currently the world's tallest building. The foundation work on Kingdom is scheduled to begin before the end of December and take about 10 months, the company said. The contract for the work is worth approximately EUR25 million, the company said.

Courtesy: Saudi BAURER

Oregon Abandons Sustainable Skyscraper Dream



Portland is abandoning its plan to build a sustainable skyscraper, according to a statement from Mayor Sam Adams. The Oregon Sustainability Center lost its impetus after a series of rejections and reductions in the scale of the project. The originally proposed Sustainability Center was supposed to be a 13-story perfectly energy efficient "living building," with a price tag of $90 million dollars, according to local media. The design called for the building to produce 100 percent of its needed energy renewably and on-site.

A scaled back, 7-story version of the building resurfaced last month, reducing costs by millions of dollars and increasing the private backing. Despite this amendment, the Mayor has cancelled the project entirely.

"The proposed Oregon Sustainability Center was set to be a laboratory for green technology," Mayor Adams told local press in a statement. "Nonetheless, it's become clear to me this week that I don't have the votes necessary to pass the project through Portland City Council, and we won't be moving forward."

Courtesy: Oregonlive

USD 123 Million Contract for Arabtec

Arabtec has received an USD 23 million contract to build Europe's future tallest tower, the 463-meter (1,520-foot) Lakhta Center. The long-delayed Gazprom project found new life when it was relocated to a more acceptable site after initial opposition. Arabtec has been working on the project since 2008, when Gazprom awarded it a US$2.7 billion contract to build the tower and the surrounding complex.

Havvada Island - A Home for a New Community

A century after the Republic of Turkey was proclaimed; Prime Minister Recep Tayyip Erdogan presents the Canal Istanbul project and offers to re-visit the map of the city. One billion cubic meter of soil may be carved out of the main land in order to create the canal. Turkish developer, Mister Serdar Inan, proposes to reconstitute the soil to create an island off the shore of Istanbul and a home for a new community.

A green island made of 6 hills of different sizes circling the downtown center of the land. Each hill up rises on top of a mega structural sphere that supports the residences on the hillsides and a community life at the center. Each hill is designed as a mega-dome structure

based on engineering and structural principals studied and implemented in architecture since ancient times and further developed as a geodesic dome in the last century by Buckminster Fuller. The design proposed for most of the island relies on compression and tensional integrity; it maximizes the material utilized to build the structure and infra-structure of the Island.

The urban planning optimizes the slopes of the hills and their panoramic view on each side for greater residential areas. Traditionally, communities have built their residential areas around the center of the town where political decisions or trade was made, and spiritual and religious temples were built - the community would grow around those centers with extended skirts of residence buildings and neighborhoods. The center valley of the Island offers ideal space and planning for parks and recreation centers. The buildings are covered with green living-roof and fade in the natural-type of environment in an organic way, while contributing to the constant energy recycling of the Island.

FINALLY! First Pics from the Ryugyong: North Korea's Skyscraper We Thought They'd Never Finish

Finally, after decades of waiting and endless speculation, North Korean authorities have opened the doors to the now mythical Ryugyong hotel, giving the world its first peek inside. These exclusive pictures, reveal the as-yet-unfinished central dining room that's so huge it dwarfs all that stand inside it.

Construction on the Ryugyong began in 1987 and was due to be completed two years later, however delays and mis-management prevented Kim Il Sung's dream of building the tallest building in the world from being realised. Building work halted in 1992 and for nearly two decades it remained a dormant triangle on Pyongyang's skyline, treated by many as a symbol of North Korea's economic failure.

Work resumed in 2008 after heavy investment from Egypt's Orascom group, who are also responsible and heavily invested in North Korea's mobile telephone industry. It has been previously reported that Orascom have installed a powerful telephone ariel in the top of the Ryugyong. Though the building's interior is yet to be completed, North Korean officials claim it will eventually contain the country's premier restaurants, hotel accommodation, apartments, and business facilities.

Hotel view from a distance

Picture from outside the hotel

Inside the hotel

Though the building's interior is yet to be completed, North Korean officials claim it will eventually contain the country's premier restaurants, hotel accommodation, apartments, and business facilities.

Looking towards the top

Courtesy: nknews

2012

ASCE 6th Congress on Forensic Engineering

31 October - 3 November, San Francisco, USA (Co-sponsored by IABSE)

www.asce.org/event

International Conference on Sustainable Development of Critical Infrastructure

15 November, Shanghai, China - IC-SDCI (Co-sponsored by IABSE)

www.wfeo.net

International Conference & Exhibition- "Implementation Challen-ges and Way Forward for Construc-tion and Infrastructure Sector during 12th Five Year Plan (2012-2017")

30th January to 01st February, 2012, India Habitat Centre, New Delhi

Construction Industry Development Council

Phone: 011-26489992, 26234770,

Email: jr@cidc.in, cidcvks@gmail.com

www.cidc.in

2013

40th IFAWPCA Convention

January 6th to 9th,2013
Gokulam Park Hotel &
Convention Centre, Kochi
Builders' Association of India Cochin Centre
Phone : 0484-2205928
E-mail : ifawpca2013@gmail.com

ISFGE 2013
Fourth International Seminar on Forensic Geotechnical Engineering

10 - 12 January, 2013, Bengaluru, India.
Email: isfge2013@gmail.com

Int. Conf. on Advances in SPIN 2013 Johannesburg, S. Africa Cement and Concrete Technology in Africa (ACCTA 2013)

28 - 30 January

www.spin.bam.de/en

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Seminars: February 4-8, 2013

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Las Vegas, Nevada USA

www.worldofconcrete.com

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February 5 to 8, 2013

MMRDA Grounds in the Bandra Kurla Complex, Mumbai, India

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Phone: +91 22 42554701

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International Conference on Environmental and Civil Engineering (ICECE 2013)

February 14-15, 2013

Renaissance Kuala Lumpur Hotel, Kualalumpur, Malaysia

www.waset.org/conferences/2013/kualalumpur/icece/registration.php

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UKIERI Concrete Congress Innovations in Concrete Construction

5 - 8 March, Dr. B R Ambedkar National Institute of Technology, Jalandhar (Punjab) India
Email: spsingh@nitj.ac.in, uccnitj@gmail.com

www.ukiericoncretecongress.com

[RE]Thinking Lightweight Structures - TensiNet

March 08 10, 2013, Istanbul, Turkey

www.iass-structures.org

Annual International Conference on Architecture and Civil Engineering (ACE 2013)

March 18 19, 2013, Singapore

Global Science and Technology Forum

Theme: Infrastructure Design for Regions in Rapid Development, Infrastructure Design for New Vertical Cities, Infrastructure for new Cities and Landscapes, Design for Urban Infrastructure: Parks, Buildings, Streets and Systems.

www.ace-conference.org/index.html

8th International Conference on Fracture Mechanics of Concrete and Concrete Structures (FraMCoS-8)

March 24-28, 2013 in Ciudad Real, Spain
E-mail: congress.framcos8@uclm.es
Tel: +34 926 295 300 Ext. 6372
www.framcos8.org

Fib Symposium: Engineering a Concrete Future: Technology, Modeling and Construction

20 - 24 April, Group Israel, Tel-Aviv, Israel

www.fib2013tel-aviv.co.il

IABSE Conference Assessment, Upgrading and Refurbishement of Infrastructures

6 - 8 May, Rotterdam,
The Netherlands IABSE

www.iabse.org

1st International Conference on Concrete Sustainability

27 - 29 May, JCI Tokyo, Japan
www.jci-iccs13.jp

International RILEM Conference on Multi-Scale Modeling and Characterization of Infrastructure Materials

June 10-12, 2013 in Stockholm, Sweden

Highway and Railway Engineering,

KTH Royal Institute of Technology
Brinnellvagen 23, Stockholm, Sweden
T :46 (0)8 790 87 17, F :46 (0)8 411 84 32
info@rilem2013.org
www.rilem2013.org

3rd Annual International Conference on Civil Engineering

Athens Institute for Education and Research
The aim of the conference is to bring together scholars, researchers and students from all areas of Civil Engineering and other related areas.
June 10-13, 2013, Athens, Greece
www.atiner.gr/civileng.htm

7th International Conference in Structural Engineering

June 18, 2013 to June 23, 2013
University of Hawaii at Manoa, HI USA
E-mail: frank.yazdani@ndsu.edu
Phone: (701) 231-7878
Fax: (701) 231-6185
www.isec-society.org/ISEC_07

ICSA 2013
2nd International Conference on Structures and Architecture 2013

24 - 26 July, Convention Centre, University of Minho Guimarães, Portugal
www.iass-structures.org

2nd International Conference on Civil Engineering and Architecture (ICCEA 2013)

August 10-11, 2013, Barcelona, Spain
IACSIT and International Journal of Engineering and Technology (IJET)
ICCEA 2013 E-mail: iccea@iacsit.net
ICCEA 2013 www.iccea.org

6th CECAR (Civil Engineering Conference in Asia Region)

August 20, 2013 to August 22, 2013
Hotel Borobudur, Jakarta, INDONESIA
E-mail: iswandi@si.itb.ac.id; info@cecar6.com
Phone: 62-22-2510715
Fax: 62-22-2512403

CC2013 The Fourteenth International Conference on Civil, Structural and Environmental Engineering Computing

September 3-6, 2013, T-Hotel,
Cagliari, Sardinia, Italy
Civil-Comp Ltd, Scotland (UK).
Tel: +44 (0)1786 870166
Fax: +44 (0)1786 870167
Email: registration@civil-comp.com

Transformables 2013

September 18 - 20, 2013, Seville, Spain
Emilio Pérez Piñero died fifty years ago. In his honor the First International Conference Transformables2013 is being organized with the aim of advancing in the research, design and building ideas on Transformability, Modularity and Lightness. Architects, Engineers, Artists, Academics
www.iass-structures.org

2013 IASS

The 2013 IASS Annual Symposium with the theme "Beyond the Limit of Man"

23 - 27 September, Wroclaw, Poland
www.iass-structures.org

26th IABSE Symposium:
Long Span Bridge and Roof Structures - Development, Design and Implementation

24 - 27 September, Kolkata, India
(Annual Meetings precede Symposium)
www.iabse.org

2nd International Symposium on UHPFRC

2 - 4 October, AFGC Marseille, France
www.afgc.asso.fr

The sixth conference on Textile Composites and Inflatable Structures (structural Membranes 2013)

October 09 - 11, 2013,
Technical University of Munich, Germany
www.iass-structures.org

ACI Fall 2013 Convention

October 20, 2013 to October 24, 2013
Hyatt Regency & Phoenix Convention Center, Phoeniz, AZ USA
E-mail: Renee.Lewis@concrete.org
Phone: (248) 848-3794
Fax: (248) 848-3701
www.aciconvention.org

SEWC 2013 Symposium
International Symposium on Innovative Architecture-Structure interaction towards Sustainable Development with Green environment

25-27 November, New Delhi.
Email: scmehrotra@mehroconsultants.com

1st R N Raikar Memorial International Conference

December 20, 2013 to December 21, 2013, Mumbai, India
Dr. Surendra K. Manjrekar
E-mail: info@sunandachemicals.com, skmanjrekar@gmail.com
Phone: 91-22-24034130
Fax: 91-22-24034133
www.ic-acirnraikarmemorialinternationalconference.com

2014

4th International fib Congress fib group India

10-14 February, Mumbai, India and Exhibition

5th International Conference on Concrete Repair

September 1, 2014 to September 3, 2014
Wellington Pk Hotel, Belfast, United Kingdom
E-mail: michael.grantham@concrete-solutions.info
Phone: 44-0-1843-606084
www.concrete-solutions.info

ANNOUNCEMENT

SEWC 2013 SYMPOSIUM

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18-20 November, 2013, New Delhi

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OCF Plot No.2, Pocket 9, Sector-B Vasant Kunj, Tel.: +90 11 26194486
Contact Point: IFP Qatar Ltd.
Fax: +91 11 46023053 Email: scmehrotra@mehroconsultants.com

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http://www.sewc-worldwide.org


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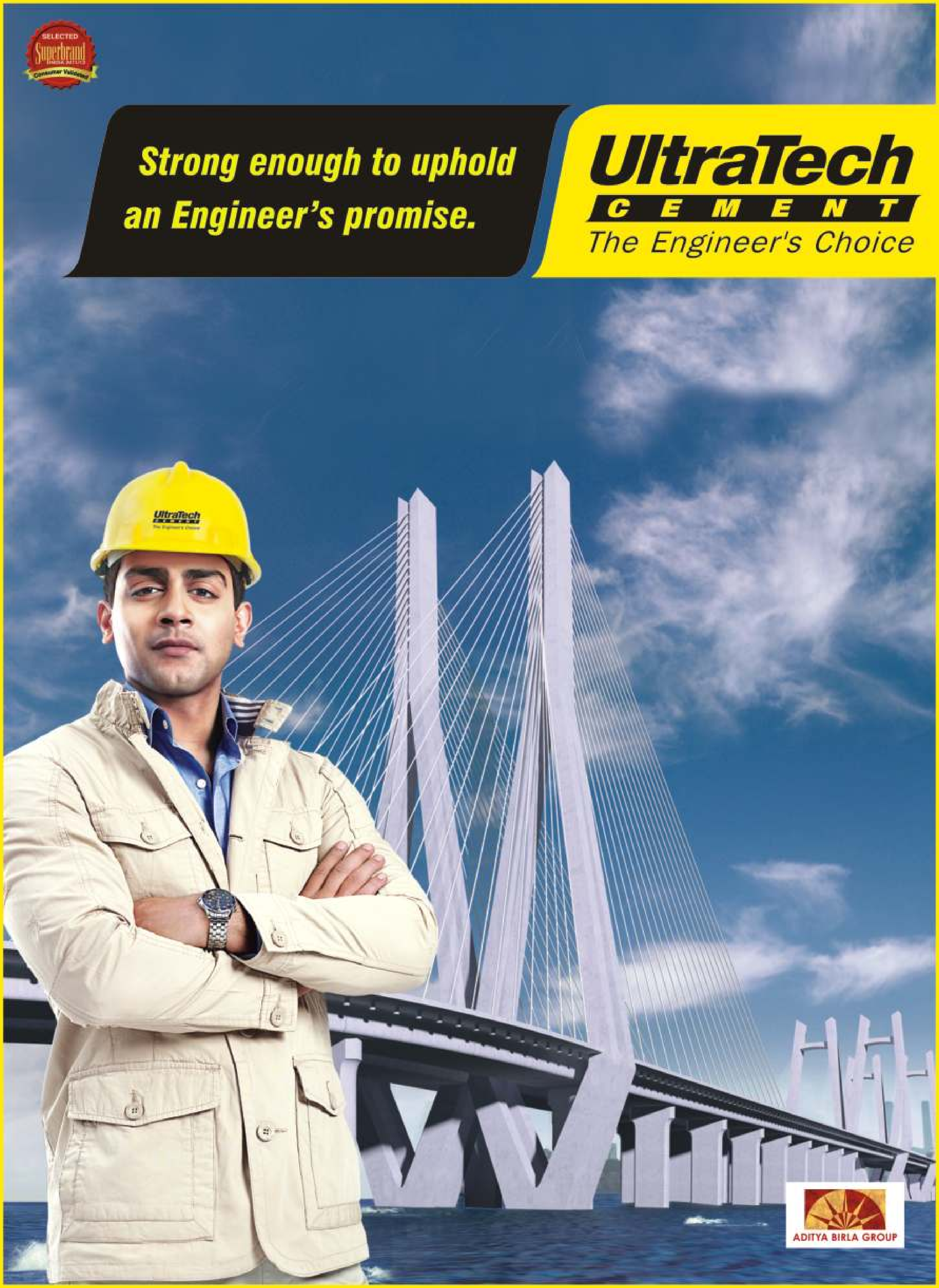
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