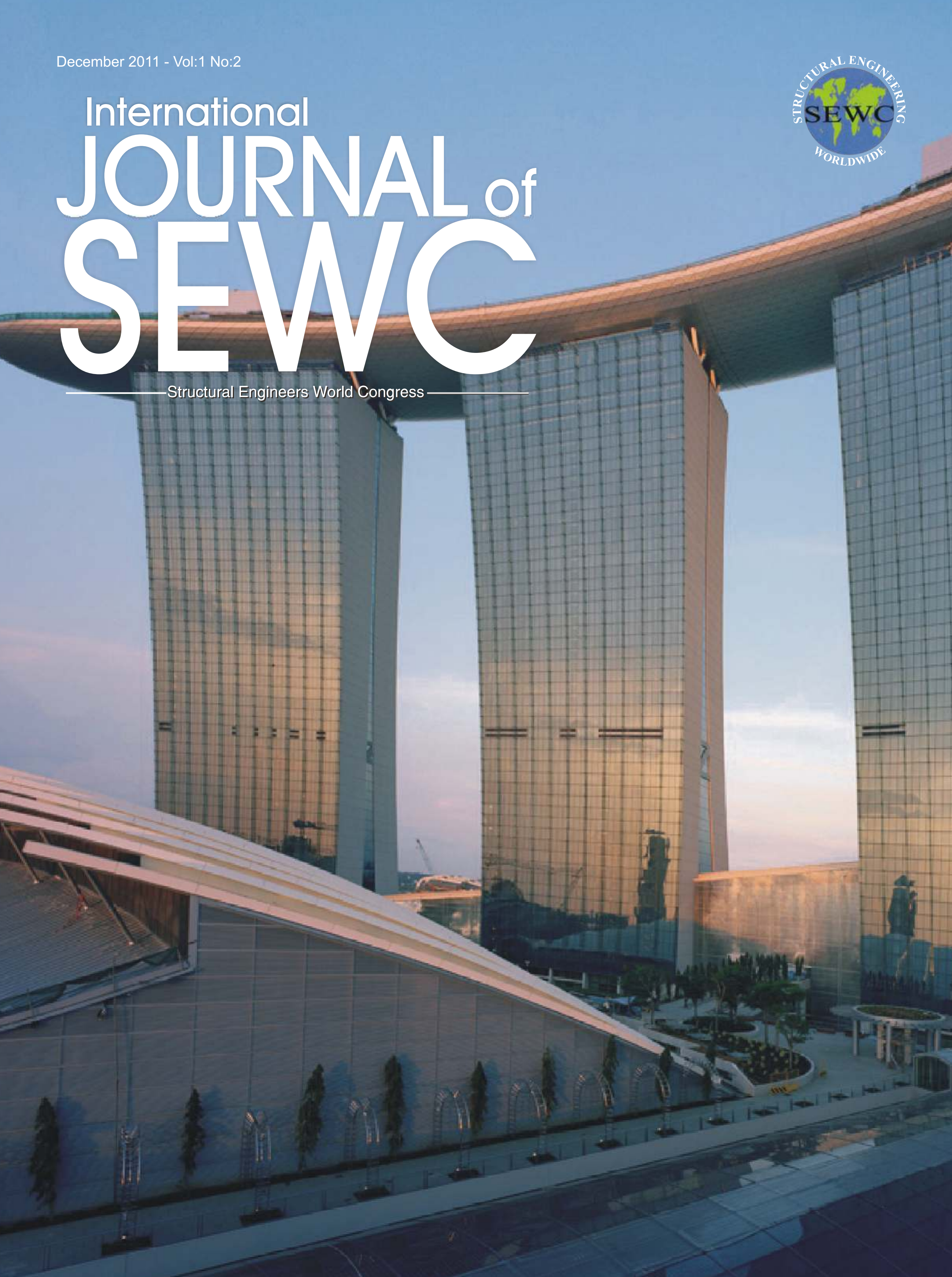
December 2011 - Vol:1 No:2

International JOURNAL of SEWC

Structural Engineers World Congress



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.

December 2011 - Vol: 1 No: 2

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Content

Journal Section

Magazine Section

Published by: Vidyashankar Hoskere on behalf of SEWC Society, INSTRUCT, 1st Floor, UVCE Alumni Association Building, K.R. Circle, Banglore - 560 001. Email: sewcsociety@gmail.com. **Designed and Printed by:** 'The Masterbuilder', 102/11, Tripti Apartments, Marshalls Road, Egmore, Chennai - 600 008, Tamil Nadu, India. Email: editor@masterbuilder.co.in

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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.

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President's Message

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

At the outset, I am happy to inform you that the response received for the first SEWC Journal released in Como, Italy during April 2011 was overwhelming and many professionals, who happened to read this SEWC Journal, have all praise for the SEWC Journal as regards to the overall quality of the publication and especially for the technical contents.

The success of the first SEWC Journal gives us immense confidence to bring out the future SEWC Journals with more variety of topics on civil, structural engineering and other related fields especially on the latest techniques that are now being practised by the structural designers. It is our endeavour to bring out the future SEWC Journals being released from time to time and this will serve as an excellent source of information on the latest developments in the field of "Structural Engineering".

SEWC Conference in COMO during 4th April to 6th April 2011 was a grand success and was well conducted by Dr. Gian Carlo Giuliani and his team. All the technical sessions and Keynote speeches were of outstanding quality. The conference venue is a wonderful place and is a part of a well known resort.

The next SEWC conference will be organized in Singapore in the year 2015 and it is being organized jointly by Association of Consulting Engineers Singapore and Precast Concrete Society, Singapore The Singapore group is enthusiastic and I wish them great success.

Editorial

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

After the successful launch of the first issue of the journal we are stepping into the next issue. The first issue was received very well by the structural engineering fraternity. It is really a challenge to sustain an international journal, especially a journal of the kind we have launched, which strikes a balance between the science of structural engineering and its application.

"Sky is the limit" could be an ideal phrase for the developments in structural engineering. We see the developments in 3D. There are structures far below the ground and even the sea, structures tall enough to reach the sky, long and wide enough to make it far beyond the human eye can comprehend. Concrete structures are becoming massive to warrant the well known size effect. With all the above outstanding features they have to resist fastest of the winds and strongest of the earthquakes. It is really worth continuing research in structural engineering for its applications are entering newer avenues in the coming decades.

Pier Luigi Nervi: Architecture as Challenge

Mario A. Chiorino[1] and Cristiana Chiorino[2]

Summary

Pier Luigi Nervi (1891-1979) is one of the greatest and most inventive structural engineers of the 20th Century. With his masterpieces, scattered the world over, Nervi contributed to create a glorious period for structural architecture.

A wide research and investigation program was initiated in 2009, on the occasion of Nervi's 30th anniversary, concerning his work and his complex figure. The many facets of his personality were explored: designer, builder, researcher and creator of new construction techniques, and, last but not least, professor and lecturer in some prestigious universities around the world, as well as author of books debating the relations between art and science in contemporary construction.

Based on the results of these extended analyses an international itinerant exhibition, "Pier Luigi Nervi - Architecture as Challenge", was created.

The paper presents a synthesis of this investigation-exhibition campaign and throws the light on some of Nervi's most celebrated works of genius and on some of his most innovative approaches to design and construction , giving evidence to the complexity of this extraordinary figure of structural artist.

Keywords

Pier Luigi Nervi, Structure, Architecture, Art and Science of Building, Concrete as an Art Form

Theme

Structure and Architecture, History of Architecture and Engineering

1. Introduction

Described by Nikolaus Pevsner as "the most brilliant artist in reinforced concrete of our time," Pier Luigi Nervi (1891-1979) is one of the greatest and most inventive designers and builders of the 20th Century, at the intersection between the culture of architects and the polytechnic culture of engineers. With his masterpieces, scattered the world over, Nervi contributed to create a glorious period for structural architecture.

His personality is composed of many facets: designer, builder, researcher and creator of new construction techniques, and, last but not least, professor and lecturer in some prestigious universities around the world, as well as author of books debating the relations between art and science in contemporary construction [1] .

A wide research and investigation program was initiated in 2009, on the occasion of Nervi's 30th anniversary, concerning his work and his complex figure. Based on the results of these extended analyses an international itinerant exhibition, "Pier Luigi Nervi - Architecture as Challenge", was created [6] (figure 17).

A synthesis of the results of this investigation-exhibition campaign is presented in the following and the light is frown on some of Nervi's most celebrated works of genius, and, especially, on some of his most innovative approaches to design and construction, giving evidence to the complexity of this extraordinary figure of structural artist.

2. Education, training and foundation of construction company

Pier Luigi Nervi graduated in Civil Engineering at Bologna University in 1913, in a fertile period for scientific, technical and architectural ideas. Thanks to the contribution of a few pioneers, builders, and designers, like Wayss, Hennebique and Maillart, in the early years of the 20th centuries the new technique of reinforced concrete was fast developing and it was associated, from the very beginning, with a more or less conscious search for stylistic results [3].

It is against this background that Pier Luigi Nervi's professional career began. Like the pioneers who had preceded him, after an initial period of training in the technical office of a construction company, Nervi set up his own construction business in 1920. Nervi was to maintain this dual role of designer and builder throughout his life. He has been described as having an engineer's audaciousness, an architect's imagination and a businessman's practical realism. His use of the most advanced technical solutions always went hand in hand not only with the pursuit of formal elegance, but also with an equally strong attention to the technical and economic aspects of the building process.

3. Early works

The stadium in Florence (1930, figure 1) was the first great work which attracted the attention of critics and the public, both in Italy and abroad [4]. Besides the intrinsic beauty of the project, characterized by the elegance and strong visual impact of the curved tapered corbels of the cantilevered roof and the spatial sculptural forms of the helicoidally warped stairs, the competition was won on account of the low cost of the construction .

Figure 1: P.L. Nervi, Stadium in Florence, 1930

The series of great hangars for the Italian air force at Orvieto and Orbetello (figure 2) followed between 1935 and 1940.

Here Nervi probably inspired by some earlier works in steel and laminated wood also for hangars and temporary exhibition halls, especially in the German area [5,8] designed a daring geodetic roof with intersecting arched ribs to enclose an internal space which is dramatically simple in structure. The first group of hangars was constructed using traditional scaffolding and wooden forms for the concrete structure. In those which followed, he used precast elements for the ribs, which were then connected on-site. The use of precast components would afterwards become a constant in Nervi's work. He always sought to exploit to the maximum the outstanding compositional and structural freedom which this technical process offers.

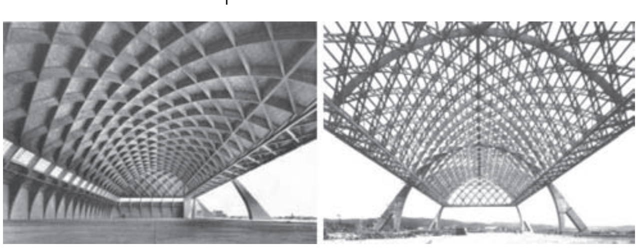

Figure 2: P.L. Nervi, Hangars in Orvieto, 1935 and Orbetello, 1940

4. Early model testing

The hangars were also the first structure for which, in addition to static calculations, Nervi had recourse to tests on reduced scale models. The test were performed by Prof. Guido Oberti (1907-2004) at the Politecnico diMilano, in the Model and Construction Testing Laboratory created by Arturo Danusso (1880 - 1968), using celluloid elastic models on a 1:30 scale (figure 3). Nervi would maintain this procedure for most of his later works (see Section 7).

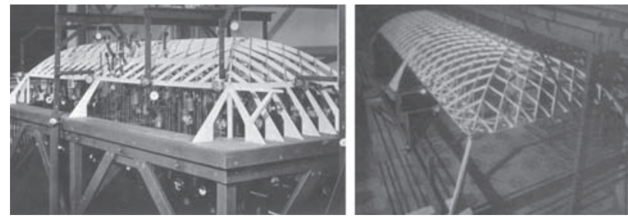

Figure 3: Celluloid models of the first and second series of hangars tested at the Model and Construction Testing Laboratory at the Politecnico di Milano (Courtesy CESI- SMES Archives, Seriat, Bergamo)

5. Ferrocement: a new technique for structural architecture masterpieces

In his first important post-war work - the astonishing central hall of the Turin Exhibition Complex built in 1948 - to make the precast elements for the magnificent transparent 94 meters barrel-vault, Nervi used the new technology of ferrocement (figure 5). This technique, originally adopted by Nervi for the walls and roofs of an experimental warehouse at the site of his construction company at La Magliana, near Rome, and for the hulls of small ships (figure 4), consists in a thin layer of concrete reinforced by a thick mesh of small diameter wires, exhibiting remarkable ductility and resistance to cracking.

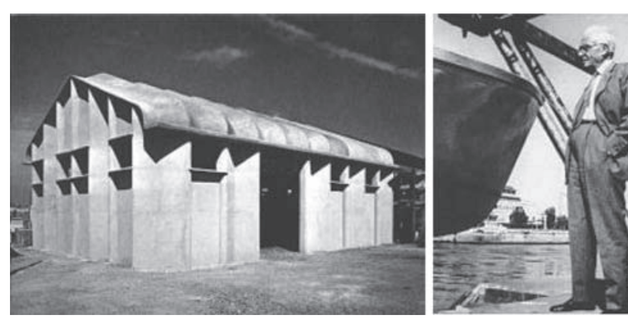

Figure 4: (left) P.L. Nervi, experimental warehouse in ferrocement, La Magliana, Rome, 1945; (right), Pier Luigi Nervi in front of one of his boats in ferrocement.

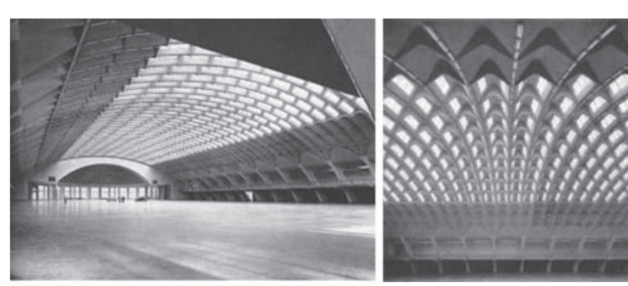

Figure 5: P.L. Nervi, Central hall (Hall B) of Turin Exhibition Complex, Turin, 1948.

Ferrocement can be used to mould elements of any geometric shape undulating in the case of the Turin vault and in the great ribbed dome of the 1960 large Sports Palace in Rome (figure 6) to be connected by cast-in-situ concrete. For the 55 x 157 metres dome of the Hall C of the same Turin complex, designed and built by Nervi in 1950, the ferrocement elements become thin diamond-shaped tiles (2 cm thick) functioning as

formwork for the concrete cast on their upper surface and within the contact channels at their sides, after their assemblage. The result is a particularly elegant mesh of reinforced intersecting ribs (figure 7).

The same pattern characterizes the structural fabric of the vaults and domes of some of Nervi's most famous later works: the Kursaal at Ostia (1950), the Ballroom at the Chianciano spa (1952), the small Sports Palace in Rome (1957, with Antonio Vitellozzi, figure 8), where the extremely elegant layout of the concrete ribs seems to be inspired by the geometrical network of the heart of sunflowers, the Leverone Field House and Thompson Arena at Dartmouth College (New Hampshire, USA, 1962 and 1975, with Campbell and Aldrich), the Norfolk Scope Arena (Virginia, USA, 1965-71, with Williams and Tazewell& Associates, at the time the largest dome in the world with its diameter of 135 m., figure 9) and St Mary's Cathedral in San Francisco (1963-71, with Pietro Belluschi, figure 10). In this last work the ferrocement tiles and the mesh of concrete ribs adapt to the elegant hyperbolic paraboloid surfaces of the dome [7].

Figure 6: P.L. Nervi with Marcello Piacentini, Sports Palace in Rome, 1958-60

Figure 7: P.L. Nervi, Hall C of Turin Exhibition Complex, Turin, 1950

In the Gatti Wool Mill (1951) the tiles are used to build a flat floor: the design of the ribs that mark the ceiling is derived from the lines of the principal bending moments, again resulting in a particularly refined formal outcome that is found in a number of other subsequent projects.

6. Maturity works and international recognition

Nervi's first important work outside Italy was the UNESCO Headquarters in Paris (1953-58, in cooperation with Marcel Breuer and Bernard Zehrfuss), its most characteristic feature being the fascinating folded structure in exposed concrete of the walls and roof. A series of other prestigious commissions followed Besides those already mentioned above, the list includes: George Washington Bridge Bus Terminal in New York (1962),

Montreal's Victoria Square Tower (1961-66, with Luigi Moretti, figure 11), Australia Square and MLC Center Towers in Sydney (1964-72, with H. Seidler) and the Italian Embassy in Brasilia (1979). In Italy, the most celebrated works of his later period include: the Pirelli Tower in Milan (1955-59, with Arturo Danusso and Gio Ponti, figure 12), the complex of works for the 1960 Rome Olympics (besides the two Sports

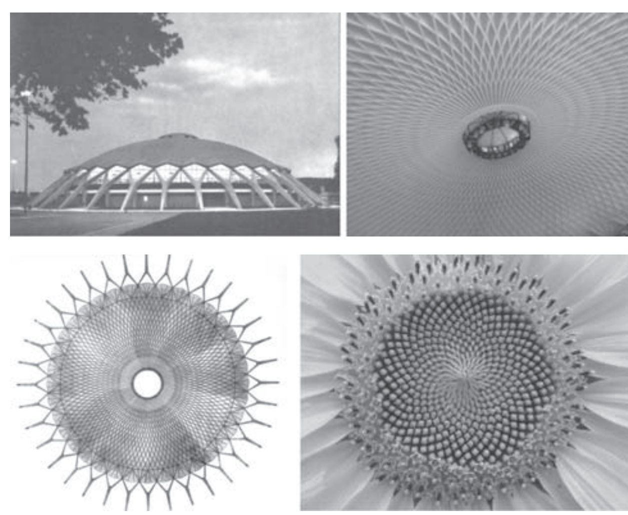

Figure 8: P.L. Nervi with Antonio Vitellozzi, small Sports Palace in Rome 1957

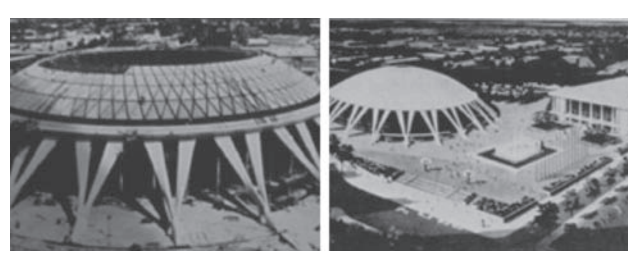

Figure 9: P.L. Nervi , with Williams and Tazewell & Associates, Norfolk Scope Arena, Virginia, USA, 1965-71

Palaces, the Flaminio Stadium and the Corso Francia Viaduct), the Palazzo del Lavoro in Turin (1959-61) (figure 13), with its geometrically fascinating columns whose striped slanting surfaces are covered by a steel umbrella-like structure (designed by Gino Covre); the Ponte Risorgimento in Verona (1963-68) with the sculpturally highly effective shape of the concrete girder, and the Papal Audience Hall in the Vatican (with Antonio Nervi, 1963-71, figure 14), which recalls the themes of the Turin Exhibition Hall of 20 years earlier while enhancing them to create an imposing composition.

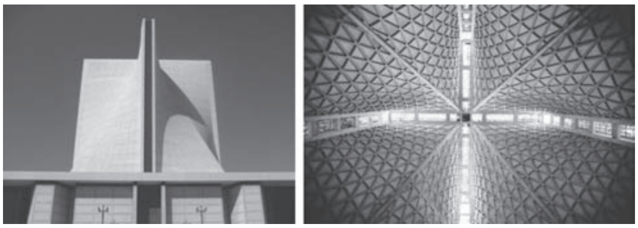

Figure 10: P.L. Nervi with Pietro Belluschi, Mc Sweeney, Ryan & Leew and Leonard F. Robinson (structural engineer), St. Mary Cathedral, San Francisco

Figure 11: P.L. Nervi with Luigi Moretti, Victoria Square Tower, Montreal,1961-66

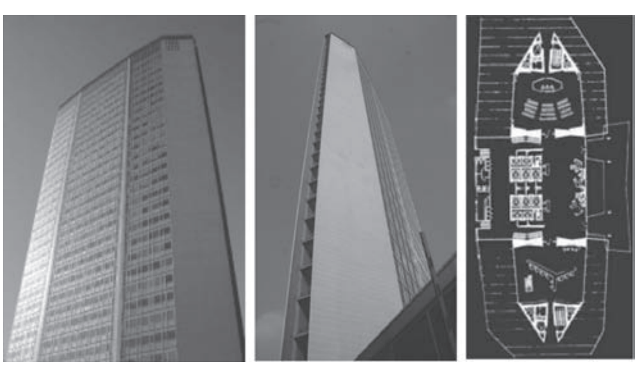

Figure 12: P.L. Nervi with with Arturo Danusso and Gio Ponti, the Pirelli Tower in Milan, 1955-58

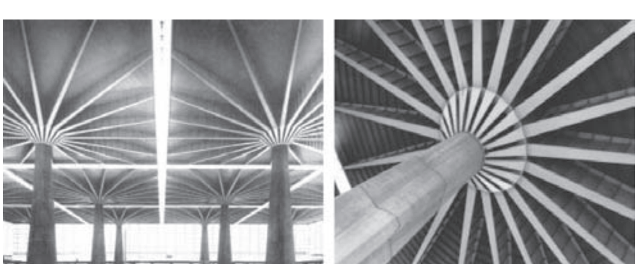

Figure 13: P.L. Nervi with Antonio Nervi and Gino Covre, Palazzo del Lavoro in Turin (1959-61)

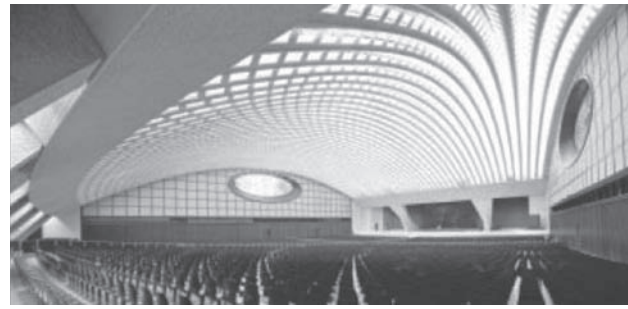

Figure 14: P.L. Nervi with Antonio Nervi, Papal Audience Hall, Vatican City, 1963-71

7. Experimentation in the work of Pier Luigi Nervi

Experimentation played a major role in the work of Nervi. Understood as the direct observation of the structural response, both on full scale prototypes and life-size scale constructions as well as, in particular, on reducedscale models, experimentation was considered as the best strategy, as an alternative to or in addition to the best available computational approach, to overcome the difficulty consisting, at the time, in the recognition of the practical impossibility, of basing the procedure for safety checks of complex constructions on adequately accurate, and computationally feasible theoretical models.

A strategy also found - not coincidentally - in Torroja, and in other leading exponents of structural architecture in the twentieth century including, to cite just a few, Franz Dischinger, Antoine Tedesko, Felix Candela, Heinz Hossdorf, and Heinz Isler [2].

It is important to note that this happened during a period in which great changes were affecting the very foundations of engineering, in the specific field of structural mechanics. Beginning in the 1930s, and increasingly over the following two decades, new formats were developed for the ascertainment of the static safety of constructions, with regard in particular to constructions in reinforced concrete. The focus gradually shifted, in fact, from the analysis of their behaviour in service, in general quite adequately examined by means of elastic calculations, to the study of their ultimate resistance, in order to arrive at a more realistic assessment of their margin of safety with respect to collapse, a margin that in most cases proved to be superior to that measured in the elastic field due to the favourable plastic adaptation of the materials close to the point of rupture.

In this early period, the technique of experimentation on models, too, had to adapt with intelligence to this complex context in which the very criteria of evaluation of the safety of constructions were being modified, overcoming many difficulties in order to be able to provide an effective response. In particular the difficult constraints of scale effects had to tackled with ingenuity and tcompetence. Experimentation on models, which for engineers like Nervi and Torroja became an essential phase of their design path, thus ended up by becoming in itself an extremely refined art especially when applied to works produced with a material characterized by complex behaviour such as reinforced concrete an art in which it was necessary once again, almost in the same way as in real construction, to be able to combine technological expertise and imagination.

After the interruption of activity during the war , the scientific collaboration between Nervi and Oberti initiated at he Laboratory of the Politecnico di Milano continued for more than 30 years within the new research laboratory of ISMES created by Danusso in Bergamo, with the support of Italcementi, the leading cement corporation, and of SADE one of the major hydro electrical companies engaged in the programs of constructions of dams, a type of structures whose design still largely depended on scale model tests

One of the most complex models produced and tested within the new facilities was that for the reinforced concrete frame of the Pirelli Tower in Milan, the 135-metre tall skyscraper, whose structural project was entrusted to Nervi and Danusso (1955-56). This time the large spaces and the avant-garde equipment available at the ISMES allowed Oberti to create a model in large scale (1:15) almost 10 metres tall (figs. 15), produced in micro-concrete of pumice-stone and cement, and resting on a layer of rubber to simulate the deformability of the soil, a model capable of representing the specific features of the structure and thus of being tested beyond service conditions up to failure. The increased interest among the scientific community also in the dynamic

response of structures to the environmental action of the wind and of earthquakes led Nervi, Danusso and Oberti to consider the advisability of carrying out - before the failure tests - a series of tests in the dynamic field in order to verify the effects of the action of the wind.

The tests fully confirmed the validity of the structural configuration that Nervi, this time together with his now elderly friend Danusso, had conceived to give solidity and strength to the elegant form designed by architect Gio Ponti. The variants introduced in the project and in a second series of tests, following the results of the first series, were minimal, aimed at obtaining an increase in the flexural rigidity of the main vertical cantilever - formed of pairs of large dovetailed pylons - through the insertion of architraves joining the pylons themselves even in the upper two thirds of their height, and at improving the torsional behaviour which was affected by the eccentric position of the elevator towers.

In 1962 the ISMES hosted the tests on a celluloid model in small scale (1:53) (figure 15) for the 145 metre tall skyscraper in Place Victoria, Montreal, at the time the highest building in reinforced concrete in the world, designed by Luigi Moretti with a structural project by Nervi. In this case the attention was focussed above all on the dynamic tests in the elastic field for the study of the response to the action of the wind and of earthquakes.

These tests were followed by those carried out in 1964-65 for another prestigious building in North America: St. Mary's Cathedral in San Francisco, designed by Nervi as structural consultant together with architect Pietro Belluschi. In this case the tests were as many as four. The model in small scale (1:100) for the wind tunnel tests was followed by two models in medium scale (1:40 and 1:37) in resin (fig. 16 left) for the static and dynamic tests, with special attention to seismic tests due to the building's location. These tests were accompanied by checks on numerical models based on early applications of the finite elements method by the engineering

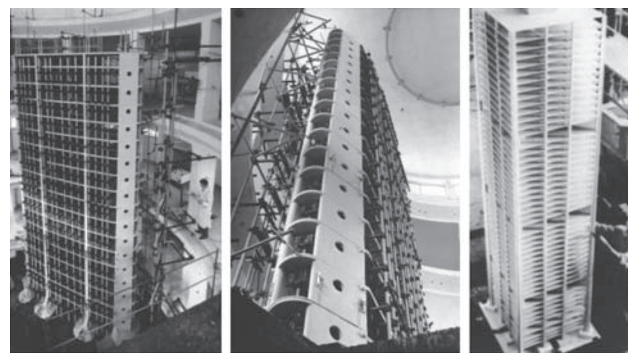

Figure 15: left and center: micro-concrete reduced scale models (1:15) of Pirelli Tower for tests beyond the elastic domain; right. elastc reducede scale model (1:53) of Victoria Square Tower, Montreal

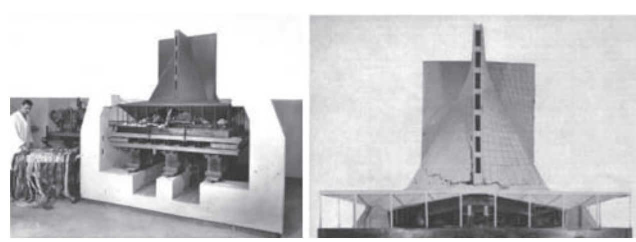

Figure 16: left: elastic model (1: 37) of St. Mary Cathedral under dynamic test; right: micro-concrete reduced scale models (1:15) of St. Mary after the ultimate limit state teswt.

studio of Leonard Robinson responsible for the final design and by the review board of Californian experts responsible for the control of the project. Finally, a large scale model (1:15) in micro-concrete was constructed for the failure tests (fig. 16 right).

In the second half of the 1960s ISMES hosted the experiments on models for a number of important projects in USA and Europe. In 1967, for the Cultural Center in Norfolk, Virginia (figure 9), Nervi used a model in 1:100 scale in the wind tunnel at the Polytechnic of Turin and an elastic model in resin in 1:50 scale for static and dynamic tests (fig.13). These experiments were followed in 1968-69 by the tests on a model in large scale (1:6.6) in reinforced micro-concrete of a large segment of the hyperbolic paraboloid roof for Newark International Airport, New Jersey, and the dynamic tests on two elastic models in resin in 1:40 scale for the B.I.T. offices in Geneva, for which Nervi was consultant engineer. The elastic model of the Rupert C. Thompson Arena at Dartmouth College (New Hampshire, USA) in 1:50 scale (fig. 16) was the last of the models for Nervi's projects to be tested in Bergamo in 1970-71.

While the new computational techniques based on numerical modelling were appearing in those years, it is fair that the art of building, that extraordinary designers like Nervi and Torroja developed in what was an unrepeatable season of architectural engineering, was largely indebted to the art of experimentation. Perhaps justifying Oberti's frequent citation, with undissimulated satisfaction, of the saying attributed by Vasari to Michelangelo: "The most blessed monies that are spent by those who would build are on models."

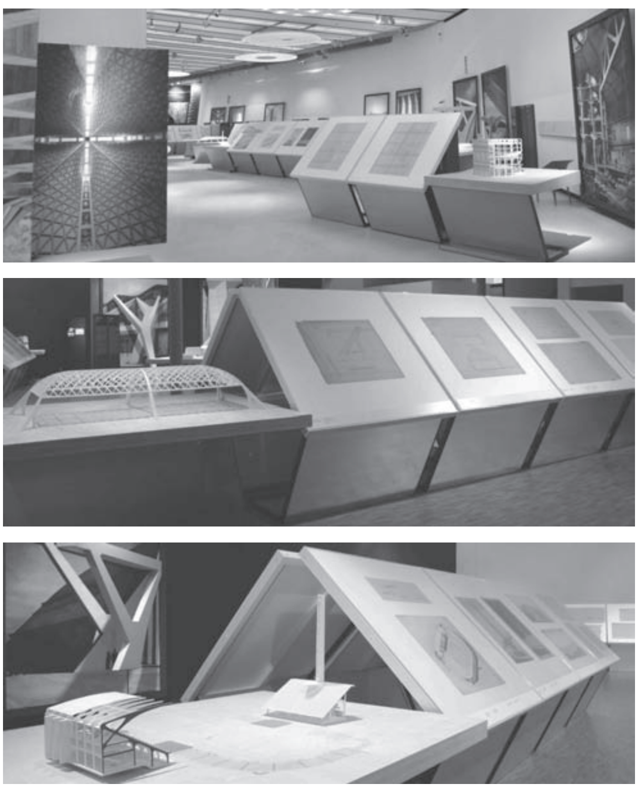

Figure 17: International Exhibition "Pier Luigi Nervi - Architecture as Challenge"; MAXXI, Museum of the Arts of the 20th Century, Rome, December 2010-March 2011 (first bove), and CIVA, Centre International pour la Ville, l'Architecture et le Paysage, Brussels, June-August 2010 (below).

8. Philosophy of structures

When, with his profound knowledge of construction techniques, Nervi discusses the answer to the fundamental question "Science or art of building?" that titles his most famous book published in 1945, far from underestimating the importance of the mechanics of structural systems, he undoubtedly emphasizes the priority of the intuitive moment in the conception of structural architecture: "The conception of a structural

system is a creative action only partly based on scientific data; static sensitivity entering in this process, although deriving from equilibrium and strength considerations, remains, in the same way as aesthetic sensitivity, an essentially personal aptitude". In this vision - that he shares with another master of structural architecture of the 20th century like Eduardo Torroja, who declared a few years later in Razón y Ser de los Tipos Estructurales (1957) that "the birth of a structural complex, the result of a creative process, the fusion of art and science, talent and research, imagination and sensibility, goes beyond the realm of pure logic to cross the arcane frontiers of inspiration" - Nervi fears that the forced need to use analytical models for the reliability assessment of structures might limit a designer's inventiveness. He believes that structural imagination

frequently transcends the possibilities - at the times confined by the lack of modern computerized structural analyses - of analytically rigorous verification. This struggle for a design freedom was also the principal justification for his keen interest for experimental research on mechanical scale models.

These concepts were at the base of Nervi's extensive and interesting writings focusing especially on the language of architecture and the relationship between structure and form, and on the ethical value of building in a correct way. These were also the typical themes of the university teaching he regularly carried out at Rome's School of Architecture and of some of his important speeches and lectures at prestigious universities from Buenos Aires (1951) to Harvard (Norton lectures, 1961-62). They characterized as well his exchanges of ideas and professional collaboration with those who shared his culture and mind-set, such as Mario Salvadori, structural engineer and professor at Columbia University.

The true art of Nervi, and of these other eminent protagonists, is the ability of composing the fracture between art and techniques in spaces that border poetry, without renouncing, in the conversion of the inspiration into a design and of the design into a construction, to the modus operandi of the engineers, but rather emphasizing it with original and innovative contributions.

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Form, Structure and Energy

Martínez-Calzón Julio[1]

Summary

This paper introduces and analyses a new concept, named tensibility, which allows assessing or guiding the quality of the response of a structural system integrated in a given formal arrangement. This tensibility concept makes possible to estimate the efficiency loss of deformation-resistance capacity of the structures needed for free forms compared to those needed for other forms which are more significant or canonical and reduce the required internal strain energy by adhering compromisingly to the structural meaning.

Keywords

Special constructions; structural proficiency; internal strain energy; canonical forms; free forms; significant forms; icons; mannerism; manipulation; tensibility.

Theme

Structural & Architectural Design

1 Introduction

Coinciding with the intensive and active use of computers, we have lately been witnessing a deep transformation of architectural and engineering forms of many important, recently built, buildings and bridges. We, structural engineers, are all being affected by a new way of understanding and categorizing these forms and the structures which give them stability and strength.

But rarely do we stop and reflect on the internal strain energy the structure must develop in order to cope with the loads and maintain the deformation below the acceptable limits required to ensure the comfort and functionality of the structural system. In these new free forms, the required energy is greater than in traditional forms, or, technically speaking, 'canonical forms' - as we shall see later.

There's an awareness of having been, or perhaps still being, immersed in a mannerist age that manipulated previous concepts and processes, which orderly determined the tendency to conceive our constructions. This manipulation, usually superficial and banal, wanted to achieve special, notorious, surprising and increasingly exciting forms at any price, in a sort of methods-and-models marketing. It was clear propaganda for companies which demanded such constructions, by way of icon or symbol which, however ephemeral, would boost the sales of their products. These forms were beyond any architectural or cultural value buildings might exhibit. And it's even worse when similar assignments were made by politicians who, through these kinds of nonsensical constructions, put getting a personal "image" before the respectful and rigorous use of funds contributed by citizens who elected and trusted them.

It is not the author's intention to tackle these social-political issues or to examine these kinds of behaviour, which are the responsibility of society at large, but to focus on a balanced assessment of what this set of concepts (forms, bearing structures and the energy they need to develop) means for the domain of the technology and, more profoundly, on what its repercussions are for the broader concept of culture of societies.

This proliferation of forms that are exclusively ruled by originality and image, and which are definitively unrelated to any commitment towards their resistant behaviour - even in works of the greatest importance - come to determine a way of operating defined as "free forms", very often used in buildings but also increasingly present in footbridges and bridges.

This paper aims to assess qualitatively, and in some cases quantitatively, what basic meanings this impulsive presence of free forms might add to the structural field and what would be their correlation with the internal strain energy mobilised by those structures.

2 Scope

Because if this trend line of designs were to be enhanced, some questions would undoubtedly arise:

- To what extent could the ratio originality/cost be acceptable?
- Where do the limits of such originality have to be set? Because it's clear that it cannot be zero in view of the fact that, in their positive development, our societies need some margins for design, cult to beauty, novelty, etc. and don't have to be subjected solely to utility fac-

tors, minimum cost, etc.; some concessions might therefore be acceptable.

It's obvious that these limits, if any, should be determined by concepts which are different from the ones this analysis of the creativity form-structure-energy seeks to establish. For the public administration, these concepts would probably be much more in line with the character and socioeconomic requirements of what each country, society and time establishes in their budgets and priorities. For privately owned companies, they would have to fit with their market and return-oninvestment strategies.

With regard to bridges: neither can we, the engineers, be acquitted of formal, even formalistic, attitudes and one might observe an appreciable trend to use gratuitous solutions, which are remote and even sometimes deviant from the structural practice, solely to achieve originality, without caring to entail important cost overrun, so to achieve the media coverage (press, TV, etc.) that would make up for the economic setback.

By means of example: it seems difficult to admit in a technical context, that works such as the Alamillo Bridge (figure 1), or the more recent Bridge Pavilion in Zaragoza (figure 2), with a per square meter cost between 6 and 12 times

Fig. 1: Alamillo Bridge in Sevilla (S. Calatrava); elevation

Fig. 2: Zaragoza Pavillion-Bridge (Zaha Hadid); Bird's eye view

higher than what could be considered as reasonable, have been accepted with hardly any criticism by the engineers in charge of their completion, regardless of the fact that more or less technical juries elected those designs. And this is important because these decisions affect heavily our relationship with society even if the real cost of completion is zealously hidden or disguised.

To illustrate the above-said, let us compare two types of cross-sections which are frequently used in footbridges and bridges and which allow assessing and criticizing the formal design decisions.

I) Circular tubular cross-section and its comparison with the corresponding inscribed rectangular one (figure 3)

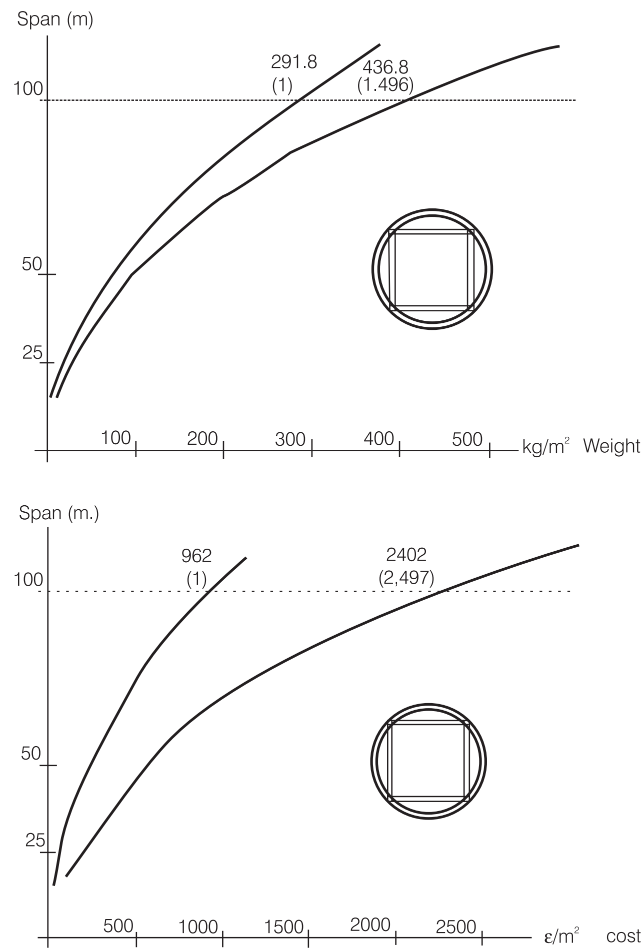

Fig. 3: Comparison of Cross Sections I

II) Lenticular cross-section and its comparison with the inscribed rectangular box section (figure 4)

These examples witness that the use of these forms under bending stresses determines an average weight increase of about 40% and an average cost increase of about 140% for the tubular cross section; whereas these numbers are respectively 120% and 225% for the lenticular cross-section.

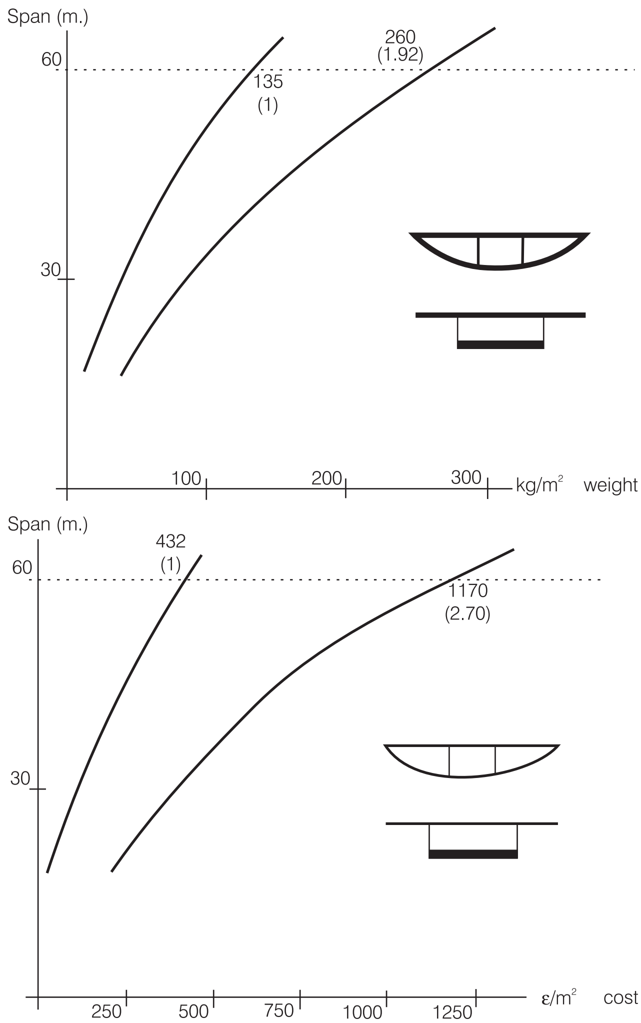

Fig. 4: Comparison of Cross Sections II

These are significant values that should be known by the technicians which have to assess the possibility and suitability of these solutions.

However, there's no need to dismiss the whole domain of free forms because during their study and structural design, some positive aspects may arise for the structural technique. In order to establish a way to carry out this active assessment of constructions and forms in a weighted manner, it's advantageous to introduce a new criterion: tensibility. A factor that correlates the three categories we've been using: form, structure and energy.

3 Tensibility

Thus, the factor tensibility is defined as the intrinsic ability of a structure or building type to actively drive the stresses induced by all loads on it, into its bearing points by making maximum use of its formal configuration and the arrangement of its composing elements, so to achieve energeticdeformation behaviours close to the canonical ideals - those of minimum internal strain energy of the deformed system which responds to the acting loads.

Each structural arrangement adjusted to a certain architectural form can thus reach a certain tensibility factor. The skill to perceive or measure this factor and the ability to increment it and to estimate the potential limit of a given formal configuration reveals the grade of structural proficiency of the structural engineers.

In order to achieve that, the engineers have to rely on an intense and intuitive knowledge of the broader and deeper properties and resources of structural systems: joints, couplings, links, compatibilities, intelligent devices, material properties, staged behaviour, etc.; which might come from other systems or be adaptations of them and on which they can rely with great praxis and skill so to enhance the understanding and optimization of the resistant plexus of the system.

Thus, the most operative possibilities for the application and assessment of the tensibility factor are obtained by means of the qualified use of concepts such as:

Mega-structures; bring into play the total dimensions of a building or area

- The interrelationship of substructures or subsystems:

- Cores - slabs
- Bearing facades - slabs
- Special blocks (shear walls, cable structures, local tubes, etc.)

- The possibility to operate on elements:

- Prestressed diagonals
- External prestressing
- Ties

- Deformational incompatibilities:

- Bending-torsion, equivalent to elastic restraints
- Coupled cantilevers, equivalent to elastic bearings
- Active preloading (prestressing, raising of bearing points, etc.)

- The redirection of stresses:

- Transformation of structural areas: provisional restraints; joint-blocking; etc.
- Larger but more operational load paths
- Tensegrities and semi-tensegrities, obviating bending

Mega-bars and mega-sections

- The use of hybrid solutions: adequate combinations of building types and materials

- A precise understanding of the slenderness and the restraints required to ensure local and global stability
- The use of intelligent devices: dampers, stabilizers, semi-concordant links, etc.

On the contrary, conditions and situations of anti-tensibility are:

- Those which set the structure according to forms and surfaces which lack of active arrangements
- Forms with abrupt changes in areas which are remote from bearing points or partial unloading systems
- Angulations or deviations of the structure which determine notable breaks of the stress flow.

Because regardless of the socio-cultural value of any of these new free forms, the current key role of structural engineering is to actively help achieve the full potential of the original creative idea - if any - pursued by such forms. And, this mental attitude arouses: to achieve new tensible possibilities which enhance the basic conceptual approach, and to establish nuanced reductions which, without notable distortion, free the original idea from its inappropriate lines with low tensibility profiles.

By following these guidelines, the final work will shed the fundamental underlying architectural spirit of the creation itself as well as an additional, deeply harmonic intensity of its form, partially conceptual and partially psychological, in its essential adherence to the laws of nature.

And it will achieve it, not in the same way as the classic or canonical forms did, but in a new and typical manner, as befits its rising existence and as excellent and worthy as them.

Structural engineering is primarily interconnection and integration of the bearing system into a conglomerate that combines materials, space, form and energy and which is mechanically as close as possible to what in the realm of living beings would be a skeleton that is harmonically entangled in its global metabolism, equivalent to the architectural action. And that also, and without distinction, can be applied to the engineering works by changing the architectonic variables for the engineering ones of our works.

The structural density, in conceptual sense, measured by tensibility, guides, towards the transcendent as if it were a kind of compass; whereas the path of the mere form never reaches the perennial and magisterial in the field of construction.

This approach will allow us to compare clearly:

- On the one hand, the current avalanche of free form proposals which present society seems to claim;
- On the other hand, an effective engineering which is able to carry out the mastery of the new process and direct the performance of this society towards deeper and more consistent positions.

A clear vision of what the tensibility concept encompasses may allow transmutation of possible free, soulless, forms into other similar ones which tend towards significant forms thanks to proficiency in the field of structures and matter; a proficiency that has to be compositional, analytical and constructional.

4 Examples

In an attempt to clarify the aforesaid, six construction projects are presented wherein this proficiency has been achieved through the use of tensibility:

4.1. Gas Natural's New Headquarters. Barcelona Architects: EMBT

A building complex wherein two great cantilever bodies are to be highlighted, one of them a prism with an average span of 42 m; the other one has a variable depth and a maximum span of 32 m.

In the first one, the mega-structure condition and the concept of global suspension from the top of the central core, determine clearly the used tensibility ideas followed while minimizing the use of functional areas by integrating the main structure in the facades.

In the smaller body, tensibility is expressed by driving the deviating horizontal components of the sloping elements towards a rear core. Due to the eccentricity of the axis of the cantilever, the floor slabs have to work as horizontal deep girder structures able to drive the vertical torsional moments, originated from that eccentricity, towards the concrete core.

Tensibility is also making good use of all the vertical cores together with the stiffness of the composite slabs so to achieve that this system works as a whole against wind and seismic loads.

Fig. 5: General view of the complex

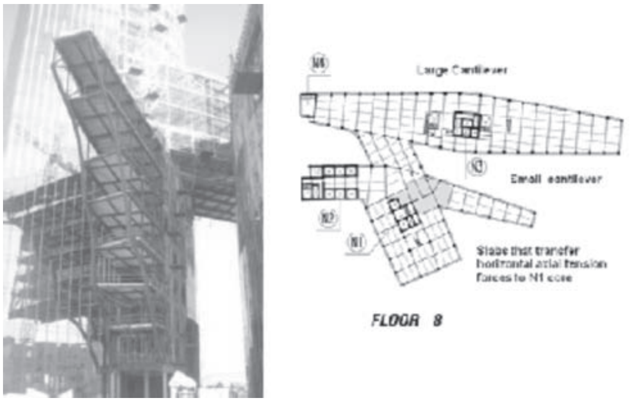

Fig. 6: Small cantilever body under construction and floor plan

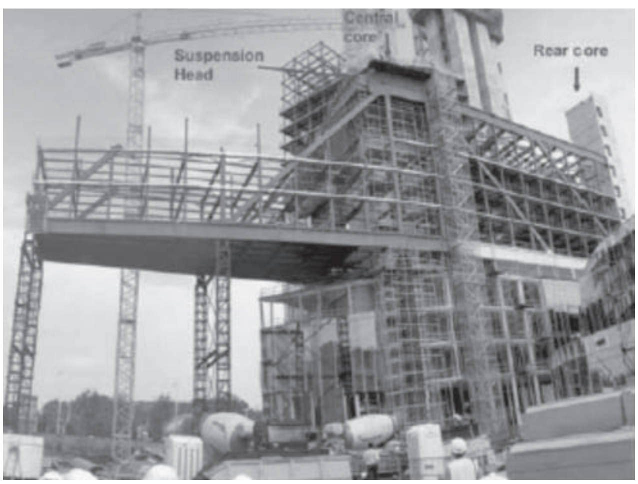

Fig. 7: Erection of large cantilever building showing the megastructure under construction

Fig. 8: Structural arrangement of the suspension head on top of the central core

4.2. Torre Espacio. Madrid Architects: Pei, Cobb, Freed & Partners

Use of prestressed diagonals in the two large facade trusses of a high-rise building so to reduce, by means of staged pre-stressings, the bending moments which occur during the erection process. This has been defined earlier as one of the specific mechanisms of tensibility.

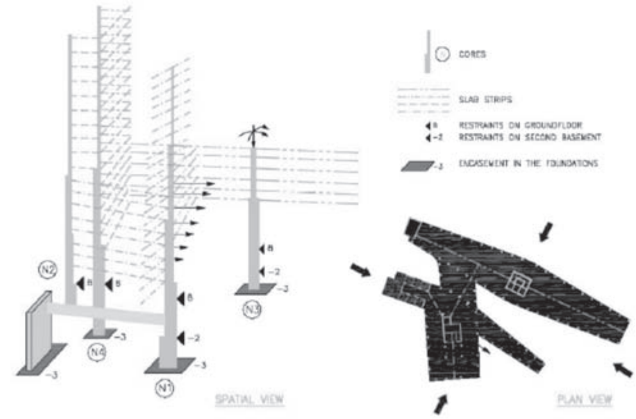

Fig. 9: Global structural arrangement

Fig. 10: Tower with open concourse on ground level

Fig. 11: View of the truss girders in the building

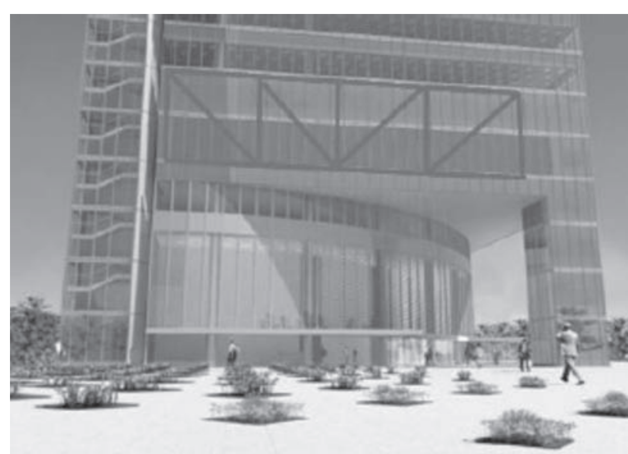

Fig. 12: Large truss girder with prestressed diagonals

4.3. New airport terminal roofs. Alicante (Spain) Architect: Bruce Fairbanks

Set of 36 m sided square domes, bearing on 13.50 m high cylindrical tubular composite columns with an external diameter of 0.9 m.

The resistant system is composed of 4 circular bowstring arches made out of steel boxes and with a radius of 36 m set at the four sides. They bear monolayer domes built out of two sets of round, orthogonally arranged, hollow tubes and a system of diagonal rods. In this structure, the tensibility of the system is developed by assuming several geometrical and mechanical conditions; such as:

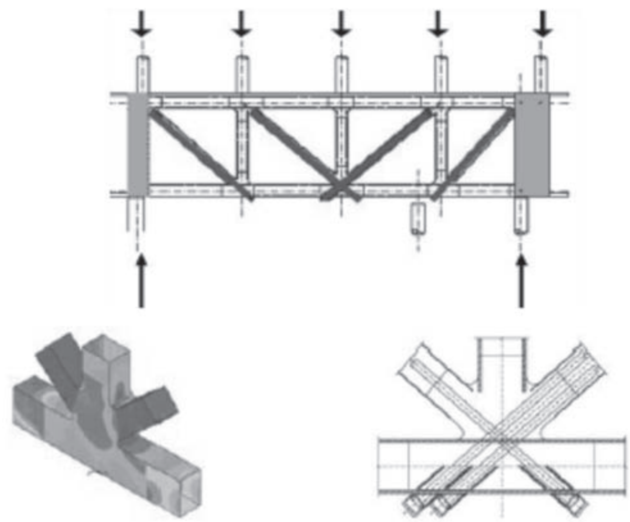

Fig. 13: Arrangement of the prestressing strands in the diagonals

- Identical tubes are completely pre-manufactured off-site with the appropriate thicknesses for each area and placed parallel so to form a pseudo spherical dome which is geometrically very close to a spherical one. Diagonal rods cross the small gap between both sets, achieving a global structure that behaves mainly as a membrane but is able to resist laminar bending and shear.

- Specially designed joints made out of a piece of moderately thick tube with square crosssection which holds in its vertices four ear plates. These ear plates allow establishing a pin connection with the jaw-end of the solid diagonal rods of various diameters 12 to 32 mm.

- Each self-supporting dome is assembled on ground level and, once finished the assembly, lifted by means of a crane onto the outside end of provisional gantry-beams so to be launched to their final location by means of simple bogies.

Fig. 14: Bird's eye view of the construction process

Fig. 15: Detail of the joints between dome layers

Fig. 16: Placement of domes through launching on provisional girders

Fig. 17: Inside view of the dome system

4.4. La Pallaresa complex. Barcelona Architects: Iberian Arquitectes

The three buildings that form the complex have important cantilevers at the front in such a way that the lower horizontal soffits of their first floors are at the same level; hence creating at the ground level a great common open concourse which neatly unifies the three buildings.

The structural tensibility of these buildings has been exploited maximally by using megastructures in all their facades so to let them constitute, together with the central concrete core and the solid floor slabs, a spatial bearing structure which is characterized for lacking great cantilever trusses acting as upper suspending lintels or lower bearing members; hence avoiding systems which impose unfavourable functional conditions on the buildings.

The reinforced concrete facades are perforated by a staggered window pattern; the lateral ones being large cantilevers able to drive, in to most favourable manner, the self and front facade loads towards the foundations thanks to the ideal diagonal grid present in its geometry when modulated each two storeys. These facades are thus structural as well as functional, acting as the building envelope and protected with the adequate coating.

The construction procedure also requires some key tensibility concepts: propping only the first four floors and three heights of facade so to let the subsequent storeys contribute progressively to the global resistant system as they are being built. Monitoring displacements through hy-

Fig 18: The three buildings with horizontally aligned cantilever soffits

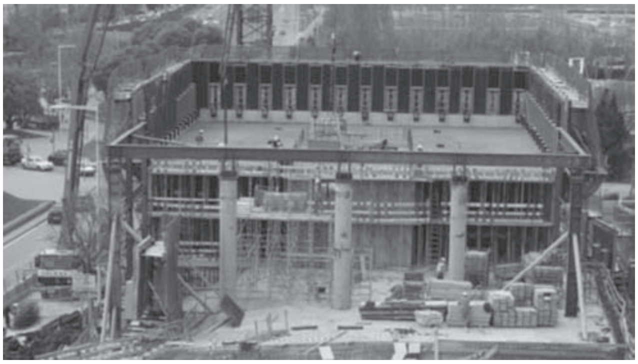

Fig.19: Temporary props and formwork arrangement at the beginning of the construction of the hotel cantilever façade

Fig 20: Hotel structure after prop removal

Fig. 21: Diagonal lines in the facade

Fig. 22: The complex at the end of construction

Fig. 23: Final general view

draulic jacks during prop removal allows controlling the response of the structure, comparing the theoretically predicted values with the real ones and hence ensuring a correct behaviour and construction.

4.5. Zerozero tower. Barcelona Architect: Enric Massip

High rise building of 24 storeys with a spearhead shaped floor plan, a central concrete core and facades composed of two interconnected bearing layers:

- An inner one made out of 16 cm wide vertical columns tightly placed at 1.35 m from one another

- An outer one made out of rectangular tubular elements with a 0.68 m by 0.24 m cross-section, placed diagonally and randomly as branches, called bamboos.

The prestressed concrete floor slabs connect the aforesaid double layer to the core so to constitute a tube-in-tube system that bears all vertical and horizontal loads.

The tensibility of this structure is found in the fact that, although the bamboos were initially only thought as ornamental, both layers of the facade were interconnected so to let them work simultaneously and efficiently: the first one carrying mainly the vertical loads and the other one - made out of diagonal members - driving the horizontal loads towards the foundations, via a process of compatible interaction between both systems. In the most unfavourable direction, this double facade constitutes about 34% of the total bearing capacity of the building; with about 18% in the bamboos and about 16% the inner layer; resulting in adequate overall dimensions and slender wall thicknesses for the core as well as an acceptable global sway response.

Fig. 24: Model showing the double facade system

Fig. 25: Arrangement of the double façade members

Fig. 26: Inner patio with special horizontal bracing beams

Fig. 27: Longitudinal final view

4.6. Bridge over the Avenue of the Americas. Montevideo (Uruguay)Engineer: Julio Martínez Calzón

The tensibility feature of the emblematic and iconographic design demanded by the administration is achieved by placing the single sculptural pylon in the centre of gravity of the curved deck's dead load. That way, the pylon is subjected to a vertical resultant with minimum bending moments which only appear, either positive or negative, in case of asymmetric live load and due to wind, thermal gradients, etc.; fairly small values compared to the possible total loads. Additionally, the prestressing of the bottom half of the pylon is actively improving the bending response of the composite system of steel tube filled with concrete that receives that prestressing. The foundation, which is set adequately eccentric to the pier, provides also an adequate tensibility arrangement.

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Fig. 28: External view with pylon

Fig. 29: Continuity of the deck

Fig. 30: Arrangement of the stays in the pylon arms

Fig. 31: General view at sunset

Interactive Design and Dynamic Analysis of Tensegrity Systems

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

Summary

A real time implementation of a discrete element method is presented with applications in interactive design and dynamic non linear analysis of lightweight reticulate space structures. It is validated on a representative example by comparison with two other static analysis codes adapted to the behavior of tensegrity systems.

The advantages and versatility of the approach are demonstrated through examples in stability analysis and deployable structure simulations.

Keywords

Tensegrity, dynamic behavior, interactive design.

Themes

Structural & architectural Design - action engineering - steel & composites

1. Introduction

Tensegrity systems are a particular class of reticulate space system known since the fifties with the work of K. Snelson [1] and the definition given by R. B. Fuller [2] after the contraction of the term "tensional integrity" that qualifies the state of these systems. More recently, they were defined [3] as a discrete set of compressed elements inside a continuum of tense elements in a stable equilibrium state. Thus, tensegrity structures are self-equilibrated systems made of bars and cables in a selfstress state, which brings the possibility to create thin, lightweight and transparent structures (figure 1).

However, the design of these systems is generally complex because the form and the internal forces needed to ensure the rigidity and stability are closely related. Form-finding approaches have been developed to consider these constraints during the design, using force density methods [4] or by a combination of modules [5] but they lead usually to regular geometry. Recently, a free form composition process was described, leading to irregular configurations using a step-by-step construction process [6].

Although these systems still fascinate architects and are the subject of an extensive literature, their development remains restrained by a lack of knowledge and confidence when compared to more conventional construction principles because these systems do not fit well in the existing structural analysis engineering process. In order to improve this situation, analysis methods need to be adapted to get the better of these systems by taking into account more easily the initial selfstress, local non linear effects and large displacements. In addition, due to their lightness, the dynamic behavior of these systems is also to be guaranteed and properly determined, which may require an additional specific analysis procedure, getting the whole design process more complex.

The methodology presented in this paper relies on the use of a dynamic simulation in order to solve at the same time the analysis and design problem. Actually, tensegrity systems are very similar to molecular systems. Thus, they can be modeled as a set of nodes in elastic or more complex interactions representing bars and cables. Computing power being more and more affordable, complex discrete systems can now be simulated in real time on current desktop computer. It is then possible to simulate continuously the evolution of such a system as it is reacting to internal and external forces while building its structure, which corresponds to incorporating a dynamic simulator within a structural modeler. This approach, extensively used in computer graphics domain for clothes and multi-bodies simulations [7], was also chosen by several authors for the visualization and simulation of elementary modules or regular structures, but with limited editing or analysis possibilities [8][9]. This methodology was also highlighted and applied successfully in form-finding of tension structures [10] and robotic simulation [11].

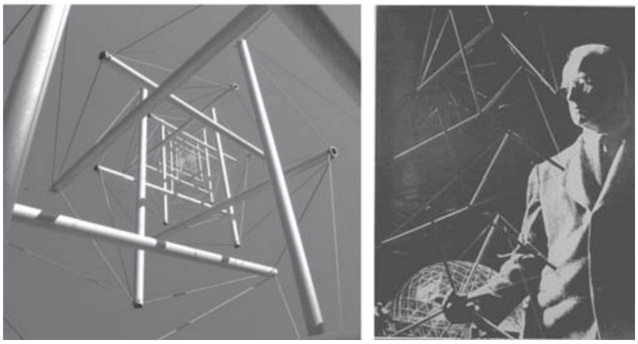

Figure 1: Snelson's Needle Tower and R.B Fuller's Tensegrity mast

In this paper, an implementation of these principles in the structural analysis field is presented. Its possibilities and advantages over the conventional analysis process usually disconnected from the architectural design, are demonstrated. Indeed, the dynamic analysis is conducted in real time and allows interactive form adjustments, editing of structural properties and can also be adapted for use in quasi-static and kinematic analyses.

2. Tensegrity systems static analysis

Tensegrity systems are self-stressed space reticulated systems where the nature the elements is adapted to the set of internal forces, which allows gains in weight and transparency. Though, the global self-equilibrium and stability depend strongly on the existence of a compatible selfstress state, whose level and distribution must be designed in order to keep cables in tension and bars in compression, in conformity with their mechanical abilities [12].

Static equilibrium of reticulate systems corresponds to the equilibrium of all the degrees of freedom under internal Nij and external Fj forces (figure 2).

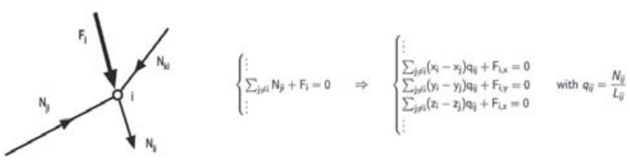

Figure 2: Global static equilibrium

Using density force factors qij the static equilibrium can write in matrix form [13] as Aq = f (1)

where A is the n*m equilibrium matrix that expresses the equilibrium along the n degrees of freedom of the normal forces of the m elements that constitute the structure. The column vector q represents the selfstress state vector, composed of the density force factors qi, defined as the ratio of the normal effort over the length of element i. Selfstress state verify the global static equilibrium of the system under no external load therefore,

it is the solution of the problem (1) with f=0, which is by definition the kernel of the equilibrium matrix A. Thus, this vector space can be used to generate a convenient selfstress state able to fulfill all the rigidity and stability design requirements [14].

Because the behavior of tensegrity system is subject to several sources of non linearity, mainly due to the unilateral behavior of cables and second order transverse rigidity induced by normal forces, particular precautions must be taken in the global analysis, which is often delicate with conventional structural analysis codes. To do so, advanced computation software [14] have to consider rest length state variables, geometric second order terms in the rigidity matrix of elements, slackening of cables and implement nonlinear schemes like Newton-Raphson.

3. Analysis using a dynamic model

Actually, if we accept to simplify the angular momentum of bars and to neglect their flexibility by concentrating masses to the nodes, tensegrity systems can be viewed as a set of masses interconnected by tensed or compressed elements, which corresponds to a discrete mass-spring system. Thus, these systems fall in the range of applications of discrete elements methods in which equations of motions can be integrated using calculation schemes that allows the computation of trajectories for systems with complex interactions and behaviors.

3.1. Dynamic simulation

In our approach, tensegrity systems are defined by nodes and elements entities. Their dynamics are determined using an explicit fourth order Runge-Kutta (RK4) time integration scheme. This method, which is a classic in numerical simulations, is easy to implement while providing high performance. The system is defined kinetically by its dynamic state X(t), which is the sets of the positions and the speeds of the nodes (equ. 2) at a given time t.

$$x(t) = \begin{Bmatrix} x_i(t) \\ x'_i(t) \end{Bmatrix} \quad (2)$$

During the RK4 processing, the dynamic state X(t) is successively computed from the combination of firstorder derivatives of the state of the system X'(t) = f(X,t), which rely actually on the determination of forces to the nodes (equ. 3).

$$f(x,t) = \frac{dx(t)}{dt} = \begin{Bmatrix} x_i(t) \\ x''_i(t) \end{Bmatrix} = \begin{Bmatrix} x'i(t) \\ \frac{1}{m}\Sigma F_i \end{Bmatrix} \quad (3)$$

The structure of the system is defined by the set of elements, associating two node's indices and a rest length. Their mechanical behavior is explicitly defined in the calculation of normal efforts from the knowledge of their dynamic state and of their rest length. In our case, we use a Kelvin-Voight model with a bilateral behavior using two distinct young's modulus E for traction and compression (figure 3).

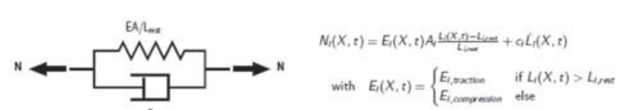

Figure 3: Generic rheological model of elements

In figure 3, Ai is the cross section, Li(X,t) and Li,rest are respectively the actual and rest length, and ci the damping factor for the element i. This approach potentially allows, using threshold values, to simulate plasticity by modification of the rest length or local rupture. Actually, the implementation of others rheological behaviors or interactions between entities are only limited by the possibilities of translating them as forces to the nodes or structural modifications. For convenience and rigorous modeling of structures using many regular elements in the manner of Building Integrated Models approaches [15], nodes and elements are associated to a class number, or family, for which all the shared physical properties like cross section area A, young's modulus E, mass density or more are described.

3.2. Visualization and interactions

The simulation program presented in this paper, designated as "ToyGL", is based on the C++ GLUT libray, which allows running the simulation computations in parallel to a visualization of the structure using OpenGL with possibilities of user inputs using mouse and keyboard (figure 4).

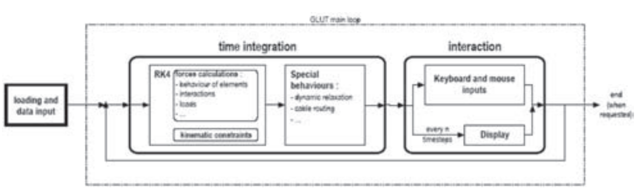

Figure 4: General architecture of the simulation program

In addition to adjusting global simulation parameters, the user is also allowed to change the structure, by adding or removing selected elements or to adjust their properties and rest length. With these tools, one can build from nothing or modify an existing virtual structure in real time. The architecture of this program allows applying special behaviors after each time step calculation. In our case, a dynamic relaxation algorithm is implemented in order to cancel nodes speed when reaching local maximums of kinetic energy, which provides an artificial damping for accelerating the convergence towards a stationary state in form-finding processes or quasi-static analyses. To illustrate the possibilities of this approach, we present in the following some applications in design, analysis, kinematic study and robustness test.

4. Interactive design

In this first example, we present the construction of a simplex module, a three bar regular tensegrity system. Elements are made of steel, with a Young's modulus of 210 000 MPa and mass density of 7 850 kg/m3 while each nodes weights 1 kg. The cross section area of cables is 1 cm2 and that of bars is 5 cm^2.

The different steps of construction are illustrated in figure 5. Starting from a simple node, upper layer cables are created in the shape of a triangle. Then, nodes are moved upward and restrained in order to keep the layer in shape. Then, vertical cables are created and let hanged under gravity load. The lower layer and the bars are added next, leading in step 6 to a finalized tensegrity module. An additional step allows obtaining a regular module by setting rest length to the homogenous values of 1 m for cables and 1.47 for bars, which leads to a final regular selfstress state that exhibits credible admissible normal stress levels of 90.8 MPa

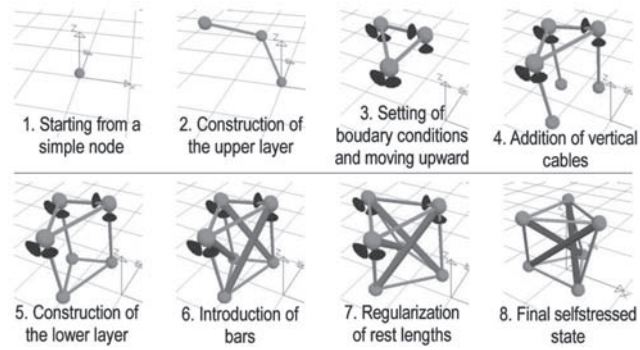

Figure 5: Virtual construction process of a simplex module

in layer cables, 157.4 MPa in lateral cables and 46.2 MPa in struts.

In this example, the elements have realistic dimensions and are made of stiff material so the structure is very rigid. Thus, the timestep dt of the explicit integration scheme has to be adapted to avoid numerical instabilities. A typical rule is to take dt as a fraction of the highest vibration mode period Tmax :

$$dt < \text{fraction of } T_{max} \quad \text{with} \quad T_{\max} = 2\pi\sqrt{\frac{m_{\min}}{k_{\max}}} \quad \text{and} \quad k_{\max} \leq \sum_{\substack{\text{Converging}\\ \text{elements}}} \frac{EA}{l} \qquad (4)$$

where mmin is the minimum node's mass in the system and kmax the maximum axial rigidity that can be estimated as the cumulative stiffness of converging elements to the more connected node. In our case, by considering the unfavorable case of two opposite bars converging to a 1 kg node, we find a Tmax of 0.5 ms, which validates the choice of a 0.1 ms timestep for this particular problem.

5. Real-time remodeling

In addition to creating realistic structures, another design possibility is by remodeling an existing system that can be generated beforehand using different approaches, either graphically using a CAD software or programmatically [16]. Applying changes to rest lengths or editing elements categories lead then to potentially interesting derived geometric configurations. At the same time, the effects on the stability of the system are instantly noticeable and indicators on the selfstress state can help to do adjustments that are

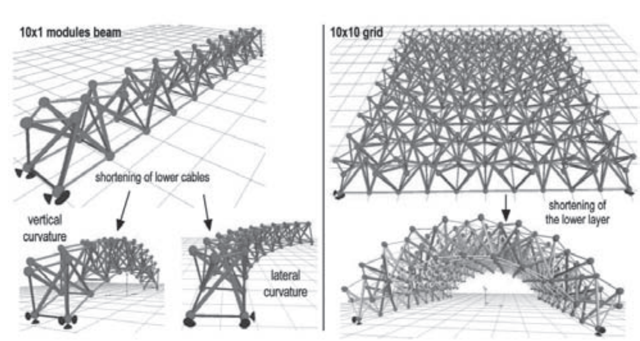

Figure 6: Geometric transformation of a tensegrity beam (a) and a plane grid (b)

necessary to balance the distribution of normal forces toward acceptable levels.

To illustrate this, we present example structures formed by an assembly of 4-bars "quadruplex" tensegrity modules, shaped as cubes with a side of 1 m. The material characteristics remain that of the first example in order to exhibit realistic structures. The first example is a 10 modules beam simply supported on both ends (figure 6.a) in which curvature is induced by a 5 cm shortening of the lower layer of cables. Because the selfstress state is largely perturbed by this change, upper cables are relaxed by increasing their rest length of 33 mm in order to obtain realistic normal effort levels. In the end, the beam became an arc with a camber of 1120 mm stabilized by a selfstress state whose values range from 16.4 kN in compression to 22.2 kN for the more tense cable. Applying the same principle but between the two lower longitudinal cable lines, we can generate an arc in the horizontal plane. We present also in figure 6.b the case of a plane grid becoming a spatial barrel vault.

6. Validation on a static analysis case

Beyond the design possibilities, the approach of tensegrity systems using dynamic simulation presented in this paper allows a large range of analysis. Since stationary state is a particular case of dynamic motion, quasi-static analyses can be achieved using a dynamic relaxation mechanism that accelerates the convergence toward a static equilibrium.

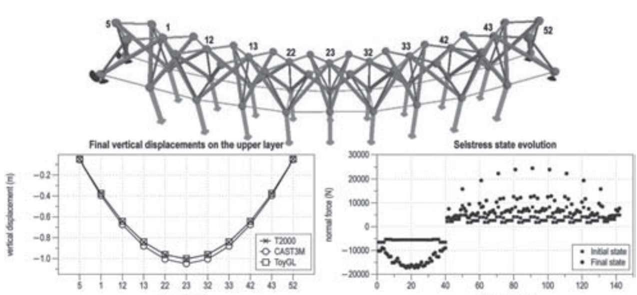

Figure 7: Quasi-static analysis results

To illustrate and validate this process, the tensegrity beam presented in figure 6 is taken as an application in a comparison between different codes. A selfstress state is given to the generated structure by a shortening of the cables joining the upper to the lower layer by 1 mm. This structure is simply supported at its ends and loaded with a vertical force of 100 N at each lower node (Figure 7). The results exhibit a rather sane behavior. Indeed, the level of selfstress increases in every element under the external load in conformity with their nature. This means that no cable slackening can happen and that the initial shape is that of minimum internal energy. Values from the displacements and selfstress are compared to those obtained from two other codes : Tensegrity 2000 (T2000), dedicated to tensegrity systems [14], and CAST3M, a general purpose research calculation code [17].

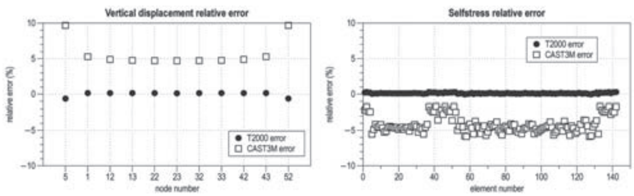

Figure 8: Comparison with T2000 and CAST3M codes

We can see in figure 8 that the differences on the final geometry and the selfstress state are very small when compared to T2000, which validates the proposed approach for this kind of analysis. Though, results from the CAST3M code differs more sensibly, which raises the difficulties in adapting a general code to all the mechanical qualities of tensegrity systems. Besides, if we look at the computation times, this method appears very competitive as a motionless equilibrium state can be reached within 1 to 2 seconds on a standard notebook computer in the case of this simple structure.

7. Kinematic study

A domain for which the dynamic simulation is particularly adapted is the analysis of mechanisms and kinematics, especially in the case of deployable structures. Indeed, the global stability must be studied for all intermediate geometric configurations and complex interactions, like contact, have to be considered. In addition, the actuators and devices that induce movements can be properly simulated with this approach. To illustrate this, we present in figure 9 the folding of the previously defined tensegrity beam formed by 10 quadruplex modules. Special behaviors are implemented in order to simulate active routing cables [18] that are passing thru several nodes along the structure, and to implement contact between elements.

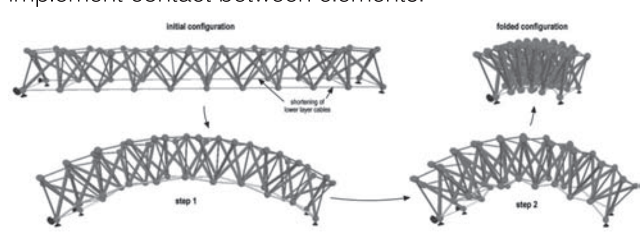

Figure 9: Folding steps of a 10 modules tensegrity beam

The shortening of the two continuous longitudinal cables in the lower layer controls the folding process presented in figure 9 that leads to a contraction to 25% of the initial length. Though, in the folded state, the selfstress state exhibits high normal forces values, which can't be realistic and have to be considered by relaxing upper cables during the process or using softer elements but with consequences on the static behavior.

8. Structural robustness

The dynamic approach presented here is also perfectly suited to study the effects of transitory loads or local failures. In figure 10, we present the qualitative results of simulated rupture in several sets of cables of the 10 modules tensegrity beam presented earlier, with colors illustrating the normal forces levels.

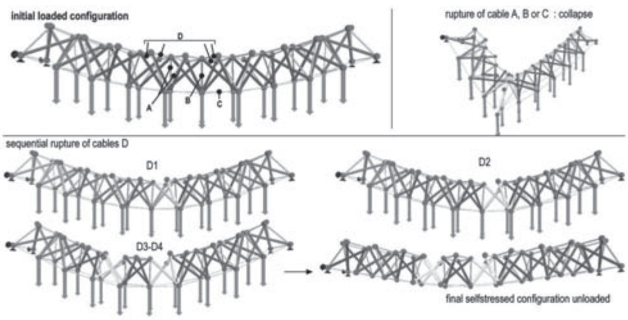

Figure 10: Evolution of the tensegrity beam under different local cable failure scenarios

These results confirm that failures in cables A-B (vertical cables) and C (lower layer) induce systematically a collapse of the structure. However, upper layer cables are less sensible and we observe that the system remains stable even after several upper cables failures, which provokes a vanishing of the local selfstress states associated to the concerned modules. Although we observe an appreciable robustness of the tensegrity beam, its stability still depends on the existence of a few components that are to be designed with higher exigencies.

Conclusions

We presented in this paper an explicit dynamic model and its use as a versatile method for the design and analysis of tensegrity systems. Because these systems are assimilated to discrete mass-spring systems, the simulation of their dynamic behavior can be implemented efficiently. Associating the calculations with an interactive visualization interface, it become possible to create and modify structures in real time, with a direct feedback on their behavior. This approach opens new possibilities, tending toward a more physically-based design, but also giving access to traditionally complex analysis, such as the assessment of kinematic solutions for deployment or structural robustness under accidental local failures.

The perspectives offered by this approach are vast. For instance, creep and thermal effects can be implemented in the local behaviors of elements, which may enrich the analysis range and the prediction of selfstress loss in the long term. This dynamic model can also be used in parallel with experimental vibration analysis to identify material parameters such as damping or to validate active control strategies [19][20] on lightweight tensegrity structures. This model can also be extended with the implementation of semi-rigid joints between bars, which may open to the study of flexible structures. Proof of its high flexibility, a version adapted to biomechanics is actually used for the simulation of living cells.

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Recent Development in Design of Stress Ribbon Pedestrian Bridges

Jiri Strasky[1]

Summary

Stress-ribbon bridges consist of very slender concrete deck segments placed over bearing cables in the shape of a catenary. These bridges are characterized by successive and complementary smooth curves that blend into the environment. A disadvantage of the classical stress-ribbon type structure is the need to resist very large horizontal forces at the abutments, which determines the economy of that solution in many cases. For that reason, two new self anchored structural systems that combine the stress-ribbon with the arches have been developed. In the first one the stress ribbon is supported by an arch, the second one combines a curved stress ribbon with a curved flat arch.

Keywords

stress ribbon, flat arch, external cable, precast segments, plan curvature, progressive erection.

Theme

development - design - analysis - construction - testing

1. Introduction

Stress-ribbon bridges consist of very slender concrete deck segments placed over bearing cables in the shape of a catenary. Prestressing the deck segments stiffens the structure, providing stability to the cables. These bridges are characterized by successive and complementary smooth curves - see Fig. 1.

The curves blend into the environment and the curved shape, the most simple and basic of structural solutions, clearly articulates the flow of internal forces. The main advantage of these structures is their minimal environmental impact because they use very little material and can be erected without falsework or shoring, which could disturb the natural environment. Because there are no bearings or expansion joints, the bridges require only minimal long-term maintenance. The bridges built in the Czech Republic and in the USA were discussed in a paper presented at the International Bridge Conference in Pittsburgh in 1999 [1]. Problems connected with the design and construction of the stress-ribbon structures and a survey of such bridges built all over the world is presented in [2].

Usually the deck of stress ribbon structures is assembled of precast segments of a double tee cross section that is stiffened by diaphragms at joints. The bearing and prestressing cables are situated in troughs made in edge girders [2]. Local bending moments that originated in the stress ribbon deck near supports are resisted by a cast-in-place saddles designed as partially prestressed members.

An excellent example of this type of the structure is the Lake Hodges Pedestrian Bridge recently completed in San Diego - see Figs. 1. This, so far the longest stress ribbon bridge, has three continuous spans of equal span length of 100.58 m. The sag in all spans is 1.41 m [4].

Experience with performance of bridges built so far and parametric dynamic analyses have proved that the stress ribbon deck can be even lighter. Therefore we have developed a new type of the cross section formed by slender slab that is carried and prestressed by external cables - see Fig. 2.

The stress ribbon is assembled of precast segments from high strength concrete; the external cables are formed by monostrands that are lead in stainless pipes. At supports, the pipes are composite with the stress ribbon deck and create a composite member that is able to resist a positive bending moment originating there - see Fig. 3.

Figure 1: Lake Hodges Pedestrian Bridge

2. Stress Ribbon Bridges Supported or Suspended on Arches

A disadvantage of the classical stress-ribbon type structure is the need to resist very large horizontal forces at the abutments, which determines the economy of that solution in many cases. For that reason, a new system that combines an arch with the stress-ribbon has been developed. The stress-ribbon is supported or is suspended on an arch - see Figs. 4 and 5. The structures form a self-anchoring system where the horizontal force from the stress-ribbon is transferred by inclined concrete struts to the foundation, where it is balanced against the horizontal component of the arch.

Figure 2: Segments on external cables

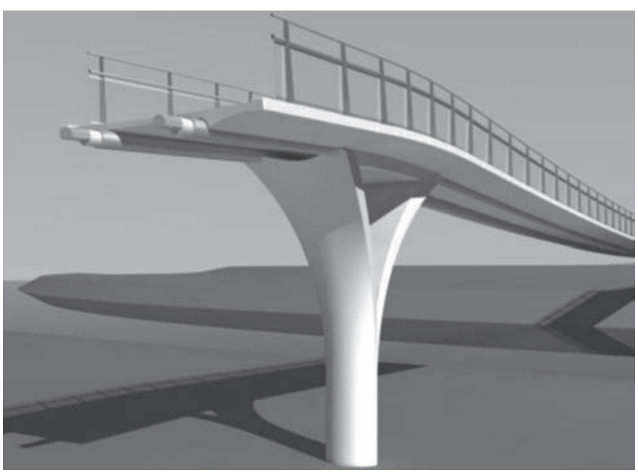

Figure 3: Stress ribbon & Pier table

The stress-ribbon structure supported by the arch was carefully analyzed and tested at the Brno University of Technology. Then the studied system was applied to a design of four bridges that have been built in the Czech Republic. A stress-ribbon structure suspended on two inclined arches was applied in the design of the 92-meter-long McLoughlin Bridge built in Portland, Oregon.

1.1 Structural system

The development of the self-anchored stress-ribbon structure supported by an arch is evident from Fig. 4. It is clear that the intermediate support of a multi-span stress-ribbon can also have the shape of an arch (see Fig. 4a). The arch serves as a saddle from which the stress-ribbon can rise during post-tensioning and during temperature drop, and where the center "band" can rest during a temperature rise.

In the initial stage, the stress-ribbon behaves as a two-span

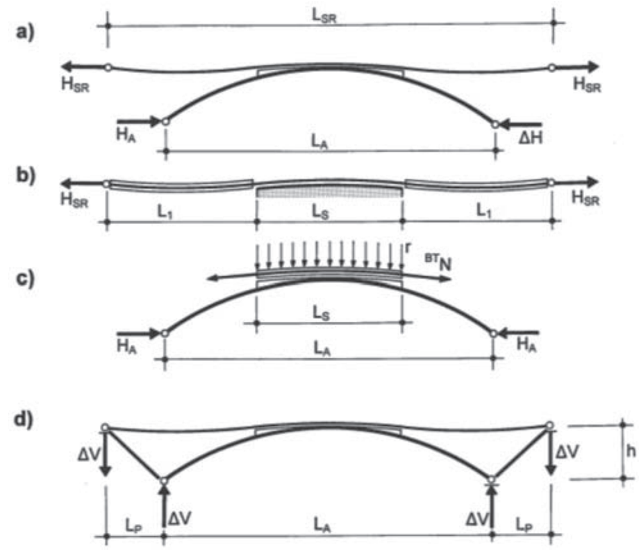

Figure 4: Stress ribbon supported by arch

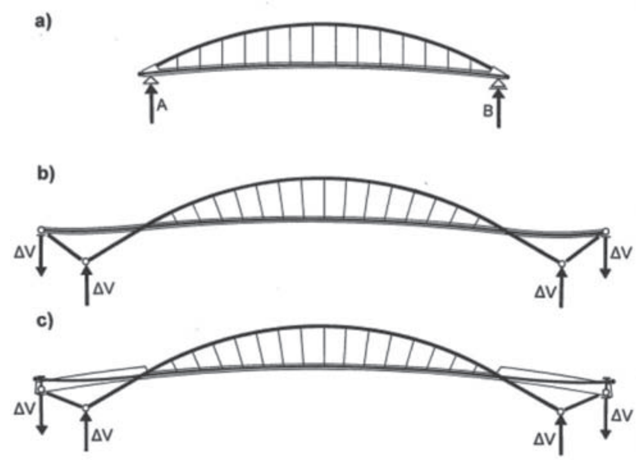

Fig. 5: Stress ribbon suspended on arch

cable supported by the saddle that is fixed to the end abutments (see Fig. 4b). The arch is loaded by its self weight, the weight of the saddle segments and the radial forces caused by the bearing tendons (see Fig. 4c). After post-tensioning the stress-ribbon with the prestressing tendons, the stress-ribbon and arch behave as one structure.

The shape and initial stresses in the stress-ribbon and in the arch can be chosen such a way that the horizontal forces in the stress-ribbon HSR and in the arch HA are the same. It is then possible to connect the stress-ribbon and arch footings with inclined compression struts that balance the horizontal forces. The moment created by horizontal forces HSR.h is then resisted by the ?V.LP. In this way a self-anchored system with only vertical reactions is created (see Fig. 4d).

It is also obvious that the stress-ribbon can be suspended from the arch. It is then possible to develop several self-anchored systems - see Fig. 5. Fig. 5a shows an arch fixed at the anchor blocks of the slender prestressed concrete deck. The arch is loaded not only by its self weight and that of the stress-ribbon, but also with the radial forces of the prestressing tendons. Fig. 5b shows a structure that has a similar static behavior as the structure presented in Fig. 4d. Fig. 5c shows a similar structure in which the slender prestressed concrete band has increased bending stiffness in the portion of the structure not suspended from the arch.

1.2 Model test

The author believes that a structural system made up of a stress-ribbon supported by an arch increases the potential application of stress-ribbon structures. Several analyses were undertaken to verify this. The structures were checked not only with detailed static and dynamic analysis, but also on static and full aeroelastic models. The tests verified the design assumptions and behavior of the structure under wind loading that determined the ultimate capacity of the full system.

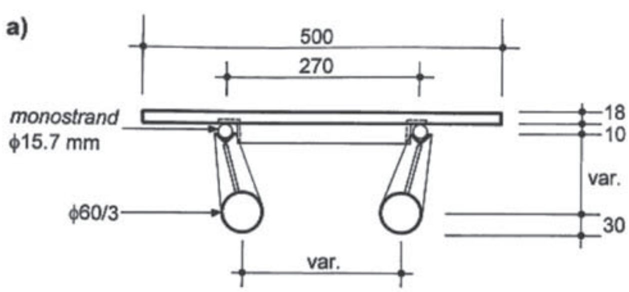

Figure 6: Static model - cross section

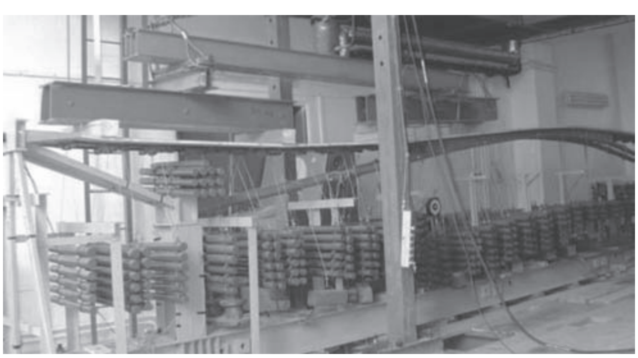

Figure 7: Ultimate load

The model tests were done for a proposed pedestrian bridge across the Radbuza River in Plzen, Czech Republic. This structure was designed to combine a steel pipe arch having a span length of 77 m and the deck assembled of precast segments. The static physical model was done in a 1:10 scale. The shape is shown in Figs. 6 and 7. The test has proved that the analytical model can accurately describe the static function of the structure both at the service and ultimate load. Results of the tests were utilized in a design of following structures.

1.3 Olomouc Pedestrian Bridge, Czech Republic

The bridge crosses expressway R3508 near a city of

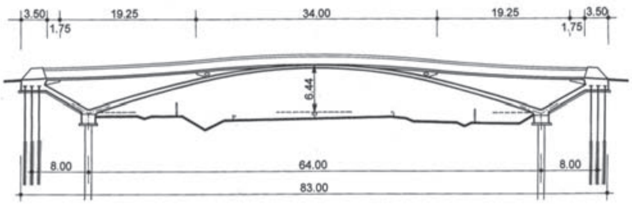

Figure 8: Elevation

Olomouc. The bridge is formed by a stress-ribbon of two spans that is supported by an arch (see Fig. 8). The stress-ribbon of the length of 76.50 m is assembled of precast segments 3.00 m long supported and prestressed by two external tendons (see Fig. 9). The precast deck segments and precast end struts consist of high-strength concrete of a characteristic strength of 80 MPa. The cast-in-place arch consists of high-strength concrete of a characteristic strength of 70 MPa. The external cables are formed by two bundles of 31-0.6" diameter monostrands grouted inside stainless steel pipes. They are anchored at the end abutments and are deviated on saddles formed by the arch crown and short spandrel walls. The steel pipes are connected to the deck segments by bolts located in the joints between the segments - see Fig. 2. At the abutments, the tendons are supported by short saddles formed by cantilevers that protrude from the anchor blocks. The stress-ribbon and arch are mutually connected at the central of the bridge. The arch footings are founded on drilled shafts and the anchor blocks on micro-piles. Although the structure is extremely slender, the users do not have an unpleasant feeling when standing or walking on the bridge. The bridge was built in 2007

Figure 9: Completed structure

1.4 Pedestrian Bridge across the Svratka River in Brno, Czech Republic

Another such bridge was built across the Svratka River in a city of Brno, Czech Republic. Close to the bridge an old multi span arch bridge with piers in the river is situated. It was evident that a new bridge should also be formed by an arch structure, however, with bold span without piers in the river bed. Due to poor geotechnical conditions, a traditional arch structure that requires resisting of a large horizontal force would be too expensive. Therefore, the self anchored stress ribbon & arch structure represents a logical solution - see Fig. 10.

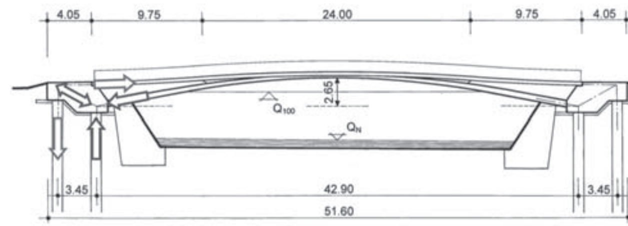

Figure 10: Completed bridge and elevation

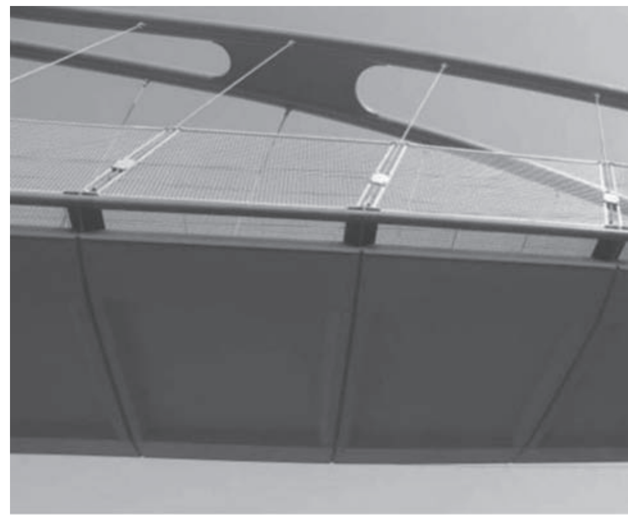

Figure 11: Suspension of the deck

Since the riverbanks are formed by old stone walls, the end abutments are situated beyond these walls. The abutments are supported by pairs of drilled shafts. The rear shafts are stressed by tension forces, the front shafts are stressed by compression forces. This couple of forces balances a couple of tension and compression forces originating in the stress ribbon and arch. The arch span L = 42.90 m, its rise f = 2.65 m, rise to span ration f/L = 1/16.19. The arch is formed by two legs that have a variable mutual distance and merge at the arch springs. The 43.50 m long stress-ribbon is assembled of segments of length of 1.5 m. In the middle portion of the bridge the stress ribbon is supported by low spandrel walls. The stress ribbon is carried and prestressed by four internal tendons of 12 0.6" dia monostrands grouted in PE ducts. The segments have variable depth with a curved soffit. The stress-ribbon and the arch were made from high-strength concrete of the characteristic strength of 80 MPa.

Although the structure is extremely slender, and first bending frequencies are close to 2 Hz, the users do not have an unpleasant feeling when standing or walking on the bridge. The bridge was built in 2007.

1.5 McLoughlin Boulevard Pedestrian Bridge, Portland, Oregon, USA

The McLoughlin Boulevard Pedestrian Bridge (see Fig. 12) is a part of a regional mixed-use trail in the Portland, Oregon metropolitan area. The bridge is formed by a stress-ribbon deck that is suspended on two inclined arches. Since the stress- ribbon anchor blocks are connected to the arch footings by struts, the structure forms a self-anchored system that loads the footing by vertical reactions only (see Fig. 5c). The deck is suspended on arches via suspenders of a radial arrangement; therefore, the steel arches have a funicular/circular shape.

The stress-ribbon deck is assembled from precast segments and a composite deck slab. In side spans, the segments are strengthened by edge composite girders. The deck tension due to dead load is resisted by bearing tendons. The tension due to live load is resisted by the stress-ribbon deck being prestressed by prestressing tendons. Both bearing and prestressing tendons are situated in the composite slab. The bearing tendons that were post-tensioned during the erection of the deck are formed by two bundles of 12 by 0.6" diameter strands that are protected by the cast-in-place slab; deck prestressing tendons are formed by six bundles of 10 by 0.6" diameter tendons that are grouted in ducts.

Edge pipes and rod suspenders make up part of the simple hanger system - see Fig. 11. The suspenders connect to "flying" floor beams cantilevered from the deck panels to provide the required path clearance. The edge pipes contain a small tension rod that resists the lateral force from the inclined suspenders on the end of the floor beams. Grating is used to span the gap between the edge pipe and the deck panels. Construction of the bridge commenced in March, 2005, and was completed in September, 2006.

3. Curved Stress ribbon & Flat arch bridges

Recently, several noteworthy curved pedestrian bridges, which decks are suspended on their inner edges on suspension or stay cables, have been built. However, curved stress ribbon and/or flat arch bridges have not been built so far. Therefore we have decided to study these structures analytically and verified their function on a static model.

The above curved structures have to be designed in such a way that for the dead load there is no torsion in the deck. One possibility is to supplement the deck with stiff L shaped members and suspend the deck at their top portions. The cable geometry has to be designed in such a way that suspenders or stay cables direct to the deck's shear centers. This approach was also used in a design of our stress ribbon structure - see Fig. 13. The structure is formed by slender deck that is supplemented by steel L frames supporting the slab. The tops of their vertical portion are connected by steel pipes in which the strands are placed. Radial forces originating in the strands during their post-tensioning load the structure - see Fig. 13b. The vertical components of these radial forces balance the dead load - see Fig. 13c, the horizontal components balance the dead load's torsional moment and load the structure by horizontal radial forces - see Fig. 13d. Since the stress ribbon is fixed into the abutments, these forces create uniform compression in the deck.

Figure 12: Completed bridge

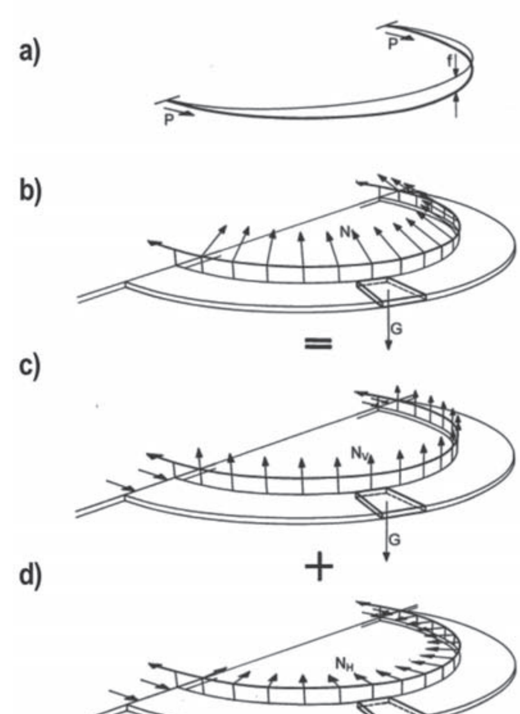

Figure 13: Curved stress ribbon

Our design has been developed from the above approach. However, for resisting of the effects of the live load, we have to add additional cables that have very complicated arrangement. Therefore we have decided to add a torsionally stiff member of the pentagon cross section. Resulting arrangement of the structure is evident from Figs. 14. The curved pedestrian bridge of the span of 45 m is in a plan curvature that in pathway's axis has a radius of 45 m. The maximum longitudinal slope at the abutments is 7%. Both the concrete slab and the steel girder are fixed into the anchor blocks. The external cables are situated in the handrail pipe and are anchored in end concrete walls that are fixed into the anchor blocks. Horizontal forces are resisted by battered micropiles - see Fig. 15a.

The detailed static and dynamic analyses have proved that the structure is able to resist all design loads. The first bending frequency is $f1 = 1.437$ Hz. The forced vibration done

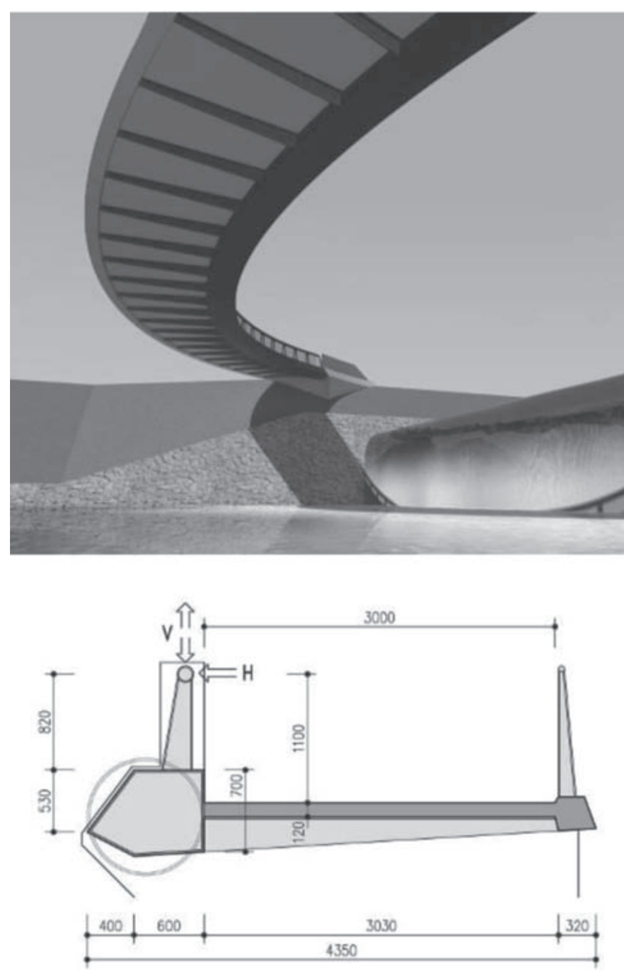

Figure 14: Stress ribbon & arch structure

according to [3] has caused maximum acceleration amax = 0.578 m/s2 that is close to allowable acceleration alim = 0.589 m/s2. Therefore we propose using two dampers, to be sure the pedestrians would not have an unpleasant feeling when standing or walking on the bridge.

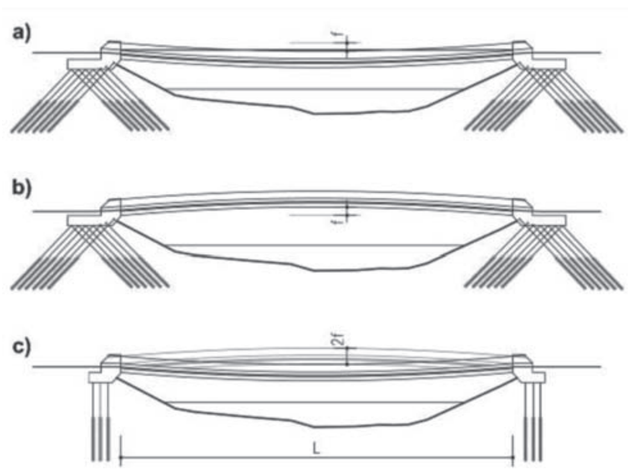

Figure 15: Stress ribbon & arch structure

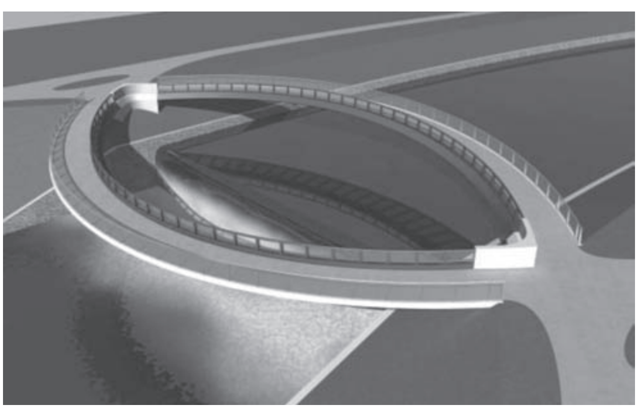

Figure 16: Stress ribbon & arch structure

The function of the studied structure was verified on a static model built in the scale 1:6 - see Figs. 17 and 18. To reduce the longitudinal horizontal force, the curved stress ribbon was tested together with a structure formed by a curved flat arch of a similar arrangement. It is evident that actual structure can be built similarly. The horizontal force from the stress ribbon can be balanced by the horizontal force originating at the flat arch - see Fig. 15b. In this way a very economic structural system can be created. The analyses according to [3] have proved that the dynamic behavior of the curved flat arch is similar to the curved stress ribbon structure; the first bending frequency f1 = 1.437 Hz.

To simplify construction of the model, the steel box of the stress ribbon and flat arch was substituted by steel pipe that were fixed into the end anchor blocks. The steel pipes were produced with steel L shaped members. Their horizontal part supported the concrete slab; their vertical parts supported steel pipes in which the monostrands were placed.

To guarantee a model similarity, the curved steel pipes and transverse L members were loaded by concrete block and steel rods representing the self weight of the structure. This load was suspended on the steel structure. The live load was represented by additional concrete block and cylinders placed on the stress ribbon deck.

The construction of the model was carefully monitored. The structure was tested for 2x3 positions of the live load (see Fig. 17) and for the ultimate load (see Fig. 18). The live load was situated on the left side, right side on the whole length of the deck. The measured deformations and strains were in a good agreement with the results of the static analysis. At the end, the model was loaded by an ultimate torsional load that was situated along whole length of both structures. The both structures proved that have a sufficient margin of safety.

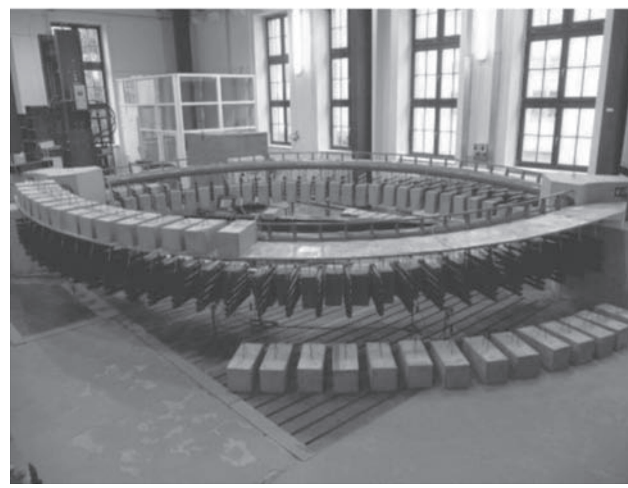

Figure 17: Model test - Service load

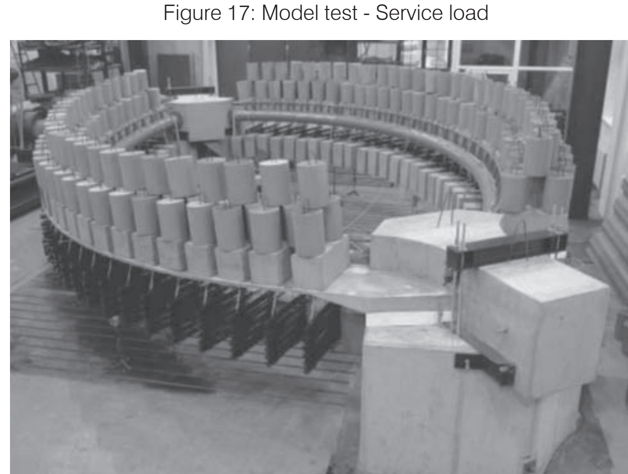

Figure 18: Model test - Ultimate load

Conclusions

The continuous development of the stress ribbon structures is being done at the Faculty of Civil Engineering of the Brno University of Technology with a collaboration of engineering firm Strasky, Husty and Partners, Brno. The design of

the bridge utilized results of the research projects of the Czech Ministry of Industry FI - IM5/128 "Progressive Structures from High-performance Concrete". Paper originated with the financial support of the Ministry of Education, Youth and Sports of the Czech Republic, project No. 1M0579, within activities of the CIDEAS research centre.

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Solar Updraft Power Plants and Solar Chimneys (Power Towers)

Wilfried B. Krätzig[1]; Martin Graffmann [2], Reinhard Harte [3], Ralf Wörmann [4]

Abstract

The present lecture will start with an explanation of the working principles of Solar Updraft Power Plants, in which Power Towers form the huge central chimneys as air pressure sinks. It then elaborates on the loadings of these dominating RC shell structures, detailing the wind load as the dominating action. After this, we change to response characteristics of power towers, followed by their essential design principles. Further, vibration modes and instability phenomena are explained, and the non-linear structural response behavior - materially and geometrically combined - is exemplified as indicator for determination of failure limit states of such structures with design-service-lives of around 100 years. After mentioning the favorable electricity costs we close with further advantages of solar updraft power plants compared to other renewable energy sources.

Keywords

RC shell structures, renewable energies, high-rise towers, shell instabilities, shell dynamics.

Theme

Industrial buildings - construction - wind loading - reinforced concrete

1 Introduction: What are Solar Updraft Power Plants?

Solar Updraft Power Plants (SUPPs) like in Figure 1 form the most sustainable and economic resources for natural electric power generation in arid zones, presently known. They work free of CO2-emissions, using solar irradiation as fuel. The CO2-footprint from plant construction and operation delivers 10 to 20 g CO2/kWh of produced electricity, depending on the plants' design-service-lives. (Modern coal power plants produce around 750 g CO2/kWh). The leveled electricity costs drop down below unbelievable 10 €ct/kWh.

Their general working concept is illustrated in Figure 2 (Schlaich [8]). Such power plant essentially consists of the collector area (CA), the turbo-generators as power conversion units (PCUs), and the solar chimney (SC). In the CA, a large glass-covered area, solar UV-irradiation heats the collector ground as absorber and consequently warms up the air inside the collector, which streams towards its center. There, in the PCUs, the kinetic energy of the permanent stream of warm air is partly transformed into electric power. Source of the gain of kinetic energy is the uplift of the warm air in the huge SC, the power tower, creating a pressure sink at the PCUs' outlet. The updraft in the SC is roughly spoken proportional to the tower height.

Such SUPPs are supposed to deliver sufficient efficiency in locations with high solar irradiation of more than 2.0 MWh/m2a, valid in the earth's sun-belt between ±35° latitude. The efficiency of a SUPP there depends on the size of the CA (air temperature) and on the height of the SC (air pressure difference): A plant with CA diameter of 7.000 m and with SC height of 1.500 m is estimated to deliver a maximum electric power output of about 400 MWp, on mid-days in summer time. For a smaller SUPP in the KALAHARI desert in South Africa, the power output over 24 hours for different months of the year is shown in Figure 3 (Pretorius [6]).

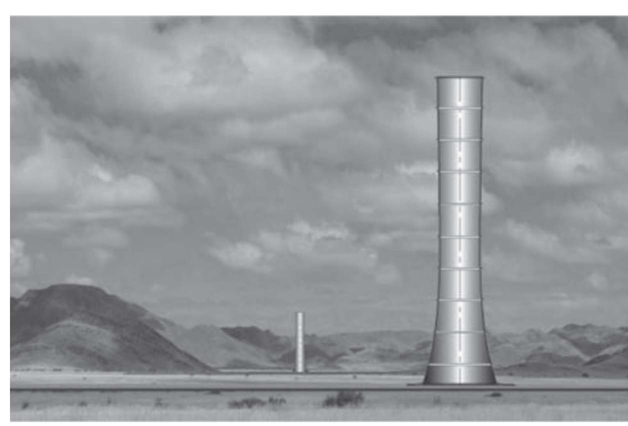

Figure 1: Computer vision of solar updraft power plants

From this typical power output curves of SUPPs we observe, that obviously this power generating technique possesses a natural storage property: It also delivers electric power after sun set. This heat storage property can be extended by various technical means - a horizontally separated CA or controllable water storage devices - such that SUPPs can serve base load power as well as peak load one (Pretorius [6]).

Solar updraft power generation was first proposed in 1903

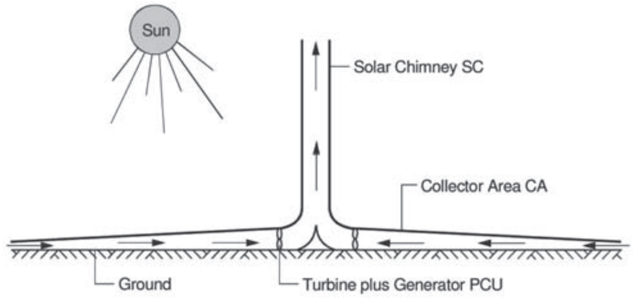

Figure 2: Schematic solar upwind power plant

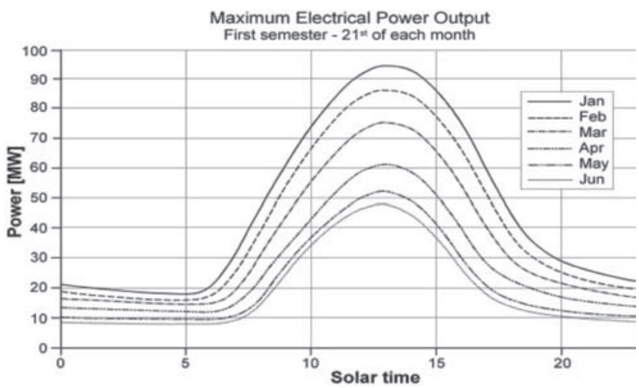

Figure 3: Power output of SUPP from summer to winter seasons

by the Spanish officer I. CABANYES. Starting in 1982, the German engineering professor J. SCHLAICH constructed the first experimental SUPP with a 200 m high SC, a 240 m wide CA, and a maximum power output of 50 kW in MANZANARES/SPAIN, see Figure 4. SCHLAICH's team operated this plant very successfully for more than 7 years, such that the basic data source for this modern type of power generation stems from the 1990s (Schlaich [8]). Recent aspects of SUPPs are summarized in (Schlaich et al. [7]), actual ones in (Wikipedia: Solar updraft towers) including a discussion of their pros and cons. In spite of the mentioned predominantly positive experiences with the MANZANARES experimental plant, regrettably no professional SUPP has ever been erected up to now.

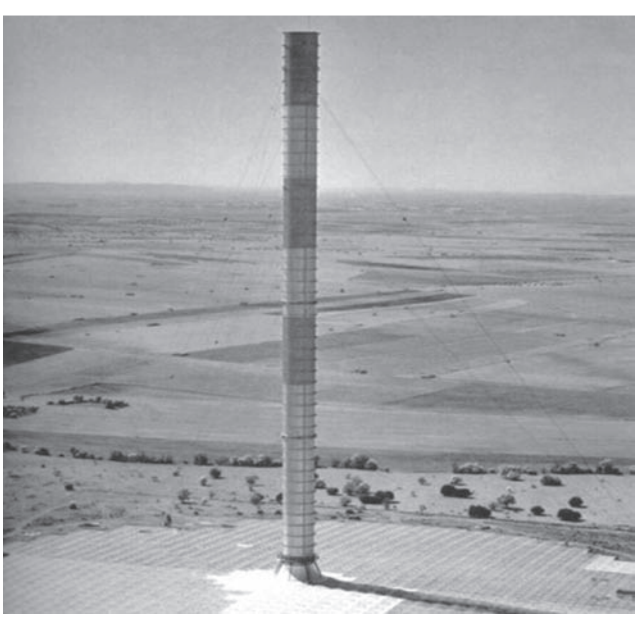

Figure 4: J. Schlaich's SUPP prototype from 1982 at Manzanares/ Spain

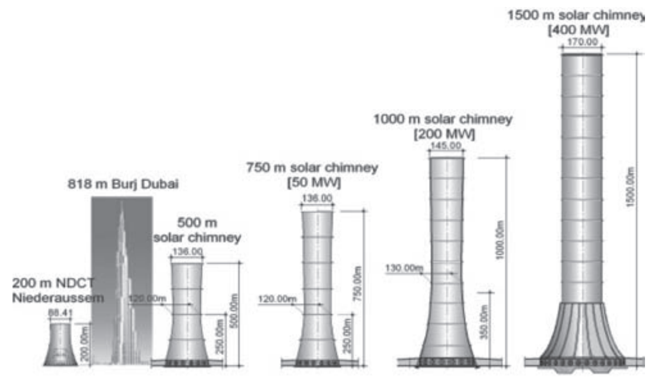

Figure 5: From the world-highest Natural draught cooling tower to pre-designs of future SUPPs

2 Loadings of power towers

Characteristic landmarks of SUPPs as in Figure 1 and 5 are the huge central SCs or power towers. Depending on the planned power output, they are designed as thin RC shell reaching to enormous heights, as Figure 5 elucidates. Such thin shell structures are loaded by the following load actions:

- dead weight D,
- Wind loading W,
- Temperature effects T from arising production states of the plant,
- Earthquake excitations E in case of seismically sensitive plant locations,
- Soil settlements S of external origin, unfavourably to SCs because of their considerable size,
- Construction loads M from guys of the central top-slewing crane serving the construction site on tower top, from additional deposit platforms, from elevators for material and personal transportation, or from anchoring of the self-climbing scaffold and formwork.

Due to the enormously planned heights of SCs (Figure 5), the wind actions form by far the most significant loading element in the tower design. Wind loading consists of the external pressure distribution W_e

$$w_e(z,\theta) = C_{peo}(\theta).\varphi.q_b(z), \qquad (1)$$

and the internal suction W_i

$$w_i = s_i = c_{pi} \cdot q_b(H). \qquad (2)$$

In (1), cpe(O) abbreviates the normalized pressure distribution over the circumference O, R the dynamic amplification, and qb(z) the design wind pressure over height z. In (2), cpi stands for the internal suction coefficient and qb(H) for the stagnation pressure on tower top.

The wind structure above 250 m, especially the stochastic modelling of its gustiness, is rather unknown in their properties as loading element. For a first attempt, qb(z) can be taken as the envelope of the maximum velocity pressure values over tower height. Such approximation will overestimate the wind actions because of the gustiness, a stochastic loading process in time and space. As remedy to overcome this uncertainty, we prefer experimental measurements in boundary layer wind-tunnels, in which the wind-flow are modelled with the same statistical mean values as in nature. The great importance of a correct understanding of the wind gustiness for a safe tower design can be visualized from the vibration properties in Figures 10, 11 with severe interactions of membrane and bending vibrations [van Koten et al. [2]).

3. Typical response characteristics and design consequences

Figure 6 represents a pre-design of Kratzig & Partners Consultant Engineers, a SUPP with a power tower of 1 000 m of height. Combined with a collector of 6 000 m of diameter this plant shall produce a peak power of 200 MWp (equal to an annual work of 600 GWh/a) for 2.240 MWh/m2a of solar irradiation. At the tower waist at 400 m of elevation its shell diameter is 133 m wide, at the upper rim 145 m. Below 400 m the tower shell widens in a strength-optimized hyperbolic shape to a foot diameter of 260 m. The wall thickness of the highperformance RC 70/85 varies from 0.25 m to 0.65 m, as plotted in Figure 6 on the right. In addition to the upper edge member, 9 intermediate and pre-stressed RC ring-stiffeners are fixed on the outer shell face.

16 turbo-generators deliver the plant power, they are placed directly on the tower foundation around the footing perimeter, to reduce air inflow losses and avoid differential settlements between machinery and tower (Backstrom et al [1]). The foundation itself is constructed as a closed ring of around 20 m of breads, depending of the soil conditions.

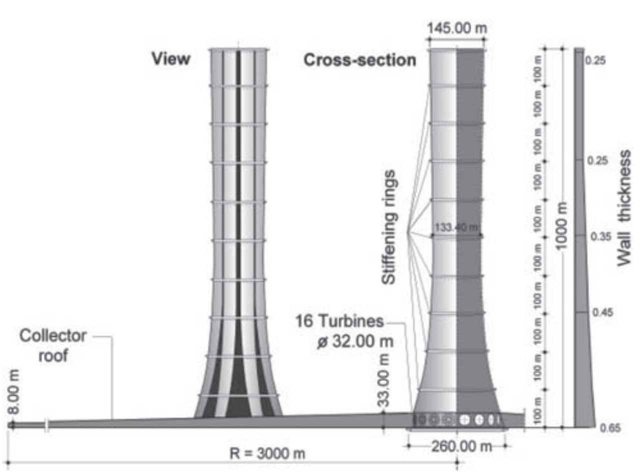

Figure 6: SUPP with SC of 1 000 m of height for a 200 MWp solar plant

The lower widening of the shell from 130 m in the throat to 260 m at the base serves two important purposes:

a) it provides the space for the guide vanes in the SC, fabricated of Teflon-coated glass-fiber membranes, to take care for an injection of the warm air with a minimum of losses, and

b) it reduces strongly the wind shear stress between the air inlets by suitable meridional shape optimization. But such shaping of the shell alone would never deliver an economic tower design. In addition, the shell structure requires the mentioned strong ring-stiffeners in order to constraint the meridional wind forces from shell-like distributions towards beam-like ones, and to guarantee a sufficient buckling safety of the SC.

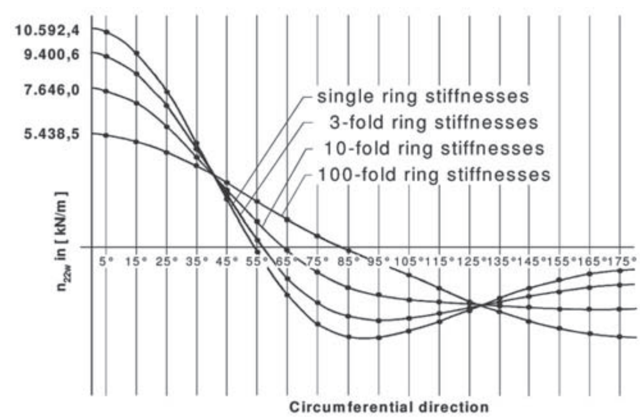

Figure 7. Distribution of meridional wind forces at 280 m of elevation for the 1 000 m tower with different ring stiffness

First attempts of this tedious optimization process are shown in Figure 7. There several distributions of meridional wind forces at 280 m of tower height are plotted in dependence of the varying equal stiffness of all 9 applied ring beams. The reference (single) beam stiffness belongs to a cross-section of 0.40 m 3.00 m, increasing from top of Figure 7 to below over 3-fold, 10-fold finally to 100-fold values. In this Figure one observes, how the wind force distributions approach the cosine-shaped optimum of beam-like stresses. With these changes likewise the maximum tension stresses are reduced, which are chiefly responsible for most of the meridional reinforcement in the tower wall.

Figure 8 elucidates the same optimization process for the selected tower, plotting the maximum tension forces from wind load n22W against the tower height. Compared to the (negative) dead load forces -n22D one observes, how the tension stresses can be minimized by increase of the ring-stiffness, likewise the amount of meridional reinforcement is reduced, an important cost argument. So finally we accentuate, that any global optimization of a power tower comprises the main global tower dimensions, the meridional shape of the SC as well as the stiffening rings. Such cost-optimized designs of SCs are based on rather low shell thicknesses because of the transportation and built-in costs of the in situ concrete at great heights. But such savings require ring-stiffeners of increased stiffness, an enormous technical construction challenge.

The design of such a SC in Europe clearly is based on the safety concept of the Euro Codes, especially on EC 2. So it distinguishes limit states of serviceability and limit states of failure. The German cooling tower guideline (VGB-610 Ue [10]) provides valuable supplements for thin shell structures. Structural analysis and dimensioning are based on large integrated FE models with around 200 000 dofs, in which all foundation properties are included because of possible influences on such large structures (Kratzig et al. [3]). A real challenge of all detailed design considerations is the life-durations of power towers of 100 years.

5 Instability and vibration behavior of solar chimneys

Tower instabilities and tower dynamics require extremely

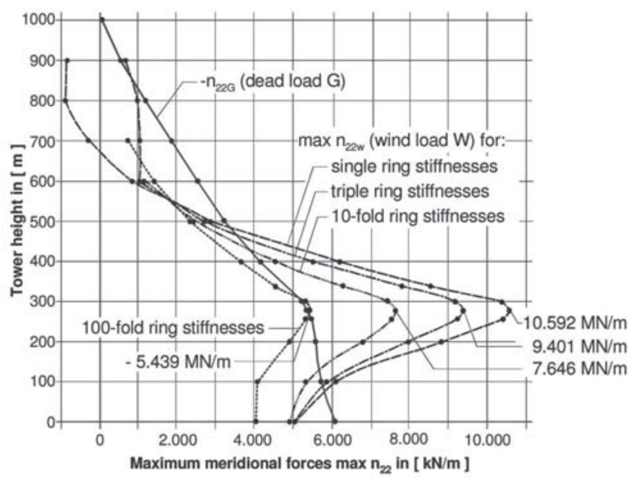

Figure 8: Maximum wind tension forces versus tower height for different ring stiffness

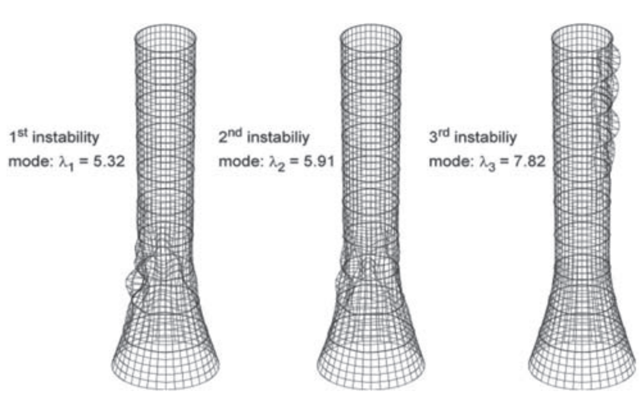

Figure 9: First three instability modes of the tower of Figure 6 under load combination ($D+W_e+W_i$)

detailed investigations in the design process. An interesting insight into the load-bearing behavior of power towers can be gained from those instability modes and their corresponding buckling safeties. For elucidation, the three lowest modes of the SC from Figure 6 are pictured on Figure 9. Under the load combination (Dead weight D + External wind We + Internal suction Wi), structural parts with high compression in the lower third of the tower tend to instability, by expectation. But instability deformations can also be observed in the shell fields below tower top, there a consequence of the chosen low shell thickness. This behavior requires strong ring-stiffeners. It should be mentioned that for construction processes using slip-form-techniques, the shell thickness there should be increased.

This brief overview on linear instability phenomena gives evidence to the suspicion that solar towers may be rather sensitive to random geometrical imperfections in each section of their height, which possibly can be built-in during the construction process of the chimney. Presently detailed studies hereto are missing, so we recommend as imperfection limits the restrictions from the German cooling tower guideline: Due to this, maximum imperfections shall not exceed half of the shell thickness at each position (VGB-610 Ue, [10]). To be fair we should emphasize, that this important topic for shell instability belongs to those questions, which are to a high extend presently rather unanswered for solar towers.

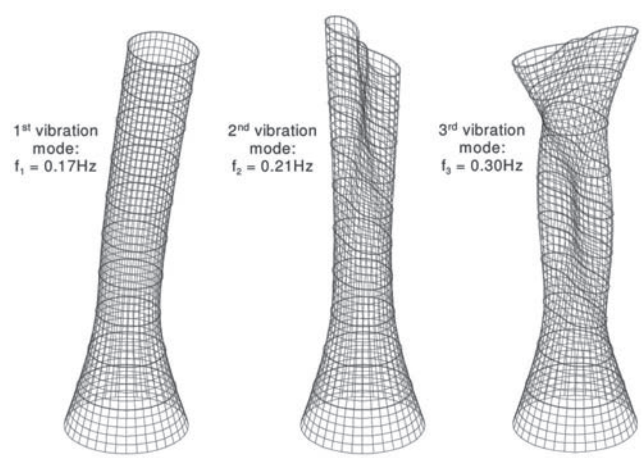

Figure 10: First three natural vibration modes of 1 000 m tower

Further interesting response aspects can be observed from the natural vibration behavior of power towers, e.g. from the vibration modes for that 1 000 m SC from Figure 6, plotted on Figures 10 and 11. As well known, any dynamic wind response can be decomposed into the elements of a function space containing a closed set of natural vibration modes as basis, so these mode shapes describe the dynamic deformation capabilities of a power tower. The lowest vibration mode on Figure 10 obviously characterizes a beam-like dynamic behavior, the next ones are typical shell-like deformation. In wind-induced vibrations both types are coupled, such that membrane and bending shell responses

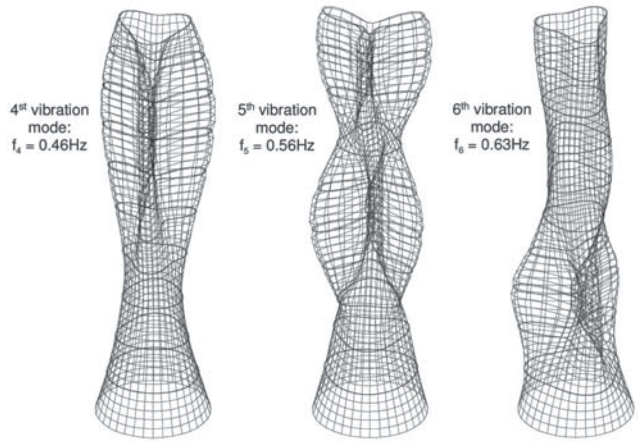

Figure 11: Fourth to sixth vibration mode of 1 000 m tower

appear in combined manner. Obviously from those many sign changes of the shell vibration modes over tower height, observable especially in Figure 11, we conclude, that there may appear complicated local vibrations from gust excitations, which renders the dynamic wind response of SCs unintelligible: Somewhere over the tower height of 1 000 m there might always be some suitable wind gustiness to excite the tower shell.

Because of the great tower size, the natural vibration frequencies are extremely low, so they are much more closely neighbored to the maximum of the VON KARMAN wind power spectrum than for usual engineering structures. Thus storm actions with possible local cracking will endanger these structures more severely, they have to be tackled with great care and increased safety requirements.

On the other hand, coherent seismic excitations will not lead to severe stresses, since the natural frequencies are far away from the maximum of seismic power spectra. If a power tower is damaged by ageing or deterioration, wind actions generally will increase (high tuned for wind), while seismic responses will decrease (low tuned for earthquakes). But this recognition holds only for coherent (synchronous) seismic excitations. Non-coherent (a-synchronous) seismicity in contrast may lead to high stresses from travelling waves in the tower footing, which renders underground conditions safety-relevant. In addition, the extremely low natural frequencies of SCs determine unmistakably evidence of danger of kinetic instabilities under wind excitation.

5 Nonlinear tower response

Linear elastic analyses of power towers, treated up to now, deliver only crude approximations of real failure responses. This holds also true for their instabilities. Closer to real failure behavior are non-linear simulations, well suited to the constitutive behavior of RCs. As well known, this property results in non-linear responses (material nonlinearity), at least beyond certain wind-load intensities. Also aging of the material leads to nonlinear processes, an important aspect for the intended high life-duration of 100 years. In addition, in the column-like SC second order effects might be activated during wind deformation (geometrical non-linearity), so any careful response study of them should comprise both kinds of non-linearity.

In contrast to classical linear elastic FE-technology by inclusion of material nonlinearities, all applied 2D finite shell elements need to have at least a virtually layered structure. This means, that each element GAUSS-point consists of a chain of integration sub-points, one in each layer. At these sub-points, on material point level, the constitutive relations of the material in question including their deterioration components have to be coded and numerically evaluated (Kratzig et al. [4]). The material point level, our lowest macroscopic simulation level, collects all constitutive information usually in a complete 3D environment xi: x1,x2,x3. Suitable 3D constitutive laws for material components including deterioration descriptions because of ageing have to be described there and then mapped into 2D. Certainly only those material properties can be incorporated via the computer analysis into the final response, which are included at this level. For the RC simulations applied here these material properties are: nonlinear elastic-plastic - -behavior for concrete in compression including material micro-cracking, leading to stress release in the post-strength range,

- elastic-brittle behavior for concrete in tension with rather low tension strength leading to macroscopic crack-damage, modeled here in smeared crack manner,
- elastic-plastic material modeling for reinforcement steel,
- nonlinear inelastic bond behavior between steel and concrete, limited by inelastic slip damage.

The final FE-analysis follows a multi-level iteration technique as described first in (Kratzig [5]). For each new load-step, the iteration starts with a new global vector of out-of-balance forces, climbing down at each element in each GAUSS-point via increments of the global degrees of freedom, then increments of the element degrees of freedom, finally to increments of the continuum strains at material

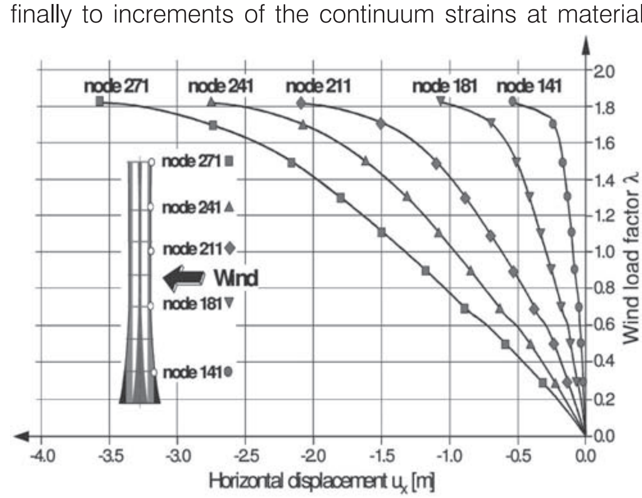

Figure 12: Non-linear tower displacements versus wind-load

point level. There the coded constitutive laws transform strain increments into stress ones, after which the path returns back up again to the global structural level for the next iteration step, with an improved new internal force vector. Even with very fast PCs, such analyses require rather intensive computing efforts.

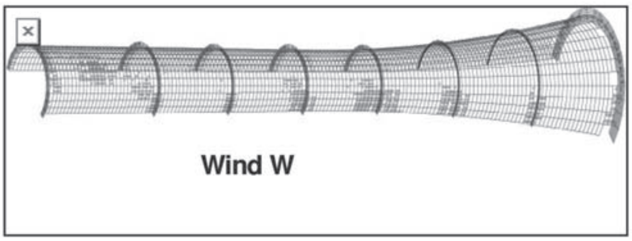

Figure 13: Simulated crack patterns just before failure

By such an analysis technique one can follow a certain load combination, e.g. D + We , up to structural failure of the SC. Figure 12 demonstrates the result for a rather slim 750 m power tower demonstrating a final displacement of 3.60 m at the top, just before failure at a wind-load factor of = 1.85. This factor is related to the standard wind (= 1): If interpreted as global safety factor, it seems just sufficient for a safe design.

In order to recognize the reasons for the obviously non-linear response, we record the RC tension strains during the simulation, interpreting values above tension strength as cracks with corresponding crack widths. The crack patterns of such simulation are responsible for the non-linear behavior, in Figure 13 they are shown for the last equilibrium state, just before failure. Due to the pre-stressing of the ring stiffeners, most crackpatterns in the tower shell are located on luff-side, in the ring-stiffeners on lee-side.

Such cracking studies had been applied in this design to further reinforce areas with early cracks, in order to increase systematically failure safety and life-duration of the power tower. As a whole, non-linear simulations provide excellent information about the real deformation behavior of SCs, and about the stress-redistributions under cracking, which enable us to achieve very economic and sufficiently safe designs.

6 Summary and outlook on solar updraft power technology

The present paper offered an overview over some response and design problems of solar chimneys, the central air pressure motor of solar updraft power plants. As the reader will agree, these shell structures contain enormous engineering challenges, solely because of their great height. As just described, further considerable efforts are required, until sufficiently safe and economic towers can be constructed. But looking into the future of SUPPs, they will comprise a great step in mankind's strive for endurable and sustainable supply of electric power. Although their efficiency is rather low, due to the low temperatures of their working medium air, SUPPs contain several severe advantages, compared to other renewable power technologies:

- Despite of relatively high investments, the production prices for electricity are extremely reasonable: The leveled electricity costs LECs can be pushed down below 0.09 €/kWh.
- The process in question requires no water, neither for steam formation nor for cooling of the worked-off steam, so SUPPs are the only plant type employable in arid areas with typically rare water resources;
- SUPPs possess a natural storage capability of heat, which easily can be extended to energy-storage for 24 and more hours;

Further important aspects are beyond the described purposes of SUPPs as pure electricity plants. The climate in the outer collector area favors a use as greenhouse, ideally for farming and agriculture. Thereby not only additional CO2 can be chemically bounded in the harvested crops, mankind can also develop a new important source for food-supply and survival by SUPPs (Stinnes [9]).

Acknowledgement

The present manuscript extends the work of Kratzig, Harte, Graffmann, Montag: Power Towers: On structural engineering problems of solar updraft chimneys, presented at the SEMC 2010 in Cape Town. Intensive enduring co-operation with Prof. Dr-Ing. Hans-Jurgen Niemann, Niemann & Partners Engineering Consultants, Bochum, in the field of wind actions on power towers is gratefully acknowledged.

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Performance of High Rise Buildings under the February 27th 2010 Chilean Earthquake

René Lagos C[1]

Abstract

The current practice of seismic design of high rise reinforced concrete buildings in Chile is reviewed. After the March 3rd1985 Chilean earthquake (Ms 7.8)9,974 residential buildings over 3 stories high were built in the central area of the country affected bythe February 27th 2010 earthquake(Mw 8.8). Of this, 20% had 10 stories or more and an estimate of 3% had over 20 stories up to 52, the tallest at the time of the earthquake. Statistics indicate 4 buildings collapsed (between 4 to 18 stories), and about 40 buildings were severely damaged and had to be demolished. This represents less than 1% of the total number of new residential buildings built during this period in the affected area.

These numbers show that shear wall buildings constructed in Chile performed well.Nevertheless, the earthquake produced significantstructural damage on somenew high rise shear wall buildings in Santiago, Viña del Mar, Chillán and Concepción.

Observed damage was concentrated near the base, on L or T shape walls, presentingcrushing and spalling of concrete and buckling of vertical reinforcement at boundary regions, extending horizontally deep into the wall length. Vertically the observed damage only affects a small portion of the story high. Shear failures at the web were also observed.

This represents an opportunity to draw lessons from the evaluation of the seismic performance of modern concrete high rise buildings, designed with modern code provisions, under a major earthquake event. As a consequence of the observed damage, changes have been made in the Chilean Codes. The Concrete design code historically has followed ACI 318, with some specific exceptions, but new provisions have been added for the design of special concrete shear walls.

The most important changes include limits to:

- The level of axial stress allowed.
- The slenderness of the walls.
- Spacing of transverse reinforcement.
- Splices in longitudinal reinforcement.
- Amplification of shear forces for design.

Case Studies: Two examples of tall buildings with successful behavior during the earthquake are presented, both located in Santiago, Chile:

- Costanera Center Tower 2, a 62 story, 300 meters high office building.
- Titanium Tower, a 52 story, 192 meters high office building with dampers as energy dissipation devices.

Subject

Structural& Architectural Design

Action

engineering

Keywords

earthquake - seismic design - high rise buildings - shear wall - concrete - damage - Chile

1. Introduction

Chile is located in the southern part of South America between the Andes Mountains and the Pacific Ocean. It has an average of 200 km wide and4270 km long. Along the shore lineis the Pacific trench, wherethe Nazca Plate penetrates under the South America Plate generating frequent subduction type earthquakes usually followed by tsunamis.

On February 27, 2010 a magnitude Mw 8.8 subductioninterplate earthquake impacted the central part of Chile including the cities of Concepción, Viña del Mar and Santiago, affecting an area of 500 km long and 200 km wide, where 40 % of the country population lives. It is the sixth world largest magnitude earthquake recorded by mankind.

Between 1985, when a magnitude Ms7.8 earthquake affected approximately the same area of the country, and 2010, 9,974 buildings over 3 stories high were built in this area according to construction permits issued (ComitéInmobiliarioCChC). Of this, 20% had 10 stories or moreand an estimate of 3%had over 20 stories up to 52, the tallest at the time of the earthquake.

The statistics show that among the engineered buildings, there were 4 collapses (between 4 to 18 stories), and about 40 buildings were severely damaged and had to be demolished(ref [1]).This represents less than 1% of the total number of new residential buildings built in this period, affected by the earthquake. The rest only suffered nonstructural damage and in some cases minor reparable structural damage.

Over 20 stories, there were no collapses. An estimate of less than 10 buildings presented visible permanent inclinations of around H/300. Some of these have been already fixed.

These numbers show that shear wall buildings constructed in Chile performed well. Nevertheless, the earthquake produced significant structural damage on some new high rise shear wall buildings in Santiago, Viña del Mar, Chillán and Concepción. Also important damage on nonstructural components was observed.

This represents an opportunity to draw lessons from the evaluation of the seismic performance of modern concrete high rise buildings, designed with modern code provisions, under a major earthquake event.

On buildings, observed damage was concentrated near the base, on L or T shape walls, presenting crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions, extending horizontally deep into the wall length. Vertically the observed damage only affects a small portion of the story high. Shear failures at the web were also observed. Damage in slabs was observed, due to wall rotation at doorways in upper stories.

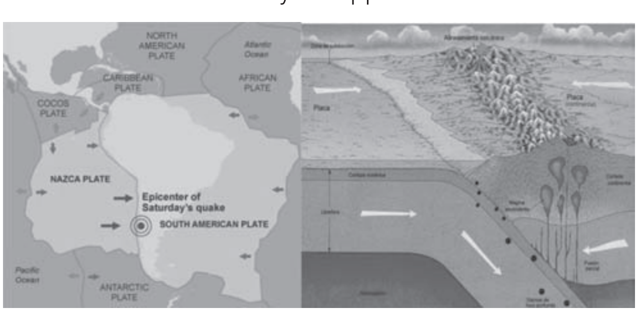


Figure 1: Subduction type earthquake, the Nazca Plate moves under the South American Plate

2. High Rise Structural Systems in the Chilean Practice

High rise buildings in Chile can be classified according to their use in two main categories: residential and office buildings. The main difference is that the later requires large open spaces in plan, while the first must have partitions for occupant privacy. As a consequence the typical structural systems adopted are:

Figure 2: Geographical area affected by earthquake

Residential Buildings three Stories or More
Authorized Between 1985 and 2009

Number of Stories	Country Region						Country Total
	V	Metro	VI	VII	VIII	IX	
3 story	598	3.412	390	202	673	75	**5.350**
4 story	213	517	37	23	93	32	**915**
5 story	143	548	20	17	63	37	**828**
6 story	53	75	3	3	22	15	**171**
7 story	22	290	2	13	20	7	**354**
8 story	63	330	5	2	12	5	**417**
9 story+	413	1.310	10	27	102	77	**1.939**
Total	**1.505**	**6.482**	**467**	**287**	**985**	**248**	**9.974**

Source:Estimate of the Chilean Chamber of Construction based on constructions permits statistic from INE

Figure 3: New buildings affected by the earthquake

2.1. Residential Buildings (fig. 5)

2.1.1. Floor system: flat concrete reinforced slab. Spans: 5

to 8 m., thickness: 14 to 18 cm supported on shear walls and upturned beams at the perimeter.

2.1.2. Vertical and lateral system: concrete shear walls.

2.2. Office Buildings:(fig. 4).

2.2.1. Floor system: Flat post tension slab. Spans 8 to 10m., thickness: 17 to 20 cm.

2.2.2. Vertical and lateral system: Concrete shear wall core and concrete special moment resisting frame (SMRF) at the perimeter.

Parking facilities for residential and office buildings are always placed below street level requiring normally several underground levels offloor space accounting for 30 to 40 % of the total construction area.

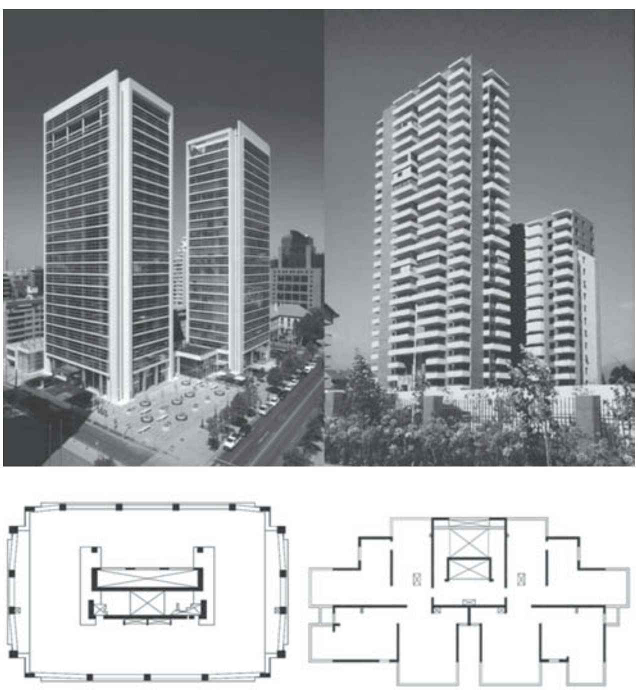


Figure 4: Typical office building & Typical residential building

Figure 5: Typical residential neigbourhood in Santiago Chile

2.3. Structural Indicators

Several indicators have been widely used throughout the years in Chile to evaluate the structural characteristics of buildings and their correlation with successful seismic performance.

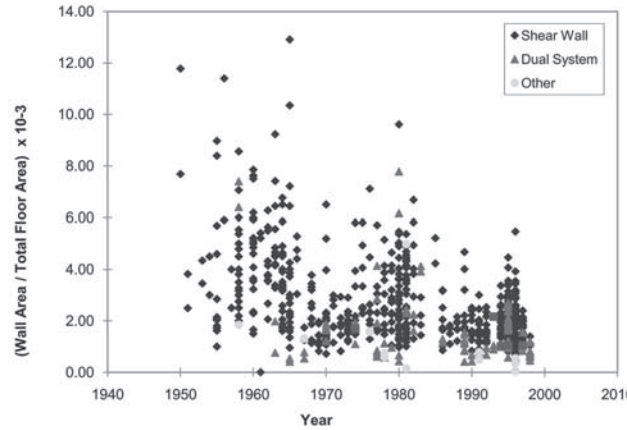


Figure 6: Wall Density Indicator for Chilean buildings between years 1950 and 2000

2.3.1. Wall Density Indicator:

Figure 6 illustrate the evolution of this parameter calculated as the wall area in the first floor on each principal direction divided by the total floor area above this level (ref. [2]). In the years shown in figure 6, the average is close to 0.2%. In the last 10 years, buildings have increased the number of stories and the quality of concrete used. This has led to a continuous reduction of the wall density indicator to values around 0.1% and in cases as low as 0.05%. A wall density value around 0.1% in each principal direction of the lateral load system has proven to be adequate to provide sufficient lateral stiffness and resistance for earthquake forces. The main difference between office and residential buildings is that office buildings have shorter wall length and wider thickness than residential buildings. On residential buildings it is easy to accommodate long partitions turned into thin structural walls.

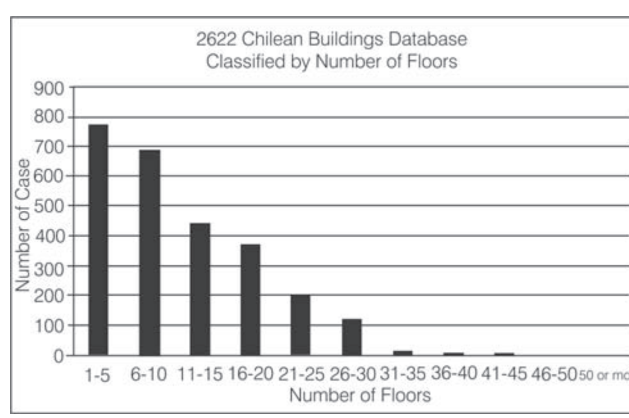


Figure 7: Chilean building database classified by number of floors.

2.3.2. Bio-Seismic Profile

These typologies have been evaluated by Guendelman (ref.[3])based on the computation of seismic Indicators from a database of 2622 buildings built in Chile between 1950 and 2009. This methodology proposed by Guendelman has been called "Bio-Seismic Profile".Post-earthquake field observations have led to identify the range forthese param-

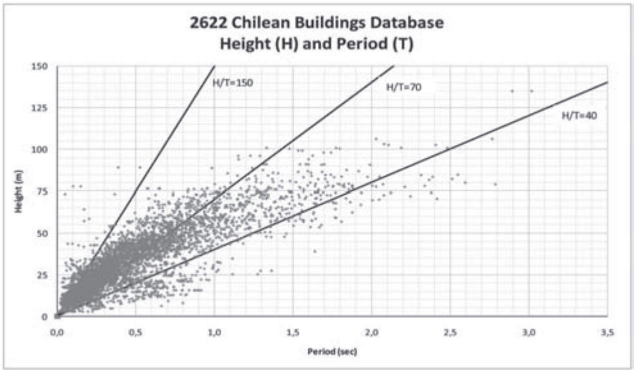


Figure 8: Chilean building database showing parameter H/T

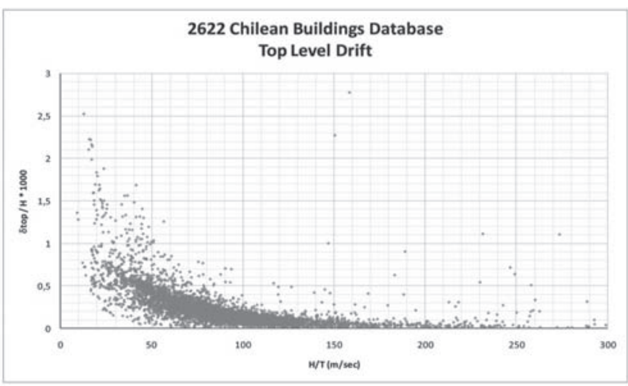


Figure 9: Chilean building database showing top displacement vs. H/T parameter

eters associated to satisfactory seismic behavior. The computation of the seismic indicators is performed using the results of the seismic analysis.

2.3.2.1. Stiffness Indicator H / T:

It is the quotient of the Total Height of the building (H) divided by the First Translational mode period of the building calculated from spectral analysis (T). The units are meters/sec. which represents a velocity. Historical values (fig.8) are in the range of 20 - 160 m/sec. Values below 40 m/sec. apply to flexible mostly frame buildings; values between 40 and 70 m/sec. represent normal stiffness buildings; and values over 70 m/sec. pertain to stiff buildings. Values of H/T above 40 usually comply with the code drift limitation of h/500. This makes unnecessary to account for P-? effects and have shown adequate seismic performance.

2.3.2.2. Modal Coupling Indicators:

Modal coupling between different modes of vibration usually result in amplifications in building seismic response. To minimize this undesirable phenomenon, practice has shown that when natural periods differ in at least 20% among them,dynamic amplifications of the response due to modal coupling are minimized.

2.3.2.3. Structural Redundancy Indicator:

This parameter counts the number of the most relevant resisting lines in each principal direction. A number no less than4 lines on each direction has been considered as adequate.

2.3.2.4. Ductility Demand Indicator or Effective Spectral Reduction Factor: R**

R** = Elastic Response Base Shear / 1.4 Design Response Base Shear Values of 4 or less have proven successful behavior in past earthquakes. Design for values larger than 4 should be investigated with approximated nonlinear analysis such as push-over analysis.

3. Building Code provisions in Chile

3.1. Chilean Code NCh433.Of96has the provisions for design and analysis methods of high rise buildings under seismic forces:

3.1.1. Type of analysis:

Modal spectrum linear elastic analysis.

Modal damping: 5% of critical damping

Modal superposition method: CQC

3.1.2. Building mass from:

DL + 0.25 LL

3.1.3. Accidental Torsion Analysis:

Accidental eccentricity at level k:

$e_{kx} = \pm 0.10$ by (Z_k / H) in the x direction

$e_{ky} = \pm 0.10$ bx (Z_k / H) in the y direction

3.1.4. Earthquake Load combinations:

1.4 (DL + LL ± E)

0.9 DL ± 1.4 E

3.1.5. Drift limitations:

Interstory drift at Center of Mass $\delta_{cm} \leq 0.002$

Interstory drift at any point i $\delta_{cm} - 0.001 \leq \delta_i \leq \delta_{cm} + 0.001$

3.1.6. Base shear limitations:

$IA_0P/6_g \leq$ Base shear $\leq 0.35SIA_0P/_g$ for concrete buildings

3.1.7. Seismic Zoning:

SeismicZone	GeographicArea	A0
Zone 1	Andes Mountainsstriparea	0.20 g
Zone 2	Central strip of Chile between the Coastal Mountains and the Andes Mountains	0.30 g
Zone 3	Costalstriparea	0.40 g

3.1.8. Types of foundationsoils

SoilType	Description	S	T_0	T'	n	p
I	Rock	0.90	0.15	0.20	1.00	2.0
II	Dense gravel, and soil with vs ≥ 400 m/s in upper 10 m.	1.00	0.30	0.35	1.33	1.5
III	Unsaturated Gravel and sand with low compaction	1.20	0.75	0.85	1.80	1.0
IV	Saturated cohesive soil with qu< 0.050 Mpa	1.30	1.20	1.35	1.80	1.0

3.1.9. BuildingCategory: Importance factor

Building Category	Description	I
A	Governmental, municipal, public service or public use	1.2
B	Buildings whose content is of great value or with a great number of people.	1.2
C	Buildings not included in Category A or B	1.0
D	Provisional structures not intended for living	0.6

3.1.10. Design Spectrum: (fig. 10)

Parameter	Formula	Comments
Design Spectrum	$Sa=\frac{IAo\alpha}{R^*}$	I : importance factor A_0: zone maximum effective acceleration R*: reduction factor α : amplification factor
Amplification factor	$\alpha=\dfrac{1+4.5\left(\frac{Tn}{To}\right)^p}{1+\left(\frac{Tn}{To}\right)^3}$	T_n : vibration period of mode n T_0, P: soil parameters
Reduction factor	$R^*=1+\dfrac{T^*}{0.10To+\left(\frac{T^*}{Ro}\right)}$	R_0 :structural system parameter R_0= 11 for shear wall and braced systems T*: period of mode with largest translational mass in the direction of analysis

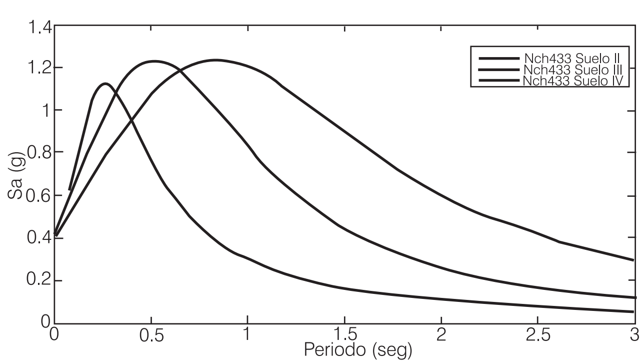


Figure 10,Chilean Code NCh433.Of96, Design Spectrum for seismic zone 3, for soil type I, II and III

4. Damage in R/C buildings in Concepción and Viña del Mar after the 27-F Earthquake in Central Chile.

The earthquake presented some unexpected characteristics such as the low frequency content, not seen before in Chile. This affected considerably the response of high rise buildings. Records obtained in Concepción by the University of Chile (ref.[4])show high destructive potential and are similar to others obtained in Viña del Mar and Santiago in soft soils. The Chilean Code NCh433.Of96 Earthquake Resistant Design of Buildings has been updated recently. The new acceleration elastic response spectra reflectan increment in the expected displacements for the maximum considered earthquake MCE.

The earthquake produced important structural damage in a number of high rise shear wall buildings in Santiago, Viña del Mar, Chillán and Concepción. Damage was concentrated at the base, onL or T shape walls with compression failures and buckling in vertical reinforcement at poorly detailed boundary elements. Shear failures at the web were also observed.

Figures 11 to 14 show typical plan configurations of damaged buildings. The architecture of this buildings present similar characteristics such as a central corridor with transverse walls of rectangular L or T shape. The walls are continuous from top to foundation. Atstreet and underground parking levels the walls frequently present penetrations or length reduction to facilitate parking. This representsa common structural irregularity on modern high rise buildings. Site observations indicate that stress concentrations around these areas initiate the failure mechanism in wall damage(fig. 16).

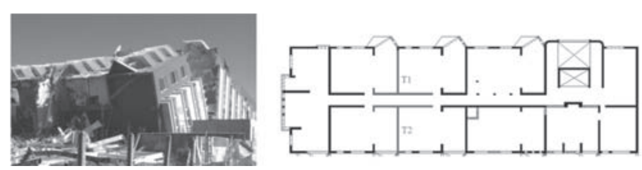


Figure 11: Building A, Concepcion: 15 story collapse

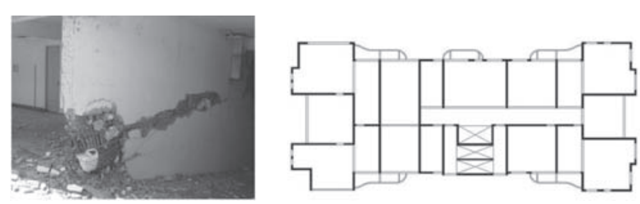


Figure 12: Building B, Concepcion: Compression, tension and shear failure on first and second floors

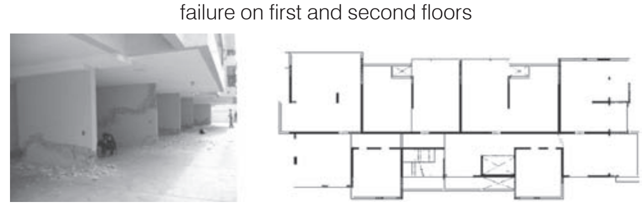


Figure 13: Building C, Viñadel Mar: general wall failure at first floor: 40 cm out of plumb at the top.

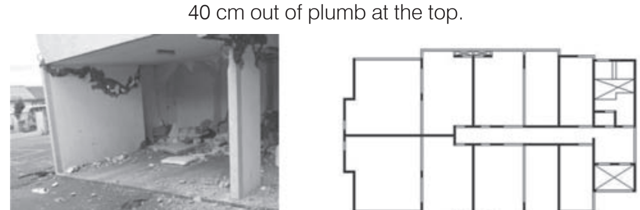


Figure 14: Building D, Concepcion: compression failure at first floor: 40 cm out of plumb at the top

Figure 15: Buckling and fracture of vertical reinforcement at web and edge of an L shaped wall with opening

Figure 16: In building B; buckling of vertical reinforcement at wall discontinuity.

5. Chilean Code changes after the 2010 earthquake: 5.1. Design of concrete special structural walls:

Several changes for the design of concrete special structural walls were incorporated in the new emergency revisionof the Chilean Code recently published. These provisions are intended to prevent crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions and at the same time prevent shear failures.

5.1.1. Design for bending and axial load of shear walls:

- General behavior: the configuration of the entire cross section (section T, L, C, etc.) should be considered.
- In determining the resistance of this type of sections, the contribution of all the longitudinal reinforcement in the cross section must be considered.
- Limiting axial load:at any cross section in a wall, unit elongation of the longitudinal reinforcement must exceed 0,004 when the concrete reaches the opposite extreme fiber shortening unit equal to 0.003. This is to avoid compression failures in favor of tension and more ductile types of failures.
- Alternatively, the preceding requirementcould be satisfied if the axial compression load on the wall does not exceed 75% of the axial balance load.
- Design shear forces in walls: shear forces from analysis must be amplified by 1.4 unless they are obtained using capacity design method.
- Slenderness: wall thickness must be greater than 1/16 of the unbraced length.
- Bar splices in longitudinal reinforcement: transverse reinforcement must be provided at lap splices. " Bar buckling: spacing of transverse reinforcement must be ? 6 longitudinal bar diameter.
- Bar spacing and confining reinforcement in boundary elements must comply with:

Figure 17: Confinement details

6. Conclusions

High rise concrete buildings constructed in Chile in the past 25 years performed well during the 2010 earthquake. Nevertheless, the earthquake produced significant structural damage on some new high rise shear wall buildings. As a consequence of the observed damage, changes have been made in the Chilean Codes (ref. [5, 8, 9, and 10]). The Concrete design code historically has followed ACI 318 [6],with some specific exceptions,but new provisions have been added for the design of special concrete shear walls. These provisions are intended to prevent crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions,and at the same time prevent shear failures when shear wall buildings are subjected to large displacement demands. A natural consequence of the more stringent design requirements has been an increase in the construction cost of the structures. Buildings of natural period less than 1,0 sec.(15 story) have not suffered significant differences, but for buildings of natural periods above 2.0 sec.(25 story), the cost of the structure has increased up to 10%.

7. Case Study: Costanera Center Tower 2, Santiago Chile

7.1. Introduction

Costanera Center Tower 2 is an office building, part of a multi-use development of 700,000 m2 of construction area, including four towers for office, apartments, hotels, a six level podium for retail, andsix levels of underground parking and service spaces. The architects are Pelli-Clarke-Pelli (USA) and AlemparteBarreda (Chile) and the structural engineers Rene Lagos Engineers (Chile).

Currently the tower is under construction in Santiago de Chile and will be finished during 2012. The total construction cost for the project is US$ 600,000,000. The complex is located in the intersection of Costanera Av. and Nueva TajamarStreet on a site 240 by 250 meters.

CostaneraCenter Tower 2, the tallest, has 110,000 m2of floor area, 62 occupied stories and a total high of 300 meters above street level plus19meters below ground. The tower floor plate starts at its base with a dimension of 47 meters by 47 meters and tapering to 40 meters by 40 meters from the 20th floor up. For the earthquake, on February 27th 2010, the building structure was under construction at approximately 50% of the total eight. The building responded elastically and no damage occurred.

Figure18: Costanera Center Tower 2, Santiago Chile

7.2. Structural System Description.

The structural system consists of a central reinforced concrete core, formed by shear walls with coupling beams. At the perimeter of the plan there is a reinforced concrete frame. They are linked by horizontal diaphragm slabs. The floor framing system consists of steel beams spanning between core and perimeter frame acting compositely with the concrete slabs over metal deck.

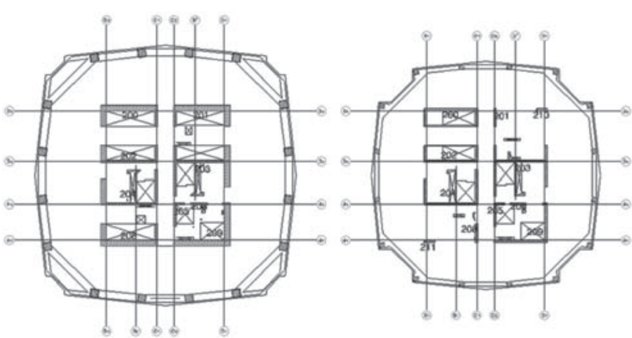


Figure19:structural system for levels -5 to 51 and 52 to 62

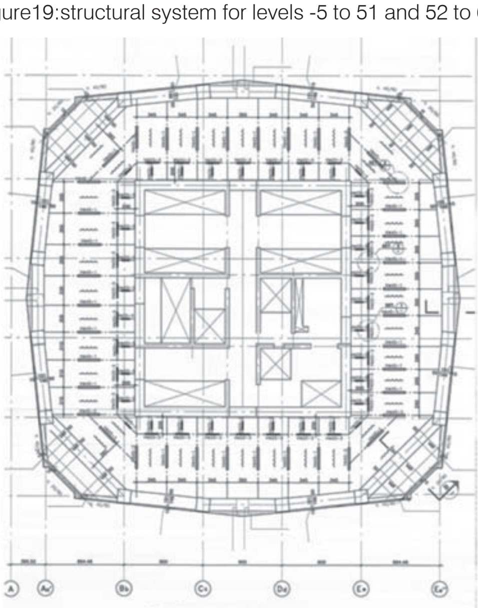


Figure20: floor system, composite slab + steel beam

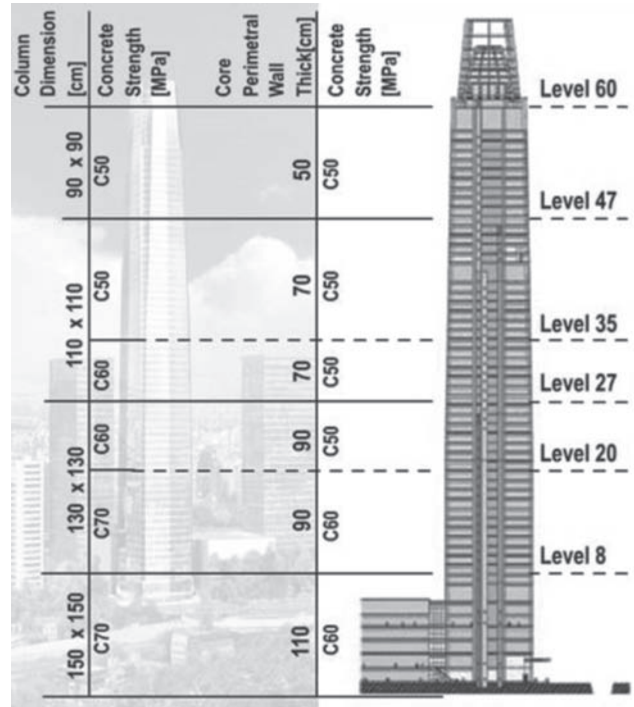


Figure 21: Lateral structural system dimensions

7.3. Lateral Response.

The lateral load resisting system for Tower 2 essentially relies on the flexural and shears deformation of the core and the axial stiffness of the columns. These factors control the efficiency of the system regarding stiffness. For lateral loadresistance, the shear forces and overturning momentsfrom seismic and wind loads are mainly resisted by the concrete core which takes more than 90 % of these forces. Figure 19 shows a typical structural floor layout.

The fundamental periods of the Tower are: T1 (N-S) = 7.21sec., T2 (E-W)=6.57sec. and T3 (rot)= 3.95sec. The use of outriggers at the mechanical floors was evaluated for increased lateral stiffness but finally disregardedbecause the core stiffness was enough to provide a driftlimit of H/800 which is less than H/500 required by the code.Figure 24 shows the drifts and lateral displacements for the seismic design loads.

7.3.1. Wind

Two approaches were used to evaluate wind load response for the tower:

7.3.1.1. Chilean Code NCh 432.Of71

Calculation of the Action of Wind on Structures

7.3.1.2. Wind tunnel test.

The study was done by Rowan Williams Davies & Irwin Inc. (RWDI) in accordance with ASCE Standard ASCE/SEI 7-02.

The adopted wind velocity for the 50-year return period corresponds to a wind speed of 38 m/s at 10 m above ground in open terrain, interpreted to be a 3 second gust value. The predicted wind-induced accelerations at the top occupied floorwere within the ISO based criteria for the 1, 5 and 10 year return periods. The 10 year accelerations were also within thecommonly used criterion of 20 to 25 milli-g for an office tower.

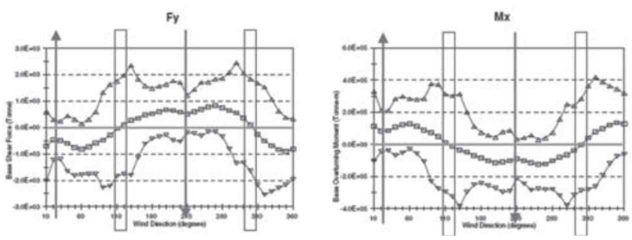


Figure 22: Wind tunnel test structural wind response.

7.3.2. Earthquake

Two approaches were used to evaluate the dynamic response of the building under earthquake loading:

7.3.2.1. Dynamic Response Spectrum Analysis (DRSA)

according to the Chilean Code NCh 433.Of96 Seismic Design of Buildings. The building is located in Chilean Seismic Zone 2 with soil type II (Gravel). The minimum design base shear force for this condition is 5% of the building weight (DL + 0.25LL).

The code requires drift ratios at the center of mass ? h/500 for inter-story drift, and H/500 for overall drift. According to the code, buildings are expected to behave in the inelastic range and be able to withstand the design earthquake without collapse. Nevertheless Tower 2 has a fundamental period of 7.2 seconds and the DRSA shows the elastic response for the building has a base shear of 6% (< 5% x 1.4). This means that the building would not enter the inelastic range under this earthquake level and the forces attracted are resisted by elastic displacements. Tower 2 is expected to remain in the elastic range under this level of earthquake.

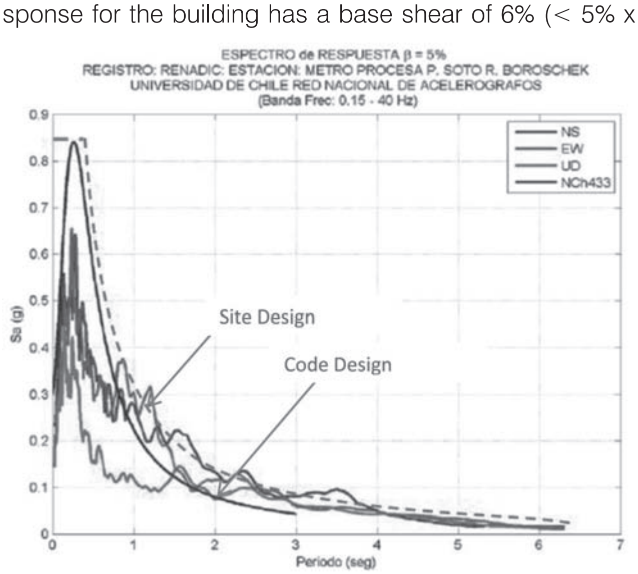


Figure 23: Comparison of Spectrums 27-F vs. Site Spectrum and Chilean NCh433. Code Spectrum

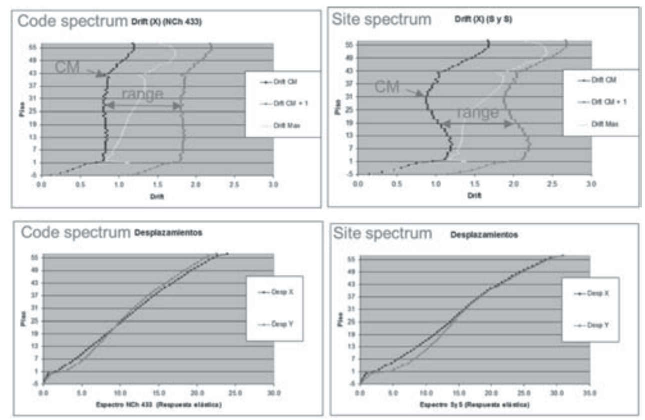


Figure 24: Code spectrum vs. Site spectrum displacements and drifts

7.3.2.2. Dynamic Time History Analysis:

A specific seismic hazard assessment study for the site was performed by S y S Consulting Engineers (Chile). The hazard is controlled by the subduction between Nazca and South American plates. For design, three subduction earthquakes were defined:

- Off-shore interplatesubduction earthquake of Richter magnitude Ms = 8.5 and at the hipocentral distance of 130 Km.This earthquake is characterized by a peak horizontal ground acceleration (PGA) of 0.42 g. " Far off-shore interplatesubduction earthquake of Richter magnitude Ms = 8.5 and at the hipocentral distance of 320 Km.This earthquake is characterized by a PGA = 0.18 g.
- Intermediate depth intraplatesubduction earthquake of Richter magnitude Ms = 8.0, with the epicenter at only 20 Km East of the building site and a hipocentral depth of 80 Km. The PGA for this earthquake is 1.2 g. Elastic acceleration response spectra were estimated for the three earthquakes.

For the time-history dynamic analysis of the building, design artificial accelerograms were generated for the two last types of earthquakes. Figure 25 shows the lateral displacements for the time-history elastic analysis performed with the artificial earthquake records.

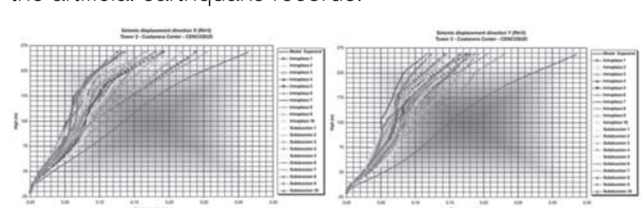


Figure 25: Lateral displacements for time-history elastic dynamic analysis in X and Y directions

7.3.2.3. Static Non-linear analysis

A push-over analysis was performed by Guendelman and Capacity-Demand curves were determined, indicating the building would behave essentially in the elastic range under the design earthquakes defined in the Assessment of Seismic Study.

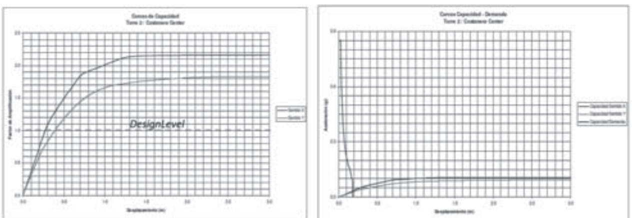


Figure 26: Building Capacity curves and Capacity-Demand for the Design Earthquakes

8. Case Study: Titanium La PortadaTower, Santiago Chile

8.1. Introduction

Titanium La Portada is a 52 story 192 meters high building, plus a 7floor underground parking and a podium 11 stories high around the office tower. The total building area is 130,000 m², and the construction cost was US$ 120,000,000. The architects of the building are SENARQ (Chile) and the structural engineer is Alfonso Larrain y Asoc. Ingenieros.

The building is located in the corner of Andres Bello Av. andVitacura Av. on a site of 60 by 100 meters, three blocks away from the Costanera Center complex.

The tower typical floorhas a lens shaped plan, 61.0 meters long by 34.0 meters wide (fig. 28).On February 27th 2010, the building was finished and ready for occupancy. The building responded elastically and no structural damage occurred. The seismic dampers performed as expected reducing the response of the building. Post-earthquake inspection of the dampers revealed some plastic deformations on the bending plates, without need for replacement.

Figure 26: Titanium La Portada Tower, Santiago Chile

8.2. Structural System Description.

Themain structural system is formed by a central concrete wall core around elevators and stairs, and a concrete special moment resisting frame (SMRF) at the perimeter of the plan.At the far ends of the plan, cross type steel bracings three stories high with energy dissipation devices were used throughout the complete high of the building for reduced seismic performance. In the longitudinal direction, energy dissipation devices were placed in two shear walls as shown on figure 26 in sectors 1 and 2. The floor structural system is solved by hollow prefabricated one way slabs supported by the core on one side and the perimeter beam on the other, The prefab elements are topped with 5 cmconcrete with an embedded steel reinforcing mesh, carefully anchored at both ends for structural integrity of the diaphragm.

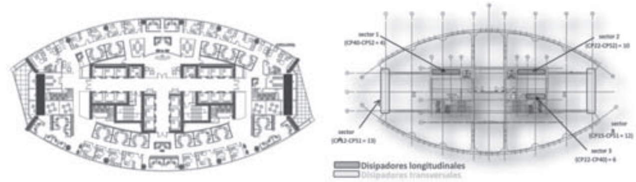
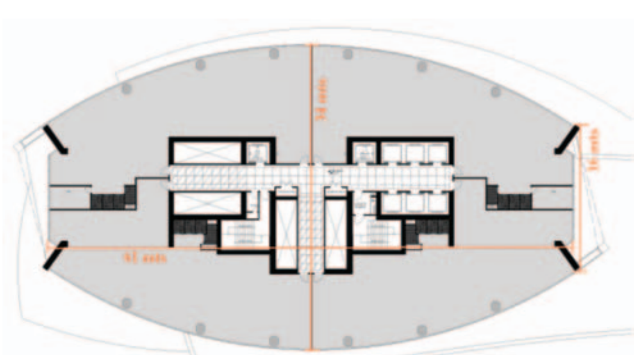


Figure 26: Architectural and structural plan showing location of energy dissipation devices (UFP Type)

Figure 27: Transversal and longitudinal energy dissipation devices (plastic hysteretic plate bending)

8.3. Lateral Response.

The lateral load resisting system for Titanium Tower relies on the flexural and shears deformation of the core, the axial stiffness of the columns, the cross bracing and damping characteristics of the energy dissipation devices. These factors control the efficiency of the system regarding stiffness. For lateral load resistance, the shear forces and overturning momentsfrom seismic and wind loads are mainly resisted by the concrete core which takes more than 90 % of these forces.



Floors	Concrete fc' MPa	Column cm.	Walls x_{dir} cm.	Walls y_{dir} cm.
-7 to -4	60	100/140	60	70
-3 to -1	60	100/120	60	70
1 to 5	60	100/100	60	70
6 to 15	60	95/95	55	65
16 to 20	40	95/95	55	65
21 to 25	40	90/90	50	60
26 to 30	40	85/85	45	55
31 to 35	40	80/80	40	50
36 to 46	40	75/75	35	40
47 to 52	40	70/70	30	35

Titanium Tower: Structural Elements

Figure 28: Typical floor structural plan

The lateral deflections in the longitudinal direction are controlled by seismic forces compared to wind forces. In the transverse direction, the building has the largest exposure to wind and consequently wind loads deflections are larger than seismic lateral deflections.

Figure 28 shows a typical structural floor layout. This complies with the strength and stiffness requirements of the Chilean Seismic Code for stability and drift control. The fundamental periods of the building are: T1 (N-S) = 5.88 sec., T2 (E-W) = 4.61 sec. and T3 (rot) = 3.70 sec.

8.3.1. Drift control:

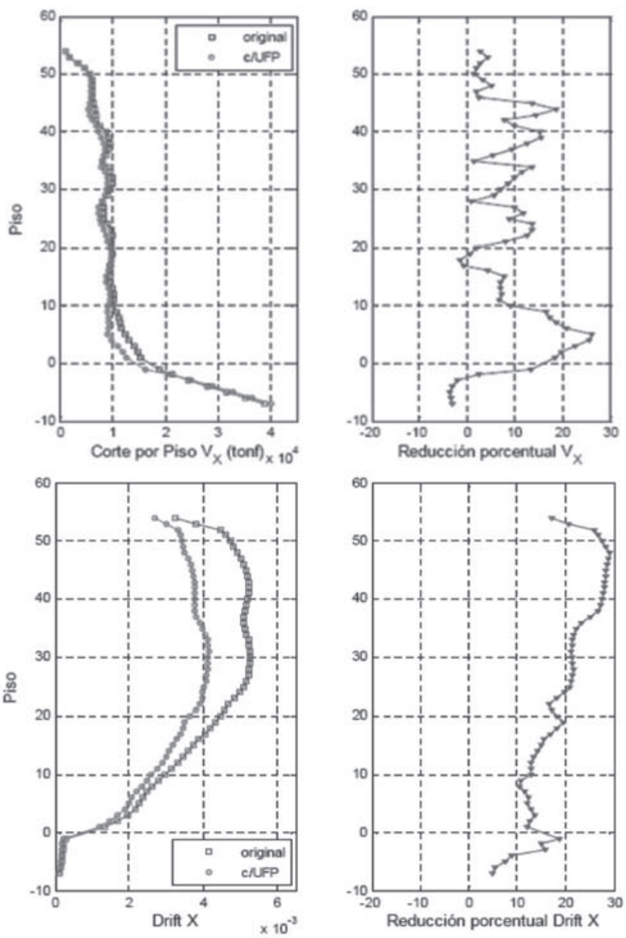


Figure 29: Shear force and drift reduction in longitudinal direction due to dampers for NS Melipilla earthquake.

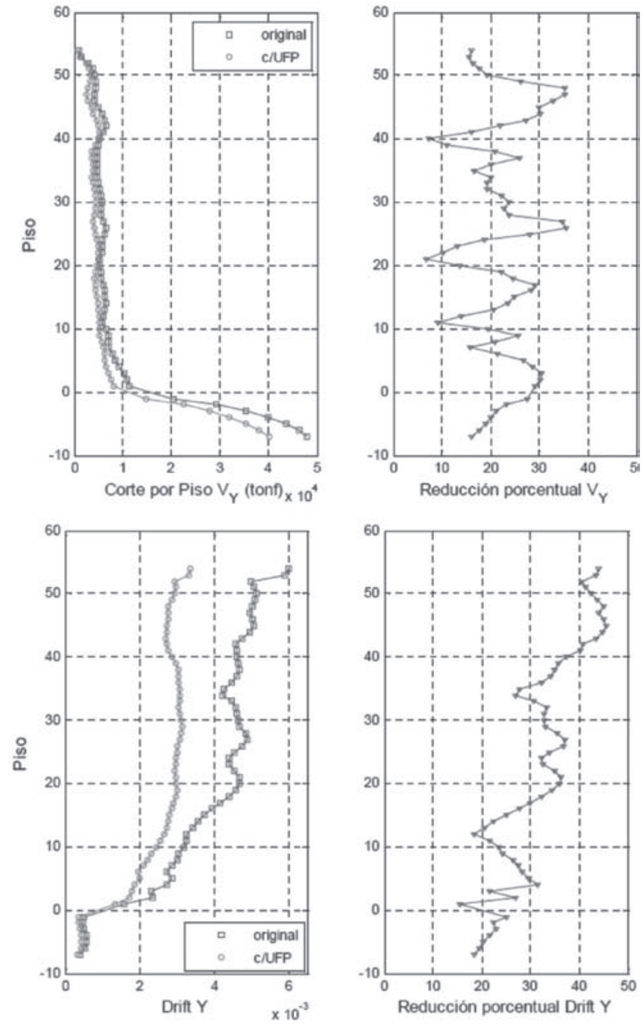


Figure 30: Shear force and drift reduction in transverse direction due to dampers for NS Melipilla earthquake.

Response reductions obtained for the NS component Melipilla earthquake (Ms 7.8), due to the use of dampers,are in the range of 10 to 30% in the drifts in the longitudinal direction and in the range of 20 to 45% in the transverse direction.

9. Final remarks on seismic performance of high rise buildings

In concrete shear wall structural systems and dual systems, frequently used in high rise buildings in the range of 20 to 40 story, the use traditional design practice based on fundamental mode response and force reduction factors, have proven to provide reliable seismic performance when designed according to modern codes such as ACI 318, assuming the local seismic code provides a good estimate of the demand.

For buildings subject to large displacement demands, additional requirements to those in ACI 318 should be considered for the design of concrete shear walls, to prevent crushing and spalling of concrete and buckling of vertical reinforcement at boundary regions. Observations in the Chilean 2010 earthquake indicate the need for stringent requirements in:

- The level of axial stress allowed.
- The slenderness of the walls.
- Spacing of transverse reinforcement.
- Splices in longitudinal reinforcement.
- Amplification of shear forces for design.

The use of energy dissipation devices has improved the seismic performance of high rise buildings. Of special importance is the reduction of damage obtained in non-structural components. Observations in Chile indicate this type of damage in many cases was the reason for buildings to become inhabitable, involving high repairing costs. Future trends should move into the adoption of these technologies as standard solutions.

Nevertheless, the adoption of a performance based philosophy instead of prescriptive type codes is setting out the best practice principles for the seismic design of high rise buildings. This type of codes have been in use in Japan and China for some years and permitted in the United States in the IBC code.

Seismic performance of modern generation tall buildings with complex geometries cannot be predicted with elastic methods of analysis using global force reduction methods. Nonlinear response-history procedures must be used to predict their behavior and guarantee their target performance under moderate and severe earthquake conditions.

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Scenario-based time-dependent definition of seismic input: an effective tool for engineering analysis and seismic isolation design

Giuliano F. Panza[1,2]; Antonella Peresan[1,2]; Franco Vaccari,[2]; Fabio Romanelli[1,2]; Alessandro Martelli[3]

Summary

For the protection of cultural heritage and the design of strategic buildings and critical infrastructures, when it is necessary to consider extremely long time intervals, the standard probabilistic estimates of seismic hazard (PSHA) are by far not applicable. The heuristic limitations are, indeed, a major limit of PSHA, whose results, extrapolating ground motion with an infinitely long return period from a few hundreds years of the available earthquake catalogues, may turn out to be a purely numerical artefact. A viable alternative, capable of minimizing the draw backs of PSHA is represented by the use of a scenario-based methodology, named neodeterministic approach (NDSHA), which relies on observable data and is based on the physical modelling of seismic waves generation and propagation processes. The NDSHA naturally supplies realistic time series of ground motion, which represent also reliable estimates of ground displacement readily applicable to seismic isolation techniques, useful to preserve historical monuments and relevant man made structures.

In addition, an integrated NDSHA approach has been developed that allows for the operational definition of time-dependent scenarios of ground shaking, through the routine updating of formally defined earthquake predictions. The integrated NDSHA procedure, which is currently applied to the Italian territory, combines different pattern recognition techniques, designed for the space-time identification of strong earthquakes, with algorithms for the realistic modelling of ground motion. Accordingly, when an alarm is declared, a set of scenarios of expected ground shaking at bedrock, associated with the alarmed areas identified by means of the algorithms CN and M 8 S , can be readily computed by means of full waveform modelling, both at regional and local scale, considering all possible earthquake sources within the alerted areas. F or the relevant sites, further investigations can be performed taking into account the local soil conditions, in order to compute the seismic input (realistic synthetic seismograms) for engineering analysis.

The practical application of seismic input modelling for seismic isolation purposes, and particularly for the protection of historical buildings, has been already carried out for example for several sites, including the cities of Trieste (NE Italy) and Valparaiso (Chile) and the towns of Marigliano and Ercolano (Naples, Italy). A recent example of the practical adv antages that can be provided by the time-dependent definition of ground-shaking scenarios is given by the Cathedral of Santa Maria di Collemaggio, which was severely damaged during the L 'Aquila earthquake (April 6 , 2009). Based on the ongoing alert for the nearby CN region and the relevant ground shaking expected at the site, the restoration and protection of the cathedral by means of dampers could have been timely completed, possibly limiting if not preventing the occurred damage.

The relevance of the realistic modelling, which permits the generalization of empirical observations by means of physically sound theoretical considerations, is evident, as it allow s for the optimisation of the structural design with respect to the site of interest. Moreover, the time information given by the intermediate-term middle-range earthquake prediction can enhance preparedness and planning of risk mitigation actions.

Keywords

Seismic hazard; earthquake scenarios; seismic input; earthquake prediction; seismic isolation

Theme

Structural Design - Action engineering/earthquake -Isolators

1. Introduction

Most of the seismic zonation adopted by the current regulations, either on a national or a regional scale, have been defined according to a well-established but obsolete, conventional probabilistic approach, PSHA (Cornell [1]) and hence they are basically affected by the limitations of such methodology (Panza et al. [2]). Specifically, probabilistic seismic hazard maps are: a) strongly dependent on the available observations, unavoidably incomplete due to the long time scales involved; b) not considering adequately the source and site effects, since they resort to convolutive techniques (e.g. ground motion attenuation relations, now very popular under the acronym GMPE, Ground motion prediction equations), which cannot be applied when dealing with complex geological structures; c) time-independent, being based on the assumption of random occurrence of earthquakes (Bilham [3]).

Lessons learnt from recent destructive earthquakes, including the L 'Aquila (2009), Haiti (2010) and Chile (2010) earthquakes, provided new opportunities to revise and improve the seismic hazard assessment (SHA). There is the need, however, of a formal procedure for the official collection and proper evaluation of seismic hazard assessment results, so that society may benefit from the scientific studies and may not be misled by the incorrect hazard assessment results and even non-scientific results. An effective improvement of current methodologies for SHA requires understanding the limits and uncertainties of the available tools and related estimates. Models currently used for SHA are generally verifiable, but their validation has been often limited by the available data and accumulated knowledge. Cross-checking with available observations and independent physics based models is thus recognized as a major procedure for the necessary validation of SHA models.

Recently Kossobokov and Nekrasov a [4] show ed that the worldwide maps resulting from the Global Seismic Hazard Assessment Program, GSHAP (e.g. Giardini et al. [5]), are grossly misleading, as proved by fatal evidence by all the deadliest earthquakes occurred since 2000 year (table 1). In fact, the probabilistic GSHAP maps that w ere published in 1999, giving peak ground acceleration (PGA) values with 10% probability of being exceeded in 50 years, have been disproved by the seismicity of the past ten years. T he comparison between the expected PGA values provided by GSHAP and the actual maximum PGA experienced during the period 2000 -2009, performed in terms of related intensities, showed major inconsistencies, particularly severe as earthquakes of greater and greater size were considered. Moreover, the authors analyzed the seismicity of the past century, as well as the seismicity of the decade 1990-1999, when GSHAP was developed, and found that the performance of the maps was equally poor. Thus GSHAP fails both in describing past seismicity, as well as in predicting expected ground shaking.

In view of the very unsatisfactory performances of most of the traditional probabilistic approaches, the need for developing time-dependent scenario-based seismic hazard maps, from a mid-term anticipatory perspective, is evident. The neo-deterministic approach, NDSHA (Peresan et al. [6] and references therein), allows us to integrate the available information provided by the most updated seismological, geological, geophysical and geotechnical databases for the site of interest, as well as advanced physical modelling techniques, to provide reliable and robust basis for the dev elopment of a deterministic design basis for cultural heritage and civil infrastructures in general (Field et al. [7], Panza et al. [8] [9]). Neo-deterministic refers to scenario-based methods for seismic hazard analysis, w here attenuation relations and other similarly questionable assumptions about local site responses, all implying some form of physically not sound linear convolution, are not allowed in, but realistic synthetic time series are used to construct earthquake scenarios. The NDSHA procedure provides strong ground motion parameters based on the seismic waves propagation modelling at different scales-regional, national and metropolitan - accounting for a wide set of possible seismic sources and for the available information about structural models. The scenario-based methodology relies on observable data and is complemented by physical modelling techniques, which can be submitted to a formalized validation process.

Table 1. List of the ten deadliest earthquakes occurred during the period 2000-2010, and the corresponding intensity differences $\Delta I_0 = I_0(M) - I_0$(mPGA), estimated among the values observed and predicted by GSHAP. $I_0(M) and I_0$ (mPGA) are computed from the observed magnitude M and the predicted PGA, respectively, using existing relationships (after Kossobokov and Nekrasov a [4]).

To sum up: 1) Statistical analysis is a must for seismic hazard assessment, because this is the nature of earth sciences and the results from any approach should be testable by observations; 2) PSHA is not a VALID probabilistic approach because a) it is not based on valid earthquake source model (single point source which is no longer valid for earthquakes having safety concern), b) mathematical pitfalls (i.e., incorrect use of statistics and probability theory such as G-R curve and ground motion attenuation), c) misinterpretation and use of the annual probability of exceedance (i.e., the probability of exceedance in one year-a dimensionless quantity) as a frequency (per year-a dimensional quantity); 3) scenario-based (NDSHA) hazard analysis is a statistical approach because the uncertainties are considered, but quantified in a different way than PSHA and the associated occurrence frequency can be assessed; 4) DSHA is also a statistical approach although it is called deterministic. Its major draw backs are a) to rely upon attenuation relations and b) obscure occurrence interval (frequency).

The evolving situation makes it compulsory any national or international regulation to be open to accommodate the most important new results, as they are produced and vali-

Location	Date	Magnitude	Intensity difference	Casualties
Port-au-Prince (Haiti	12.01.2010	7.3	2.2	222.570
Padang (Southern Sumatra, Indonesia)	30.09.2009	7.5	1.8	1.117
Menchuan (Sichuan, China)	12.05.2008	8.1	3.2	87.587
Yogyakarta (Java, Indonesia)	26.05.2006	6.3	0.3	5.749
Kashmir (North India and Pakistan border region)	08.10.2005	7.7	2.3	86.000
Nias (Sumatra, Indonesia)	28.03.2005	8.6	3.3	1.313
Sumatra-Andaman (Indian Ocean)	26.12.2004	9.0	4.0	227.898
Bam (Iran)	26.12.2003	6.6	0.2	31.000
Boumerdes (Algeria)	21.05.2003	6.8	2.1	2.266
Bhuj (Gujarat, India)	26.01.2001	8.0	2.9	20.085

dated by the scientific community. An example is provided by the Ordinance of the Prime Minister (OPCM) n. 3274/ 2003, plus its amendments and additions, which have enforced the current Seismic Code in Italy: in the Ordinance it is explicitly stated that the rules of the code must be revised as new scientific achievements are consolidated. Destruction and casualties caused by the L 'Aquila earthquake (April 6, 2009 ; M6.3), despite it took place in a w ell know n seismic territory of the Italian peninsula, are just a sad reminder that significant methodological improvements are badly needed toward a reliable assessment of ground shaking and engineering implementation.

2. The neo-deterministic approach

NDSHA is an innovative procedure that supplies realistic time histories from which it is possible to retrieve peak values for ground displacement, velocity and design acceleration in correspondence of earthquake scenarios (e.g. Parvez et al. [10]; Paskaleva et al. [11]). The procedure is particularly suitable for the optimum definition of the characteristics of the modern anti-seismic devices, when the accelerometric data available are not representative of the possible scenario earthquakes and when non-linear dynamic analysis is necessary. By sensitivity analysis, know ledge gaps related to lack of data can be easily addressed, due to the limited amount of scenarios to be investigated.

Where the numerical modelling is successfully compared with records, the synthetic seismograms permit the microzoning, based upon a set of possible scenario earthquakes. Where no recordings are available the synthetic signals can be used to estimate the ground motion without having to wait for a strong earthquake to occur (pre-disaster microzonation). In both cases the use of modelling is necessary since the so-called local site effects can be strongly dependent upon the properties of the seismic source and can be properly defined only by means of envelopes. In fact, several techniques that have been proposed to empirically estimate the site effects using observations convolved with theoretically computed signals corresponding to simplified models, supply reliable information about the site response to non-interfering seismic phases, but they are not adequate in most of the real cases, when the seismic sequel is formed by several interfering waves.

One of the most difficult tasks in earthquake scenario modelling is the treatment of uncertainties, since each of the key parameters has its own uncertainty and intrinsic variability, which often are not quantified explicitly. A possible w ay to handle this problem is to vary systematically (within the range of related uncertainties) the modelling parameters associated with seismic sources and structural models, i.e. to perform a parametric study that takes into account the effects of the various focal mechanism parameters (i.e. strike, dip, rake, depth etc.). The parametric studies w ill allow us to generate adv anced groundshaking scenarios for the proper evaluation of the site-specific seismic hazard, with a complementary check based on both probabilistic and empirical procedures. Once the gross features of the seismic hazard are defined, and the parametric analyses have been performed, a more detailed modelling of the ground motion can be carried out for sites of specific interest. Such a detailed analysis should take into account the source characteristics, the path and the local geological and geotechnical conditions.

The NDSHA (e.g. Panza et al. [8] [9]) represents one of the new and most adv anced approaches and it has been applied successfully in many areas worldwide (e.g. Parvez et al. [12]; Panza et al. [2]). This approach addresses some issues largely neglected in traditional hazard analysis, namely how crustal properties affect attenuation: ground motion parameters are not derived from overly simplified attenuation functions, but rather from synthetic time histories. Starting from the available information on the Earth's structure, seismic sources, and the level of seismicity of the investigated area, it is possible to estimate peak ground acceleration, velocity, and displacement (PGA, PGV, and PGD) or any other parameter relevant to seismic engineering, which can be extracted from the computed theoretical signals. NDSHA allow s us to obtain a realistic estimate of the seismic hazard w here scarce (or no) historical or instrumental information is available. Synthetic seismograms can be constructed to model ground motion at sites of interest, using know ledge of the physical process of earthquake generation and wave propagation in realistic media. The signals are efficiently generated by the modal summation technique (e.g. Panza et al. [9]), so it becomes possible to perform detailed parametric analyses that permit to account for the uncertainty in input information.

The estimates of seismic hazard obtained according to the NDSHA and to the probabilistic (PSHA) approaches have been compared for the Italian territory (Zuccolo et al. [13]). The NDSHA provides values larger than those given by the PSHA in high-seismicity areas and in areas identified as prone to large earthquakes, while lower values are provided in low-seismicity areas. T he PSHA expected ground shaking estimated with 10% probability of being exceeded in 50 years (associated with a return period of 475 years) appears severely underestimated (by about a factor 2) with respect to NDSHA estimates, particularly for the largest values of PGA. When a 2% probability of being exceeded in 50 years is considered (i.e. return period of 2475 years) PSHA estimates in high-seismicity areas become comparable with NDSHA; in this case however, the overall increase related with probabilistic estimates leads to significantly overestimate the hazard in low-seismicity areas. These observations point out one of the basic limits of PSHA estimates, particularly severe as far as building codes are concerned, that is the overly dependency of ground shaking on earthquakes recurrence (i.e. on the probability threshold selected for the maps).

From an anthropocentric perspective, buildings should be designed so as to resist future earthquakes. When an earthquake with a given magnitude Moccurs, it causes a specific ground shaking that certainly does not take into account whether the event is rare or not; thus ground motion parameters for seismic design should not be scaled depending on earthquake recurrence. With reference to Italy, based on recent experience from L 'Aquila earthquake and in line with the spirit of the Ordinance OPCM n. 3274/2003, which enforced the current Seismic Code shifting from a -emergency/rescue" to a -prevention" perspective, the PSHA should be used only to classify the territory on the basis of the probability that, in a given area, an earthquake with a given magnitude may occur in a given time interval (disastrous [say 500 years]; strong [say 140 years]; frequent [say 70 years]; etc.). Accordingly, when considering two sites A and B prone to earthquakes with the same magnitude, say M = 7, given that all the remaining conditions are the same, the site w here the recurrence is lower appears naturally preferable; nevertheless parameters for seismic design (DGA, PGA, PGV, PGD, etc.) must be equal at the two sites, since the expected magnitude is the same (M = 7). The evaluation is obviously different from a merely statistical point of view, which may apply to insurances but it is certainly not inline with the Ordinance OPCM n.3274/2003.

3. Time-dependent scenarios at regional scale

Based on the NDSHA an operational integrated procedure for seismic hazard assessment has been dev eloped (Peresan et al. [14] and [15]; Zuccolo et al. [16]) that allows for the definition of time-dependent scenarios of ground shaking, through the routine updating of earthquake predictions. Accordingly, when an alarm is declared, a set of scenarios of expected ground shaking at bedrock, associated with the alarmed areas identified by means of formally defined algorithms, can be readily computed by means of full waveform modelling, both at regional and local scale, considering all of the possible earthquake sources within the alerted areas. For the relevant sites, further investigations can be performed taking into account the local soil conditions, in order to compute the seismic input (realistic synthetic seismograms) for engineering analysis.

The intermediate-term medium-range earthquake predictions are performed by means of the algorithms CN and M8S (Keilis-Borok and Rotwain [17] and Kossobokov et al. [18]). CN and M8S algorithms belong to a family of fully formalized procedures for intermediate-term middle-range earthquake prediction, which are tested in several regions worldwide since more than twenty years. The results of the global scale real-time experimental testing of these algorithms (e.g. Kossobokov et al. [19]; Rotwain and Novikova, [20]) indicate the possibility of practical earthquake forecasting, although with limited accuracy, i.e. with a characteristic alarm-time ranging from a few months to a few years and a linear uncertainty in space of hundreds of kilometres. B y the algorithms definition, in fact, the dimensions of the monitored areas increase proportionally to the length L of the source of the events to be predicted, in order to account for possible long-range interactions in seismic activity. Thus it appears particularly relevant to combine the space-time information from CN and M8S algorithms with observations independent from seismicity, such as those provided by the morphostructural analysis. Based on the morphostructural zonation, in fact, the pattern-recognition technique can be used to restrain the alerted areas to the more precise location of large events, independently from any transient seismic information. In Italy, the identification of the areas prone to strong earthquakes has been performed by Gorshkov et al. [21], [22] for two magnitude thresholds, M 6 .0 and M 6 .5 ; the identified seismogenic nodes are used, along with the seismogenic zones (Meletti and Valensise [2004]), to characterise the seismic sources for ground motion modelling. The results from practical application of the integrated neo-deterministic procedure, which permits to compute different kind of scenarios at different space level, are illustrated considering:

- national scale scenarios at bedrock associated with the alerted regions;
- scenarios at bedrock associated with each single seismogenic node within the alerted region;
- detailed scenarios, that take into account local soil conditions, associated with each single seismogenic node within the alerted region.

The proposed approach complements the traditional approach to seismic hazard estimates, since it supplies routinely updated information that can be useful in assigning priorities for timely mitigation actions and hence it is particularly relevant for Civil Defence purposes.

3.1 Scenarios at bed rock associated with the alerted regions

The application of CN and M8S algorithms to the Italian territory is described in detail in Peresan et al. [24]. F or the application of the algorithm CN a regionalization composed by three macro-zones, defined strictly based on the seismotectonic zoning and taking into account the main geodynamic features of the Italian area, is considered. For the application of M8S algorithm (i.e. the spatially stabilized variant of M8), seismicity is analysed within a dense set of overlapping circles, with radius increasing with the magnitude of the target events and covering the monitored area. An experiment is ongoing since 2003, aimed at a real-time testing of M8S and CN predictions for earthquakes with magnitude larger than a given threshold (namely 5.4 and 5.6 for CN algorithm, and 5.5 for M8S algorithm) in the Italian region and its surroundings. Predictions are regularly updated every two months and a complete archive of predictions is made available on-line (http:// www.ictp.trieste.it/ www users /sand/prediction/ prediction.htm), thus allowing for a rigorous testing of the predictive capability of the applied algorithms. The results obtained during more than seven years of real-time monitoring already permitted a preliminary assessment of the significance of the issued predictions (Peresan et al. [15]). So far, 12 out of the 14 strong earthquakes, occurred within the monitored territory since 1963, have been correctly preceded by a TIP declared by CN algorithm, with less than 30 % of the overall space-time volume occupied by alarm; the confidence level for such predictions is above 99%. Similarly, the algorithm M8S correctly identified 14 of the 23 earthquakes with magnitude M5.5 + (i.e. between 5.5 and 6.0), occurred since 1972 within the monitored territory, with a space-time volume of alarm of about 34%; the confidence level of M5.5 + predictions has been estimated to be above 98% (no estimation is yet possible for higher magnitude levels).

According to the neo-deterministic procedure, the expected ground motion is modelled at the nodes of a regular grid with step 0.2° x 0.2° point, starting from the available information about seismic sources and regional structural models. Ground shaking scenarios associated with the alerted area are defined considering altogether the set of possible sources included in the region, following the procedure described in (Peresan et al. [15]). In such a way an alarm (which consists of space, time and magnitude information about the impending earthquake) can be associated with maps describing the seismic ground motion caused by the potential sources in the alerted region.

In this paper we provide the example of an alarm declared by the CN algorithm for the Northern region (figure 1), which turns out to be relevant in connection with the L 'Aquila earthquake (April 6, 2009). Figure 1 a, b and c illustrate the maps of PGD (peak ground displacement), PGV (peak ground velocity) and DGA (design ground acceleration), associated with the CN Northern region; any other parameter of

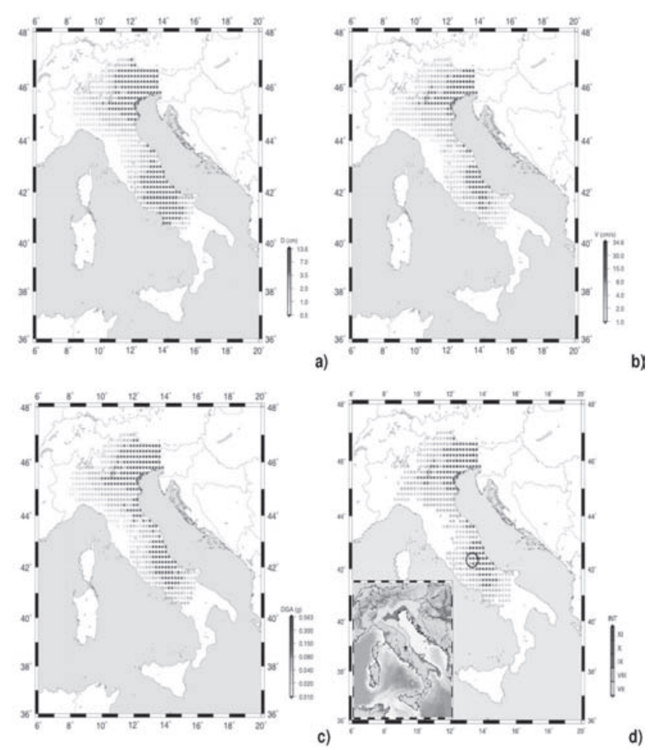

Figure1: a) map of PGD, b) PGV, c) DGA, d) Intensity scenario (MCS >VII) associated to an alarm in CN Northern region, as defined for the period 1 March 2009 - 1 May 2009. The minimum value of DGA reported in the map is 0.01 g. The alarmed region is shown as insert, along with the epicentre of the L'Aquila earthquake (blue star); the circle in the intensity map indicates the area within 3 0 km from the epicentre of the M = 6.3 earthquake occurred on April 6, 2009.

interest for seismic engineering can be mapped as well. Maps of expected intensities (figure 1 d) can also be obtained (e.g. Zuccolo et al. [13]), using the available relationships among the computed horizontal ground motion and the observed macroseismic intensities (e.g. Panza et al. [25]).

CN and M8S predictions, as well as the related time-dependent ground motion scenarios, are routinely updated every two months and are made available to the Civil Defense of the Friuli Venezia Giulia Region since 2006. The L'Aquila earthquake occurred outside the areas alerted by CN and M8S algorithms for the corresponding magnitude interval (figure 1 d), therefore it turns out to be a failure to predict; the epicentre, however, was localized just outside (about 10 km) the region alerted by CN algorithm for an earthquake with magnitude M >5 .4 . Thus, the time-dependent ground shaking scenario defined for the period 1 March 2009 - 1 May 2009, correctly predicted the macroseismic intensities, as large as IX (MCS), observed for this earthquake (Peresan et al. [15] and references therein).

3.2 Scenarios at bed rock associated with earthquake prone areas

The space uncertainty typical of the intermediate-term middle-range predictions is quite large. An attempt to constrain the location of the impending events is possible through the combined use of seismological, geological and morphostructural information. In fact, pattern-recognition can be used to identify the sites capable to generate the strongest events inside the alerted areas, independently from any transient seismic information. The areas prone to strong earthquakes are identified based on the morphostructural nodes, which represent specific structures formed around the intersections of lineaments. Lineaments are identified by the Morphostructural Zonation (MZS) Method (Alekseevskaya et al. [26]), that, independently from any information about seismicity, delineates a hierarchical block structure of the studied region, using tectonic and geological data, with special care to topography. The boundary zones between blocks are called lineaments and the nodes are formed at the intersections or junctions of two or more lineaments. Among the defined nodes, those prone to strong earthquakes are then identified by the pattern recognition on the basis of the parameters characterising indirectly the intensity of neo-tectonic movements and fragmentation of the crust at the nodes (e.g. elevation and its variations in mountain belts and watershed areas; orientation and density of linear topographic features; type and density of drainage pattern). For this purpose, the nodes are defined as circles of radius R = 25 km surrounding each point of intersection of lineaments. The morphostructural zonation of Italy and surrounding regions, as w ell as the identification of the sites w here strong events can nucleate, has been performed by Gorshkov et al. [21], [22] considering two magnitude thresholds: M 6 .0 and M 6 .5 . In order to have a picture of w hat should be expected if a strong earthquake occurs during a TIP, the scenario of ground motion at bedrock can be computed considering a single node prone to a strong earthquake. In addition research is in progress showing that, by focussing on specific faults included within alerted nodes, it is possible to perform parametric studies, which permit to single out the relevance of source-related effects, like directivity. In figure 2 we supply an example of scenario corresponding to the fault ITIS038 from the database DISS3 (Basili et al. [27]), which is included in the node I26 (Gorshkov et al. [21]). The rupture process at the source and the consequent directivity effect (i.e. radiation at a site depends on its azimuth with respect to rupture propagation direction) is modelled by means of the algorithm dev eloped by Gusev and Pavlov [28] and Gusev [29], that simulates the radiation from a fault of finite dimensions, named PULSYN (PUL se-based wide band SYN thesis).

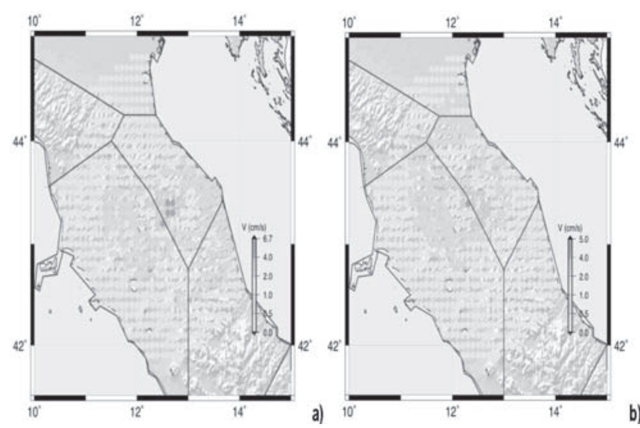

Figure 2: Ground shaking scenarios at bedrock (peak ground velocity, PGV) a) for source directivity south-east; a) for source directivity north-west. The fault ITIS038 from the database DISS3 (Basili et al. [27]), within the node I26 from Gorshkov et al. [21], is considered.

4. Seismic input at local scale and microzonation

While waiting for the accumulation of new strong motion data, a very useful approach to perform immediate microzonation is the development and use of modelling tools. These tools are based, on the one hand, on the theoretical knowledge of the physics of the seismic source and of wave propagation and, on the other hand, exploit the rich database, already available, that can be used for the definition of the source and structural properties. Actually, the realistic modelling of ground motion requires the simultaneous know ledge of the geotechnical, lithological, geophysical parameters and topography of the medium, on one side, and tectonic, historical, palaeoseismological, seismotectonic models, on the other, for the best possible definition of the probable seismic source. The initial stage for the realistic ground motion modelling is thus devoted to the collection of all available data concerning the shallow geology, and the construction of a three-dimensional structural model to be used in the numerical simulation of ground motion. To deal both with realistic source and structural models, including topographical features, a hybrid method has been developed that combines modal summation and the finite difference technique (e.g. Fäh and Panza [30]), and optimizes the use of the advantages of both methods. Wave propagation is treated by means of the modal summation technique from the source to the vicinity of the local, heterogeneous structure that we may want to model in detail. A laterally homogeneous anelastic structural model is adopted, that represents the average crustal properties of the region. The generated wavefield is then introduced in the grid that defines the heterogeneous area and it is propagated according with the finite differences scheme; source, path and site effects are all taken into account, and it is therefore possible a detailed study of the wavefield that propagates even at large distances from the epicentre.

4.1 Applications to cultural heritage and seismic engineering analysis

The proposed methodology has been successfully applied to many areas worldwide (e.g. Panza et al. [8], [9]) and, for the purpose of seismic microzoning, to several urban areas in the framework of the UNESCO/IUGS/IGCP projects -Realistic Modelling of Seismic Input for Megacities and Large Urban Areas" (e.g. Panza et al. [9]) and in the framework of various scientific networks like -Seismic Hazard and Risk Assessment in North Africa", "Seismic microzoning of Latin America cities" and -Seismic Hazard in Asia". Several examples of application of the NDSHA approach can be found in the topical volume on -Advanced seismic hazard assessment" (Panza, Irikura, Kouteva, Peresan, Wang and Saragoni Editors; Pure and Applied Geophysics, vol. 168 ,

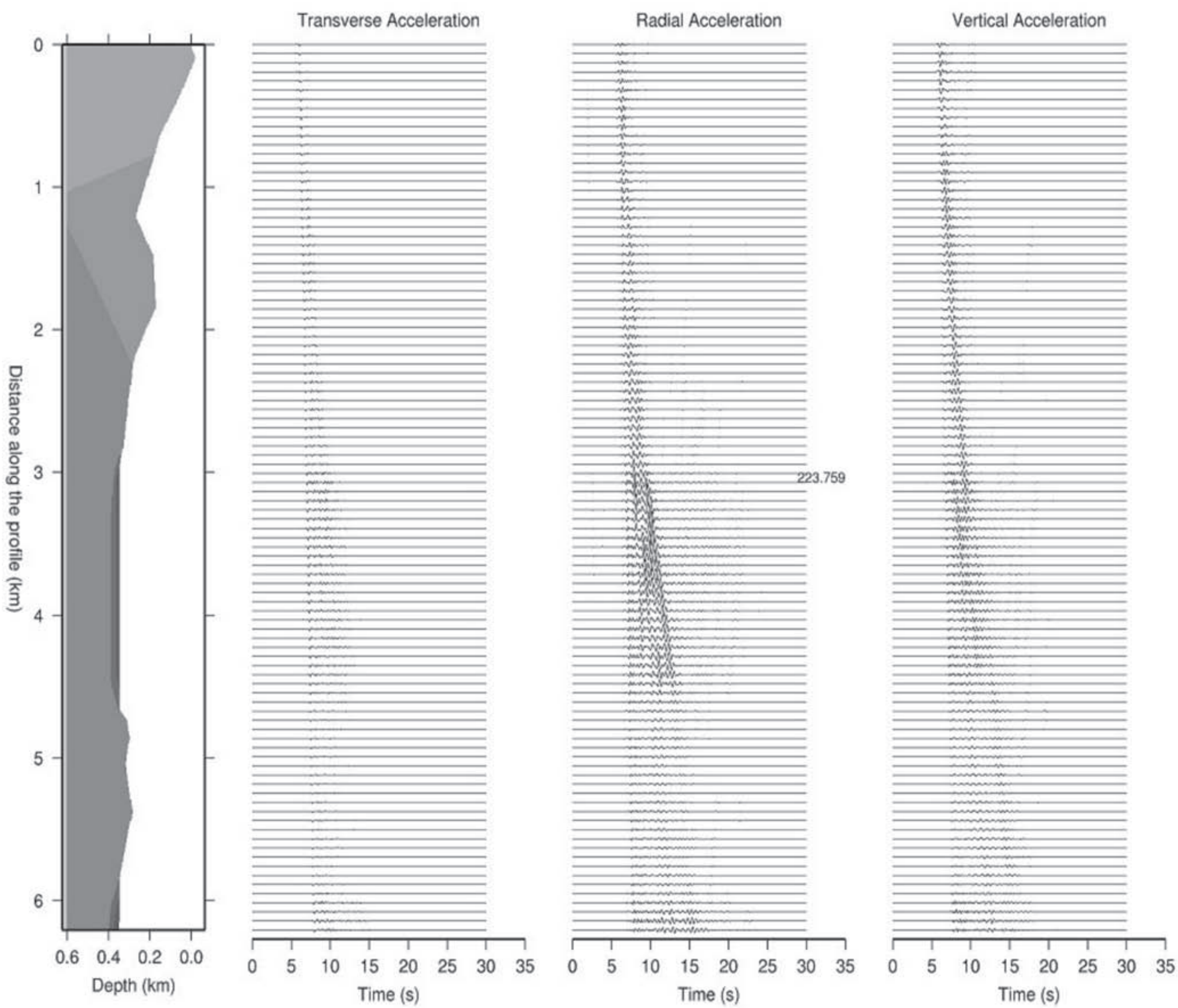

Figure 3 : modelled acceleration along the profile. From top to bottom: vertical, radial and transverse component of motion. The lower panel shows the considered 2D structural profile. All the synthetic seismograms are normalized to the maximum peak acceleration (223 cm/s2), obtained for the radial component about 3 km far from the beginning of the profile (after Vaccarietal. [3 2]).

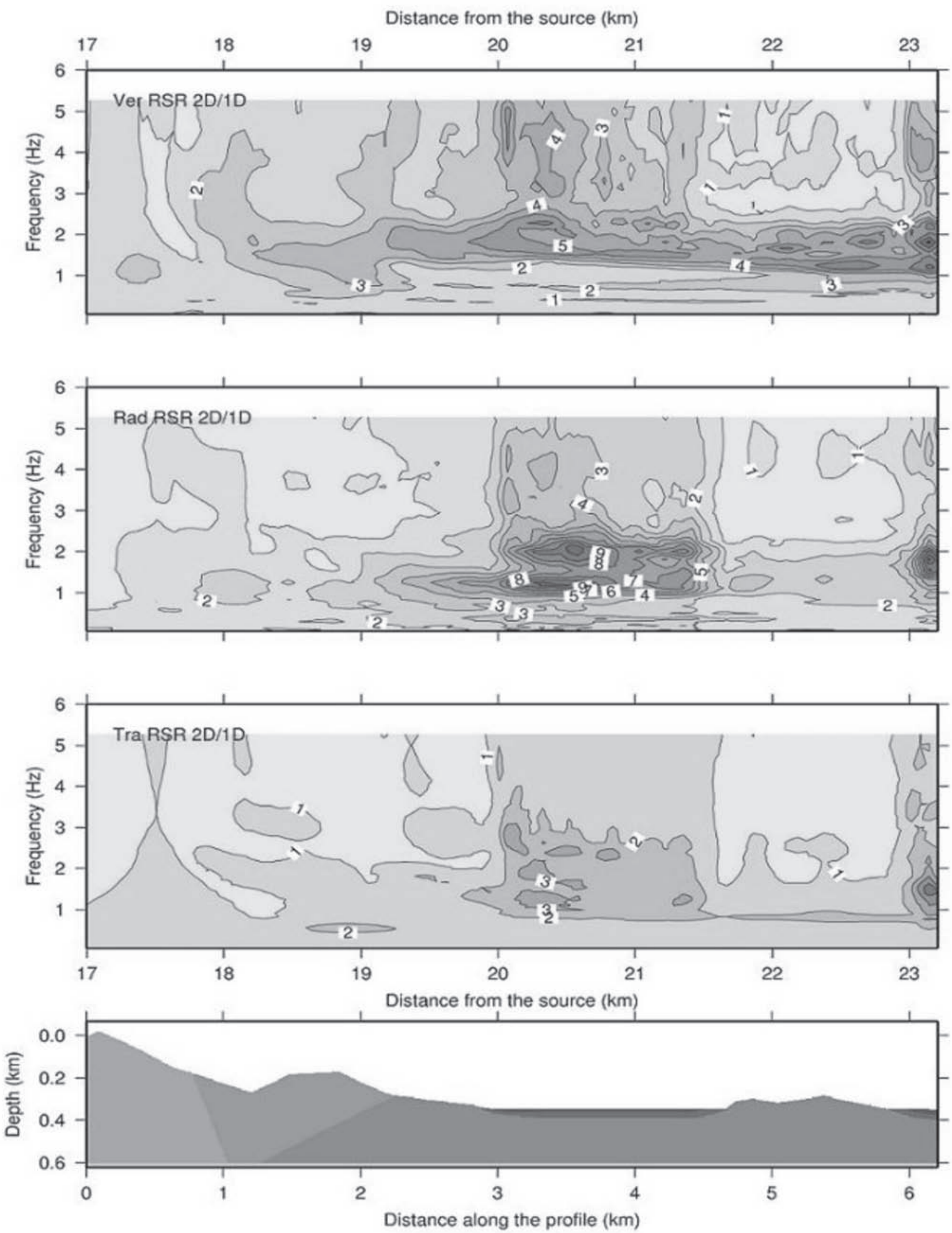

Figure 4 : modelled spectral amplification along the profile, computed from the synthetic accelerograms as response spectra ratios (2D /1 D bedrock). From top to bottom: vertical, radial and transverse component of motion. The lower panel shows the considered 2D structural profile (after Vaccari et al. [32]).

2011). The methodology has been applied to assess the importance of non-synchronous seismic excitation of long structures as well. To take into account the local variability of the ground motion, due to source effects, local lateral heterogeneities and attenuation properties, can be crucial for the realistic definition of the asynchronous motion at the base of bridge piers (e.g. Romanelli et al. [31]).

A pilot application of the proposed approach, including a detailed evaluation of the expected ground motion accounting for site effects, has been carried out, among others, for the city of Trieste (NE Italy). This analysis, described in detail by Vaccari et al. [32], has been performed using a detailed definition of the mechanical properties of the site (and of the seismic source, modelled as an extended source. The rupture process at the source and the consequent directivity effect (i.e. radiation at a site depends on its azimuth with respect to the rupture propagation direction) has been modelled by means of the algorithm, dev eloped by Gusev and Pavlov [28] and Gusev [29]. Along the profile the ground motion is modelled with broadband synthetic accelerograms (maximum frequency 5 Hz) computed by the hybrid technique described in detail by Panza et al. [8]. For the engineering analysis, attention has been focused on Palazzo Carciotti, a masonry neoclassical palace, which has a prominent artistic relevance within the city (Vaccari et al. [33]). The palace is located in the ancient part of Trieste, where soft superficial sediments of poor geotechnical characteristics are present. Ground motion modelling (figure 3) show ed that these site conditions may lead to a peak ground acceleration value of 0 .2 g, greater than the value obtained at bedrock (figure 4). The computed synthetic seismograms have been used as seismic input for the engineering seismic safety appraisal of the building, performing three kind of dynamic analysis, namely: a) a modal analysis, using response spectrums calculated both from synthetic seismograms and according to Eurocode 8, b) a push-over analysis, c) a dynamic push-over analysis. The analysis evidenced that a strong earthquake ($M = 6.5$) occurring in the proximity of Trieste (21 km far from the considered site) could seriously compromise the stability of the structure.

A further application of the NDSHA approach to the definition of the seismic input has been carried out for the municipality of Nimis (Italy), aimed at the design of residential seismically isolated buildings (Zuccolo et al. [16]). The seismic input has been defined considering different levels of detail for the earthquake source, both for a bedrock model and taking into account the specific site conditions. The horizontal response spectrum, calculated in the centre of the municipality by modelling the most dangerous source, advises against the construction of a building with a fixed base, but it is compatible with the seismic isolation, and it has been, therefore, used for the design of a residential seismically isolated building. The maximum displacement for the isolation system has been estimated about 17 cm, a value much lower than that provided by the code design response spectrum (28 cm).

The practical application of seismic input modelling for seismic isolation purposes, and particularly for the protection of historical buildings and cultural heritage, has been already carried out for the tow ns of Marigliano and Ercolano (Naples, Italy), where the computed ground motion has been used for the design of an innovativ e 3D-Isolation system under the Herculaneum Roman ship, as described by Indirli et al. [34]. In addition, the use of seismic input modelling has also been planned to check the data used to design the new seismically isolated building of the Romita High School which is under construction in Campobasso, after the demolition of the two most unsafe pre-existing blocks of such a school (Martelli and Forni [35]), as w ell as for defining the input data for the design of the new headquarters building of the ENEA Centre of Bologna, which should be soon erected on a seismic isolation system in the new -Technopole" of Emilia-Romagna Region in Bologna.

The NDSHA approach has been also applied at a local scale for the City of Valparaiso, in the framework of the -MARVASTO "Project (Indirli et al. [36]), coordinated by ENEA, with the participation of Italian (ENEA, Universities of Ferrara and P adua, ICT P) and Chilean (University Federico Santa Maria in Valparaiso, University of Chile in Santiago) partners.

Finally, a recent example of the practical adv antages that can be provided by the time-dependent definition of ground-shaking scenarios is given by the Cathedral of Santa Maria di Collemaggio, which was severely damaged during the L'Aquila earthquake (April 6, 2009). Based on the alert for the nearby CN region (namely Northern region, figure 1 d), that was ongoing since March 1, 2009, and on the relevant ground shaking expected at the site, the restoration and protection of the cathedral by means of dampers could have been timely completed, possibly limiting if not preventing the occurred damage (Martelli and Panza [37]).

5. Conclusions

The performances of the standard probabilistic approach to seismic hazard assessment (PSHA) proved to be very unsatisfactory, when considering the largest earthquakes worldwide occurred during the last decade (e.g. Kossobokov and Nekrasov a [4]). Lessons learnt from recent destructive earthquakes show that a single hazard map cannot meet all the requirements from different end-users. Nowadays it is recognised by the engineering community that peak ground acceleration (PGA) estimates alone are not sufficient for the adequate design of special buildings and infrastructures, since displacements may play a critical role and the dynamical analysis of the structure response requires complete time series of ground motion. Moreover, when dealing with the protection of cultural heritage and critical structures (e.g. nuclear power plants), where it is necessary to consider extremely long time intervals, the standard PSHA estimates are by far unsuitable, due to their basic heuristic limitations. Therefore the need for an appropriate estimate of the seismic hazard, aimed not only at the seismic classification of the national territory, but also capable of properly accounting for the local amplifications of ground shaking (with respect to bedrock), as w ell as for the fault properties (e.g. directivity) and the near-fault effects, is a pressing concern for seismic engineers.

A reliable characterization of the seismic input is essential for the design of seismically isolated structures, which is based on displacements (Martelli [39]; Martelli and Forni [35], [38]). Accordingly, it is necessary to accurately define the maximum displacement at the period relevant to the isolated structure and the energy content at the low frequencies, which should be expected at the specific site. Since the safety of the isolated structures fully relies on the deformation capability of the isolators to withstand the earthquake and, in several countries, the design forces acting on the superstructure and foundations are somewhat lowered to account for the isolators effects, the design displacement should not be underestimated; how ever, overestimating the expected displacement may also lead to design an isolated structure with an overly rigid behaviour at low excitations, inadequate in case of smaller earthquakes, which are more frequent and therefore are likely to affect the structure during its lifetime. In addition, since seismic isolation requires creating a structural gap compatible with the design displacement, overestimating the expected displacement might lead to renounce applying this technique, especially w hen dealing with the retrofit of existing buildings, where usually there are strict space constraints. Finally, for structures of considerable linear dimensions (e.g. bridges and also some buildings), it is necessary to account for the possible asynchronous ground motion along the base of structure, independently on whether the structure is seismically isolated or conventionally designed.

A viable alternative capable of minimizing the draw backs of traditional SHA is represented by the use of the scenario earthquakes, also named NDSHA, characterized at least in terms of magnitude, distance and faulting style, and by the treatment of complex source processes. The relevance of the realistic modelling, which permits the generalization of empirical observations by means of physically sound theoretical considerations, is evident, as it allows for the optimisation of the structural design with respect to the site of interest. NDSHA naturally supplies realistic time series of ground motion, which represent also reliable estimates of ground displacement readily applicable to seismic isolation techniques, useful to preserve historical monuments and relevant man made structures. It is evident, in fact, that deriving displacements by double integrating the estimated accelerations (as it is done in PSHA) may introduce significant errors; moreover, the peak ground acceleration period is quite different from that of displacements (usually larger). The considerations abov e are particularly relev ant to countries like China and Italy, where, following the Wenchuan (2008) and L 'Aquila (2009) earthquakes, seismic isolators and other anti-seismic dev ices are widely applied (Martelli and Forni [35]).

Current computational resources and recent advances in seismic hazard assessment, along with the acquired knowledge on the response of different structural typologies (including seismic isolators and dissipation systems) supply effective tools for seismic risk mitigation. The proposed time-dependent approach complements the traditional approach to seismic hazard estimates, since it supplies routinely updated information about the expected seismic input. The time information associated to the scenarios of ground motion, given by the intermediate-term middle-range earthquake predictions, can be useful to public authorities in assigning priorities for timely mitigation actions, such as the seismic safety appraisal of strategic buildings and structures. Urban planners and Civil Defence may also highly benefit from the proposed advanced method for seismic hazard assessment, in order to properly evaluate the vulnerability of urban settings and to develop prevention plans, paying special attention to the structures that must be efficiently operating after the earthquake (i.e. hospitals, fire stations, pipelines and other distribution networks, etc.). The aforesaid remarks and proposals are part of those contained in two resolutions concerning recommended modifications of the Italian and European design rules for the isolated structures that are being discussed (February 2011) at the Commission for the Environment, Territory and Public Works of the Italian Chamber of Deputies (Alessandri et al. [40]; Benamati and Ginoble [41]).

Acknowledgments

This research has been developed in the framework of the ASI -Pilot Project SISMA and has been partly supported by the Regione Autonoma Friuli Venezia Giulia (Civil Defence - ICTP Agreement DGR 2226 dd. 14.9.2005 and DGR 1459 dd. 24.6.2009).We are indebted with Virgilio Fiorotto and Christian Marsonfor very stimulating and useful discussion.

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Art and Structural Engineering Art of Structural Engineering

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Summary

Beyond the classical dispute between architects and engineers, it is a matter of fact that art and engineering are not only compatible, but also for some cases indissociable. This communication aims to contribute to the debate. In the first part the experience of artists is exemplified in terms of influence on structural engineering : Pevsner and Snelson opened the way to complex surfaces and innovative structural composition. Following David P. Billington proposal for a so called “Structural Art”, some few examples are described enhancing the symbolic dimension of realizations, whose morphology reflects artistic processes. Some explanations provided by cognitive sciences help to understand the mental process of designers and the required conditions for a creative process. Historic examples of artistic movements like “Constructivism”, “Bauhaus” and the experiment of “Black Mountain College” illustrate the benefice of crossing experiences of designers, whatever can be their field.

Keywords

Art, structural engineering, structural composition

Theme

Structural and architectural design

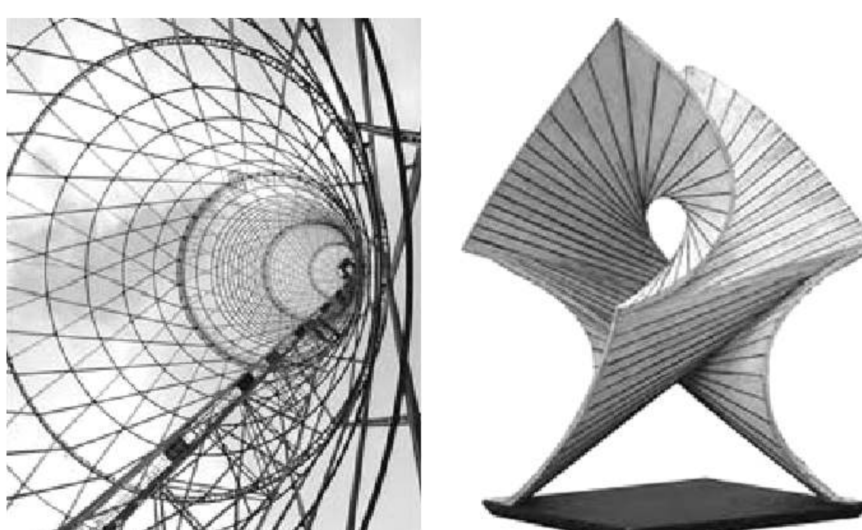
1. Introduction

Beyond the classical dispute between the architect and the designer, there is another one debate which is of interest for structural engineers: can we speak of Art when Structural Engineering ends with some solutions which are recognized as pieces of art, like it is for the Eiffel Tower, and more recently the Millau Viaduct ? This communication intends to provide some arguments following previous publications reflecting our interrogations (Motro [1]), and willingness to associate in the same project at the training level architects, artists and engineering (Aanhaanen [2]). We firstly present some cases enhancing a close relationship between art and structural engineering, and secondly we describe some of the features and conditions of situations where structural engineering becomes an art.

2. Art and Structural Engineering

2.1. Pevsner and Xenakis : From Constructivism to Pavillon Philips in Brussels

Constructivism is an artistic movement born in Russia at the beginning of the XX° century. Brothers Pevsner and Gabo wrote its manifesto. This movement proclaims a geometrical construction of the space, using especially elements such as the circle, the rectangle and the straight line. This way of thinking adapts itself also well to the sculpture as to the design even in the architecture. A somewhat recent exhi-



A B

Figure 1: Constructivism : A-Hyperbolic Tower Moscou 1922 (Choukhov)
B-Developable surface (Pevsner)

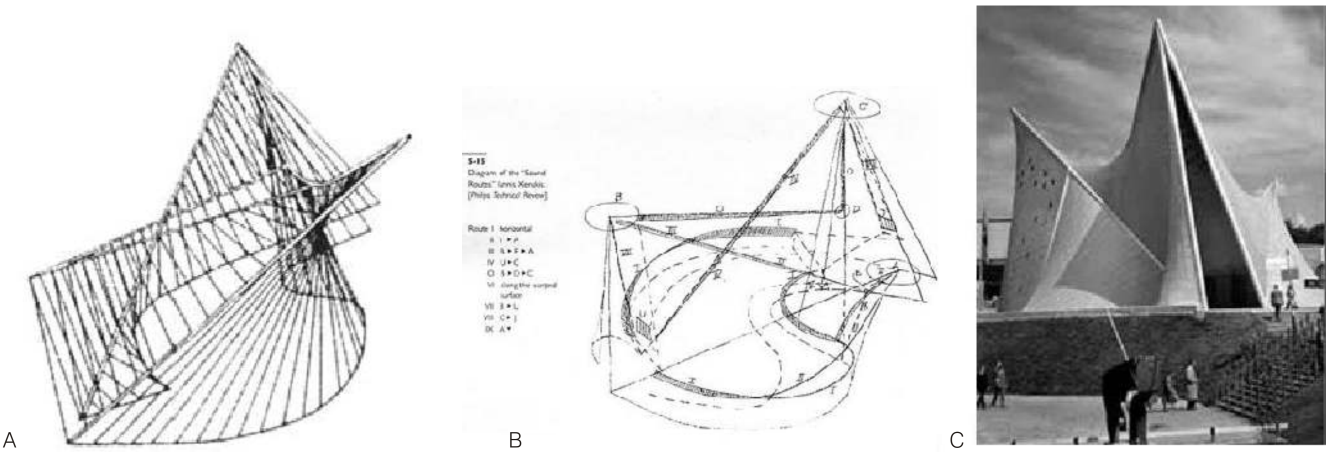


Figure 2 : Pavillon Philips : A Ruled surface - B Sketch by Xenakis - C Realization

bition held at the Guggenheim Museum, "The Russian and Soviet Avant-Garde, 1915-1932", gives more details on the work of constructivists who organised their first exhibition manifestation, the Obmokhu (the Society of Young Artists) in May 1921. Rodchenko, one of the constructivists, claimed in January 1921 (Lodder [3]):

"All new approaches to art arise from technology and engineering and move towards organization and construction".

Artists and engineers are indistinctly members of this group. Vladimir Grigorievitch Choukhov, a famous Russian engineer was member of the Constructivism. He developed and achieved several hyperbolic towers. In the field of metallic structures design (Figure 1 A). Choukhov is one of first to develop practical methods of calculation of the efforts and the elastic deformations of the beams, the shells and the membranes. His designis based on ruled surfaces.

Following the same geometrical principle Antoine Pevsner realized many sculptures based on ruled surfaces and sometimes developable (Figure 1-B). Such artistic achievments opened the way to double curved surfaces generated by straight lines.

In 1958, for the Universal Exhibition in Brussels, Yannis Xenakis designs the so-called "Pavilion Philips". Yannis Xenakis, mathematician, musician and architect worked at that times with Le Corbusier. It is a matter of fact that the two men disagreed during this event, but this dispute is beyond our text. What we want to underline is the formal analoghat we want to underline is the formal analog between this Pavilion and the sculptures presented by Pevsner. It is obvious that the geometrical design, based on ruled surfaces (Figure 2 A) is clearly in the line of Pevsner's sculptures. The sketch of Figure 2 B, even if difficult to read has been drawned by Xenakis (his name is mentioned on it), the third illustration (Figure 2 C) represents the completed structure.

It is interesting to note that the double negative curvature surfaces result from the assembly of double negative curved paving stones poured on sand on the ground of a rough dimension of 1 m 50 aside. These paving stones are 5 cms in thickness; they are supported by a double network of prestressed cables 8 mm in thickness. Following this realization, and keeping the same concept of surfaces with negative double curvature, Yannis Xenakis will design steel and membrane system for his famous "Polytope" raised behind the Pompidou Center in Paris.

2.2. Kenneth Snelson, Forces made visible

Kenneth Snelson is an artist who is at the key point between art and structural engineering. Many papers have been devoted to his pionneer work in the field of tensegrity structures. His controversy with Buckminster Fuller he met in Black Mountain College in the early fifties is beyond the scope of our paper. He himself explained his view in a public letter inserted as annex of the book that I devoted to tensegrity (Motro [4]). The most important thing to note is that he had a pure artistic behaviour and he reached a first concretisation of the tensegrity concept by working successively on three sculptures ending in the well known "Double X". Every "X" component described in his patent is strut like element. Using an assembly of three (Figure 3) gives access to the classical simplex (Motro et al [5]).

In 2009 Snelson had an exhibition in Marlborough Gallery, Chelsea, New York. The title was "Kenneth Snelson Forces Made Visible, and this is also the title of the book edited for this opportunity (Hartney [6]). The best title for the work of this artist who is able to make forces visible. Forces are a mechanical concept useful for engineers who want to size their structures and they are by nature invisible. On the

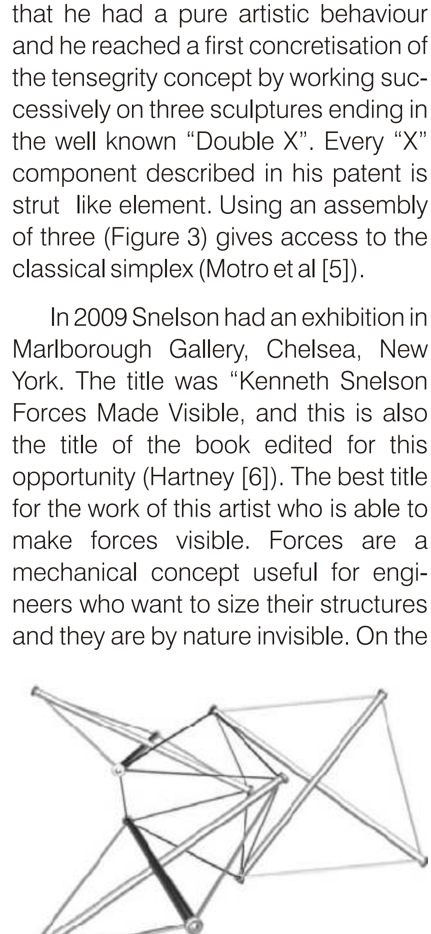
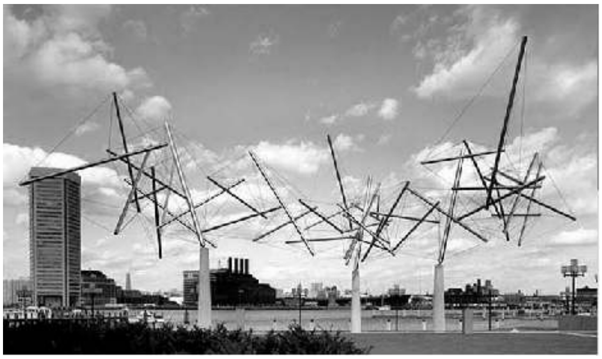


Figure 3 : Simplex generation by assembly of three "X" components


A


B

Figure 4 : Snelson sculptures A Easy Land, Boston B Tensegrity Tower

other hand forms are visible and measurable, and they are the product of the artistic process. Why are forces made visible? Some keypoints may be put forward:

» By differenciating clearly cables and struts, Snelson's sculptures provide an information on whether tension or compression is present..This is not the same for classical reticulate systems

» If the level of tension and/or compression can be qualitatively evaluated according to the external diameter size of components, it is insufficient since this level is depending upon the material, and the thickness of tubes, or the arrangement of cables.

» The very specific structural composition surprises and fascinates everyone seeing them for the first time: struts seem to float in the air. And this is also a key point, since people, and engineers more than the others, are surprised by this new kind of flow of forces. They are accoutumated to gravity effects, and in this case gravity seems to be absent. The artist provokes interrogations by submitting an unknown process for transmitting forces.

» Last but not least: art is a source of emotional feeling. In case of tensegrity systems people feel that at every end of each strut, cables contribute to the equilibrium. And it cannot be possible without an amount of stress of tension in cables, and compression in struts: struts are "lifted" inside the structure by a continuum of cables. But or course it is not easy to understand how all these forces are distributed, we only know that the whole is in equilibrium, stressed and stable

Finally it can be said that the artistic work by Snelson obliges the structural engineers to question their structural approach, and to enrich it.

3. Art of structural Engineering

3.1 Structural art

Some engineers brought up their practice at the level of the art. Several were celebrated during the exhibition The Art of Engineer, Builder, Contractor, Inventor, held in Paris 1997 (Picon [7]).

"The Tower and the Bridge" (Billington [8]) has as subtitle "The New Art of Structural Engineering". In this book many famous engineers are presented in their artistic way of designing: Thomas Telford and Gustave Eiffel, Robert Maillart, Felix Candela and Heinz Isler among others. Let Billington speak about this structural art:

"The conservative, plodding, hip-booted technicians might be, as the architect Le Corbusier said, "healthy and virile, active and useful, balance and happy in their work, but only the architect, by his arrangement of forms realizes an order which is a pure creation of his spirit…it is then that we experience the sense of beauty". The belief that the happy engineer, like the noble savage, gives us useful things but only the architect can make them into art is one that ignores the centrality of aesthetics to the structural artist"

Following this affirmation he describes the three "dimensions of structure": scientific, social and symbolic. The symbolic dimension is closely related to aesthetics. These classical virtues Firmitas, Utilitas, Venustas enhanced by Vitruve could appear as an old history without any interest. Nevertheless if "Venustas" is recognized as one of the characteristics of artistic manifestation, some engineers practiced a real art.

3.2 Morphology in question

There is always a close relationship between the resulting morphology of a design process and the personality of the designer. The following quotation is generally attributed to Victor Hugo:

"La forme c'est le fond qui remonte

Figure 5 : Garabit Viaduct Gustave Eiffel

Figure 6: Millau Viaduct Foster and Virlogeux

à la surface" A direct translation would be without meaning. The idea is that every form is somewhat the visible result of an invisible content. The external appearance is insufficient to conclude, and it is a matter of fact that in the case of the Millau Viaduct (Figure 6), known as Foster's project, the resulting morphology is strongly related to Michel Virlogeux's work. We can perhaps claim that Virlogeux has been the soul of this project. The soul assesses the expression of the designer through the resulting morphology among other parameters. At the beginning of the process there is at least one symbolic choice that is not related to scientific or social dimensions. Michel Virlogeux said that in this case the main idea was to design a viaduct not to span the river Tarn, but to span the entire valley.

For other cases the morphologic appearance allows one to identify the designer. Isler's (Figure 7) morphologies are characteristics of their strong expression as designers, but the morphology is also the result of a strong coupling between form and forces: they are somewhat "funicular shapes" as Gaudi's Colonia Güell.

3.3 Which way to Art of Structural Engineering?

3.3.1 New design situations

Nowadays, the scientific aspects of design are helped by the increasing power of available numerical tools, but if designers can use them at the different stages of the process, since the initial idea to the realization of the project, they cannot reduce their work to manipulation of tools. They are now freer for expressing their own personality by a continuous process, and as artists can experiment, they may simulate by prototypes and numerical modeling the continuous materialization of their project. Besides, let say, the classical artists they have to take into specific dimensions as claimed by Billington: scientific (in terms of mechanics of material and structures), and social (adequacy with the ongoing progresses of technology, the cost necessities, and the apparition of new constraints like environmental ones). Nevertheless some of them are sufficiently imaginative and also creative to submit new solutions evolving from their experience and meeting the actual constraints. If the design process in structural engineering is governed by the scientific dimension, true designers do not provide the same solution to a given problem. Their own experience and way of thinking are conditioning the quality of their proposal. There are many differences with classical manifestations of art like size of construction and the necessity of permanence in terms of security, but the mental process is of the same kind. Similarly training conditions are also very important, and common training with artists and architects may contribute to increase the level of their art of engineering.

3.3.2 Contribution of the cognitive sciences

According to the preceding remarks, it appeared interesting to try to identify some characteristics of the mental behavior of a designer regardless of his training as architect, engineer or artist. Taking advantage of cognitive sciences results we could investigate more precisely some aspects of the design process. One major issue is related to the mental representation for the designer of an existing or a projected object. Perception, and concep-

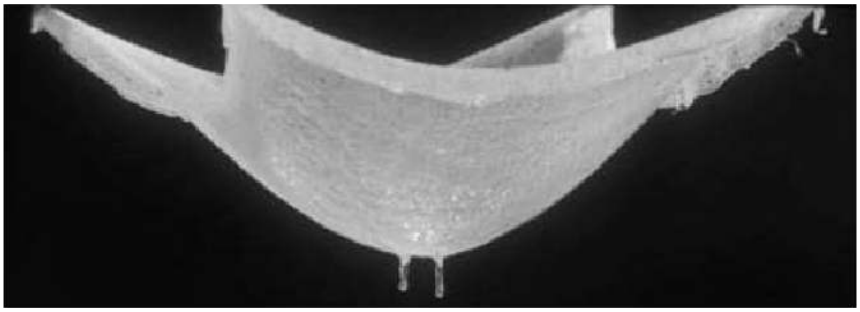


Figure 7: Concrete Shell by Isler and experimental funicular model

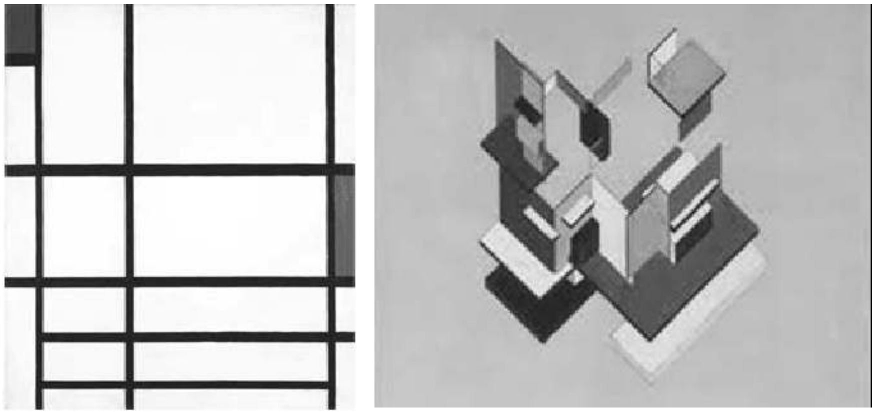

A B

Figure 8: A Piet Mondrian painting B Project by Theo Van Doesburg.

tion are two faces of this issue. The perception of real 3D objects is firstly the source of a mental representation in the so called "working memory" of the designer. These mental representations are progressively stored in the long term memory of the designer. The link between mental and physical worlds, during design process and morphogenesis, takes advantage of "a knowledge tank" (so called "Long term memory"). This tank is filled step by step during designer's life according to his perception of the physical world and his own skills.

During the conceptual design phase mental representations of previously perceived objects arise in the working memory of the designer, coming from his long term memory. This long term memory is like "a memory tank" which is filled by perception operations and training, and which is also dependent of heredity, culture, training, travels, discussions…An iterative work is then operated and requires a progressive materialization of the projected solutions, generally by means of 2D representations (computer's screen or sheet of paper) and/or by means of physical models. This iterative process is a sequence of problem solvings in working memory. They give access to solutions (hypotheses) which are analyzed and compared with known solutions stored in the long term memory . This long term memory is continuously enriched by perceptions and training. A major point is that the designer is not always filling his long term memory with elements, he is also building links between the information of increasing complexity, he is building procedures and he memorizes these procedures. On the basis of this cognitive approach undertook by Silvestri [9], and associated with an experimental study (concerning more than thirty people), we have some information useful for a better understanding of the mental process of designers.

3.3.3 Creative Movements

We evoked at the beginning of this paper the role of Constructivism. There was also a similar attempt with "De Stijl" with Pietr Mondrian and Theo Van Doesburg (Figure 8). A true and free dialog between architects, engineers and artists was clearly fruitful in other places where they worked together like the Bauhaus or Black Mountain College inviting designers to exchange their thoughts for a better mutual understanding. In Black Mountain College (Figure 9), where Snelson was sculptor student in 1948, the experimentation was the main adopted principle as it can be understood by reading the essay by Diaz [10] who writes :

These three models of experiment- the methodological testing of the appearance and construction of form in the interest of designing new visual experiences (Albers), the organization of aleatory processes and the anarchical acceptance of accident (Cage), and "comprehensive, anticipatory design science' that propels, teleologically current limited understanding towards a finite totality of universal experience (Fuller)- represent important incipient yet disparate directions of post-war art practice, elements of which would be sampled, if not wholly adopted, by Black Mountain students and subsequent practitioners.

Again, like for the famous Bauhaus, cross pollination between creative people creates the best condition for improving the experience of every one, and enriching his knowledge gained by very separate practices that have in common creativity and artistic attitude in common. Generally engineers who shared this kind of training have more chance to reach the Art of Structural Engineering.

Conclusions

At the era of computers and numeric models, the engineers have an impressive set of tools that they can

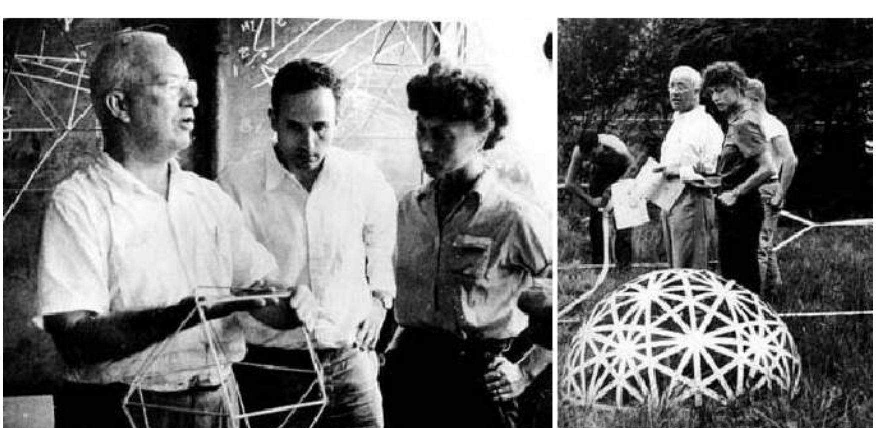


Figure 9: Buckminster Fuller teaching at Black Moutain College

use during their design process. But they remain, as human, the key element able to make their practice an art. Some of them reached this level for the benefit of mankind. Memory, experimentation, own culture, imagination, creativity are the prerequisite for this art of structural engineering that needs to provide appropriate answers to the three dimensions quoted by D. P. Billington : scientific, social and symbolic.

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New Stadia Structures

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Summary

South Africa's successful application for hosting the World Cup has resulted in the construction of several new stadia. The construction or modernization of these new arenas coincides with novel ideas for the design. In all cases the roofs play the major role when it comes to the question of unique design and easy recognition.

The roof is the most important element to create the stadium look. Since the cable net structures for the Olympic Stadium roof in Munich, we use all our knowledge and creativity to make the roof of a stadium the star of the event.

This can easily be observed in South Africa. The stadium roofs in Johannesburg (Soccer City), Durban, Port Elizabeth and Cape Town were designed by structural consulting engineers schlaich bergermann und partner under the lead of Knut Göppert, are already widely known for their special designs.

Roof structures for enormous spans had to be developed, often with a depth of more than 50 m. Spatial load transfer structures are state of the art when it comes to efficient and sustainable engineering solutions.

Fabrication and erection aspects influence the early design ideas and lead to unique installation processes.

The authors present the four projects in detail and provide a view into the future of stadium roof design.

Keywords

Stadia, roof, cable net structure, special design, spans, efficiency, sustainable engineering solutions.

Theme

buildings - construction - design

1. Soccer City Stadium Johannesburg

The earth coloured 'African Pot', the traditional calabash, as a melting pot of cultures can be seen as the unique Pan-African symbol. This was the main idea of the architects for the design of the biggest stadium on the African continent.

The stadium can accommodate 94 000 spectators. The three tier levels can be reached via curved inner and outer ramps.

The Stadium is located in close neighbourhood to Soweto. Historical parts of the upper tier level of the old stadium (built in 1987), where Nelson Mandela was present during important events have been retained and conserved, whereas the whole rest of the stadium has been newly built.

Specially arranged coloured façade-panels with 6 colures and 3 textures on the surface changing from darker to lighter colours on the top as well as the light coloured upper membrane blend into the natural surroundings of the typical local 'mine-dumps'.

Starting from the architects images and ideas, schlaich bergermann und

Figure 1: Image by Steven Wilbrenninck/Interbeton

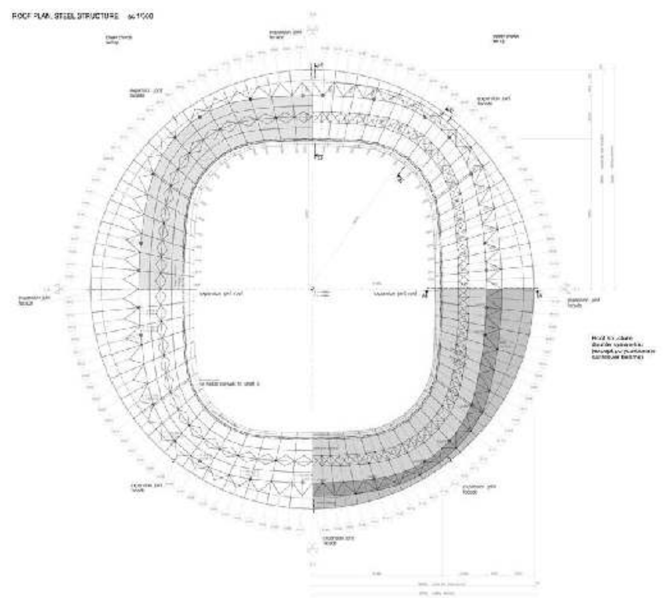

Figure 2: Image by sbp

Info

Client : City of Johannesburg

Structural Engineers :
Roof and Facade: schlaich bergermann und partner, Stuttgart

Concrete Structure :
PDNA, Johannesburg

Architects :
Boogertman and Partners (Bob van Bebber / Piet Boer), Johannesburg

Wind Engineering :
Wacker Ingenieure, Birkenfeld

Contractors:

Main contractor : JV GLTA/Interbeton South Africa / Netherlands

Main contractor roof :
Cimolai, Pordenone / Italy

Membrane Subcontractor :
Hightex, Rimsting / Germany

Fibre concrete subcontractor :
Rieder, Austria

Capacity : 94.000 seats

Steelwork : 9.000 to

Roof area : Upper membrane: 23.000 m^2 PTFE/Glass, Lower mesh membrane: 25.000 m^2 PTFE/Glass

Vertical mesh membrane : 2.000 PES/PVC

Glazing :
12.000 m^2 Polycarbonate, t =12 mm

Facade area : 35.000m^2 fibre reinforced concrete, t=13mm

Start design - Completion :
April 2006 February 2010

partner designed and developed the optimised structure for the huge roof and the façade structure in close corporation with the structural engineers of the concrete tiers structure, PDNA of Johannesburg.

The overall shell geometry (roof and façade) derives from a torus with an outer diameter of 300 m which was defined in section be several radiuses.

As the opening of the roof and the surrounding spatial ring truss (the most important structural element) follow the rectangular shape of the field and the arrangement of the tiers, both geometries combined create an impressing 3 dimensional curved structure.

The 800 m long spatial ring truss with its 3 chords (circular sections) is clad with polycarbonate panels. It is supported by 12 concrete shafts and 16 columns only.

The roof cantilever truss (open sections) length over the tiers is 38m. The cantilevers are clad with arch supported PTFE membrane on the top side and an open mesh Glass/PTFE membrane on the bottom side. For the cladding of the inner edge of the roof also polycarbonate panels were used. Different from many different arch supported membrane roofs for stadia, the soccer city stadium uses a radial arch arrangement. This considers the comparative small panel size as well as the architectural idea of the "finger formed clay pot".

The slim shell structure of the façade, designed as curved steel beam is supported on inclined concrete columns and fixed on the top to the spatial ring truss.

The glass-fibre reinforced 13 mm thick concrete façade panels 1,2 m x 3,6 m have been fabricated with natural colours and different textures and arranged exactly according to the architects special patterning design.

2. Moses Mabhida Stadium, Durban

As part of the City of Durban´s redevelopment program the projected World Cup stadium was chosen to create an icon for the KwaZulu Natal region and Durban, being the 2nd largest city in South Africa. The ambitious plan to gain international attention enabled the lead Architects von Gerkan Marg und Partner, Berlin, the lead structural engineers schlaich bergermann und partner, Stuttgart and BKS, Durban to design an outstanding stadium of unprecedented scale and beauty and therefore won the design competition.

The scope of work for the structural engineers did also include all erection engineering, the checking of all workshop drawings and surveys, the site and fabrication supervision as well as the technical lead for the tender process and the implementation phase for the client.

The multipurpose stadium with a possible capacity of 85.000 seats features a unique roof structure of 46.000m^2 of Glass/PTFE membrane being prestressed against a cable net. The cable net is tensioned against two steel compression rings along the perimeter of the stadium and a mayor arch structure with 103m height and 360m distance between its foundations.

Governed by the high wind loads in close proximity to Durban´s coastline the membrane structure required a

Figure 3: Image by Markus Bredt

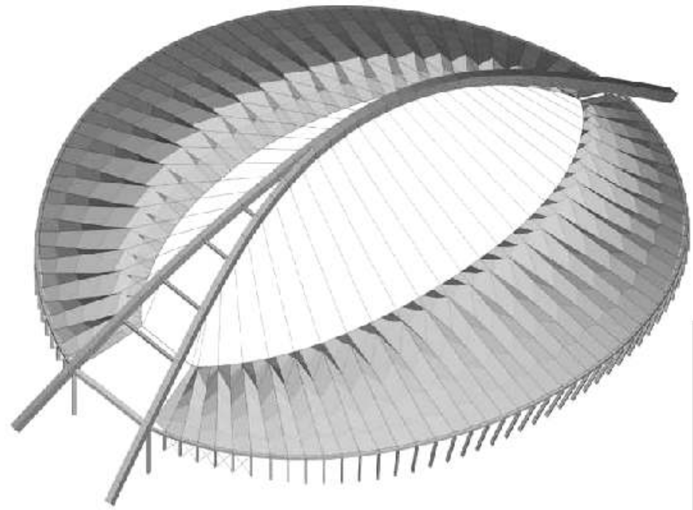

Figure 4: Image by sbp

rather dense cable support structure to minimize the membrane stresses. To achieve a global safety factor of greater than 5, as stipulated in the European design guide for tensile surface structures, the distance in plan between ridge and valley cables needed to be reduced to a maximum distance of only 8m at the outside perimeter (compression ring). Often the installation procedures reduce the material strength remarkably, especially when effective quality control in missing. To achieve the required safety factor in reality, a detailed investigation into the strength deteriation due to, manufacturing, handling, packing and installation was undertaken, to overcome the critical strength reduction due to folding of the glass/PTFE material. In order to not accept mishandling the breaking strength of the virgin material and the installed material was defined and tested in several stages, even on installed material by taking out sub panels for testing.

Info

Client :
City of Durban - eThekwini Municipality

Structural Engineers :

Roof : schlaich bergermann und partner, Stuttgart

Concrete Structure :
BKS Durban, South Africa

Architects : gmp Architekten, Berlin

Wind Engineering :
Wacker Ingenieure, Birkenfeld

Contractors : Main contractor: JV WBHO / Group 5, Main contractor Roof: Pfeifer Seil- und Hebetechnik, Memmingen, Membrane subcontractor: Birdair, Buffalo

Capacity : 70.000 World Cup mode, 54.000 legacy temporary seats will be substituted by conference facilities, 85.000 Olympic mode

Roof area : 39.000 m^2 (vertical projection), 46.000 m^2 (membrane surface)

Steel work : 2.860 to main arch, 2.700 to compression ring and columns, Cable structure: 550 to

Start design Completion :
March 2006 November 2009

The form found and most effective structural shape of the stadium roof drains 75% of the rain water directly towards the gutter located at the compression ring. The remaining water is firstly running towards the tension ring, before it can be redirected naturally towards the compression ring in the areas underneath to the arches. To achieve this the membrane shape and the gutter located on the tension ring had to be form found in a specific manner. All membrane connections close to the gutter were connected using continuous cables and cable clamps requiring local membrane cut outs with unsupported membrane edges. These locations were designed and tested using bias cut and straight cut membranes.

The installation of the membrane panels was following structural design criteria that were derived using the local wind data of prevailing winds. Due to the open shape of the partly installed roof the wind loads during construction and therefore membrane stresses were higher than for the final building, even though reduced wind pressures for construction stages were used. Several construction stages, also for the membrane installation, had to be assessed in the wind tunnel.

3. Nelson Mandela Bay Stadium Port Elizabeth

The Port Elizabeth, one of the South African cities selected to host the games of 2010 Fifa World Cup had the special challenge of building a world class sports arena.

A German design team started in 2005 the planning of the stadium, aiming to design a signature landmark that could be at the same time a structurally and economically meaningful building.

Contractors from South Africa, USA, Australia, Japan and Kuwait worked during 42 months in its construction, until its completion in April 2009.

Particular boundary conditions, as the frequent wind and an extremely corrosive environment, due to high temperatures combined with a high degree of humidity and salt content of the air

Figure 5: Image by Grinaker-LTA

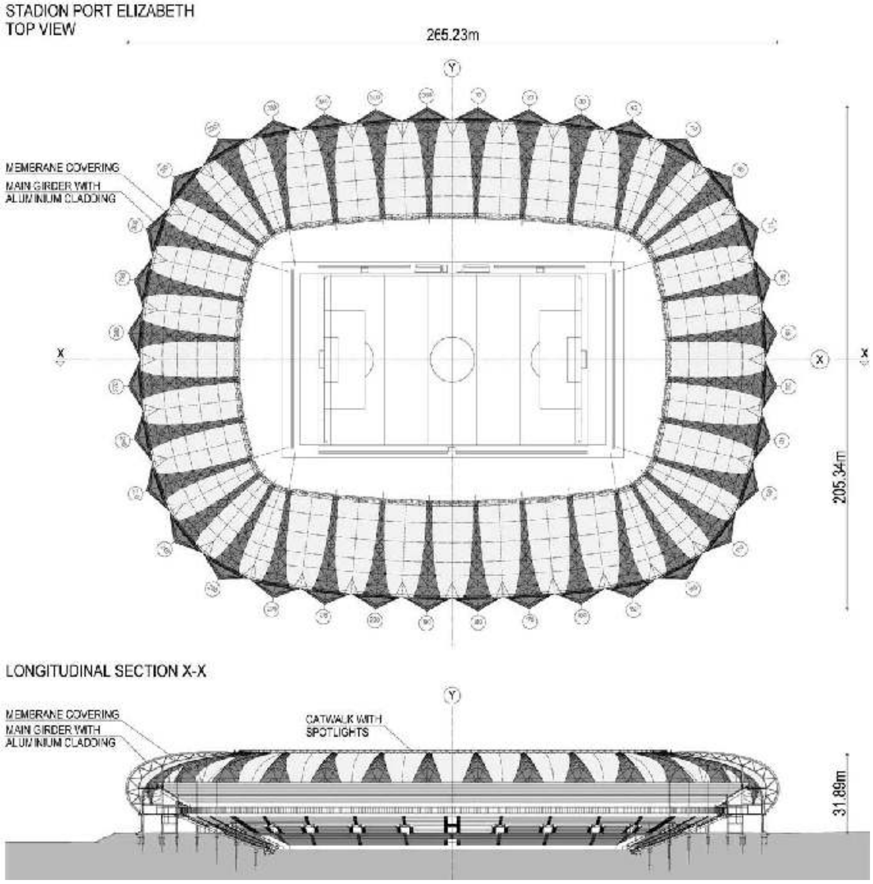

Figure 5: Image by sbp

required individual solutions from the very initial design of the roof until its final completion.

The architectural planning team was inspired by the privileged site, an elevated platform next to the North End Lake, to create a building that could be remarkable and visible from afar.

Roof and facade had a fundamental role in the planning process integrated in an interesting interplay of concave and convexes forms, both created not only the stadium identification, but functionally they also provided a wind shelter for the internal stands. In addition to the wind tunnel tests to determine the wind loads acting onto the roof, a wind comfort study for the stands and the field was performed to provide the maximal comfort to the spectators as well as the owner's confidence in the project. Again, schlaich bergermann und partner's approach to control all stages of a project, from first design ideas until the last bolt being placed, was a fundamental contribution to the success of the project.

Thirty-six triple chord steel girders of spatial tubular framework, cantilevering over the grandstands, carry the roof and simultaneously articulate the unique outer appearance of the stadium like petals of a flower growing on top of monumental facade columns and tapering off towards the centre of the stadium, all together forming a calyx in a ridge and valley shape.

The girders, clad with aluminium standing seam sheeting, form the ridges. Fabric panels of PTFE coated glass fibre membrane, spanning between the girders, form the valleys and dewater the roof. The alternation of

Info

Client : Nelson Mandela Bay Metropolitan Municipality

Design Team Roof : von Gerkan Marg und Partner, Berlin architects, schlaich bergermann und partner, Stuttgart - structural engineers, Iliso Consulting, Port Elizabeth - structural engineers, local co-ordination Wacker Ingenieure, Birkenfeld wind engineering

Contractors : General Contractor Joint Venture Grinaker-LTA / Interbeton, Steel and Membrane structure: Taiyo Membrane Corporation Birdair, Australia/USA, Aluminium Cladding: CC George Roofing, Cape Town, South Africa, steel manufacture: ABJ, Kuwait

Capacity :
48600 seats, thereof 45 940 permanent

Covered area : 30000 m²

Steel Structure : 2300 t

Membrane :
22000 m² Glass-PTFE membrane

Aluminium Cladding : 22500 m²

Costs Roof : ca. 22 mio €

Start design Completion :
November 2005 - April 2009

Figure 6: Image by Bruce Sutherland

translucent and opaque material is visible as a series of illuminated surfaces: at day from inside - at night from outside. A ring beam connecting the tops points of the girders forms the inner edge of the membrane bays and carries a circular walkway integrating the flood light system.

Polysyloxane based corrosion protection with high UV resistance has been chosen for the structural steelwork; movable and accessible parts are duplex coated. All relevant details of connections of the membrane and the aluminium sheeting had to pass a long term salt spray test to demonstrate their applicability.

Port Elizabeth stadium construction performed an interesting experience in the coordination of international suppliers and contractors, in a complex but successful example of globalisation. The steel framework girders were prefabricated in Kuwait and shipped to Port Elizabeth in parts. Next to the bowl, the girders were assembled on falsework templates considering tight tolerances. After surveying, the 45m long 25m wide curved 55t trusses were lifted in position on top of the R/C structure by a crawler crane. Due to a maximum wind speed for the crane activity, most lifts had to be done in the early morning hours and had to be finished before the wind frequently started to increase. The membrane panels, fabricated in Japan, were unfurled on a temporary net spanning between the girders and pre-tensioned in steps. A big part of the aluminium sheeting has been installed by climbers sheet by sheet since only the area next to the facade was accessible by a movable shoring tower.

Besides accommodating sports events like a 2010 quarter final, the stadium also houses conference rooms, offices, gastronomy and corporate boxes.

The major legacy of Port Elizabeth stadium is, therefore, its remarkable contribution to the revitalization of the quarters around North End Lake, transforming the area with its architectural presence and opening its doors for the public, even beyond the final whistle of the 3rd place playoff match in South Africa this year.

4. The new Cape Town Stadium

The new Stadium in Cape Town is set into the spectacular scenery of the Table Mountain, Lions Head, City Bowl and Atlantic Ocean. Although the location within the Greenpoint Common Area was controversial at the beginning set in-between a golf course, cricket grounds, tennis courts, etc. the completed building shows a respectful integration into its environment and surroundings. The design was driven by two main criteria: first, the city set forth specific criteria, which limited the maximum height of the building. The second parameter was the dominating impression of the horizontal silhouette of the Table Mountain. A simple spoke-wheel roof, as has been done many times before for other stadiums, was not possible due to the required height of the columns, which would have exceeded the limit by far. Even a cantilevered roof would have required a construction height at the outer edge that would have exceeded the set limit. Furthermore, to create a counterpoint to the Table Mountain, the eaves of the building should have an intentional curvature to their shape.

The final result is a roof design consisting of a strongly undulating compression ring, a suspended cable net and an elevated truss girder structure. The latter one stabilizes the "soft" cable net in case of unbalanced loads and lifts up the actual roof surface to an elevation that allows for natural dewatering to the outside. Since the suspension roof requires additional weight to bear uplift forces, the entire cladding system of the roof was designed as glazing, a first for any stadium roof. A mesh membrane, located underneath the truss girders and spanning between the radial cables, closes the roof void from below and positively influences the visual appearance, acoustic behaviour and wind exposure level. Only the cantilevering part of the structure, from the ring cable to the inner roof edge remains uncovered from below. The glazing above this area is clear, whereas on the rear part of the glazing (with the membrane underneath) a layer of white print was applied to its lower surface. Great importance was also attached to the design and appearance of the roof surface, since it is visible from many highly frequented observation points as well as from the higher situated properties of Greenpoint, and therefore acts as a "facade".

The lateral facade consists of an off-set steel structure made of vertical and horizontal beams, which are connected to the concrete structure behind by diagonal struts. The horizontal beams, curved in plane, are arranged in front of the facade surface and therefore form a visible horizontal division of 14 strips. The steel structure is covered with a silver coloured mesh membrane. Its doubly-curved single panels form an almost chameleon-like skin, which intriguingly changes the appearance of the stadium in colour and translucence depending on the exterior lighting conditions. Due to the seaside location and typically extreme wind conditions in Cape Town the wind loading was, of course, the governing aspect of the design of the structure. A particularity for the design was that not only the pressure values according to the standards had to be taken into account, but also the specific topographic conditions at that particular location between Table Mountain, Signal Hill and the close sea. The final wind loads acting on the structure had to be investigated by two different wind tunnel tests. The first one used a topographic model to determine the specific local wind and gust conditions, while the second one applied these first results to the model of the stadium building.

Info

Client : City of Cape Town

Structural Engineers :
Roof and Facade: schlaich bergermann und partner, Stuttgart

Concrete Structure :
BKS Bellville, South Africa

Architects :
Stadiumarchitects (gmp Berlin, Louis Karol, Point Architectsboth Cape Town)

Wind Engineering :
Wacker Ingenieure, Birkenfeld

Contractors :

Main Contractor : JV Murray&Roberts WBHO, roof: JV Pfeifer-Birdair, Facade: JV Hightex Mostostal

Capacity : 68.000 (WC), 55.000 afterwards (temporary seats will be substituted by lounges)

Roof area : Glazing ~37.000 m², ~9000 single panes (2x8mm TVG)

Lower membrane :
35.000 m² mesh fabric (PES/PVC)

Facade Area :
~27.000 m² mesh fabric (Glas/PTFE)

Start design - Completion :
April 2006 December 2009

Another challenge specific to this structure was the interaction between cladding (glazing) and deformation. Despite the stiffening effect of the truss girders, the resulting deformations of the structure are not negligible; this means that there is significant amount of warping and bias/distortion in the substructure of the glazing. To allow for this, numerous tests had to be conducted to develop the bearing details for the glazing, which allow for the required relative movement between the glass panes and sub structure.

A perfect interplay between design architects and the structural engineers from schlaich bergermann und partner as well as the smooth cooperation between the engineers of the roof and façade structure with our colleagues responsible for the reinforced concrete structure formed a strong base for the successful completion of the project. Again, it has been proven that it is fundamental for projects of this complexity to have design, erection engineering, site and fabrication supervision as well as the checking of all workshop drawings concentrated in the hands of the lead structural engineers of the roof.

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The 4D Visualization of Civil Engineering Structures

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[3]Emeritus Professor, Howard University, Washington, DC

Abstract

Digital media has become the dominant vehicle for teaching, learning, entertainment, and in general for the presentation of information. Although 'Digital Media' brings 'clarity and connectivity' to once isolated academic sectors, it is also true that 'Media Modeling' has come with a danger unforeseen; i.e. Validity of the media. Digital Media has the equal ability to propagate 'untruths' as well as the truth sometimes making information selection a process in itself. In the not so distance past, it was required to have great construction skill sets, construction management talent, civil engineering expertise and 'real world' experience to build a successful building. With the advent of the Internet a blueprint can be had and printed within in minutes. But education gained in this manner, lacks academic details and/or valid construction theory. Because these designs tend to have a strong tendency for simplification core civil engineering principles are sometimes compromised.

The new challenges for education in the field of civil engineering will be to make visualization, and its 4-D methodologies inclusive of the discipline, in regards to both the presentation and the preparation, and still be true of intent while staying void of 'false intent'. By combining analogical (visual) education techniques with the driven civil engineering philosophies into as educational construct, it is possible to perform the analysis of civil engineering structures - both virtual and physical and to provide an open 'blueprint' for teaching. Visualization techniques function mostly as 'dimensionless' entities, and make themselves available as strong Civil Engineering resources. Proper methods and presentation procedures, have the layered capacity for analysis and possess the primary focus of function to advance the execution of civil engineering structures through 4-D methodologies via education, presentation, and comprehension.

This paper presents 4-D Visualization and its educational concepts related to Civil Engineering 'Structures' - and their sphere disciplines can have a dramatic effect on civil engineering education. It is the first of series of papers and are parts of the Doctoral work of the senior author. Due to the complexity of civil engineering structures, one is in a constant state of flux and constantly needs to adapt 'newer' technologies and teaching methodologies to include programs such as AutoCAD and Maya. The logical 'educational structure' of civil engineering and its structures are thus enriched by visualization and such features capable of defining the realm of civil engineering are discussed in this series of papers [2,3,4].

1.0 Introduction

Visualization uses commonalities as the 'fourth' component in actualized 4-D modeling and analysis. Newly defined 'Civil Visualizations' and their spawned educational procedures, assist in the creation of Civil Engineering Life story learning philosophies that will represent civil engineering strategy from beginning to end by using digital media to represent the holistic summary of the event. For example, if a bridge is being re-engineered. A movie would should the original concept design, the legacy of the bridges' service would be reviewed and the new design would be discussed in great engineering detail complete with an interview from the lead engineer and sample calculation values. 'Media' and time [sequences] animation will show the bridge under natural disasters and atypical loading constraints. This type of application of 4D creates a 'open source' educational toolset that is available without special software or hardware and is not dependent its applications or to 'current' technology or time that is to say, independent of principality change, foundation principles will remain the same.

Although common in the entertainment industry, and in the medical community - digital media and its brethren graphical applications have not been used to their fullest advantage in Civil Engineering. Traditional methodologies of structures visualized - can be used to represent sophisticated and complex Civil Engineering concepts, while reinforcing 'core' themes and engaging engineering outcomes by summarizing theory via visual and/or printed media. 4-D deliverables will be a series of concise sets and methodologies, ranging from newly crafted Lifestories outcomes and civil engineering documentaries. All formats will have alternate 'traditional' response formats.

By having a 'typical' 'display (format) presentation' that is independent of Internet connectivity the flexibility can allow remote applications, such as an engineered farmer's bridge to be built in an isolated region without heavy consultation.

2. Scope for Visualization

2-D and 3-D Visualization scopes do not necessarily convey the information/data binding in combination with 4-D inclusion. Whereas 4-D brings a typical 4th dimensional element such as "time", Visualization as an observation embeds the active input of time. This 'observation participation' then acts to accelerate the learning experience by providing simple user interfaces and projections that enable students to better comprehend relationship detail and engineering nuances. In the present context, 'Visualization' becomes the preferred descriptive for the technology; it is an effort to preserve the engineering foundation while remaining independent of the computer sciences - emphasizing applying Civil Engineering in such a manner that both the instructor and student are at ease and confidant in the projected subject.

2.1 Visualization as a Learning Vehicle

Any 'Learning Vehicle' that delivers civil engineering as 'visual' knowledge attempts to accelerate civil engineering educational learning. Visual principles provide simulated 'experience' and assist conceptual reasoning by targeting civil engineering curriculum courses and by merging legacy core civil engineering values with 'new media'. This culmination of traditional responses becomes transfixed into an appropriate digital format superimposing the modern media formats. These new 'promotions' have relevancy that can be produced on common platforms with ordinary material. Properly used, these new disciplines allow Civil Engineering to become an engaging choice of the students to understand the scope of the field.

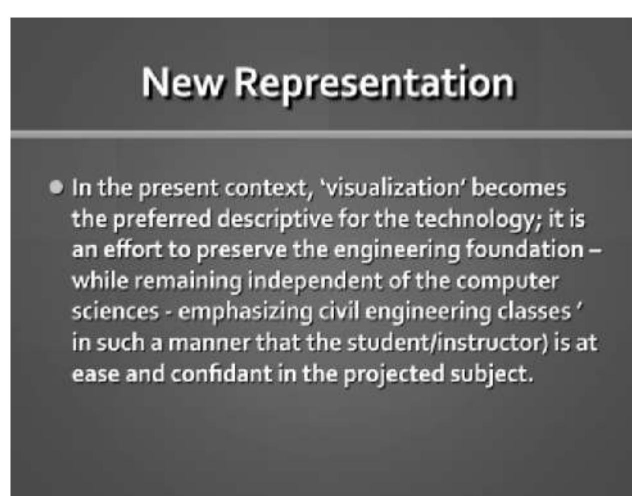
3.0 Rationales and Significance

Accuracy is always the most important aspect of civil engineering modeling and modeling seeks to simulate real world conditions for the purpose of observation, learning, and behavior recognition. 4-D modeling in civil engineering, using digital media, can help to save resources in the real world, by representing environmental forces in static/dynamic loading towers and limiting physical testing facility operational cost. Visualization, unlike the actual construction field, can provide all extremes in trail and error conditions that do not waste resources Be it hurricane forces, wall bearing loads, or the distance spacing of rebar the principles of civil engineering can be dictated by the behavior of materials versus environment on a computational project field all are ideal target to be visualized. Because engineered structure's performance under 'active' conditions require the ability to 'foresee' conceptual revelations of predicted actions therein, Visualization becomes the key to limiting failures and collapses. Visualization becomes the means in which to work out flaws and redesigns. The emergence of 'interpretive graphical toolsets' and applications that represent complex operations, make typical civil engineering tasks straightforward, accurate, and fast. New information can be learned via digital mediums that provide simulated graphical representations that in every way convey real world responses and conditions. Documentary-types movies Lifestories - show engineering theory within specific environment and help construction scheduling by allowing for more 'aggressive' planning and closer detailed work intensity.

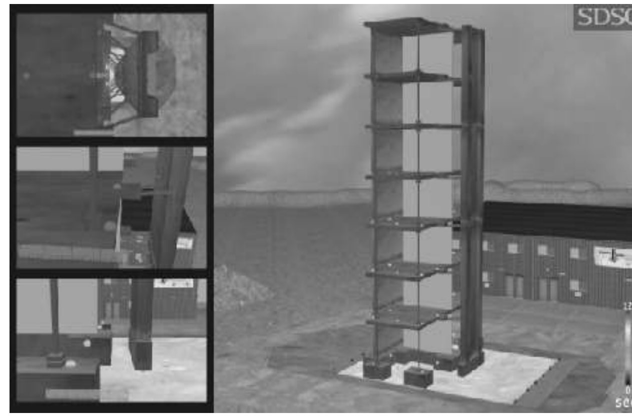
These collective methods have:

1) The capacity to represent buildings and structures.

2) Programs and Applications that



The Seven-Story Building on its Shake Table at the San Diego Super Computer Center, University of San Diego State (Courtesy: A. Chourasia)



The Seven-Story Building at 54 seconds into the test. Sphere colours show displacement and the encapsulated cells show close-ups inside the structure. San Diego Super Computer Center, University of San Diego State, (Courtesy: A. Chourasia)

accurately represent infrastructure, foundation, and curriculums.

3) Visualizations that express deformations, loading, and stresses graphically.

4.0 Graphical Techniques

Adaptive graphics provide insights for 'real-time' looks for repairs, natural disaster assessment, and ongoing structural analysis. Visualization in civil engineering generates immediate solutions that address the engineering problem in terms that are apparent while fortifying the results of a given summation because visualizations have the flexibility to project multiple possible outcomes. Visualization becomes the planning operative as well - because of its accuracy consistency in areas such as material costing and task scheduling. But more importantly, these operations coupled with well 'linked' data naturally becomes 4-D inclusive. When trying to educate or enhance Civil Engineering teaching experience, it is always helpful to have approaches that are simple and helpful in projecting various perspectives to the targeted audience. The comprehension of complex, Civil Engineering principles at a glance is always a value added resource. For example, in the field of earthquake studies, the seismic institutes have integrated visualizations into the analytical phase of the surveyed data. The calculated tables represent the stress reading outputs via traditional digital media methods. The produced graphical representations allow for a general understanding that can then be interpreted without specific expertise.

3.1 Formulation: New Representation

Techniques that exposure targeted audiences to the understanding of immersed civil engineering at a glance, are essential in the propagation of civil engineering philosophies. To explain to the public, the before mentions earthquake projections thorough series of charts and graphs is bulky. However, simple animation showing plate movement and seismic wave propagation is always effective because every recognized the action of water. One needs only to have seen a ripple in water to understand an earthquake's origin, because the animation shows each seismic plate rolling under the other. The plate 'waves' will push along simulated coastlines showing the devastating water elements and their impact on the coastal forces. The representations then allow for a general interpretation without specific expertise.

3.2 NEES

At the Network for Earthquake Engineering Simulation (NEES) high performance outdoor shake table, data is produced that combines both the textual and the visual. Each stream is based on observed, recorded, and calculated data. Similarly, both the actual picture(s) and the animated model(s) are used to define the environment(s) and their movements, likewise the stress/strain gage results become synonymous with the computational 'observations' rendered from the visualized model. In this manner, the combination of 'real world' and virtual environment become a natural analogical teaching agent.

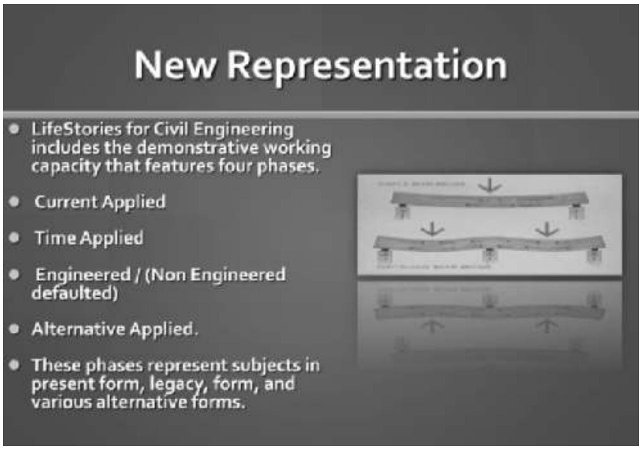
These outcomes are relevant anywhere, but they will be particularly so, in countries like India, where visualized definitions will define the strengths and the deficiencies of inner city complexes (planned or unplanned) and rural bridge construction or other places where there is no address for proper sustainability. Virtualization representations become invaluable for the showing the rapid-order changes of environmental conditions providing side-by-side comparisons for definitive analysis that pointedly summarize the not-so-subtle changes.

4.0 Types of Visualizations

There are several media formats available to be used to engineer alternate solution sets.

Some of the more common types take the form:

1) Multi-Media and Still Photography Graphs, Charts, Static Visuals, Pictures, and Images via printed media and electronic presentation.



Concept of proposed I-35 West

2) Simulation, Motion Video, and Animation Dynamic representations, usually in form of movies, short films, and documentaries.

3) WebPages and Portals Internet offerings that have attached additional information, data, and graphics, as well as encapsulated Video and Sound Recordings.

4) Printed Media Magazines, books, journals, and Papers all have the capacity to adopt and execute successful visualizations. With the development of Power Point and the Microsoft Office suite as the workhorse, it is possible to develop concise professional presentation that forwards the ideology and advantages of Visualization.

5.0 Civil Engineering Lifestories

Most civil engineering concepts are not intuitive or naturally understood holistically. Failures by structures due to natural disasters can be a nightmare in terms of analysis and remediation. As a result of the I-35 bridge failure, we have learned the devastating impact of improper loading and poor structural design maintenance. By modeling the rebuild visually, we can gain a concise visual representation that give us a real-time prospective and comprehension of the basic foundation scope and build principles.

The Lifestory projects the actual environment in terms of engineering, design, scope, and 'as-builds'. Some urban degrading infrastructure is rooted in the simple fact that there is no current data of the construct itself or existing blueprints that can used to provide 'real time' analysis based on present day conditions. Lifestory projections can show the ever-changing conditions, via visual effects representing flooding, high wind, and even contaminate exposure (such as oil). Competing designs could be shown side by side and judged by category or as one-to-one entities. Lifestories also have inherent strong abilities to match construction time line against the corresponding building proposals in essence making the process of proposal submission less subjective to non critical sway and reinforcing objective throughout the construction process.

Lifestories can, thus define and isolate urban engineering design flaws in visually simple terms that civil engineers can then propose applied solutions and their respective proposed 'execution' solutions.

5.1 Visualization as Simulation

Simulation modeling of structural conditions is conducted to represent a given environment - either by mathematical formulation or experimental work. Specific simulation techniques have advantages and imbedded disadvantages that depend on model accuracy, applied resources, and data input. Aligned with modern computers, these complex analyses are calculated efficiently. Experimentation, leveraged with off the shelf charting routines, will provide a 'mathematical vision' of the behavior, but it may still require an extensive 'skill set' and multi-expertise background to be fully understood. This 'understanding' can be a critical factor in engineering application whether it manifest in shortcomings the funding of the project or the student's capacity to understand essentials. By using similar animated models, 'Civil Engineering' virtual worlds can depict the behavior of any structure, because the animation will predictably adhere to physics and the applied loading and the building's dimensions will be appropriately scaled leading to conclusive outcomes.

Though these methods are not substitution for proper civil engineering, they will be far more useful than no engineering at all. Farmers in the outskirts of rural districts could now build irrigation systems - made from local concrete mixtures - that will not wash away with the monsoon season. Or that community footbridge could now be constructed in such a manner that it would not fail when load by automobile of the masses of local transport of harvest. Visualization is capable of conveying targeted information seamlessly. Further, it will be possible to produce legacy mapping - showing the evolution of the structure and the 'foundation' progressions. Designs fashioned, by using the data produced by the 'virtual synopsis' models, will be able to resist the effects of both natural and man-made conditions while allowing “green' as an encapsulated component. Visualization functions as a solution ambassador for engineering concepts and problems. It combines the written word (descriptive) and the Visual (Media) of civil engineering as the substituted theme. In a perfect world, visualizations that would instruct on construction sequences while its illustrations help 'engineer', would also define the limits of operation and avoid the threat of abuse collapse. While not an ideal solution for critical engineering applications, its alternative non-engineered - is by far, the worst of the prevailing construct choices. Non-engineered structures, herald collapses as a function of time. Visualization is a movable 'engineering environment' that integrates concepts via first-person participation. Execution is the summation of the 'good' words.

Models designed for theoretical limits, via visualized environments, can be solidified by 'virtual modeling' integration. One of the natural advantages conveyed by graphical approaches is the development of aggressive strategies that reinforce, refine, and reference the infrastructure principalities. Methods that represent control systems, bridge maintenance schemas, disaster resistance scenarios, and urban sprawl can provide a 'real-time' look at repairs, (showing visual progression), and assess maintenance fault/damage for alternative possibilities. But more importantly, this insight will produce accurate timelines, to the minute costing, and an accurate 'workload' schedule.

5.2 4-D Modeling Concepts

4-D modeling - in a simplified version - represents a 3-D field of scalar values. Although the '4th' dimension can represent various values sometimes complex to represent it is always nearly a vector of some form of time captured (as in a still photograph) or dynamic (as in a documentary). Therefore, 4-D Visualizations become similar to the traditional 3D models with additional layered cues deployed for simplicity and sometimes visual 'brevity'. Understanding legacy conditions through visualization and analogical means becomes the component for projecting realms and environments that simulate real world conditions and consequences of environment.

Simulation modeling for any existing engineering conditions is conducted to represent a given environment - either by mathematical formulation or experimental work. Specific simulation techniques have the advantage of computer accuracy and continuous real-time data input. Aligned with modern computer networks, these complex analyses are calculated efficiently and accurately.

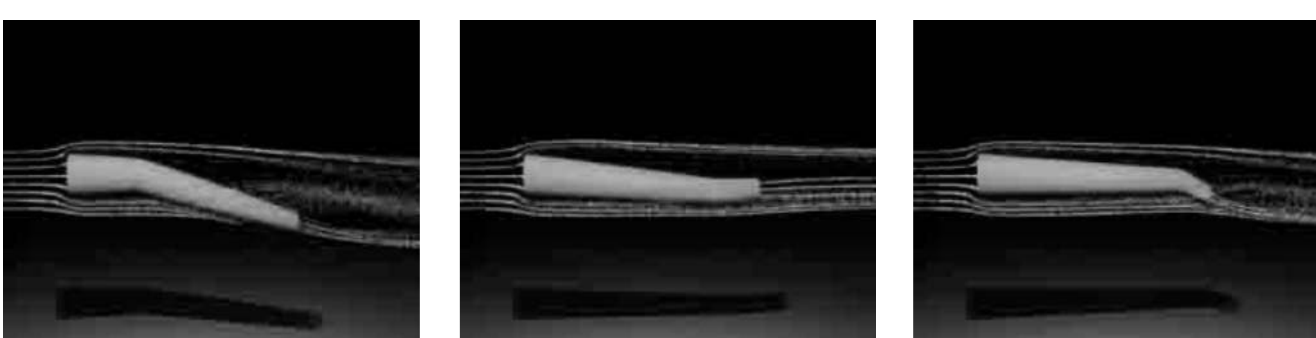
6.0 Concluding Remarks

Experimentation leveraged with 'faulty assumptions can provide a 'mathematical vision' of behavior that is contaminated from its the origins. It is imperative for the 'class' to have clear foundation concepts. In the above figure, we have 'flow' over a simple wing. By description, it is complex and somewhat ambiguous. But by visual display, the concept is not only easy to comprehend - it is ready for discussion, development, analysis, and group readdress for possible re-engineering. By using similar still model sequences, civil engineering's worlds can depict the behavior of structures, because the visualizations will adhere to physics laws and principles providing concise conclusive outcomes.

By using visualizations, Lifestories, and their associated graphical techniques, corresponding engineering principles and educational concepts can be shown for direct educational and scientific analysis. Visualization Education provides the following Instructional advantages:

1) It interprets graphs and charts.
2) It makes qualitative assessments openly understood.
3) It produces simple quantitative assessments and translations.
4) It constructs maps and directives that are easily understood.
5) It translates visually, complex information into three dimensions.
6) It conceptualizes changes in respect to time providing a visual, virtual, legacy.

Visualization has the innate ability to show representations via the recorded and the observed. But it can also portray deltas of change, simplifying the collective of scientific data and calculations into the interactive educational assets. 'Cause and effect' are isolated onto displays and printed media, highlighting the effects of loading and



(Examples of progressive simulation)

strain into the apparent. Visualization, unlike simulation, is the direct representation of actual data and its linked behavior. But as in all models, visualization's purpose is to allow an experience' that will contribute to the manipulation of not only the design, but produce a working solution.

Keywords: Visualization, Animation, Civil engineering structures, graphical applications, analytical environment, graphical tools, 4-D animation, visualization techniques, virtual graphic volumes, and open source.

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IASS Bestows Prestigious Eduardo Torroja Medal To R. Sundaram

Mr. R. Sundaram, one of the most well-known architects and structural engineers to have emerged out of India in recent times, will be honoured with the “EDUARDO TORROJA MEDAL”, conferred by the International Association for Shell and Spatial Structures (IASS), Madrid.

According to an official communication from IASS' President, Prof. John F. Abel, the Executive Council of IASS has voted to confer upon Mr. Sundaram the prestigious Torroja Medal in recognition of outstanding and distinguished contributions to design, construction, or research of shell and/or spatial structures.

The IASS, founded by Eduardo Torroja in 1959, has as its goal the achievement of further progress in the field of shell and spatial structures, through an interchange of ideas among all those interested in lightweight structural systems such as lattice, tension, membrane and shell structures.

Mr. Sundaram will be formally conferred with the Eduardo Torroja Medal at the IASS-APCS Symposium 2012 to be held in Seoul, South Korea during 21-24 May 2012 [www.iass2012.org]. He has also been invited to present a plenary lecture during the occasion.

Mr. Sundaram is the current President of Structural Engineers World Congress Inc. [SEWC Inc.] till 2015.

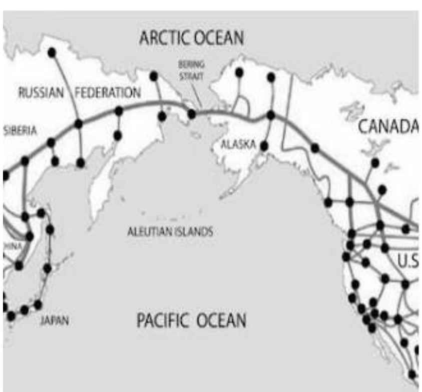
Among other awards received by him, recently he is honoured by the Prestressed & Precast Concrete Society, Singapore, by conferring him a “Life-Time Honorary Membership” of the Society.

Sundaram, Chairman & Managing Director, Sundaram Architects Private Limited

Green light for £60bn GBP Bering Strait tunnel

It was first mooted as long ago 1905 by Tsar Nicholas 11, but this week the Kremlin finally gave the green light for a 65 mile (106 km) tunnel linking Asia and North America, taking the epic project a step nearer reality.

The conference in Yakutsk was hosted by Yegor Borisov the Governor and the project was ratified by President Medvedev's top officials including Aleksander Levinthal the deputy federal representative for the Russian far East.



It's been hailed as the greatest railway project of all time but admittedly, there are still a small few details outstanding such as funding to iron out but Russia is determined to pursue its claim to the huge fossil fuel and mineral wealth in the arctic and develop its trading ties with China. Experts forecast that the completed service could carry 3% of the world's freight and earn £7billion GBP per year. Engineers have said the project could reach break-even in seven years.

A 500 mile, £900m GBP link from the Trans-Siberian railway to Yakutsk is already in construction and will be completed in 2013, nudging towards the Russian goal of a further 2360 miles by 2030. This will provide strategic links from the mineral rich territory in the north to key freight lines in Russia and China.

World's Busiest Single Runway Set for Resurfacing

Gatwick Airport has awarded a significant contract to civil engineering contractor Volker Fitzpatrick to resurface the airport's main runway.

The main runway - the heartbeat of the airport - is resurfaced every 11 years to ensure it remains in the best possible condition. Gatwick will also take the opportunity to carry out additional works at the same time, such as, replacing the airfield lighting and improving the drainage system.

The project will involve resurfacing an area of 400,000m^2 - the equivalent of 100 football pitches - with approximately 65,000 tonnes of asphalt. Around 1,900 runway and taxiway lights, fed by 530 kilometres of electrical cabling, will also be replaced. Carrying out work on Gatwick's main runway is a major challenge with up to 53 aircraft landing and taking off every hour. However, the project will be delivered during the quieter night period to ensure passenger flights are not disrupted.

Tinted Wrap for Olympic Stadium

As the final preparations of the Olympic Games come closer, the London Olympic Stadium will soon unveil the wrap, sponsored by Dow. Embracing the temporary, the design by Populous adopts fresh sustainability ideas to create a compact, flexible and lightweight building inspired by the stage sets at outdoor events.

The design responds to a different type of brief which is in keeping with the International Olympic Committee's desire to ensure a 'Green Games', showcasing a more sustainable approach to hosting the event. In August 2012 the eyes of the world will be on London and the Olympic Stadium, making it the most viewed building in history.

The preparation for these games are an example of how innovative architec-

ture and the practical issues of building delivery are compatible as all venues are further ahead in construction than any past games reassuring to all spectators and athletes.

Courtesy: World Architecture news

MCD Move to Empanel Structural Engineers

With rise in illegal constructions in the city raising question marks over the safety of such buildings, the Municipal Corporation of Delhi has invited applications through a public notice for empanelling structural engineers whose assistance can be sought by citizens to ensure security of their building structures. The public notice seeks to empanel structural engineers to "enable the citizens to avail of their services for the purpose of structural safety certificate and structural design, as are required in support of building plan application, occupancy or completion certificate and regularisation of building or structure".

The move comes a few weeks after the civic agency opposed a Delhi Government proposal making it mandatory for property owners to furnish structural safety certificates for sale of properties on the grounds that it does not have enough structural engineers to inspect buildings and issue the necessary certificates.

After the objection, the Delhi Government had withdrawn the proposal.

Public Sotice

The public notice issued by the civic body stated that the minimum qualifications for a structural engineer would be graduate in civil engineering from a recognised Indian or foreign university, or corporate member of civil engineering division of Institution of Engineers (India) with minimum three years experience in structural engineering practice with designing and field work.

The issue of structural safety gained ground after a building with illegal floors collapsed in Laxmi Nagar in November last year claiming 70 lives.

Courtesy: The Hindu

Philippines Declares September as Structural Engineering Month

President Benigno Simeon Aquino III has declared September 2011 as "Structural Engineering Month" to urge Filipinos to use structural design in promoting public safety. In Proclamation No. 164, Aquino said recent natural disasters worldwide highlight the importance of promoting public safety through safe structures like buildings.

"Recent global developments related to natural disasters highlight the importance of promoting public safety through the design and construction of safe and sound engineered structures such as buildings, bridges, towers, and other structures," Aquino said in his proclamation.

In a proclamation signed on May 9 Aquino also noted the need to "increase public awareness about the role and the social and moral responsibilities of Civil Engineers specializing in structural design in relation to promoting public safety."

Benigno Simeon Aquino III

He also pointed out the Association of Structural Engineers of the Philippines, Inc. (ASEP), an affiliate society of the Philippine Institute of Civil Engineers (PICE), is marking its 50th anniversary this September.

"For the past 50 years, ASEP has been a partner of the government in its commitment to the protection of life and property by promoting responsible structural engineering through various undertakings, such as effective programs for disaster risk reduction and management and continuing professional development," he said.

Thus, he said there is a need to "express our appreciation for the work of our structural engineers and to spread awareness of their vital contribution to the country."

Courtesy: www.president.gov.ph

Emulating Nature for Better Engineering

According to Carmen Torres-Sanchez of the Department of Mechanical Engineering, at Heriot-Watt University, Edinburgh and Jonathan Corney of the Department of Design, Manufacture and Engineering Management, at the University of Strathclyde, Glasgow in the natural world, the graduated distribution of porosity has evolved so that nature might transfer forces and minimise stresses to avoid whole structure failure. For instance, a crack in the branch of a tree will not lead to the felling of the tree in the same way that a broken ankle will not lead to collapse of the whole leg. "Porosity gradation is an important functionality of the original structure that evolution has developed in a trial and error fashion," the team explains.

It is not just tree trunks and bones that have evolved graduated porosity, beehives, marine sponges, seashells, teeth, feathers and countless other examples display this characteristic. Researchers would like to be able to emulate the way in which nature has evolved solutions to the perennial issues facing engineers. In so doing, they will be able to develop structures that use the least amount of material to gain the lowest density structure and so the maximum strength-to-weight ratio.

"Many engineering applications, such as thermal, acoustics, mechanical, structural and tissue engineering, require porosity tailored structures," the team says. If materials scientists could develop porous materials that closely mimic nature's structural marvels, then countless engineering problems including bridge building and construction in earthquake zones, improved vehicle and aircraft efficiency and even longer-lasting more biocompatible medical prosthetics might be possible.

Unfortunately, current manufacturing methods for making porous materials cannot mass-produce graduated foams. The collaborators in Scotland, however, have turned to low power-low frequency ultrasonic irradiation that can "excite" molten polymers as they begin to foam and once solidify effectively trap within their porous structure different porosity distributions throughout the solid matrix. This approach allowed the team to generate polymeric foams with porosity gradients closely resembling natural cellular structures, such as bones and wood. The technology opens up new opportunities in the design and manufacture of bio-mimetic materials that can solve challenging technological problems, the team adds.

The researchers anticipate that using more sophisticated ultrasound energy sources as well as chemical coupling agents in the molten starting material will allow them to fine tune the formation of pores in the material. This is an area of current interest because it would facilitate the design of novel texture distributions or replicate more closely nature porous materials, the team concludes.

Courtesy: Science Daily

A Vertical Forest

Bosco Verticale is a 'Vertical Forest' project designed by Stefano Boeri, and is presently under construction at Milan, Italy. The architect aims to revolutionize urban architecture by implementing reforestation and naturalization, with forest vegetation grown skywards. Known to be the first such project in the world, the two apartment buildings that are under the plan measure 110 metres and 76 metres. Covered with plants including shrubs, flowering plants and more than 900 trees, the buildings will help balance the microclimate in the urban surroundings. A noteworthy fact is that if each unit is placed separately on land, the resulting forest would span 10,000 sq. metres.

The diverse plants will absorb carbon dioxide and dust whilst producing humidity and oxygen. Besides, the building will be safe from noise pollution and harmful radiations. Grey water from the building will be recycled via a filtering system to water the plants. The energy requirements of the building will be met by setting up photovoltaic solar systems. With an estimated cost of 87.5 million US dollars, the first phase of the BioMilano intends to set up a green belt around Milan, which is placed among the most polluted European cities.

Concrete Dam Blown for Recreating WWII Air Attack

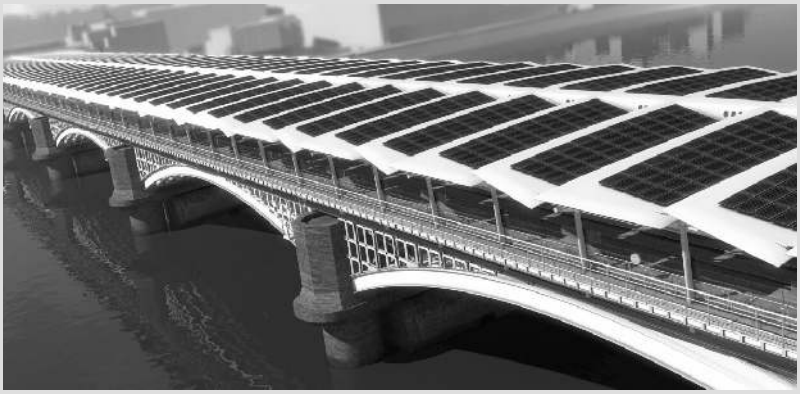
If you are someone who wanted to replicate a World War II style air raid using a 'bouncing bomb' to blow up a dam, what would you do? May be use special effects in order to recreate the scene. However, Hugh Hunt has gone a step ahead and built a concrete damp and then blew it using the bomb.

Almost 440 concrete blocks were used for the particular dam project. Hugh wanted to replicate an air attack on Germany's Ruhr Valley in 1943 and placed an ad for finding places where the scene could be filmed. The ad was seen and responded to by James Bellavance, who at that time was the economic development officer with the District of Mackenzie, B.C. The location chosen was near Williston Lake, which was about 13 km north of the community. Interestingly Bellavance had prior experience as a building contractor and also in cement production.

A geotextile liner was used in the construction of the temporary dam in order to hold back the water pressure. The entire construction took about five weeks time, with one week being lost by the crew due to heavy rains. About 175,000 cubic metres of soil was excavated to 40 feet below the lake level near the dam, for the project, according to Bellavance.

Since a WWII era bomber could not be found, a similar looking DC-4 was used to bomb the target, using dummy bombs. Around 230 kgs of plastic explosives were placed at the base of the dam, which were detonated at the same time when the dummy bomb landed, making up the concrete to blow up spectacularly for filming.

Blackfrairs Station to Get Solar Makeover



The historic Blackfriars station in London is all set to get a solar makeover. Over 4,400 solar panels will be installed, making it the largest solar-powered bridge in the world. The refurbishment project, undertaken by Solar-century, along with other public transportation and facility up-grades is expected to lower carbon emission, reduce conges-tion, and improve the passenger environment and experience significantly.

The project, featured and reported on popular website Inhabitat, aims to bring the steam-era station into the 21st century, is part of the ambitious £5.5 billion Thameslink Program by Network Rail to improve the interchange between the London underground and the national rail. It requires 14,000 tons of materials, most of which will be transported by barge rather than roads to reduce the environmental impact.

The project, when completed, is expected to double the station's capacity and allow nearly 24 trains to run the tracks every hour. It will also give people access to various local attractions like the Tate Modern and the Global Theater.

The CEO of Solarcentury, Darry Newman, says that station buildings and bridges are an integral aspect of Britain's urban landscape and that it is great to see that Blackfriars will be generating green, completely renewable energy for many years to come. He also adds that the success of Blackfriars' solar makeover will influence public perception greatly and it will be an important step toward a clean energy future.

A Slope Shaped Indoor Ski Resort

An incredible indoor ski resort is coming upon the outskirts of Stockholm, Sweden. Although there are other similar facilities available in different parts of the world, this particular project the Skipark 360 is perhaps the first time that something on this scale is being attempted.

One of the highlights of the resort which has been featured in popular online source Inhabitat is the 160 meters vertical drop. A 3.5 km cross-country skiing tunnel would make the resort even suited for a world cup, according to the news report on the website. There would be a slew of other facilities and amenities for the entire family in the resort.

The resort that has been designed by C.F.Moller will be powered by a variety of alternative sources of energy including, hydropower, solar and wind. On completion the resort would be competing with other similar large resorts for being one of the greenest resorts in the world.

A Floating Island Nation Planned

Owning his or her own floating island is perhaps every human being's secret desire. If you have the money to splurge you could be living out your fantasies , thanks to Project Utopia created by BMT Nigel Gee a British design group.

The project will consist of a floating island consisting of not less than 11 decks. Featured on popular website

dvice, the island is meant for billionaires who have the wherewithal to spend millions but cannot find independent islands for themselves.

The Utopia is not exactly designed for travelling at great speeds, with its four legs propelling it at a leisurely pace. The entire concept is cross between a luxury yacht and a massive island on high seas.

Shotcrete 2012

This platform has gathered shotcrete specialists for twenty years, in a surrounding field where the exchange easily takes place. Topics of the conference will be accelerators and early strength, composite shell tunnel linings, water tightness of shotcrete linings, leaching behavior, sulfate attack, shotcrete deformation behavior, sprayable membranes, manipulators, fiber reinforced shotcrete, testing of fiber reinforced shotcrete, polymer modified shotcrete for repair works, the use of white cement for shotcrete, shotcrete with special surface appearance, shotcrete for the Darwin Cocoon, nozzleman training. An exhibition will take place parallel to the conference, presenting special materials, products and equipment for the production and testing of shotcrete.

Conference Language: German, English Summaries of all presentations will be available. Some presentations will be in English.

Details

Date: 12th January - 13th January 2012
Contact Point: Mrs Agneta Kusterle Kusterle
Tel.: 43-650-8244610
Email: spritzbeton@kusterle.net
Website: www.spritzbeton-tagung.com

Location: Congress Centrum Alpbach, Tirol Alpbach, Austria

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Details

Date: 8th February - 11th February 2012
Contact Point: Mrs. Agneta Kusterle Kusterle
Tel.: 202-266-8111

Location: Orange County Convention Center Orlando, FL USA

Ultra-High Performance Concrete and Nanotechnology

Third International Symposium on Ultra-High Performance Concrete and Nanotechnology for High Performance Construction Materials, March 7 - 9, 2012, University of Kassel, Kassel, Germany

Details

Date: 7th March - 9th March 2012
Website: http://www.spp1182.de/

Location: University of Kassel Kassel, Germany

International Equipment EXPO

JW Marriott San Antonio Hill Country Resort & Spa San Antonio, TX USA

Details

Date: 14th March - 17th March 2012
Tel.: 469-359-6000, Fax: 469-359-6007
Email: jhall@adsc-iafd.com

Location: Jan Hall ADSC 8445 Freeport Pkwy, Ste. 325 Irving, TX 75063

ACI Spring 2012 Convention

Details

Date:18th March - 22nd March 2012
Tel.: (248) 848-3795 Fax: (248) 848-3701

Location: American Concrete Institute, 38800 Country Club, Dr Farmington Hills, MI 48331-3439

ACI-KC 3rd International Conference

The ACI-Kuwait Chapter invites you to attend its third international conference and exhibition that will be held in Kuwait on 26-28 March 2012. The theme for the conference is, "Towards the Development of Resilient Sustainable Buildings and Structures". The Organizing Committee invites authors to submit papers related to the theme.

Details

Dates: 26th March - 28th March 2012
Contact Point: Dr. Moetaz El-Hawary
Tel.: 96524989260
Email: hmoetaz@yahoo.com

Location: Hilton Hotel Kuwait, Kuwait

ICSA 2013

The "2nd International Conference on Structures and Architecture 2013" will be held in Guimarães, Portugal on July 24-26.x Convention Centre of the Campus of Azurém of the University of Minho Guimarães, Portugal

Details

Dates: 24th July - 26th July 2013

2013 IASS

The 2013 IASS Annual Symposium with the theme "Beyond the Limit of Man" will be held in Wroclaw, Poland, on September 23-27. Click here to obtain more information on IASS 2013 Symposium.

Details

Dates: 23rd Sep - 27th Sep 2013
Website: www.iass-structures.org

Location: Wroclaw, Poland

SEWC 2013 Symposium

International Symposium on Innovative Architecture-Structure interaction towards Sustainable Development with Green environment Organised by: Structural Engineers World Congress (SEWC-India) Supported By: Consulting Engineers Association of India (CEAI) Address for Communication

Details

Date: 24th Dec. - 27th Dec. 2013
Contact Point: IFP Qatar Ltd.
Tel.: +90 11 26194486
Fax: +91 11 46023053
Email: scmehrotra@mehroconsultants.com

Location: OCF Plot No.2, Pocket 9, Sector-B Vasant Kunj, New Delhi-110070.

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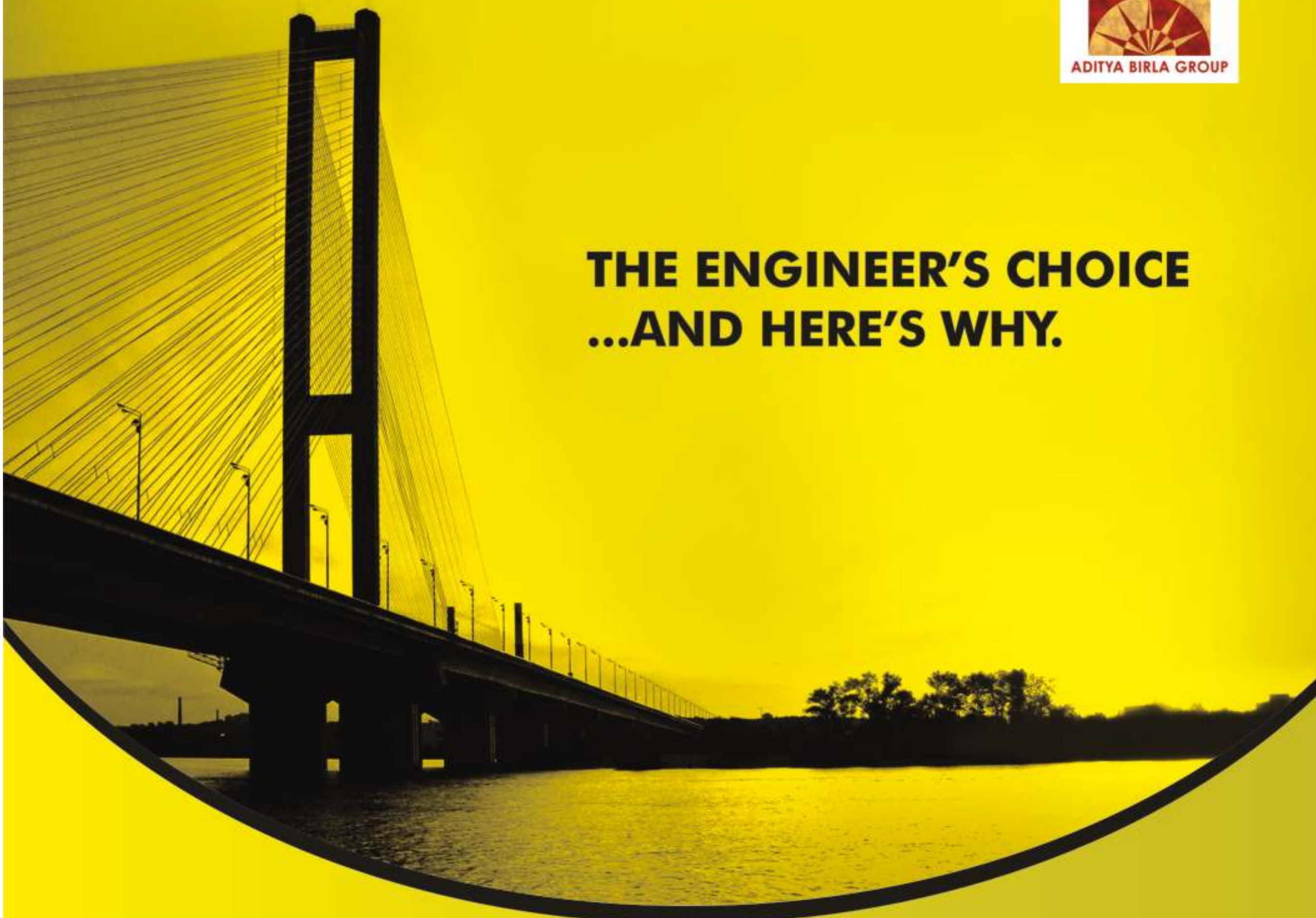

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