Europe and the Mediterranean: Towards a Sustainable Built Environment

Edited by Ruben Paul Borg, Paul Gauci, Cyril Spiteri Staines
SBE 16 Malta

Europe and the Mediterranean
Towards a Sustainable Built Environment

International Conference

16\textsuperscript{th} March – 18\textsuperscript{th} March 2016

SBE Malta
Sustainable Built Environment
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Sustainable Built Environment Malta
SBE Malta (Sustainable Built Environment Malta) is an organisation committed to Sustainable Development, education and research in green buildings and sustainable built environment. SBE Malta acts as the National Chapter of iiSBE, the International Initiative for a Sustainable Built Environment (www.iisbe.org). SBE Malta was set up in 2012 and registered as a voluntary organisation with the Commissioner for Voluntary Organisations in Malta. It is also registered as a legal entity with the Government of Malta. The primary objective of SBE Malta is the advancement of environmental protection and improvement by promoting Principles of Sustainable Development and Sustainability in the Built Environment. SBE Malta was set up as the Green Building and Sustainable Built Environment organisation in Malta, to establish relationships with professionals, public and private organisations at the local and the international level; to participate in international organisations; to promote the advancement of education; to conduct and promote research (www.sbemalta.org).

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SBE Malta - Sustainable Built Environment

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## Contents

Foreword  
Sustainability, the Circular Economy and the Construction Industry  
*Karmenu Vella*

### Chapter 1: Sustainable Built Environment: Introduction

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Introduction</td>
<td>1</td>
</tr>
<tr>
<td>Sustainable Built Environment &amp; Social Justice</td>
<td>3</td>
</tr>
<tr>
<td>Ruben Paul Borg</td>
<td></td>
</tr>
<tr>
<td>Sustainable Built Environments add Value and Enhance the Environment</td>
<td>5</td>
</tr>
<tr>
<td>Leo Brincat</td>
<td></td>
</tr>
<tr>
<td>Sustainable Built Environment</td>
<td>7</td>
</tr>
<tr>
<td>Alex Torpiano</td>
<td></td>
</tr>
<tr>
<td>The Building Industry Consultative Council and a Sustainable Built</td>
<td>9</td>
</tr>
<tr>
<td>Environment</td>
<td></td>
</tr>
<tr>
<td>Charles Buhagiar</td>
<td></td>
</tr>
<tr>
<td>Sustainable Development: Environment &amp; Planning</td>
<td>11</td>
</tr>
<tr>
<td>David Pace</td>
<td></td>
</tr>
</tbody>
</table>

### Chapter 2: Materials and Structures

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Structures for a Sustainable and Resilient Built Environment</td>
<td>15</td>
</tr>
<tr>
<td>Petr Hajek</td>
<td></td>
</tr>
<tr>
<td>Composites: A Sustainable Alternative Repair and Rehabilitation System for Ageing Infrastructure</td>
<td>23</td>
</tr>
<tr>
<td>Ayman Soliman Mosallam</td>
<td></td>
</tr>
<tr>
<td>Quality Specifications for Roadway Bridges: Standardization at a European Level (TU1406 COST ACTION - BridgeSpec)</td>
<td>43</td>
</tr>
<tr>
<td>Jose C. Matos</td>
<td></td>
</tr>
<tr>
<td>Influence of Recycled Sand and Gravel on the Rheological and Mechanical Characteristics of Concrete</td>
<td>51</td>
</tr>
<tr>
<td>Zine-el –abidine Tahar, El-Hadj Kadri, Tien-Tung Ngo and Adrien Bouvet</td>
<td></td>
</tr>
<tr>
<td>Effect of Silica Fume on High Performance Concrete Strength</td>
<td>65</td>
</tr>
<tr>
<td>Ali Mansor, Ahmed Hamed and Ruben Paul Borg</td>
<td></td>
</tr>
<tr>
<td>High Performance Foam Concrete Produced in Turbulence Mixers</td>
<td>71</td>
</tr>
<tr>
<td>Genadijs Šahmenko, Aleksandrs Korjakins and Eva Namsone</td>
<td></td>
</tr>
<tr>
<td>Effect of Fine Clay Brick Waste on the Properties of Self-Compacting Concrete</td>
<td>79</td>
</tr>
<tr>
<td>Ali Mansor, Ahmed Hamed and Ruben Paul Borg</td>
<td></td>
</tr>
<tr>
<td>Possibilities of Redevelopment of Embankment Dams by Suitable Grouting Mixtures</td>
<td>87</td>
</tr>
<tr>
<td>Rostislav Drochytka and Magdaléna Kociánová</td>
<td></td>
</tr>
<tr>
<td>Steel Jacketing of RC Columns: Reliability of Capacity Laws for Concrete</td>
<td>93</td>
</tr>
<tr>
<td>Liborio Cavaleri, Fabio Di Trapani and Marco Filippo Ferrotto</td>
<td></td>
</tr>
<tr>
<td>Seismic Vulnerability and Fragility of Industrial Steel Buildings affected by the Emilia-Romagna Earthquake</td>
<td>105</td>
</tr>
<tr>
<td>Antonio Formisano, Carmine Castaldo and Ida Iannuzzi</td>
<td></td>
</tr>
</tbody>
</table>
The Hidden Value of Stone: Life Cycle Assessment of the Construction and Refurbishment of a 60-year-old Residential Stone Building
Dimitra Ioannidou, Spyridon Daglas, Gerasimos Kallivokas, Maria Ploumaki, Stefano Zerbi and Guillaume Habert

Structural Optimization Including Whole Life Cost of a Timber Building using Evolutionary Algorithms
Georgios K. Bekas and Georgios E. Stavroulakis

Innovative and Traditional Techniques for Seismic Retrofitting of an Existing RC School Building: Life Cycle Assessment and Performance Ranking through MCDM Methods
Antonio Formisano, Carmine Castaldo, Nicola Chieffo and Giovanni Chiumiento

Luca Finocchiaro and Roberta Ramaci

Fiber Reinforced Polymers Textiles for Strengthening Historical Buildings and Examples for Use in Italian and Maltese Heritage
Michelangelo Micheloni

A Study on Moisture Movement within Traditional and Contemporary Building Materials
Nadia Martinelli

Waste Minimisation: a Design Review
Nicolette Micallef

Green Building Products and Technology: EcoBuild
Ruben Paul Borg, Ivan Bartolo

Chapter 3: Sustainable Development & Spatial Planning

The Regulatory Framework in Urban Biogas Plants to Define New Steps for a Common Development of Regulatory Guidelines in EU Member States
Alessandro Pracucci, Giacomo Bizzarri and Theo Zaffagnini

"Evaluation of Vegetation in Urban Space" Barcelona Base Model Proposed to the Dominican Republic.
Gilkauris María Rojas Cortorreal, Jaume Roset, Francesc Navés, Carlos López Ordóñez and Jernej Vidmar

Sustainable Use of Heritage Resources
Joseph Falzon

Sustainable Strategies for Promoting the Horezu Pottery and the Cultural Landscape of Horezu, Romania
Smaranda Maria Bica and Elena Roxana Florescu

Kenzo’s Master Plan After the Earthquake - a Base for Regenerative Development of Skopje
Strahinja Trpevski

Depopulation in Rural Towns of Sicily: A Historical-to-Present Day Analysis
Agatino Rizzo

How Should We Deal with Socio-Economic Values in Development Decisions?
Chris E. Cloete

Sustainable Mobility, Livability and Public Space in Historic Village Cores – a Case Study of Lija, Malta
Maria Attard, Alberto Miceli Farrugia, Jacques Borg Barthet
Sustainability in Development Planning
Vincent Grech

Chapter 4: Sustainable Strategies for Cultural Heritage

Cultural Heritage Research Trends in Europe
Roko Zarnic
Towards Sustainable access, Enjoyment and Understanding of Cultural Heritage and Historic Settings
Roberto Di Giulio
The holistic e-Documentation of the Past: Risks and Challenges (Horizon 2020, INCEPTION)
Marinos Ioannides
“Hong Kong’s Campaign for Sustainable Built Environment - Revitalisation of old Industrial Buildings”
Cheung Hau-wai
Roadmap for IT Research on a Heritage-BIM Inter-operable Platform within INCEPTION
Peter Bonsma, Iveta Bonsma, Rizal Sebastian, Anna Elisabetta Ziri, Silvia Parenti, Pedro Martin Leronos, Jose Llamas, Federica Maietti, Beatrice Turillazzi and Ernesto Iadanza
Common Strategies and Avant-garde Approaches in Terrestrial Laserscanning
Daniel Blersch, Christoph Held, M. Mettenleiter, Christoph Fröhlich and Marinos Ioannides
Digital Documentation: Sustainable Strategies for Cultural Heritage
Assessment and Inspection
Federica Maietti, Emanuele Piaia and Beatrice Turillazzi
Diagnostic Integrated Procedures aimed at Monitoring, Enhancement and Conservation of Cultural Heritage Sites
Federica Maietti, Emanuele Piaia and Silvia Brunoro
3D Integrated Laser Scanner Survey and Modelling for Accessing and Understanding European Cultural Assets
Federica Maietti, Federico Ferrari, Marco Medici and Marcello Balzani
Risks and Resilience of Cultural Heritage Assets
Vlatka Rajcic and Roko Zarnic
The Contribution of GIS for the Sustainable Protection of Monuments: The Case of Erimokastro and Earantapixo Acropolis in Rhodes, Greece
Elisavet Tslimantou, Eleni Oikonomopoulou, Ekaterini Delegou, Charalambos Ioannidis, Jonas Sagias and Antonia Moropoulou

Chapter 5: Building Energy Performance

Current Status of Climate Change and Scenarios for Action
Nils Larsson
Introducing the Portuguese Sustainability Assessment Tool for Urban Areas: SBTool PT – Urban Planning
Luis Braganca and Erika Guimaraes
The Contribution of Energy Performance Certificates to Resource Savings and Environmental Protection - Lessons from Germany
Thomas Lützkendorf, Peter Michl and David Lorenz
Introduction of Sociology-inspired Parameters in the Energy Performance Certification Calculation Method

Stephane Monfils and Jean-Marie Hauglustaine

A Fuzzy Control System For Energy Management in a Domestic Environment

Vincenzo Bonaiuto, Stefano Bifaretti, Luca Federici, Sabino Pipolo and Fausto Sargeni

Performance Monitoring of Energy Efficient Retrofits – 4 Case Study Properties in Northern Ireland.

Teresa McGrath, Sreejith V. Nanukuttan, PAM Basheer and Siobhan Brown

The Proper Design of Façade Openings for Occupant Satisfaction: Variance Based Sensitivity Analysis and Optimization of Model Output

Rossano Albatici, Roberto Covi and Alessia Gadotti

Towards Sustainable Architecture: Lessons from the Vernacular. Case Study - the Traditional Houses of the Asir Province of Saudi Arabia.

Joseph Galea

Retrofit Opportunities for the Historical Back Bay Neighbourhood

Ornella Iuorio, Antonino Barbalace, John Fernandez

Thermal Conductivity of an Unplanted Sustainable Green Roof System

Christina Said, Paul Refalo, Luciano Mule Stagno and Charles Yousif

The Effect of Different Glazing Apertures on the Thermal Performance of Maltese Buildings

Trevor F. Caruana and Charles Yousif

Occupant Behaviour at the Presidential Palace of San Anton, Malta: A Study Supporting the Development of a Methodology to Enhance Energy Efficiency in Heritage Buildings

Amber Wismayer, Carolyn Hayles, M. Lawrence, N. McCullen, V. Buhagiar

Renovating Primary School Buildings in Malta to Achieve Cost-optimal Energy Performance and Comfort Levels

Damien Gatt and Charles Yousif

Chapter 6: Renewable Energy Sources

Wind Energy Technology Reconsideration to Enhance the Concept of Smart Cities (Horizon 2020 TU1304 COST Action, WINERCOST)

Charalampos Baniotopolous

Offshore wind turbines: from fixed-bottom to floating technologies

Claudio Borri, A. Giusti, E. Marino, G. Stabile

Variable Winds and Aerodynamic Losses of Transport Systems: A New Wind Energy Technology for Future Smart Cities

Tommaso Morbiato, Claudio Borri and Simone Salvadori

Wind Energy Potential in Venturi-shaped Roof Constructions

Bert Blocken, Twan van Hooff, Lourens Aanen and Ben Bronsema

The Effect of Pitch Angle on the Performance of a Vertical-Axis Wind Turbine

Abdolrahim Rezaeiha, Ivo Kalkman and Bert Blocken

On the Understanding of the Above Roof Flow of a High-rise Building for Wind Energy Generation

Hassan Hemida
Monitoring Based Identification for Structural Life Cycle Management of Wind Energy Converters
  Rüdiger Höffer, Simon Tewolde, Simona Bogoevska, Matthias Baitsch, Sven Zimmermann and Gabriele Barbanti

Design and Construction of a Small Multi-Bladed Wind Turbine for the Suburban and Rural Environments
  Martin Muscat, Tonio Sant, Robert Farrugia, Cedric Caruana and Redeemer Axisa

Reduction of Current Harmonics in Grid-Connected PV Inverters using Harmonic Compensation - Conforming to IEEE and IEC Standards
  Daniel Zammit, Cyril Spiteri Staines and Morris Apap

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Coming from a small island can confer certain advantages.

Malta's compact geography allows it to concentrate some of the key issues that Europe as a whole is facing today. On a small island, sustainability isn't an abstract question. It's a very real issue for everyone involved.

This means that an idea like the circular economy is an ideal fit for a compact economy like Malta's.

A more circular economic model, where re-use and recycling become our first instinct, will allow Europe to use its resources more wisely. It's not just an environmental necessity – it's also an economic opportunity that we can't afford to miss.

This is why the European Commission is proposing that Europe begins to rethink its economy, and ensure that it is suited to the challenges of the future.

It will involve action in numerous areas, from design and production processes to consumer awareness and greater use of new business models. And it's the sort of thinking that equally applies to buildings.

The construction sector uses half of all our materials, half of all our energy and a third of our water. It generates a third of our total waste. There are obvious gains to be had from higher recycling rates, and from a different mind-set when it comes to design.

The energy consumed when buildings are used is already a focus area for European policy. But there are so many more aspects to take into account, from the materials we use to construct buildings, and how we deal with them when we renovate or demolish buildings, to the energy and resources we use to produce construction materials, and the all-important water consumption, which is such a key element for Malta.

I hope that this compendium of research and new ideas will serve as an inspiration, not just for the construction industry, but for everyone who has an interest in retooling the EU economy for the changes of the future.

Malta has the highest proportion of built-up areas anywhere in the EU. Why not turn that to Malta's advantage, and think of the Islands as a laboratory for ideas for Europe's future?
Chapter 1

Sustainable Built Environment: Introduction
Introduction

Sustainable Built Environment and Social Justice

Ruben Paul Borg

President SBE Malta, Faculty for the Built Environment - University of Malta

Sustainable Development was re-defined as a broad political vision in 1987 by the World Commission on Environment and Development as “development that meets the needs of the present without compromising the ability of future generations to meet their own needs” (Brundtland Commission; WCED1987). However Marco Keiner (ETH, Zurich) argues that the idea of “sustainable development” was born in 1713 when Carlowitz edited the first book on forest sciences and argued that timber would be “as important as our daily bread” and that it should be “used with caution in a way, that there is a balance between timber growth and lumbering”. "For this reason, we should organise our economy in a way that we won’t suffer scarcity, and where it is lumbered we should strive for young growth at its place". The Carlowitz base law on ongoing use of resources is the central idea of the concept of sustainable development re-defined by the Brundtland Commission.

Since its release the Brundtland Commission report definition has been re-formulated according to different point of views. Disagreement exists as to the precise meaning of the term, and yet most definitions refer to the viability of natural resources and ecosystems over time, and to maintenance of human living standards and economic growth. The Swiss “Monitoring of Sustainable Development Project” MONET presented a workable interpretation of the principle of sustainable development based on justice: “Sustainable development means ensuring dignified living conditions with regard to human rights by creating and maintaining the widest possible range of options for freely defining life plans. The principle of fairness among and between present and future generations should be taken into account in the use of environmental, economic and social resources”.

Keiner argues that the popularity of “sustainability” stems also from the simple three pillar model used to facilitate the comprehension of the term and based on environmental (conservation), economic (growth), and social (equity) dimensions. It is based on basic aspects of human society, and yet does not explicitly take into account ‘human quality of life’; the concept of Sustainability has to be reviewed and questioned. Urban Planner Peter Marcuse, critically reviewed the concept of Sustainability (Marcuse, 1998) especially as it has come to be applied outside of environmental goals. Marcuse suggests that “sustainability” should not be considered as a goal for a housing or urban programme, (many bad programmes are sustainable) but as a constraint whose absence may limit the usefulness of a good programme. He even discusses how the promotion of “sustainability” may simply encourage the sustaining of the unjust status quo. One can say that Sustainability and social justice do not necessarily go hand in hand. Marcuse states that “sustainability” taken as a goal in itself only benefits those who already have everything that they want. Indeed, even focusing on environmental concerns, the problem for most of the world’s poor is not that their conditions cannot be sustained but that they should not be sustained.

Globally more people live in urban areas than in rural areas; 54% of the world population resided in urban areas in 2014. The construction industry contributes significantly in the consumption of natural resources and generation of waste to sustain the current needs of society. This puts greater pressures and responsibilities in ensuring a sustainable built environment and in safeguarding diverse communities. It is necessary to promote sustainable development in the built environment while ensuring social justice. Planning for social justice and environmental sustainability in the built environment, should be framed as part of an integrated system, and not as two separate goals. It is important to understand how they are linked and how to create virtuous circles of policy and practice.
that bring mutually reinforcing benefits in a way that, what’s good for society is good for the environment – and vice versa. One has to debate whether for example current policies to reduce carbon emissions, are shaped by the pursuit of economic growth, far more than human well-being. Widening inequalities and accelerating damage to the natural environment are rooted together; the economy cannot function without social and natural resources and yet they are treated as though they are infinitely exploitable.

The promotion of social justice and environmental sustainability calls for collective action and requires long-term planning. That means taking steps to prevent poverty and disadvantage from accumulating over time, as well as safeguarding natural resources for the future. In the long term, a healthy natural environment improves human well-being, while a socially just society is better able to safeguard natural resources and cope with the various consequences of climate change. The economically disadvantaged suffer first and most from the effects of climate change, while social inequalities drive up aspirations for resource-intensive consumption.

Policies that promote social justice and environmental sustainability at the same time, need to be promoted. This can be achieved through sensible exploitation of resources for construction, extending the life time and durability performance of structures and infrastructure, minimizing construction waste, retrofitting homes making them more efficient and improving access and safeguarding green spaces.

SBE Malta (Sustainable Built Environment Malta) was set up as the Green Building and Sustainable Built Environment organisation in Malta, as the National Chapter of iiSBE (International Initiative for a Sustainable Built Environment), and is committed to Sustainable Development, education and research in green buildings and sustainable built environment. The SBE 16 Malta conference as part of the World SBE 16 Series, addresses key areas promoting sustainable development in the built environment; resource efficiency, materials use and structural optimization; strategies for cultural heritage and retrofitting of existing buildings, strategic planning and urban design, energy efficiency in buildings, smart buildings and the exploitation of renewable energy sources. The conference, “Europe and the Mediterranean towards a Sustainable Built Environment”, in Malta at the Centre of the Mediterranean, is well placed to attempt a discussion on sustainability in the built environment in the context of cultural and economic diversities.

Indeed the objective of our research and professional activities is that of promoting sustainable development in the built environment. Yet we should keep in focus the ultimate goal, that of promoting the quality of life, safe environments and reinforcing social justice in our communities, to better promote environmental sustainability and environmental justice.
A few years ago many were inclined to consider the linkage between sustainability and the built environment as a hindrance to the latter’s development process. Although there will always be room for improvement, sustainability is nowadays ingrained locally within the notion of the built environment, particularly but not exclusively in urban densely populated areas.

Sustainable built environments are linked to various measuring gauges. Be they energy consumption linked, renewable energy generation or even water efficiency measures and rainwater harvesting methods and techniques. This is an area that not only calls for maximum and priority attention but also for continuous ongoing research and updating. If well managed, it can serve as an effective and efficient broker between industry, government and academia including also those organisations servicing the built environment industry itself.

We cannot even aspire to succeed unless we have a vision and clear objective linked to sustainable infrastructure and building design, construction and management that taken together can only enhance the performance of the built environment industry. To succeed we also need to implement a unique collaborative and strategic approach that can capitalise on all stakeholders’ strengths and positive and creative thinking and ideas.

By promoting access to better education, technology and innovative practices in this area we will also be facilitating our island’s process to become even more competitive not only in the region but also internationally.

An integrated approach can lead to sustainable development processes, regulations, governance and also community engagement. In practice this can extend beyond the sustainable buildings themselves, by leading to smart and healthy workplaces and living environments.

Public awareness about the built environment needs to be enhanced further through constantly ongoing awareness campaigns. Particularly since the built environment does not mean solely whether a building is a high rise one or not, but rather refers to all the man made surroundings that provide the setting for human activity. Ranging in scale from buildings and parks or green spaces to neighbourhoods, towns, villages and cities that can often include their supporting infrastructure.

I have come across various definitions of the built environment but to my mind the best definition that encompasses the sustainability linked to it, is that it is a material, spatial and cultural product of human labour that combines physical elements and energy in forms for living, working and recreating themselves on a day to day basis. The main reason being that the built environment encompasses both places and spaces created or modified by people including buildings, parks and even transportation systems.

Count on government’s support in your endeavours.
Sustainable Built Environment

Alex Torpiano
Dean, Faculty for the Built Environment, University of Malta

The ubiquitous definition of “sustainable development” is a quotation from the report “Our Common Future”, also referred to as the Bruntland Report, which says that “sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.” The concept of the “sustainable built environment” presumably emerges from this frame of mind - providing the built environment that we need today without compromising what our children will want tomorrow.

To a certain extent, in the context of the built environment, we have often referred to this as good “planning”- more of this later. I think that, just as I salute the scholars who have come to this meeting in Malta, and the scholarship which they wish to share with us, it is important to take a step back, to reiterate what “sustainable development” really means, and therefore what the philosophy of a “sustainable built environment” ought to embrace.

In the first instance, I think that it is important that we do not distort the meaning of the term “sustainability” to make it exclusively synonymous with green building, or energy efficient building, or even “protecting “the environment. The concept is much wider, much broader, and also much deeper. And it has more important ramifications. Unfortunately, this distortion pervades our current discourse.

When we invite our students to address the issues of “sustainability” in their design proposals, their response is often limited to providing photo-voltaic panels or wind turbines to generate renewable energy, often to heat or cool very energy-unfriendly proposals. There is very little thought given, for example, to the resources that are required to produce photo-voltaic panels in the first place, and to the issues that will arise when the photo-voltaic panels need to be disposed of. These are students, admittedly, but their approach to sustainability issues can be equally prevalent in professionals in the built environment, as well as in the organizations, government or otherwise, active (or vocal) within the built environment. Indeed, in the parlance of , for example, government-funded, and EU-funded proposed projects or tenders, it seems to be important to have the word “sustainable” or “eco-friendly” somewhere, since, apparently, the chances of approval of funding diminish if it is absent; I have seen tenders asking for “eco-friendly trenching works”, or a “low-carbon intervention” to repair the roof finish of a heritage building” – presumably they mean that the waterproofing they do today should last into the future!

At the level of university structures, and not only our University, the term “sustainable” is starting to be used very liberally, and not necessarily always within the spirit of the “Common Future” report. Whilst these structures are very welcome and valid, it may sometimes be necessary to fine-tune their focus. Whilst it is true that there is a strong link between Sustainable Development and Climate Change, for example, the link is not merely through the volume of traffic and the relative gas emissions; the linkage is much more complicated, since it embraces economics, politics, social behaviour, employment … the quality of life. And with all due respect, the Ministry for Sustainable Development, the Environment and Climate Change reinforces this limited linkage, at least in the perceptions of the general public; unless we are careful in the language that is used in the public realm, there is the risk that people miss the point of sustainable development. It is complex, and it is certainly not just about the environment. And it is difficult to understand how it could be separate from spatial planning.
In 1992, an international Commission on Sustainable Development was set up, and placed under the UN Economic and Social Council, ECOSOC. The Commission on Sustainable Development proved to be ineffective, according to many observers; to me, a revealing observation was made by one commentator: “if ECOSOC did its job properly, there would be no need for the Commission on Sustainable Development”. In other words, if we really undertook proper planning, spatial, economic as well as social, then our concern for “sustainable development” would no longer be there – we would not need to speak about it. After Rio+20 of 2012, we ought to be implementing Agenda 2030, and tracking the implementation of its sustainable development goals. But is there any general awareness of what these goals are, and what the implications could be for our country, or our region?

It is in this sense that I express my view, (which, I think, is also embodied in the spirit of the Sustainable Development legislation enacted in Malta), that rather than considering Sustainable Development as yet another, separate, theme, every ministerial initiative, every economic activity ought to be addressed through the perspective of the “Common Future”. Using an admittedly simplistic model, I would say that every ministry ought to have, within it, “sustainable development champions” who can, for every policy proposal, study the question, “how will this continue in the future, how will it affect future citizens of Malta” Because, in reality it is today, but all about tomorrow.

So, what does a “sustainable built environment” require of us? It requires us to think of the effect of EVERY SINGLE ACTION we take in the built environment, and consider how it will impinge on the future. And this is not an easy task, since effects can be complex. It might be better to illustrate with simple, and perhaps simplistic, examples, limited to the Built Environment. In the 17th century, the building rules for Valletta required that every house have a cistern in which to collect rain-water. How did this affect later generations, and how would it affect future generations if policies to collect rain-water were to be, on the one hand, ignored, or on the other, updated? If it were decided that every house-hold had to have sufficient photo-voltaic panels, so that it did not require connection to the grid, (the zero-energy ambition), how would that affect the cost of generating electricity using conventional plant - for when the sun is not present - unless energy generation were also accompanied by new forms of energy storage? If it were decided that every new development had to have sufficient car-parking for all the activities generated by that development, how would future public transport thrive? And if it did, what would we do with the car-parking facilities? If we decided that all construction had to be in limestone, how would that impinge on our limestone resources? And if we decided that all construction had to be in steel, because steel is theoretically 100% recyclable, where would the recycling take place? And if the answer to this question is that it is available somewhere abroad, what would be the carbon footprint of the transport operations?

As I am sure you are aware, the tools required to study these questions are necessarily multi-disciplinary. Some of these questions are, in fact, addressed by some of the papers presented in this Conference, and well done for that. I would hope that, by listening to research work undertaken in one aspect, researchers in other aspects could be inspired to look at their disciplines in a different way. This would be the contribution of this Conference to multi-disciplinarily. I also hope, however, that this work does not remain within the limitations of the publication of the proceedings of this Conference, since it will otherwise become just another nice book for our libraries. I am making this observations because I believe that, whilst it is important to focus on the detail of one’s discipline, as academics are wont to do, it is important that we look at the broader picture. The “sustainable development” problem is not an “academic” one, but a very real one, in the local context but also in the contexts of other places. It is therefore important for governments to engage in the real debates on these issues, and to put money into research into the complexity of the issues which are raised by a real “sustainable development” frame of mind.

As we launch the start of this intensive three-day Conference, I wish to reiterate my welcome, on behalf of the Faculty for the Built Environment, to the University, and Malta, and hope that all our visitors will enjoy, not only the proceedings, but the whole experience of their stay on our islands.
The Building Industry Consultative Council (BICC) is a public entity set up by parliamentary resolution in 1997 with the scope of bringing together all the stakeholders within the building industry to discuss issues and make proposals to government to ensure that the industry will be able to meet the challenges of the future.

Thus the council includes representatives of all the operators (both private and public) ranging from contractors to developers and estate agents, the educational institutions, the financial institutions, the workers representatives, the professionals as well as the regulatory bodies.

One of the main goals of BICC is to propose regulatory standards for more resource efficient buildings, to ensure that the workers have the necessary skills to be able to perform the tasks required and to promote eco friendly building products and services.

BICC carries out this work by means of working groups made up of experts from different fields together with the operators and professionals. An EU financed project has been carried out to identify skill-shortages. From the conclusions of this project, a set of criteria are being drawn up, which form part of the compulsory requirements which one needs to have to obtain a skill card. The skill card initiative will ensure that within a period of five years all construction related workers in Malta will have a skill card as testimony of their competence in their particular trade.

BICC is currently carrying out another EU funded project which is that of creating a website for the promotion of green buildings products and green building services (Ecobuild Malta). The website has been launched and is currently being populated with the relevant information. It is the scope of BICC that a better awareness of the need of using more sustainable products and methods of construction be created by means of this initiative not only for the professionals and workers within this very important industry but that there would be a better national appreciation of the importance of having better environmentally designed building whilst ensuring that we have the skilled workers required to produce such buildings.
Sustainable Development: Environment & Planning

David Pace
Commissioner for Environment and Planning, Office of the Ombudsman, Malta

With the introduction in 2010 of the amendments to the Ombudsman Act providing for the appointment of Commissioners for Administrative Investigations in specialised areas of public administration, specifically in Environment and Planning, citizens were given a fair and impartial way to seek redress and have their rights for a better quality of life safeguarded.

On taking up my appointment to this Office one of my first tasks was to open a dialogue with several NGOs working in the environmental and development sectors.

One such NGO was SBE Malta (Sustainable Built Environment Malta), a small but extremely committed and enthusiastic organisation working in the field of sustainable development. SBE Malta is the local Chapter of iSBE (International Initiative for a Sustainable Built Environment).

The primary objective of SBE Malta is the advancement of environmental protection and improvement by promoting Principles of Sustainable Development and Sustainability in the Built Environment.

As part of this commitment towards its objectives, SBE Malta has undertaken the organisation of SBE 16, one of a series of international events organised by iSBE. I strongly feel that while due attention is given to issues concerning our natural environment, awareness of the importance of issues related to the quality of our built environment is still not one of the topics featuring high on the national agenda.

As a result, citizens are not fully aware of their rights to enjoy quality where it matters most, namely the built environment where most of us spend the greater part of our lives.

The themes to be discussed during the SBE Conference fit perfectly within the functions and the mission of my office which include the safeguarding of citizens’ rights to a sustainable environment especially the right to information, and to participate in environmental decision-making process.

Therefore, it is my pleasure to support SBE Malta in its efforts, notably in the organisation of SBE 16 and while wishing it every success, I look forward to the conclusions and recommendations emerging from the Conference.
Chapter 2

Materials and Structures
Concrete Structures for a Sustainable and Resilient Built Environment

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Abstract. World is faced to increasing frequency of natural disasters and to increasing economical and social problems. The structures for sustainable future should be better prepared for the new conditions; they should be more resilient. Concrete is the most used man-made material due to its specific technical properties and workability. It is strong and durable material and thus suitable for structures with needed high level of resiliency when faced to the exceptional natural or man-made disaster situations. Recent development of concrete technology has lead to a significant improvement of technical and also environmental parameters. Due to optimization of production technology and design techniques, concrete is gradually becoming a building material appropriate and advantageous for construction of sustainable and resilient buildings. Already implemented realizations give clear signal that in the forthcoming era when designing and implementing concrete structures it will be necessary to take into account new requirements and criteria following from global aspects on sustainable development. The general methodology for ILCA of concrete structures based on ISO and CEN standards is presented as well.

1. INTRODUCTION

Concrete is after water the most used material in the world. The production of concrete in the industrialized world annually amounts to 1.5-3 tone. In consequence of a fact that world cement production has been 12 times increased in the second half of the last century, the cement industry produces at present about 5 - 8% of global man-made CO₂ emissions. The Figures 1 and 2 show a development of cement production in different regions of the world presented by The European Cement Association – CEMBUREAU. From the graph on Figure 1 it is evident that the production of cement is significantly growing in developing countries in Asia, Africa and CIS (Commonwealth of Independent States – some former Soviet Republics) and slightly growing in America and Oceania. However, in Europe was the production in 2014 even lower in comparison to the year 2001. The dominant position keeps China with 56.5% of total world cement production in 2014 (see Figure 2).

As a consequence of economy growth, especially in emerging countries, the demand for construction materials, including cement and concrete, is increasing. According to WWF report the cement production in industrialized and developing countries in 2050 is estimated to be almost 5 times higher than in 1990 despite the need for reduction of total CO₂ emissions (!).

The current development of built environment leads to the need of replacement of some old existing buildings with new buildings. Consequently, the amount of demolished concrete structures is gradually increasing. This creates needs and potential for replacement of natural aggregate by recycled aggregates. The use of recycled concrete - as an aggregate for new concrete mixes, leads to saving of natural resources and helps to reduce the pressure on landfilling sites.
More over high amount of concrete production and use is associated with high transport needs and demands on production and demolition processes within the entire life cycle. This all has significant impact on the environment.

![Fig 1: World cement production by region – evolution 2001-2014; source CEMBUREAU](image)

Current development of concrete, production technology and development of concrete constructions during last twenty years have lead to improvement of technical parameters and in the same time reduction of environmental impacts. New types of concrete have due to mix optimization significantly better characteristics from the perspective of strength, mechanical resistance, durability and resistance to extreme loads. The use of new high performance concretes enables construction of more sustainable and resilient buildings or other types of structures.

Considering above specified global situation it is highly important to focus on an implementation of new technologies and construction techniques especially in emerging countries and regions with the highest cement production.

2. SUSTAINABILITY AND RESILIENCY

World is faced to increasing number of natural disasters and to increasing economical and social problems, incl. terrorism. Earthquakes, floods, storms, hurricanes, tornados, fires, tsunamis, volcanic events, heat and cold waves, extreme dry weather etc. are more and more frequent. According to a WMO (World Meteorological Organisation) analysis the years 2011-2015 have been the warmest five-year period on record, with many extreme weather events - especially heat waves - influenced by climate change. The probability of natural disaster is now nearly five
times higher as it was in the 1970s, because of the increasing risks due to climate change – Figure 3a and 3b. In the same time we are faced to accelerating number of man-made disasters e.g. WTC New York terrorist attack September 11th, 2001 (Figure 3c), current increasing number of terrorist attacks by Islamic state, etc. As a consequence of natural and man-made disasters and economy situation in particular countries a human migration is increasing.


Application of sustainability principles in different kind of human activities became a common and necessary approach. This is supported by implementation of new sustainable oriented knowledge and other results from research, via education to the construction practice.

With respect to the general principles of sustainability, the three essential pillars should be considered in the design, construction, use and other life-cycle phases of any construction, as follows:

- environmental aspects,
- economic aspects,
- social aspects.

Resilience is in general the capacity to adapt the system to changing conditions and to maintain or regain functionality and vitality in the stage of stress or disturbance [5]. It is the capacity to recover after a disturbance or interruption. According to the Resilient Design Institute “resilient design is the intentional design of buildings, landscapes, communities, and regions in order to respond to natural and man-made disasters and disturbances—as well as long-term changes resulting from climate change—including sea level rise, increased frequency of heat waves, and regional drought”.

Considering increasing number of natural and man-made disasters in our changing world it is essential and urgent to modify principles of structural design and construction technology for development and maintenance of buildings and all built environment in order to be more resilient. Of course the reduction of environmental impacts (especially global warming) is urgently needed to slow down the process of climate change. Only such resilient and environmental friendly buildings and other structures in the built environment can be sustainable in the changing environmental as well as social situation of the forthcoming era.

3. ADVANTAGES OF CONCRETE STRUCTURES FROM SUSTAINABILITY AND RESILIENCY VIEWPOINTS

Main advantages of concrete structures from the viewpoints of sustainability are: (i) thermal mass (contributing to energy savings associated with cooling and heating), (ii) acoustic properties (improving air-born sound insulation), (iii) fire resistance, (iv) long term durability, (v) structural safety, including high resistance to natural effects, (vi) maintainability and (vii) flexibility. These advantages could be significant in designing new constructions as well as in old structures reconstructed for the new use. With respect to specifics of concrete presented as a strong and durable material it is possible to design and construct on this material bases robust structures with high level of resiliency when faced to the exceptional natural or man-made disaster situations.
3.1. **Specific advantages of concrete structures from environmental viewpoint**

- Secondary materials utilization - Utilizing supplementary cementitious materials in a composition of concrete mixture (fly-ash, granulated blast furnace, microsilica) it is possible to reduce the amount of embodied energy and embodied CO₂ and SO₂ emissions.
- Recycled concrete can be utilized as aggregate substitutes in earthwork construction and up to some extent as an aggregate substitute in a new concrete production.
- Precast concrete elements in “tailor-made” manner enable waste reduction in production and also on construction site.
- Thermal mass of concrete can contribute to energy savings associated with cooling and heating.

3.2. **Specific advantages of concrete structures from economy viewpoint**

- Long-term durability - Concrete in comparison to other materials (timber, steel etc.) enables longer service life of buildings. Concrete structures are usually more resistant to atmospheric action, they have a good capability of withstanding wear, and they do not subject easily to degradation processes. This also results in lower operating, maintenance and demolition cost.
- Less damages caused during disasters (due to high strength and fire safety) – lower economical impacts, lower costs for repair and reconstruction.
- Lower material cost, lower manipulation and transportation cost. Subtle concrete structures utilizing lesser amount of higher quality concrete could be cheaper, even though the unit cost of this type of concrete is higher than the unit cost of standard concrete types.
- Dismountable structures: Precast concrete structures can be designed as dismountable enabling consequential utilization of structural elements.
- Smaller thickness of peripheral structures can have a positive effect on construction economic efficiency (especially in areas with regulated size of built-up area).
- Thermal mass - Concrete structures due to their accumulative properties can in some cases contribute to decrease of operating cost for cooling and heating.

3.3. **Specific advantages of concrete structures from social viewpoint**

- High structural safety and reliability, higher fire resistance – This includes also high resistance to natural effects during exceptional cases of natural disasters (floods, storms, winds, hurricanes, tornados, fires, earthquakes, etc.) and terrorist attacks.
- Acoustic properties – Due to high specific weight of concrete there can by improved air-born sound insulation of structure (floors and/or walls separating different operational areas);
- Thermal mass – Concrete (material with high specific weight) can contribute to thermal stability of internal environment and consequently to energy savings.
- Maintainability - Concrete surface produced in high quality can be easily maintained, cleaned and it has long durability.
- Flexibility – Character of concrete technology enables significant design flexibility due to the possibility of forming almost any element shape limited only by structural reliability requirements.
- Healthiness - Concrete is not the source of toxic emissions or volatile organic compounds.

4. **ADVANCED TECHNOLOGICAL AND STRUCTURAL PRINCIPLES FOR SUSTAINABLE CONCRETE BUILDING STRUCTURES**

4.1. **Optimization of concrete mixture**

Utilizing new composite materials with significantly better physical characteristics creates realistic assumptions of achieving substantial effect from the perspective of material and energy savings. Some examples from abroad show that high performance concrete can be used for optimized shapes of RC (reinforced concrete) elements, which can be very subtle (wall
thickness of 30 mm and less) due to their mechanical parameters. Nowadays, use of concretes with compressive strength around 100 MPa is not exceptional, also UHPC (UHPC – Ultra High Performance Concrete) with compressive strength over 150 MPa is already used for some specific construction elements in building construction and in construction of infrastructure – Figure 4a. These kinds of materials enable design with reduced material consumption and thus with lower environmental impacts and consequently with higher reliability and durability. Due to higher durability UHPC could be used also for special elements for external skin of buildings (Figure 4b) or city furniture (Figure 4c).

4.2. Shape optimization

Shape optimization can result in subtle lightened cross-sections. Their lower weight imposes lower load on supporting structures. Application of high performance concrete enables additional savings due to higher reduction of cross section dimensions (Figure 5). Cross section shape can be created using moulds, various types of lightening elements (Figure 5b and 5c) or by application of light-weight concrete. Mentioned techniques can lead to material savings from 30 to 70%.

4.3. Effective structural concepts

The more efficient structures from sustainability and resiliency viewpoints could be obtained by combination of different materials and structural principles. Some of principles are demonstrated on results of experimental research performed at the Czech Technical University in Prague (CTU) see Figure 6.

The composite structures based on high performance silicates and wood represent the beneficial alternative to the timber floor structures – Figure 6a. The timber structures have problems to achieve sufficient stiffness; the lack of mass causes troubles with acoustics, inflammability of wood limits the use from the perspective of fire safety. Combination of concrete and timber elements can lead to advantageous structural static and environmental solutions.

Textile reinforced concrete (TRC) has been developed for very thin plate or shell elements (thickness 12 to 25 mm) where the use of steel reinforcement is not applicable because of a corrosion risk due to a thin concrete cover layer. The concept of using TRC was introduced in order to reduce the thickness and thus weight of elements and associated environmental impacts.
This concept is mostly used for shells with particular shapes, for strengthening of structural elements or for thin façade panels. On the Figure 6b there is a façade in Malmo, Sweden made from panels from TRC (13 mm thick TRC plate is fixed to steel bearing frame).

Subtle structural elements from high performance concrete can be integrated into building envelope of energy efficient buildings avoiding risk of thermal bridges. The envelope thickness represents restricting parameter of developer plan in regions with regulated size of built-up area. In these cases, subtle RC structures become a great advantage in the form of slender load-bearing wall or subtle RC frame. Figure 6c shows prototype of concrete structural subtle frame for construction of houses in passive energy standard.

Fig 6: a – hybrid floor made of thin UHPC slab glued to timber beams, b – external panels from textile reinforced UHPC – realization Skanska, Sweden, c – HPC subtle RC frame for residential buildings

5. INTEGRATED LIFE CYCLE ASSESSMENT OF CONCRETE STRUCTURES

Integrated design is a complex approach implementing all relevant and significant requirements into one single design process – Figure 7. This approach integrates material, component, and structure design and considers selected relevant criterions from a wide range of sustainability criterions sorted in three basic groups: environmental, economic and social and considering entire life cycle of structure.

Complex integrated approach is based on simultaneous and interactive consideration of different aspects:
• sustainability requirements (environmental criteria, economic criteria and social criteria);
• technical and functional requirements (technical performance, functional performance, durability);
• life cycle phases throughout the entire life of the structure;
• various functional units (material, component, entire structure).

Concrete is used in a wide variety of structures (buildings, bridges, roads, dams etc.), each designed with a specific kind of functionality and life span in mind. For all these reasons no single outline for an ILCA for a given concrete structure can be specified. A chart in the Figure 8 shows ILCA process applied to different types of concrete structures. In this process the key importance play regional specifics, because concrete is typically produced from regionally available materials using regionally available techniques and transport systems.

The methodology of sustainability assessment is based on the three traditional pillars of sustainability: environmental, social and economical issues:
• Environmental issues: e.g. effects on climate change, resource use, local air quality, biodiversity, raw material use and waste production;
• Social issues: e.g. occupants’ health and interior microclimate, security and safety, social and cultural issues;
• Economic issues: e.g. life cycle costing, support for local economy, externalities.
The criteria filed in the environmental issues are related with traditional concept of environmental impact of buildings and other structures.

The social issues include, on the one hand, criteria linked with the social effects of construction and operation of structure (protection of cultural heritage, service quality improve, local unemployment decrease, availability of affordable dwelling), and on the other hand criteria regarding occupants’ health, safety and comfort.

The most emphasis in the economical issues is put on the life cycle costing that comprises investment costs, operation and maintenance costs as well as the cost connected with final demolition or deconstruction of the structure. Benefits for the local economy are included in the criteria employment possibilities and added value of the site accruing from the construction.

Regional specifics should be considered when collecting embodied environmental data of different materials. The type of material sources, mining technologies, transport means, transport distance, technology of production, they have a significant influence on final unit
environmental embodied values. Relevant complex LCA of the product or entire structure should be based on local environmental data collected within the inventory phase of the LCA procedure.

6. CONCLUSION

Only resilient structures can be sustainable in the changing world. Due to optimization of production technology and design techniques, concrete is gradually becoming a building material appropriate and advantageous for construction of sustainable and resilient buildings. Especially optimized concrete structures using new types of concrete in advanced technologies can significantly contribute to needed reduction of global environmental impacts. One possible way is utilizing of ultra high performance concrete in optimized structural shapes. Mechanical properties of these materials such as high compressive strength, durability, water tightness etc. create conditions for designing subtle structures that leads to saving up to 70% of material in comparison with ordinary concrete, and consequently to reduction of embodied CO₂ emissions. In the same time such structures could be more resilient in the situation of changing environmental as well as social conditions.

Already implemented realizations give clear signal that in the forthcoming era there will be necessary to take into account new requirements and criteria for design and construction of concrete structures following from global aspects on sustainable development. The results show that the high quality of mechanical and environmental performance of new silicate composites creates the potential for wider application of High Performance Concrete in building construction focused on sustainability issues. Some of principles were demonstrated on results of experimental research performed by author and his team at the Czech Technical University in Prague (CTU) and are presented on Figures 4c, 5c, 6a,b,c.

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1. INTRODUCTION

In a recent report, the Federal Highway Administration (FHWA) estimated that to eliminate the nation’s bridge deficient backlog by 2028, we would need to invest $20.5 billion annually, while only $12.8 billion is being spent currently. To make this difference, federal, state, and local governments would need to increase their bridge investments by $8 billion annually to address the identified $76 billion in needs for deficient bridges across the United States. It was also reported that traffic congestion costs the US economy $67.5 billion annually in lost productivity and wasted fuel. This costly traffic congestion is blamed, to a major extent, on the existence of this increasing large numbers of structurally-deficient and functionally-obsolete bridge. Both weight and speed restrictions are required in the case of structurally-deficient bridges. Functionally-obsolete bridges are those designed according to older codes and load requirements and are not capable of safely accommodate current traffic volumes, vehicle sizes and weights. For this reason, there is an urgent need for developing rapid and cost-effective methods to repair structurally-deficient bridges and to upgrade the functionally-obsolete bridges. In the 2013 Report Card prepared by the American Society of Civil Engineers (ASCE) stated that “Over two hundred million trips are taken daily across deficient bridges in the nation’s 102 largest metropolitan regions. In total, one in nine of the nation’s bridges are rated as structurally deficient, while the average age of the nation’s 607,380 bridges is currently 42 years...”.

Strengthening and repairing existing highway bridges is a major challenge facing structural engineers worldwide. In the past few years, a number of innovative methodologies for upgrading

Composites: A Sustainable Alternative Repair and Rehabilitation System for Ageing Infrastructure

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Abstract: This paper provides an overview on some of the latest advances in applications of fiber reinforced polymeric (FRP) composites in construction. Three main applications are discussed in this paper, namely; (i) repair and rehabilitation of concrete, steel, masonry and wood structures using composites, and (ii) all-composite structural applications that includes buildings, bridges and hybrid systems. Research, system and product development reviews, as well as case studies are presented. The paper also discussed the issue regarding the importance of developing design codes, materials specifications, design manuals and national and international standards for composites used in civil infrastructure applications. A review of some of the major design codes and specifications documents published in different countries is also provided.
the capacity of steel, concrete and timber bridges have been developed. However, the majority of these innovative methods are still in the development stage and applications of such technologies are still considered as demonstration projects. This delay in moving innovative bridge developments from the laboratory to field application is attributed to several factors including liabilities-related concerns and conservatism on the part of department of transportation (DOT) decision makers, limitation of resources, lack of awareness of the technology and its positive impact on bridge performance and most important the absence of a standard concentrated approach for rolling out innovations. In order to speed the technology transfer process and to increase the confident level on these new technologies, assessment tools such as health monitoring and diagnostic/prognostic systems are needed to provide bridge engineers with continuous information on the performance of the repaired and rehabilitated bridges.

In this paper, selected successful as well as unsuccessful applications of composites are described. In addition a review of available standards, acceptance criteria and design guides are also presented.

2. REPAIR & REHABILITATION OF REINFORCED CONCRETE MEMBERS USING FRP COMPOSITES

The pioneering and successful application of FRP composites for repair and rehabilitation of reinforced concrete members was bridge and buildings columns. This application was extended other applications including beams, floor slabs and bridge decks, beam-column joints, pipes, tanks, shear walls and other structural members as shown in Figure (1). A comprehensive coverage for different repair and rehabilitation applications of composites is reported by Mosallam (2002). A multi-criteria systematic approach based on the analytical hierarchy process (AHP) was developed to assist decision-makers in evaluating the use of advanced materials by Elmikawi and Mosallam (1996). This tool was applied to test the use of advanced composite materials in the repair of deteriorated and damaged bridge columns. The purpose of the study is to respond to the need to repair, strengthen, and retrofit existing structures, and to avoid or reduce such need when planning for new structures.
2.1. Emergency Seismic Repair of Reinforced Concrete Bridge Shear Columns

During an earthquake of large magnitude, reinforced concrete columns may be severely damaged limiting the functional capacity of the bridge. It is essential that damaged columns be temporarily repaired to ensure continuous functionality of the bridge. One of the promising techniques is the use of FRP composite jackets to regain its lateral capacity. This system of repair must be pre-designed and be readily available for fast implementation to mitigate further damage that may be caused by aftershocks.

In a pilot study conducted at UCI, a shear-damaged RC circular column was repaired by a combination of fast setting epoxy mortar and carbon/epoxy jacket, and retested under lateral cyclic loading (refer to Figure 2). The “as-built” column severely failed in shear as shown in Figure (2). The column was subsequently wrapped with 4 plies of unidirectional carbon/epoxy composites and then retested. The repaired column showed very satisfactory performance as it developed flexural behavior up to a ductility of 4.0 as shown in the hysteresis loops in Figure (2). This clearly demonstrates the effectiveness of such a scheme of repair that can be used to provide fast emergency repair of bridge columns, thereby reducing the traffic impacts due to bridge closure.
2.2. Seismic Repair and Retrofit of Reinforced Concrete Beam-Column Joints

Lack of joint confinement in the pre-1970 construction has resulted in weakened link between the column and the beam and collapse of the whole structure. The majority of past published research work has focused on the repair and retrofit of the beam-column exterior joints using either conventional materials or off-the-shelf polymeric composites. Mosallam (2009) conducted a research study on structural upgrade of reinforced concrete column-tie beam assembly using FRP composites. An innovative externally strengthening technique for reinforcing interior RC beam-column joints developed by the author through the use of a hybrid High Performance Mortar (HPM) and CFRP laminates. The hybrid HPM/CFRP composite connectors are attached to both column and beam sides using both high-strength bolts and high-strength epoxy adhesives (see Figure 3). In addition, multidirectional E-glass/epoxy and carbon/epoxy composite laminates were designed for this purpose. In order to verify the effectiveness of this strengthening system, a comprehensive full-scale evaluation program was conducted. Test results indicated that a significant enhancement in the joint shear strength was achieved. The retrofitted beam-column specimen strengthened with high-strength carbon/epoxy composite laminates showed an improvement in its shear strength capacity by 1.34 times as compared to the control deficient specimen beam-column specimen. The experimental results indicated also that the use of high-modulus carbon/epoxy composites in the focused deficiency application of this thesis was not very satisfactory. For example, ductility of the high-modulus/epoxy retrofitted specimen was 36% lower than as compared to the high-strength carbon/epoxy retrofitted specimen. The use of HPM/CFRP technique for retrofitting joint specimen with rebar bond slippage was very successful. This innovative technique improved the shear strength of the joint 2.5 times the control deficient specimen. The use of advanced composite connector has prevented brittle shear failure inside the joint region and allowed a plastic hinge to develop away from the column face. The energy dissipation by the retrofitted specimen was 4.6 times the control specimen with discontinuous reinforcement rebars. The load-displacement for shear deficient control and retrofitted specimens is presented in Figure (4).
Recently, a comprehensive research program was developed at UCI to evaluate the effectiveness of the FRP retrofit system in restoring the loss of capacity due to the addition of the openings. The results indicated the innovative reinforcement systems designed specifically to upgrade the performance of the walls after the introduction of the openings that were not included in the original wall design performed in a satisfactory manner and were able to restore the capacities of the retrofitted walls to more than or equal to the average capacity of the original solid wall without openings. Geometrical and reinforcement details for the wall specimens evaluated in this study are described in Figure (5). Failure modes and a comparison of load-displacement envelopes of all wall specimens is shown in Figure (6). As shown in this figure, ductility of the retrofitted wall with door opening was 3.33 as compared to 5 for the control (as-built) wall with door opening due to the localized severe debonding failure of the retrofitted wall at the connection between the top spandrel and the narrow wall pier. In addition, the average ultimate load of the retrofitted wall with window opening was 1.32 times the average ultimate load of the control wall with window opening. For the retrofitted wall with door opening, the average peak load was 1.25 times the average peak load of the control wall with door opening.
a) Solid Wall Specimen

b) Wall Specimens with a Window Opening

c) Wall Specimens with a Door Opening

Fig 5: Wall Specimens Details
1 inch = 25.4 mm
1 foot = 0.3048 meters
Fig 6: Failure Modes and a Comparison of Load-Displacement Envelopes of all Wall Specimens
2.3. Flexural Upgrade of Reinforced Concrete Slabs with a Hybrid HPM/CFRP

Few decades ago, steel plates have been used to enhance flexural strength of RC floor slabs and bridge decks by externally bonded fiber reinforced polymer (FRP) composite systems have been accepted by the construction industry. While the FRP installation at the top of floor slabs or bridge decks has not met many difficulties, in most cases there are many obstacles to access the underside of the slab such as suppression system, electrical wiring and ventilation ducts. This also applies to retrofitting bridge overpasses where traffic interruption in the road below is unavoidable as well bridges over waterways where the application of retrofit systems requires special shoring and special application procedures that are expensive and cumbersome in most cases. Therefore, it might be difficult to achieve the well prepared concrete surface for the FRP application at the underside of the slab or might not have access at all for logistic reasons. Moreover, additional anchoring or equipment might be necessary to hold FRP laminates during the initial curing of an adhesive, which makes it further difficult to apply the FRP to the underside of slab or bridge deck.

Recently, a pilot project was initiated to develop an innovative hybrid composite system is proposed combining a high performance mortar (HPM) with carbon fiber reinforced polymer (CFRP) in the presence of adequate shear connectors. By integrating these two materials, the ability of the proposed system to increase the moment carrying capacity of RC slabs or bridge decks is demonstrated (Mosallam et al. 2011 and Kim et al. 2013). The proposed system can be installed on the top of the RC slabs or deck to enhance the positive moment capacity (refer to Figure 7). Full-scale experimental results indicated that the proposed system can increase the ultimate load capacity and ductility of the retrofitted RC slabs by about 164% and 122%, respectively, as compared with the original “as-built” capacities with easy installation. Based on the verification test results, the system was approved by the City of Los Angeles, California and was adopted for use for a commercial high-rise moment frame building in Los Angeles, California (see Figure 7).
2.4. Collision Protection System for Reinforced Concrete Bridge Girders

One of common damages in existing highway bridges is the localized damage at the bottom corners or edges of the reinforced concrete beams or box girders induced by an impact of trucks exceeding the allowable height clearance of the bridges. Due to collision impact of the trucks, the bottom or outer layers of concrete girders are usually peeled off (see Figure 8-a) so that the steel reinforcements are exposed to the surrounding environment and subjected to corrosion. This issue is also related to protection of reinforced concrete bridge superstructures and piers subjected to potential impact by barges and ships (refer to Figure 8-b). A collision protection or scarifying system is in pressing need, and it can protect the concrete girders and piers from such impact damage and thus ensure the integrity of the bridge structures.

a) Collision Damages of Highway Bridges by Buses and Trucks

b) Collision Damages of Highway Bridges by Ships

Fig 8: Collision Damages of Highway Bridges by Buses, Trucks and Ships
Mosallam (2004) developed an innovative functionally-degraded sandwich system (I-Lam®) to act as a scarifying impact system for reinforced concrete members. Results of the full-scale tests confirmed the success of the I-Lam system in protecting the RC beam from both localized and global damages. The as-built unprotected beam experienced severe damages with major concrete spalling and distortion of the steel reinforcement. As shown in Figure (9), minimum damage occurred to all I-Lam protected beams specimens. Only surface evenly distributed flexural hair cracks were observed, especially at the back side of the beam (tension side).

Based on the success of the I-Lam system, Ohio Department of Transportation (ODOT) has approved the installation of the system on one of problematic bridges in Ohio. Figure (10) shows the installation procedures for the I-Lam system.

**Fig 9: Comparisons of Unprotected and I-Lam Protected Reinforced Concrete Beams Before and After Impact**

2.4.2. Hybrid LDPE/FRP Collision Protection System for Bridge RC Piers

Several marine applications using recycled hybrid systems were constructed as demonstration projects by the US Army Corps, US. Navy, port authorities in USA (e.g. Port Wannimaie, Delaware Port Authority, etc.) and recently, California Department of Transportation (Caltrans). Caltrans

* US Patent Pending
introduced a new structural application for highway bridges where recycled LDPE/FRP hybrid beams (or camels) are used as a protection system for highway bridge abutments from potential impact by ships and barges. A pilot study aimed at evaluating both the service and the ultimate behavior of recycled Low-density Polyurethane (LDPE) beams reinforced with glass fiber reinforced polymer (GFRP) composites rebars (Mosallam 2009). The objective of this study was to conduct pre-qualification full-scale tests for this hybrid system for ship collision protection system for Oakland Bridge and other California bridge piers (refer to Figure 11).

Fig 10: First Field Application of the I-Lam Overheight Collision Protection System in Ohio, USA

Fig 11: Examples of Field Application of LDPE-GFRP for Bridge Collision Protection
3. STRUCTURAL UPGRADE OF STEEL MEMBERS USING FRP COMPOSITES

3.1. General:

The use of adhesives provides attractive features for strengthening existing under-rated bridge steel members. This includes the ease of applications, minimizing heavy equipment, minimizing or eliminating the need for making holes or using bolts. As a result, this approach can provide the structural engineers with quick and low cost fix for different members such as bridge steel girders and columns. The motivation of this research study was initiated by the urgent need to increase the static flexural capacity of the steel girders of the Sauvie Island Bridge.

3.2. The H-Lam System*:

The innovative sandwich system was developed by the author specifically for steel strengthening applications. The reinforcing honeycomb polymer composite panels consist of high strength composite facing sheets bonded to a lightweight high density/high strength core material. The H-Lam panels were designed such that they are both thermally and mechanically balanced. The face sheets of the composite sandwich panels are comprised of 0°/90° carbon/epoxy laminates with E-glass/epoxy thin laminates at the interface with the steel girder and the aluminum honeycomb core. The reason for using 90° cross laminates is for stability of the unidirectional laminates during both the fabrication and during service. The H-Lam system used in this application has an E-glass/epoxy cover layers to protect the carbon-based composite panel from galvanic corrosion (the galvanic corrosion occurs upon direct contact of the carbon/epoxy to steel in the presence of moisture, which in this application is unavoidable). In addition, the H-Lam panels have an E-glass peel-ply (refer to Figure 13) to protect the pretreated face sheet to be bonded to the steel bottom girder. Unlike the general-purpose epoxy used for the off-the-shelf CFRP (Carbon/epoxy Fiber Reinforced Polymer) composite strips system, which was originally developed for concrete and masonry, the H-Lam adhesive system was engineered specifically for steel strengthening application. In addition, the H-Lam system offers the choice of using either bond-only or bond/bolted joint between the composites and the steel member.

3.3. Flexural Upgrade of Sauvie Island Bridge Steel Girders using H-Lam Technology, Portland, Oregon, USA:

Based on the successful verification tests results, the H-Lam system was approved by the Bridge Department for actual bridge installation. The field application was performed on selected steel girders of a selected span of the Sauvie Island Bridge (refer to Figure 13). All composite panels, adhesives and tools were transported to the site at the same day of application. The field application took place on a Sunday to ensure minimum traffic interruption. In addition, a traffic restriction (from 12 p.m. to 9 p.m.) for all vehicles over 10 tons was posted on the bridge two weeks prior to the construction date. Temporary clamping steel/plywood fixtures were used for applying pressure to the composite panels during curing and were removed after one day of application. The application was completed in approximately five hours and the panels were instrumented with strain gages in different locations for the third ongoing health-monitoring phase. In addition to strain monitoring, several composite samples were adhered to steel using the same types of adhesives that were subjected to the same field environment. Frequent pull-off tests are being performed to monitor the long-term bond line strength at different environmental exposures. Detailed information on this project is reported by Mosallam (2006).

*US Patent Pending # 60-146,830
4. REPAIR & REHABILITATION OF WOOD MEMBERS USING FRP COMPOSITES

As compared to other structural applications of polymeric composites, limited information is available on structural behavior of wood members strengthened with polymer composites. A new generation of advanced composites for structural upgrade of wood members was developed by Mosallam (2013) similar to the H-Lam system discussed earlier for strengthening steel members. However, the fiber architecture of face sheets and core material type was different for wood applications. This repair system utilizes the concept of thin sandwich panels that is bonded and also screwed to the wood member (see Figure 14). The advantages of using sandwich panels in this application includes: (i) ease of application, (ii) increase in quality control of the prefabricated materials and shop pretreated surfaces, (iii) light-weight features, (iv) the presence of stiffened holes allows for drilling metal screws or nails that will act as both shear connectors and prior to adhesive curing as a temporary clamps, (v) superior fire properties due to higher glass-transition-temperature ($T_g$) and the use of phenolic matrix, and (vi) overall all economic advantages. Two types of composites: wet layup and sandwich panels, and two lamination schedule; unidirectional and bidirectional, and two lamination geometry, U-laminate and flat laminates were evaluated. For “flexure-shear” wood beams repaired and retrofitted with bidirectional, carbon/epoxy U-shaped wet layup laminates, a total of eight 203 mm X 203 mm X 3.0 m (8” X 8” X 10’) Douglas Fir (Dug Fir) Larch # 1 wood beams were tested to failure. Experimental results indicated that, in general, the use of composites as external repair and rehabilitation elements resulted in an appreciable increase of both strength and stiffness of the as-built wood beams. For example, test results indicated that an increase, up to 180% of the strength of pre-damaged beam repaired with carbon/epoxy composites is achieved. In addition, the flexural stiffness of the strengthened beam was upgraded to about 150% as compared to the pre-damaged beam specimen. Figure (15) presents load-displacement curves as-built wood beam and strengthened beams with flat unidirectional CFRP laminate and H-Lam system. As shown in this figure, the H-Lam strengthening system has superior stiffness, strength and toughness as compared to both the as-built and flat CFRP strengthened wood beams.
5. REPAIR & REHABILITATION OF MASONRY WALLS USING FRP COMPOSITES

One of the successful applications of FRP is upgrading the seismic performance of unreinforced masonry (URM) walls, which are the primary load carrying components of unreinforced masonry buildings. In old building constructions, these walls were primarily designed to carry gravity loads. Due to the absence of any lateral load carrying component, such constructions are generally fragile during ground excitation resulted from seismic events. In fact, a significant damage of these walls is observed in past due to earthquakes (Hess, 2007). Hence, seismic retrofitting of these buildings is
required in order to upgrade their seismic performance and improve the ductile behavior. Haroun, et al. (2005) conducted a comprehensive verification experimental program on cyclic in-plane shear of concrete masonry walls strengthened by FRP laminates. In this study, different types of composites including E-glass and CFRP wet layup laminates as well as pre-cured CFRP strips were evaluated (refer to Figure 16). The reported experimental results demonstrated the effectiveness of different FRP materials and lamination schemes in order to externally repair or retrofit URM walls. A significant gain in-plane shear capacity is observed when the walls are strengthened with FRP composites on either one or both sides of walls. The key experimental observations are: (i) the application of FRP in order to repair the pre-cracked wall resulted in 20% gain in the in-plane shear capacity in comparison with the as-built wall; (ii) a maximum increase of 35% is achieved in the in-plane shear capacity of URM walls when the walls are retrofitted with FRP; (iii) due to retrofitting, the ultimate failure mode changes from diagonal cracking of walls (brittle failure in nature) to compression failure at one of the wall toes (ductile failure in nature); (iv) the yield and ultimate displacements of the retrofitted walls are recorded as considerably higher than the same of the retrofitted wall. Figure (17) presents the load-displacement envelopes for all wall specimens. Figure (18) shows a summary of the ultimate shear strengths of different masonry walls.

Fig 16 Samples of Masonry Walls Retrofitted with FRP Composites: (a) Carbon/epoxy procured strips, (b) Carbon/Epoxy Wet Layup Laminates
6. DEVELOPMENT OF CODES, STANDARDS AND DESIGN GUIDES

In recent years, the construction industry started to realize the potential of using polymer composites in construction applications. Unfortunately, the construction industry and the civil engineers were faced with tremendous amount of difficulties to utilize these materials in the same manner they are used to for the conventional material such as steel, concrete and wood. The major obstacle, is the lack of design standards and authoritative codes for the use of these materials in construction applications. Despite the fact that there is a great deal of research and application information available from the aerospace industry for the past four decades or so, still the civil engineers are searching for ways to convince them with the reliability, applicability and the structural efficiency of such materials. For any structural system, design standards are one the essential requirements for professional engineers acceptance. The following paragraphs describes the effort of different professional organizations in publishing technical documents in this area.

6.1. United States of America (USA):

The American Society of Civil Engineers (ASCE), American Concrete Institute (ACI), and the has been involved in the development of several standard documents for different materials and systems. In 1997, the International Code Council, ICC of USA (formerly called International Conference for Building Officials, ICBO) Evaluation Service (ES) produced two acceptance criteria related to repair and rehabilitation of reinforced concrete and masonry structures; namely AC125 and AC 178 that are available at the ICC-ES website (icc-es.org). Unlike the ACI current proposed document, the AC125, focused more on applications related to seismic design. For bridge applications, the American Association of State Highway and Transportation Officials has published LRFD Bridge design guide specifications for GFRP-reinforced concrete bridge decks.
and traffic railings in 2009. Copies of this document can be obtained at the AASHTO website: www.transportation.org.

6.2. Japan:

In Japan, the Research Committee on Continuous Fiber Reinforcing Materials published a recommendation for design and construction of concrete structures using continuous fiber reinforcing materials (1997). Also, the Japanese Society of Civil Engineers (JSCE) established a subcommittee on FRP Bridges was and has published in 2004 a technical report titled “FRP bridges – technologies and their future”. Some of these documents are accessible to public at: http://www.jsce.or.jp/committee/concrete/e/newsletter/newsletter01/recommendation/FRP-sheet/document.htm

6.3. Canada:

In Canada, major efforts in establishing design and specifications for FRP composites in construction has been have been accomplished. The Canadian Standards Association has taken the lead in this effort by developing two documents focusing on FRP composites; namely (i) Design and Construction of Building Components with Fibre-Reinforced Polymers (2007), and (ii) Specification for Fibre-Reinforced Polymers (2010). For bridge applications, design information on composites was included in the Canadian Highway Bridge Design Code (2006). These documents can be obtained through the Canadian Standards Association web site: www.shopcsa.ca. Several documents were also developed by the ISIS Canada Research Network (ISIS). The following are some of the ISIS published design related publications that can be obtained from ISIS web site: http://www.isiscanada.com/publications/design-manuals/.

6.4. Europe:

In Europe, several organizations have been working on developing standard and technical documents related to FRP composites in construction applications. One of the active organization is fib (Fédération Internationale du Béton or the International Federation for Structural Concrete). Two bulletins (Bulletin 14 and Bulletin 35) were published by fib in 2001 and 2006. These documents can be obtained from the fib web site: http://www.fib-international.org/publications/fib. A technical document on design and construction of structures made of thin FRP pultruded elements was published in 2002 by the European Committee for Standardization (CEN), Brussels, Belgium. A copy of this document can be found at: http://www.cnr.it/documenti/norme/IstruzioniCNR_DT205_2007_eng.pdf

6.5. Middle East & North Africa:

The first design code for FRP composites in Strengthening and Repair applications was developed few years ago in Egypt. The code is published by the Housing and Buildings National Research Center (HBRC). Copies of this code can be obtained through HBRC website: http://www.hbrc.edu.eg/en/Home.html.

6.6. Australia:

Funded by the Australian Federal Government, the Queensland State Government, with additional support from a range of industry stakeholders, an initiative to establish a Fibre Composites Design and Development (FCDD) program was initiated with an ultimate goal of developing a design code of practice for FRP composites. Similar to the ICC-ES certification program in USA, another initiative to develop a National Constituent Certification Scheme (NCSS) to provide a mechanism for evaluating and accepting different types of FRP composites systems for infrastructure applications.
6.7. **International Organization for Standardization (ISO)**

Three ISO documents were published related to FRP composites:

- **ISO/DIS 14484**, *Performance Guidelines for Design of Concrete Structures using Fibre-reinforced Polymer Materials.*
- **ISO 10406-1** *Fibre-reinforced polymer (FRP) reinforcement of concrete — Test methods — Part 1: FRP bars and grids.*
- **ISO 10406-2** *Fibre-reinforced polymer (FRP) reinforcement of concrete — Test methods — Part 2: FRP sheets.*

These international standard documents are available at the ISO website: [http://www.iso.org](http://www.iso.org).

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TU1406 COST Action – Quality Specifications for Roadway Bridges: Standardization at a European Level (BridgeSpec)

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Abstract. Asset management strategies rely on maintenance actions to keep infrastructures at desired levels of performance. In case of roadway bridges, specific performance indicators are established for their components and, when combined, allow to evaluate the overall performance. These indicators, which can be qualitative or quantitative based, are obtained during principal inspections through visual examination, non-destructive testing and by temporary or permanent monitoring systems. After being obtained, these indicators are then compared with performance goals in order to evaluate if quality control plans are accomplished. It is possible to verify the existence in Europe of multiple methods used to quantify these indicators and how such goals are specified. COST Action TU1406 aims to establish a European guideline in this matter, addressing new indicators related to sustainable and economic performance of roadway bridges. An overview of this Action, with a special liaison to the sustainability issues, is provided along this paper.

1. INTRODUCTION

Roadway bridges are considered to be, in terms of maintenance, one of the most critical components of road infrastructures. Though they belong to the domain of public service, their management mechanism can be conducted by the state or under a private public-partnership model. In both cases, a QC plan, which compares, for each performance indicator, the assessed value with a pre-specified goal, should be accomplished.

However, it is verified that those plans vary from country to country and, in some occasions, within the same country. This is a huge problem, as large variation in the quality of roadway bridges is verified. Also, most of these plans do not incorporate any sustainability issues. Therefore, the COST Action TU 1406 aims to achieve the European economic and societal needs by standardizing the condition assessment and maintenance level of roadway bridges. Moreover, it will be important to address, in such plans, new indicators related to sustainable performance (e.g. noise, carbon retention rates, etc.). This constitutes a scientific advance as, actually, QC plans do not consider them.

In order to establish a standardized procedure for the assessment of performance indicators, namely, those that should be considered in a QC plan, as well as to define the performance goals, a network of experts is needed. Such network should incorporate people from different stakeholders (e.g. universities, institutes, operators, consultants and owners) and from various scientific disciplines (e.g. on-site testing, visual inspection, structural engineering, sustainability, etc.).

To summarize, there is a real problem which is the non-uniform way QC is actually developed for roadway bridges and the non-inclusion of any sustainability indicator. This is surpassed by establishing a guideline, which constitutes the main outcome of this Action. Such guideline will comprise specific recommendations for assessing performance indicators, as well as for the definition of performance goals, being expected the impacts expressed in Table 1.
Table 1. COST Action impacts.

<table>
<thead>
<tr>
<th>Impact</th>
<th>Description</th>
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<tbody>
<tr>
<td>Sustainability</td>
<td>Decrease the bridge lifecycle, maintenance and repair costs; Decrease of total energy consumption and carbon footprint; Increase of mechanical, durability and environmental performance.</td>
</tr>
<tr>
<td>Economic and societal</td>
<td>Improve user satisfaction; New job opportunities associated with new QC services; Improve economic efficiency; Increase competitiveness in structural engineering industry; Enhance risk management.</td>
</tr>
<tr>
<td>Well-being of general public</td>
<td>Decrease of maintenance, repair and reconstruction activities; Decrease of downtime situations; Decrease of disruptions; Increase of usercomfort.</td>
</tr>
<tr>
<td>Research community</td>
<td>Better perception of real problems; Improve the cooperation between research and practice; Establishment of reliable comparisons between countries; Improvement on research developments and practical procedures; Reduction of the gap between countries.</td>
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2. GENERAL BACKGROUND

In Europe, as all over the world, the need to manage roadway bridges in an efficient way led to the development of different management systems. Hence, nowadays, many countries have their own system. Although they present a similar architectural framework, several differences can be appointed, for example, with regard to the condition assessment procedure. These differences constitute a divergent mechanism that may conduct to different decisions on maintenance actions.

Within the roadway bridge management process, the identification of maintenance needs is more effective when developed in a uniform and repeatable manner. This process can be accomplished by the evaluation of performance indicators, improving the planning of maintenance strategies. Therefore, a discussion at a European networking level, seeking to achieve a standardized approach in this subject, will bring significant benefits.

In this context, a first step would be the establishment of specific recommendations for the assessment of roadway bridges, namely, used methods for the quantification of performance indicators. A set of reference time periods for these assessment actions should be also presented. A second step would be the definition of standardized performance goals. Finally, a guideline for the establishment of QC plans in roadway bridges would be developed. In these plans, it is emphasized the importance of advanced deterioration predictive models. Moreover, the concept of sustainable roadway bridge management, involving the evaluation of environmental, economic and social performance indicators during the whole lifecycle, is also highlighted.

By developing new approaches to quantify and assess bridge performance, as well as quality specifications to assure an expected performance level, bridge management strategies will be significantly improved, enhancing asset management of ageing structures in Europe.

3. CURRENT STATE OF KNOWLEDGE

Within last years, significant research has been developed worldwide regarding the condition assessment of roadway bridges, namely through the use of non-destructive tests, monitoring systems and visual inspection techniques. Obtained values, which will provide information regarding the assessed bridge state condition, were then compared with previously established goals. As a result, there are nowadays several ways of evaluating a bridge condition.

More recently, the concept of performance indicators was introduced, simplifying the communication between consultants, operators and owners. However, large deviations are still verified on how these indicators are obtained and, therefore, specific actions should be undertaken in order to standardize this procedure.
It is verified that QC plans should always address the assessed performance indicators and pre-specified goals. However, these latter values are even more difficult to obtain as they are highly subjective. As a result, a dispersion of QC plans is verified. Once roadway concession contracts are based on such plans, this may become an enormous problem for the future of our society.

It is known that in the past a similar problem was addressed with roadway pavements. Although this was verified worldwide, in Europe it was solved through COST Action 354 (performance indicators for pavements). In a similar way, during this Action, a network of experts in the field of roadway bridges will establish specific recommendations for assessing performance indicators as well as for the definition of corresponding goals. This activity will be supported in a data basis, gathered from different COST countries. The objective is to develop, for the first time, a guideline for the establishment of QC plans in roadway bridges.

Moreover, it will be also analyzed the possibility of incorporating new indicators related to sustainable performance of roadway bridges. Some of these indicators were evaluated with success within the COST Action C25 (sustainability of constructions: integrated approach to life-time structural engineering). The final purpose is to establish detailed recommendations for assessing them as well as for the definition of specific goals, in a similar way as for the other indicators, and then integrating it in the developed guideline.

4. OBJECTIVES

The main ambition of the Action is to develop a guideline for the establishment of QC plans in roadway bridges, by integrating the most recent knowledge on performance assessment procedures with the adoption of specific goals. This guideline will focus on bridge maintenance and lifecycle performance at two levels: (i) performance indicators, (ii) performance goals. By developing new approaches to quantify and assess the bridge performance, as well as quality specifications to assure an expected performance level, bridge management strategies will be significantly improved, enhancing asset management of ageing structures in Europe.

In order to reach this main general aim, the following more specific objectives/deliverables have been considered: (i) to systematize knowledge on QC plans for bridges, which will help to achieve a state-of-art report that includes performance indicators and respective goals; (ii) to collect and contribute to up-to-date knowledge on performance indicators, including not only technical indicators but also environmental, economic and social ones; (iii) to establish a wide set of quality specifications through the definition of performance goals, aiming to assure an expected performance level; (iv) to develop detailed examples for practicing engineers on the assessment of performance indicators as well as in the establishment of performance goals, to be integrated in the developed guideline; (v) to create a data basis from COST countries with performance indicator values and respective goals, that can be useful for future purposes; (vi) to develop a webpage with information about the Action and its participants, as well as, video-streaming from presentations at training schools, workshops and conferences, e-lectures, written material (e.g. technical reports), etc.; (vii) to support the development of technical/scientific committees; (viii) to disseminate activities, such as Short-Term Scientific Missions (STSM), training schools and other teaching activities (e.g. e-lectures), for practicing engineers and researchers, regular workshops, a conference and special sessions at international conferences.

5. TARGET GROUPS/END USERS

The target groups and end users who will exploit the outcome of this Action are: (i) public/private owners, as their assets will be maintained in an upscale level; (ii) operators, as standardized procedures for reducing maintenance costs, guaranteeing the same quality-level, will be introduced; (iii) design and consultant engineers, as the assessment of roadway bridges performance will be established in a uniform way, according to the developed guideline; (iv) equipment and software companies, as a new perspective will be given, regarding the most suitable equipment and software for the assessment of roadway bridges; (iv) academics and researchers engineers, as they will take an advantage of their involvement in the guideline preparation; (v) students, as they will benefit from COST tools (e.g. training schools) and from the contact with different stakeholders, involved in this Action; (vi) relevant European,
international and national associations, with which the main outcomes of this Action will be shared; (vii) standardization bodies and code writers, which will benefit from the developed guideline.

6. SCIENTIFIC PROGRAMME

The scientific focus of the Action is centered in the production of a guideline for the establishment of QC plans for roadway bridges across Europe. In this context, this Action deals with recent developments on bridge safety, maintenance and management, according to a lifecycle outlook, aiming to define a standardized procedure for performance assessment as well as for the establishment of performance goals in order to accomplish a pre-specified service level. Moreover, it is intended to demonstrate the applicability of the developed guideline, and other recommendations, with case studies.

The scientific work plan of this Action ensures the working progress in support of the objectives established. It is organized, based on the division of tasks (and subtasks) allocated for each WG, and according to a timetable.

6.1. WG1: Performance indicators

It is known that management systems are supported in QC plans which in turn are supported by performance indicators. Therefore, it is highly important to analyze such indicators in terms of used assessment frameworks (e.g. what kind of equipment and software is being used), and in terms of the quantification procedure itself. In this particular work group, the objectives will be the definition of:

(a) Technical indicators: the goal in the first step is to explore bridge structures performance indicators, in the course of international research cooperation, which capture the mechanical and technical properties and its degradation behavior. Moreover, environmental condition, natural aging, and material quality regarding to some indicators will be investigated and evaluated in their meaningfulness. These considerations, however, also include service life design methods, aimed at estimating the period of time during which a structure or any component is able to achieve the performance requirements defined at the design stage with an adequate degree of reliability. On the basis of the quality of input information (mainly concerning with the available degradation models), as sketched in the above description, it is possible to distinguish among deterministic methods, usually based on building science principles, expert judgment and past experience, which provide a simple estimation of the service life, and probabilistic methods;

(b) Sustainable indicators: in addition to technical performance indicators, which characterize the ultimate capacity as well as serviceability conditions, environmental-based sustainability indicators, will be also formulated. These variables characterize the environmental impact of a structure in the course of its total lifecycle, expressed in terms of total energy consumption, carbon footprint (CO2 emission), raw materials balance, etc. These indicators can be separated into direct and indirect indicators, where the former are related to the construction/maintenance itself and the latter are caused e.g. as a consequence of limited functionality;

(c) Other indicators: other sustainable indicators, economic and social based, may be used to evaluate a bridge performance. These indicators, based on the technical performance of a structure, capture additional aspects that may influence the decision process and typically represent the discounted (accumulated) direct or indirect costs associated with construction and maintenance. Summed up over the full life-time, they represent part of or the full lifecycle costs. They can, in the context of multi-objective optimization, be understood as a weighting scheme to arrive to a single objective function to be minimized.

The milestone for this task is the publication of a report on these performance indicators until the end of year 1. Such report will address a general description, how they are assessed (e.g. visual inspection, non-destructive tests and monitoring systems), with what frequency, what values are generally obtained and, finally, some general recommendations. This outcome will be one of the main inputs of WG5, being also used by WG3.
6.2. **WG2: Performance goals**

The main objective of this workgroup is to define a set of goals for previously identified indicators in WG1. These goals will vary according to technical, environmental, economic and social factors. Specific recommendations will be given in order to ensure that the definition of such goals should be the most generalized as possible. In particular, it will be established:

(a) **Technical goals:** it will be analyzed what goals are actually used for technical performance indicators in roadway bridges and its components (e.g. bearing, joint, etc.). It will be also evaluated which are being defined in the course of international research cooperation. There will be an open discussion within the experts’ network in this field, in order to determine the most important factors for the definition of such goals as well as the most suitable threshold values. It will be established goals, both for deterministic and probabilistic methods, for time-varying indicators and for different assessment procedures (e.g. visual inspection, non-destructive tests and monitoring systems);

(b) **Sustainable goals:** specific goals will be defined for sustainable indicators, environmental based. This task is much more difficult to perform than for technical indicators, as the historical data basis is much smaller. Nevertheless, an open discussion will be established within a network of experts in this field, in order to identify the most important factors for the definition of these goals as well as the most appropriate threshold values;

(c) **Other goals:** the definition of goals for other sustainable indicators, economic and social based, is extremely difficult as it largely depends on the established agreement between the owner and the roadway operator (concession model). Nevertheless, it will be important for the future of Europe to define such goals, or at least to provide some recommendations, so that standardized procedures can be implemented. In order to achieve this objective, an open discussion will be developed among a network of experts.

The milestone for this task is the publication of a report on performance goals until the end of year 2. Such report will address a description of the most important technical, environmental, economic and social factors, how to compute each goal, with what frequency, what values are generally obtained as well as some general recommendations. This outcome will be one of the main inputs of WG5, being also used by WG3.

6.3. **WG3: Establishment of a QC plan**

The desired service quality of the whole bridge can be affected by a single dysfunctional component or by the combination of several dysfunctional components. The decrease in bridge service quality clearly depends on the degree of components’ dysfunctionality. This dependency can be modelled, among others, by Bayesian nets, which provide the time variation of each bridge component performance.

However, in order to assure a desired service quality with minimum interruptions, bridge owners launch preventative actions when the risk of service impairment, interruption or losses in lifecycle costs reaches some predefined level. Implicitly the owners define herewith the accepted risk which can be different from country to country, based on social equity principles. This accepted risk depends upon the established performance goals for each component or combination of bridge components.

The QC plan mirrors these findings and is used for maintenance planning by defining a criteria for triggering maintenance interventions. Clearly, these QC plans have to be established for each individual bridge. They perform the basis for the asset management of this type of roadway infrastructure. The objective of this task is to establish a procedure, based on Bayesian nets or other heuristic rules used worldwide, which would allow the bridge owner to define a QC plan for each individual bridge.

The milestone for this task is to prepare a report with detailed explanation of the steps towards the establishment of a QC plan for different types of bridges until the middle of year 3. This outcome will constitute the basis of WG5, being also used by WG4.
6.4. WG4: Implementation in a case study

During this task a set of roadway bridges, belonging to different COST countries and preferably with identical typologies, will be identified. Then, for those bridges, it will be obtained the performance indicators (identified in WG1). Such values will be then compared with pre-specified goals (identified in WG2) and, finally, a QC plan will be implemented (detailed description at WG3). Different methodologies for obtaining such indicators, as well as different threshold values, will be used as the basis for benchmarking.

At the end of this task, a QC plan will be applied to such bridges, according to the recommendations established by WG3. The main objective of this study is to show the existing dispersion between obtained performance indicator values and its goals. It is important to note that this will reflect the existing dispersion among QC plans. Also, it will be tested and validated the implemented QC plan, according to the recommendations given by WG3. Obtained results will be discussed within a high level of network of experts in this field.

There are several ongoing national research projects in COST countries with which a close interaction may be established within the scope of this task. Namely, some of the roadway bridges which will be used as case study may be selected from those projects. Additionally, there will be several people from industry (e.g. owners, operators, etc.) involved in this working package.

The milestone of this task is to prepare a data basis from benchmarking, until the middle of year 4. Obtained results will validate the outcomes of WG1, WG2 and WG3, and will be used by WG5.

6.5. WG5: Drafting of guideline/recommendations

In this task it will be joined the work developed in other working packages (especially from WG1, WG2 and WG3) with the objective of writing a guideline, and recommendations, for the implementation of a QC plan for roadway bridges that could be adopted by several roadway agencies. The main goal will be the preparation of a document that can be easily adopted by engineers facing the management of new and existing bridges.

Therefore, the format and content should follow the existing codes / guidelines / recommendations used today by agencies. Hence, the first step will be the analysis of existing documentation and work developed in other similar research programs and by standardization committees at national and international level.

Due to the objective proposed, this working package will have a strong interrelation with all the other working packages, becoming an output for WG6 (dissemination). Finally, the milestone of this task is the development of a new guideline for the establishment of QC plans in roadway bridges until the end of year 4.

7. DISSEMINATION PLAN

The Action will enable useful synergies and disseminate the results to several target groups and end users. In order to achieve this, a specific WG6: dissemination of results, was introduced. This WG will assure the effective dissemination mechanisms to publish the progress and results of the Action. Among these tools are: (i) website, leaflets, posters, TV channels, radio stations, newsletters and online service news; (ii) workshops, conferences, training schools and STSM (Short Term Scientific Missions); (iii) Conferences, peer-reviewed articles and reports issued by the Action; and (iv) Guideline and link to standardization.

A website was developed – [http://www.tu1406.eu](http://www.tu1406.eu) – containing information about the Action itself which will be continuously updated. Any expert may join the action by filling a google form which is available in this website. Also available are a facebook page and a LinkedIn account accessible by [https://www.facebook.com/tu1406ca](https://www.facebook.com/tu1406ca) and [https://www.linkedin.com/company/tu1406](https://www.linkedin.com/company/tu1406).

Workshops, conferences, training schools and teaching activities will allow to explain the performed scientific work between researchers, industry and stakeholders, as well as the practical approach of the developed guideline. STSM are specially promoted to early-stage
researchers that encourage the synergy among institutions, accelerate the learning of students and provide academia and industry with highly trained staff.

The achievements of this Action will be published in international conferences, as they bring together researchers, academia and industry in an open-discussion forum, in peer-reviewed articles, as they are an important tool to prove the impact and accuracy of obtained results and to make them available for the future, and in technical reports (state-of-art reports and others) which will have the involvement of peer-reviewers from other countries.

The guideline to be achieved will include the establishment of QC plans in roadway bridges, comprising performance indicators assessment and its goals, as well as the obtained results. This recommendation report will be developed in close cooperation with scientific and practicing community and linked to European and international standards.

8. FRAMEWORK

The Action proposal arose due to the existing concern from owners, operators, consultants and researchers regarding the existence of multiple methodologies to assess and classify roadway bridges state condition. Within an R&D project developed in Portugal (SustIMS – Sustainable Infrastructure Management System; https://www.youtube.com/watch?v=Ls1W5oxVD8w), which aims to develop a cross-asset management system for highways [5], it was identified the idea of standardizing the existing practice.

In a first stage, a national analysis to assess the potential of the idea was performed, having been addressed two entities for the purpose: the Portuguese Association of Highway Operators and the Portuguese Roadway Agency (now Infrastructures of Portugal), that confirmed the same concern.

Having obtained a positive feedback, some contacts were performed at European level and a first team, with experts from different European countries, research fields and stakeholders, was established to work on this issue. Within this team was considered that the COST Association platform would be the most suitable framework to support this project.

The proposal as described, achieved a high rating during the evaluation process, especially due to the innovative character of the new concepts approach such as the study and evaluation of sustainable performance indicators. From the approval of the proposal resulted the Action’s Memorandum of Understanding which is available in the official website of COST Association in http://www.cost.eu/COST_Actions/tud/TU1406 and also on the Action official website.

The Action was officially started in April 16, 2015 and will last for four years, ending on April 15, 2019. After the initial kick-off Meeting, the Action will be carried out according to the timetable provided in Table 2 (in bold, current status). A first workshop was developed with success in Geneva, 21-22 September 2015. During this workshop 34 contributions were received from several experts. It was also defined the main guidelines towards the WG1 database development.

### Table 2. COST Action timetable.

<table>
<thead>
<tr>
<th>Activity/Months</th>
<th>3</th>
<th>6</th>
<th>9</th>
<th>12</th>
<th>15</th>
<th>18</th>
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</tr>
</tbody>
</table>

Currently are involved 174 experts from 44 different countries, distributed between the Management Committee and the various working groups. Some of these experts represent the Mediterranean Countries. An attempt is being now made in order to involve more people from the Mediterranean NNC (Near Neighbor Countries).
9. ACKNOWLEDGMENTS

This article is based upon work from COST Action TU-1406, Quality specifications for roadway bridges, standardization at a European level (BridgeSpec), supported by COST (European Cooperation in Science and Technology).

REFERENCES


The IABMAS Bridge Management Committee – Overview of existing Bridge Management Systems, 2014.


Influence of Recycled Sand and Gravel on the Characteristic of Concrete

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Abstract. The objective of this paper is to investigate the influence of type and dosage of recycled sand (RS) and gravel on the fresh and mechanical properties of concrete. Experimental program was conducted on concretes made with different ratio of substitutions (15, 30, 70, and 100%) of natural sand and gravel with RS and gravel. At the fresh state, slump, air content, and density were measured at the exit of the mixer, and then at 30, 60, and 90 min after mixing. Tests were also performed for compressive strength at the age of 1, 7, and 28 days, whereas elastic modulus measurements were done at 28 days. The results indicated that maintaining the workability of recycled aggregate concrete depends on sand or gravel substitution and their rates. Up to 30 min, slump values were decreased, but after that, no substantial change in slump values was observed. Air content increased and density decreased, with increasing recycled aggregate content (sand or gravel). Mechanical properties, such as compressive strength and modulus of elasticity, were lower than those of reference concrete.

1. INTRODUCTION

The growing need of building materials causes a depletion of natural resources. Thus, the use of recycled aggregates in the production of concrete and cementitious materials is a way to meet the needs, while preserving the environment in a sustainable approach.

To expand the use of cementitious materials made with recycled aggregates such as concrete, there is need for extensive investigation not only of their mechanical and durability properties but also of their placement in the formwork. Recent studies show that the behavior of fresh concrete with natural aggregates depends on several parameters including the nature of the admixture, its dosage and nature of the aggregates, and their dimensions. These parameters also affect the hardened concrete properties. The valorization of recycled concrete aggregates requires a mastery of the characteristics of recycled aggregates. Pepe et al. recommended that recycled aggregates from construction and demolition waste could be suitable for concrete production if proper procedure of ensuring the quality of recycled aggregates is maintained. de Juan et al. confirmed that removing surface impurities and reducing particle heterogeneities of recycled aggregate significantly reduces the gap between the performance of recycled aggregate (sand and gravel) concrete and ordinary concrete.

The concretes made from recycled gravel (RG) have been subjected to numerous studies. Most of them focus on the effect of the quantity of used recycled aggregate on the concrete strength at early age and long term.
Certain researchers have used RG with different ratios (from 10 to 100%) and different size 4–32 mm. They observed that the unit weight, and the compressive strength of the concretes produced with waste concrete aggregate have not much significant difference in compressive strength of recycle gravel up to 30% replacement of natural gravel (NG), and the RG concretes with 100% replacement had lower compressive strength than the corresponding natural aggregate concrete. This was explained by attached mortar can be used to establish the mortar content that adversely affects properties of the aggregate for different applications and porosity of concrete increased considerably when natural aggregate is replaced by recycled concrete aggregate and also there is also reduction in the mechanical properties of the recycled concrete Topçu. It has to be noted that these studies focus on the effect of RG but there have been few studies on the effect brought recycled sand (RS) on the mechanical properties of concrete. Thus, the concrete is composed of water, cement, gravel, sand and admixture in order to maintain the workability. So any changes in sand properties also affect the properties of mortar and concrete in fresh and hardened states.

Recently, Zhao et al. concluded that values of slump and compressive strength of mortars containing dried fine recycled concrete aggregate is always larger than that of mortars containing saturated fine recycled concrete aggregate. Raeis Samiei et al. also studied properties of cement mortars containing recycled concrete sand, and observed that cement mortars always have better mechanical properties than the corresponding cement–lime mortars, and this could possibly arise from a synergic effect of lime hydraulicity and the filler effect due to the fine fraction of RS within the mix, that lead to better densification of the lime mortars by blocking the capillary pores. In framework of mortar, we can find some authors worked on mortars that contain recycle sand and they showed statistically significant differences for replacement ratios up to 25%.

Knowing that, the properties of concrete in fresh state, which play an important role on its workability, on the other hand, it is important to enhance the recycled aggregates. We found that the analysis on properties of concretes in fresh state was not given importance in previous studies. It is also important to study the effect of RS on the rheological properties.

Therefore, this research was conducted on concretes based on a couple cement/admixture with different substitution percentages (15, 30, 70, and 100%) of the RS or RG. Also, in this work, the effect of time on concrete rheological properties at the output of mixer and respectively, 30, 60, and 90 min (named: T0, T30, T60, and T90) is done. This allows to interpret changes in the rheology of concrete with recycled aggregates vs. time.

2. MATERIALS AND METHODOLOGY

2.1. Cements

In this study, we used cement, CEM I 52.5R CE CP2 having different physical properties were used, with finesse Blaine is 4520 cm²/g. This characteristic is given in Table 1.

<table>
<thead>
<tr>
<th>Cement</th>
<th>S₀₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>SO₃</th>
<th>MgO</th>
<th>K₂O</th>
<th>Na₂O</th>
<th>Cl⁻</th>
<th>Loss on ignition (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(%)</td>
<td>19.54</td>
<td>5.32</td>
<td>3.06</td>
<td>62.98</td>
<td>3.71</td>
<td>1.85</td>
<td>0.86</td>
<td>0.19</td>
<td>0.07</td>
<td>1.18</td>
</tr>
<tr>
<td>Mineralogy of clinker (%)</td>
<td>C₃S</td>
<td>54.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>C₂S</td>
<td>19.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C₃A</td>
<td>9.9</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>C₄AF</td>
<td>9.7</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1 Composition and properties of cements used
2.2. Admixtures

The admixture used in this study was compatible with cement. This admixture was high range water reducing admixtures, was based on poly-carboxylate with solid content 22.5%, this admixture is plasticizer which is water reducer.

2.3. Fine and coarse aggregates

2.3.1. Natural Sand (NS) and recycled sand (RS)

The natural sand used for the experiments was semi-crushed washed 0/4 produced from Lafarge, Sandracourt, and recycled sand was derived from a specific production carried from the DLB production platform located in Gonesse/France, with crushing building after demolition and separate to deposit and aggregates. All aggregates (approximately 210 tons of natural aggregates and 200 tons of recycled aggregates) have been manufactured at once, and then packaged in big bags of 500 kg or 1000 kg, and palletized. Sands were first sieved through 4 mm sieve to remove any particles greater than 4 mm. The gradations of the sands are shown in Figure 1.

Fig. 1. Grading curves of Natural sand (NS) and Recycled (RS)

Fig. 2. Natural sand and recycled (0/4mm)
The recycled sand appeared coarser than the natural sand (Figures 1 and 2). It is predominantly composed of small gravel (0.63 to 6.3 mm) and a low proportion of medium sand (0-0.63 mm). The recycled sand fineness modulus (FM = 3.27) was significantly higher than that of natural sand used (FM = 2.25). Therefore, substitution of natural sand with recycled sand in the recycled concrete may promote the segregation of the latter and the loss of workability.

2.3.2. Natural gravel (NG) and recycled gravel (RG)

The natural gravel (Figure 3 and 4) used for the experiments were washed (two cuts 4/10 and 10/20) produced from Lafarge, Givet, and recycled gravel was from the same origin and platform recycled sands. The gradation of gravel used, are shown in Figures 5.

Large recycled aggregates are comparable to big natural aggregates, recycled sands unlike with a significant lag.

Table 2 shows the coefficient of water absorption of the sands (NS and RS) and gravels (NG and RG) measured according to the NF 1097-6 after 24 hours. It is observed that the values obtained on recycled are much larger than those on natural.

Recycled aggregates are of two different types (natural aggregate and mortar old cement that is hung), this old cement mortar is the main cause of the increase in absorption Gutiérrez

3. EXPERIMENTAL PROGRAM

3.1. Mixture proportioning

Ten concrete mix proportions were made. The program was divided on two series: which the first series; concretes were made with recycled sand of varying, percentage 15, 30, 70 and 100% and natural gravel with 0% of recycled gravel. The second series; concretes were made with natural sand (with 0% of recycled sand) and recycled gravel of varying, percentage 15, 30, 70 and 100%.
All tests were carried out on the concretes which contain the same amount of cement (320 kg / m³). The ratio w/c varied from 0.59 to 0.78 and the amount of a mixture varied from 0.3% to 0.8% to keep the same slump (200 ± 20 mm) (NF EN 12350-2). Right out of concrete mixer, the following tests were immediately realized after mixing and at 30, 60 and 90 minutes: the slump test, measure of air content, density. Another portion of concrete has been used to make the test specimens to analyze of mechanical properties in hardened state. Mix proportions corresponding to the two tested series are shown in Table 3.
Table 2 Coefficient absorption water of sands (NS and RS) and aggregates (NG and RG)

<table>
<thead>
<tr>
<th>Type of recycled</th>
<th>Coefficient of water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS (0/4)</td>
<td>0.9</td>
</tr>
<tr>
<td>RS (0/4)</td>
<td>10.0</td>
</tr>
<tr>
<td>NG (4/10)</td>
<td>0.5</td>
</tr>
<tr>
<td>RG (4/10)</td>
<td>5.1</td>
</tr>
<tr>
<td>NG (10/20)</td>
<td>0.4</td>
</tr>
<tr>
<td>RG (10/20)</td>
<td>5.7</td>
</tr>
</tbody>
</table>

3.2. Preparation casting and testing

Given the strong absorption capacity of the recycled aggregates, they will be used in a saturated state. It is therefore usually necessary to saturate these aggregates that are stored in the laboratory. Below mentioned procedure was followed for the purpose.

- Taking an aggregate sample for evaluation of the initial water content;
- Setting was sealed with a certain amount of aggregate (40 until 80 kg) the barrel must first be moistened (by removing any excess water with a sponge) to prevent water loss water adsorption on the surface of the barrel;
- Addition of additional water necessary to achieve the target moisture content;
- Rolling the barrel for homogenization;
- Put to rest the barrel horizontally for a minimum of 2 hours.

Natural aggregates are in turn used in their natural state, provided they are saturated.

Each concrete was manufactured in the concrete mixer with a capacity of 50 lt. The produced volume of concrete was separated in two parts; one for tests in fresh state and second one is for specimens preparation for mechanical tests in hardened state.

Tests in fresh state are: workability was realizing with slump Abrahams cone according to standard NF EN 12350-2; the air content was measured by a cast iron tank capacity 8 lt, according to NF EN 12350-7 and the density was measured according to standard NF EN 12350-6.

The other volume of the concrete, for each mix 24 samples were prepared, which consists of 9 cylinders (300 x 150 mm) and 3 beams (7 x 7 x 28 mm). The compressive strength test was performed at 1, 7 and 28 days according NF EN 12390-3 on cylindrical specimens. The beams were utilizes to measures the modulus of elasticity of cured at 28 days according to ISO 1920-1910 of 2010.
Table 3 Recycled aggregate concrete mix proportions for each combination.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Cement</th>
<th>NS</th>
<th>RS (4/10)</th>
<th>NG (10/20)</th>
<th>RG (4/10)</th>
<th>RG (10/20)</th>
<th>SP (%)</th>
<th>Water (kg/m³)</th>
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<td></td>
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<td>0.60</td>
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<td>596</td>
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<td>325</td>
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<td>70RS0RG</td>
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<tr>
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<td></td>
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<td></td>
<td></td>
<td>97</td>
<td>0.72</td>
</tr>
<tr>
<td>0RS100RG</td>
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<td>279</td>
<td>0.78</td>
</tr>
</tbody>
</table>

4. RESULTS AND DISCUSSION

Fresh concrete properties

Variation of slump with time corresponding to recycled aggregate content

Figures 6(a) and 6(b) represent values of slump at T0; T30; T60, and T90, respectively for concrete containing recycled sand (series 1) and recycled gravels (series 2).

It can be observed from Figure 6(a) that whatever the percentage of recycled sand, there is significant slump loss up to 30 minutes, which varies between 55% and 65% of control concrete, however, beyond 30 minutes, slump is no significant variation in slump values.
These results also show that with a 30% of recycled sand, the concrete has a better retention rheology versus time. It can be explained by the fact that recycled sand contain a significant amount of fine (Fig.1), which reveals an increase in the compactness of the granular mixture to a maximum value corresponding to 30% of the dosage of the substitution. More than 30% substitution, the amount of fine from the recycled sand increased, which has a negative effect on the compactness.

On the other hand, the above remark, concrete with 100% recycled sand has a bad retention rheology reports than other concretes. This is due to the effect of packing recycled sand, and its surface roughness of the particles. This may be attributed to the adherence of mortar to the sand particle and thereby the surface texture of recycled sand is more porous than natural sand. Therefore, the recycled sand has a significant water absorption coefficient. While the quantity of recycled sand increase, it absorbs a lot of mixing water that available in the concrete mixture in fresh state, therefore the workability of concrete decrease as the results show.

Similar trends were observed for slump results of concrete containing recycled gravel from Figure 6(b). Like, up to 30 minutes, slump loss was about 30% of control concrete, beyond 30 minutes, slump is no significant variation. Similar results corresponding to concrete mixed with recycled gravel were reported. However, the slump results are little affected by the percentage of recycled gravel.

**Air content**

Influence of recycled sand and recycled gravel at T0 and T90, on the air content of concrete shown in Figures 7(a) and 7(b).

Figure 7(a) shows that when the percentage of recycled sand increases up to 30%, the air content is marginally increased. After that they increase rapidly with increase of recycled sand content, which is three times bigger than the air content of reference concrete. Indeed, recycled sand has a greater porosity than natural sand, and the shape and roughness of the recycled sand prevents air bubbles to extract concrete during vibration, (air content with 100% recycled sand is 65% higher than that of the control concrete).
Figure 7(b) demonstrates that increase in air content is very marginal for any percentage of recycled gravel. The marginal increase in the air content for gravel recycled can be explained by the fact that its size is greater than that of the recycled sand, and so with the introduction of natural sand that shows no roughness or port, so air bubbles can escape.

**Density**

The effect of recycled sand and recycled gravel on the density of concrete is shown respectively in Figures 8(a) and 8(b). It can be seen that density decreases with the increase in percentage of recycled sand and also the percentage of recycled gravel. This is due to the light weight and more porous nature of old mortar adhered to recycled aggregates that takes place around the aggregates. These results confirm the results of the variation of air content; when air content increases, the density decreases.

In addition, it can be observed that the reduction of concrete density with recycled sand is in the range of 6%. Therefore, for using the recycled aggregate with low density and considering to required strength class, we can use until 30% of recycled sand to obtain the similar density with natural one.
The reduction in density of concrete with recycled gravel is in the range of 1% when compared to concrete based on natural gravel. We can also use the recycle gravel with different percentage because its density is approximately similar with that of natural one. Anyway, the using percentage choice of recycled aggregates depends on the type of construction. For example: for light weight concrete structures is needed and those with heavy dead weight (construction bridge), use of recycled aggregate must be promoted.

Hardened concrete properties

*Compressive strength*

The compressive strength of concrete at early age (1, 7 and 28 days) is obtained with cylinder 150x300 mm kept at 95% relative humidity and 20 °C. The results are shown in Figures 9(a) and 9(b).

Figure 9(a) shows that when the amount of substitution of the sand is greater than 30%, the strength at the young age and 28 days increased remarkably. Less than this amount, the resistance remain substantially constant. This is quite logical in keeping the increase in air content and the density of concrete when the percentage of RS increases. Moreover, the presence of the cement mortar cotenant in RS, a weakened of the structure of the mixture, and
secondly the presence of pore that facilitates rupture. Further, with age, compressive strength increases for all concretes either made with recycled sand or recycled aggregates.

Otherwise, for concrete with different gravel recycling percentage, the resulting Figure 9(b) shows that resistance to early age and at 28 days remains practically constant. This is due to the small variation in air content and the density of concrete when the percentage of recycled gravel increases.

![Graph showing variation of compressive strength with age for recycled sand and gravel.](image)

(a) Recycled Sand (RS)

(b) Recycled Gravel (RG)

Fig. 9. Variation of compressive strength with age according recycled sand and gravel percentage

The obtained results are coherent with the results of another authors who concluded that: the small amount of the old cement stuck on the gravel recycled that a weak structure of the mixture with recycled sands, and also the weak presence of the pores which facilitates rupture, and interlocking between the cement paste and the recycled aggregates themselves; The use of
recycled aggregate lowered the compressive strength. The strength reduction is due to adhered mortar, which can consequence in difficulties in obtaining the needed workability, and increase in deformation; With the reduction in the amount of water in the concrete, as a result compressive strength increases. we see that has a very low slump, but has good compressive strength, by reducing available water content due to the high water absorption content of the recycled aggregate, can lead to higher strength depending on mixtures.

5. CONCLUSIONS

The valorization of concretes made of recycled aggregates requires mastery of parameters in fresh state and also in the hardened one. Until today several studies have shown that the presence of recycled gravel in concrete directly affect this resistance in early age and long-term. This work is conducted on concrete with the same couple cement / admixture and the analysis of our experimental results allow finding the following conclusions:

- with a percentage of 30% of recycled sand, concrete has a good workability rheology over time. In contrast, concrete with 100% recycled sand has a low workability rheology reports by the lowest percentage of recycled sand in concrete.
- similar trends were observed for slump results of concrete containing recycled gravel. The result show that: up to 30 minutes, slump loss was about 30% of control concrete, beyond 30 minutes; slump is no significant variation in slump values. However, the slump results are little affected depending on the percentage of recycled gravel.
- when the percentage of recycled sand increases up to 30%, the air content is marginal increase. After that they increase rapidly with increase of recycled sand content
- the percentage of recycled gravel in concrete has no significant influence in air content.
- when the amount of substitution of the sand is greater than 30%, the compressive strength at early age and 28 days decreased remarkably. This percentage is lower, the compressive strength remain virtually constant. Unlike, for concretes with different percentage of recycled gravel, results show that resistance to early age and at 28 days remains practically constant whatever the amount of substitution.
- we can also observe that with 15% of recycled sand, the relative compressive strength is more important than that of the natural mix at 1 and 7 days. However, there is no effect concerning the mix with the recycled gravel at 1 and 7 days.

The results presented in this paper allowing clarifying the effect of recycled gravel and / or recycled sand on the behavior of concrete in fresh and hardened state, this information provided a scientific basis for professional in order to develop the recycled concrete aggregate in order to reuse in industrial platforms.

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Effect of Silica Fume on High Performance Concrete Strength

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Abstract. High Performance Concrete (HPC) is now a days used widely in the construction industry worldwide. To produce HPC with normal ingredients one use mineral admixtures like silica fume, fly ash and metakoline. In addition admixtures including Superplasticisers are also used. This paper investigates the effect of silica fume (SF), added in varying percentages (0, 3, 5, 7.5, 10, 12.5%), on concrete strength. Superplasticiser - (Visocrete Tempo 12) - was added to the concrete mixtures as well. Compressive strength, shear strength and tensile strength tests were conducted, and the results were discussed. Results showed that the compressive strength changes with the varying percentage addition of silica fume. The highest compressive strength (88 MPa) was obtained when the percentage of silica fume was 7.5 % of the cement weight. Beyond 7.5 % silica fume, the compressive strength started descending. Tensile and shear strength were found to vary in an inverse relationship with the increasing percentage of silica fume. A significant reduction in tensile strength and shear strength were recorded when the silica fume percentage is increased. However, the 10% ratio between tensile and compressive strength for normal concrete, was not found to be, relevant for HPC with SF.

1. INTRODUCTION

High performance concrete (HPC) has improved properties over normal concrete leading to high performance applications in construction. HPC is made with carefully selected high quality ingredients and optimized mixture designs. is characterized with low water cement ratios in the range of 0.2 to 0.45, while Superplasticisers are usually used to make these concretes workable. HPC almost always has a higher strength than normal concrete. Silica fume (SF) is a byproduct of the producing of silicon metal or ferrosilicon alloys. One of the most beneficial uses for SF is in concrete. Because of its chemical and physical properties, it is considered as a very reactive pozzolan. Concrete containing SF can have very high strength and can be very durable. This paper presents some mechanical properties of HPC with SF as an additive material. Several mixes of HPC with different percentages of SF were prepared during this investigation. Fresh and hardened concrete properties were assessed. Results after 28-days curing showed that SF can significantly enhance the compressive strength of HPC. A high compressive strength of 88 MPa was achieved. On the other hand, a considerable reduction in tensile and shear strength of HPC was recorded. The reduction in tensile and shear strength may be related to the increase in brittle characteristics of HPC.
2. MATERIALS

2.1. Cement

The ordinary Portland cement type 42.5N used to prepare the concrete specimen conforms to the Libyan standards 340-2009.

2.2. Water

A clean drinking quality water was used as mixing water was used as mixing water for the production of the concrete.

2.3. Aggregate

Two types of fine aggregate were used in this study; the local natural sand and the ground sand. The Specific Gravity, the moisture absorption and the Unit Weight were 2.5, 1.2% and 1720 kg/m³ respectively. Two different size of coarse aggregate were adopted, 10 mm and 14 mm.

2.4. Supplementary materials and Chemical admixtures

A commercial silica fume with an average particle diameter of 150 nm was used. The main field of application is as a pozzolanic material for high performance concrete. A Sika Viscocrete Tempo 12 was used in the concrete mixes.

3. EXPERIMENTAL WORK

For HPC there is no specific method of mix design. In the present investigation the relevant available sources of literature concerning HPC were considered. An experimental investigation was carried out on the HPC specimens to determine the workability characteristics and mechanical properties such as compressive strength, splitting tensile strength and shear strength. Five levels additions of SF in concrete as a percentage of the cement weight, were considered and compared to the control mix. A total of 54 cylindrical specimens were cast for the hardened properties tests. Compressive strength, indirect tensile strength and shear strength of HPC after 28 days curing were investigated. For all mixes, the total amount of Portland cement, aggregate, and mixing water were kept constant. The percentage of silica fume was increased incrementally. Details of the concrete mixes are presented in table 1.

Table 1. Details of constituents for the concrete mixes.

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Cement kg/m³</th>
<th>Silica Fume kg/m³</th>
<th>Silica Fume %</th>
<th>Water Liter</th>
<th>CA (10mm) kg</th>
<th>CA (14mm) kg</th>
<th>Total FA kg</th>
<th>Viscocrete kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>23.795</td>
<td>0</td>
<td>0</td>
<td>9.518</td>
<td>28.828</td>
<td>43.0716</td>
<td>31.517</td>
<td>0.33</td>
</tr>
<tr>
<td>2</td>
<td>23.795</td>
<td>0.7138</td>
<td>3</td>
<td>9.518</td>
<td>28.828</td>
<td>43.0716</td>
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<td>0.33</td>
</tr>
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<td>23.795</td>
<td>1.1897</td>
<td>5</td>
<td>9.518</td>
<td>28.828</td>
<td>43.0716</td>
<td>31.517</td>
<td>0.33</td>
</tr>
<tr>
<td>4</td>
<td>23.795</td>
<td>1.7846</td>
<td>7.5</td>
<td>9.518</td>
<td>28.828</td>
<td>43.0716</td>
<td>31.517</td>
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<td>28.828</td>
<td>43.0716</td>
<td>31.517</td>
<td>0.33</td>
</tr>
</tbody>
</table>

3.1. Preparation and casting of test specimens

The following mixing sequence was considered, after several trial intended at optimizing the workability of the concrete.
All the ingredients were first mixed in dry condition in the concrete mixer for one minute. Then the assigned amount of mixing water was added gradually to the dry mix and the material was mixed thoroughly for one minute also. The assigned portion of the viscocrete was poured into the mixer and mixed for three minutes.

After the mixing procedures were completed, deformability and viscosity of fresh concrete were determined through the measurement of slump flow time and diameter, J-Ring test, L-Box ratio test and V-funnel flow time test.

Standard concrete cylinders (H300mm & Ø150mm) were prepared and tested after 28 days. All specimens were cast in one layer. After demoulding, all specimens were placed in a curing tank at room temperature. The specimen were tested for the compressive strength, splitting tensile strength and shear strength.

4. RESULTS AND DISCUSSION

4.1. Compressive Strength

The results of the cylinder compression strength (ACI code) are shown figure 1. For the compressive strength, an optimum percentage of 7.5% SF was found to reach up to 88MPa. However, beyond 7.5% the compressive strength reduced. The decrease in strength beyond the 7.5% may be due to the pozzolanic reaction and filler effect of the Silica fume.

![Graph showing Compressive strength with different percentages of SF](image)

Fig 1: Compressive strength with different percentages of SF

4.2. Tensile Strength

The splitting tensile strength results, as per ACI code, are presented in figure 2. The tensile strength was noted to decrease for all the SF percentage addition levels. ACI 363R-10 (ACI, 2010) reports a study by Dewar (1964) that claims that for lower strength concrete, tensile strength may go up to 10% of the compressive strength; however, for higher strength it reduces to 5%. Some of expressions used for calculation of splitting tensile strength are presented below:

ACI 318-11: \( f_{st} = 0.56 \sqrt{f_c} \);
Carrasquillo et al. (1981): \( f_{spt} = 0.59 \sqrt{f_c} \)
Iravani (1996) found that equations suggested by Carrasquillo et al. (1981) shows good agreement with experimental results within ±10%.

Table 2 shows that the tensile strength values obtained are in good agreement with the above equations.

<table>
<thead>
<tr>
<th>Mix No</th>
<th>S. F %</th>
<th>W/C ratio</th>
<th>Compressive Strength $f_c$ (Mpa)</th>
<th>Splitting Tensile Strength $f_{sp}$ (Mpa)</th>
<th>$f_{sp}/f_c$</th>
<th>Factor of $f_{sp}$ to $f_c$</th>
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<tbody>
<tr>
<td>1</td>
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<td>0.4</td>
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<td>$0.758\sqrt{f_c}$</td>
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<td>3</td>
<td>0.4</td>
<td>66.35</td>
<td>5.1</td>
<td>7.68</td>
<td>$0.626\sqrt{f_c}$</td>
</tr>
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<td>3</td>
<td>5</td>
<td>0.4</td>
<td>70.6</td>
<td>4.77</td>
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<td>$0.567\sqrt{f_c}$</td>
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<td>88.15</td>
<td>4.46</td>
<td>5.059</td>
<td>$0.475\sqrt{f_c}$</td>
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<td>0.4</td>
<td>80.26</td>
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<td>5.133</td>
<td>$0.459\sqrt{f_c}$</td>
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<tr>
<td>6</td>
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<td>0.4</td>
<td>70.85</td>
<td>3.46</td>
<td>4.883</td>
<td>$0.411\sqrt{f_c}$</td>
</tr>
</tbody>
</table>

4.3. **Shear Strength**

The shear strength results are presented in figure 3. The reduction of shear strength of HPC, compared to the conventional concrete, may be related to the brittle behavior of HPC (ACI report 363R-27, 1997). The shear strength values obtained are presented in table 3.
Table 3 Compressive and shear strength for different % of SF

<table>
<thead>
<tr>
<th>Mix No</th>
<th>S. F %</th>
<th>W/C ratio</th>
<th>Compressive Strength $f_c$ (Mpa)</th>
<th>Shear Strength $f_{sh}$ (Mpa)</th>
<th>% $f_{sh}$/$f_c$</th>
<th>Factor of $f_{sh}$ to $f_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.4</td>
<td>48.86</td>
<td>4.87</td>
<td>9.967</td>
<td>0.69$\sqrt{f_c}$</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>0.4</td>
<td>66.35</td>
<td>4.13</td>
<td>6.225</td>
<td>0.50$\sqrt{f_c}$</td>
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<tr>
<td>3</td>
<td>5</td>
<td>0.4</td>
<td>70.6</td>
<td>3.76</td>
<td>5.326</td>
<td>0.45$\sqrt{f_c}$</td>
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<tr>
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<td>7.5</td>
<td>0.4</td>
<td>88.15</td>
<td>3.37</td>
<td>3.823</td>
<td>0.36$\sqrt{f_c}$</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>0.4</td>
<td>80.26</td>
<td>3.01</td>
<td>3.750</td>
<td>0.34$\sqrt{f_c}$</td>
</tr>
<tr>
<td>6</td>
<td>12.5</td>
<td>0.4</td>
<td>70.85</td>
<td>2.81</td>
<td>3.966</td>
<td>0.33$\sqrt{f_c}$</td>
</tr>
</tbody>
</table>

5. CONCLUSION

Based on the experimental study investigating the use of SF in HPC, the following conclusions can be drawn:

1. The use of SF in HPC, up to a certain percentage of additions, has a significant effect on enhancing the compressive strength of concrete.
2. The use of SF in HPC is reported to decrease the tensile and shear strength of concrete.
3. The optimum percentage of SF used in HPC and resulting in the maximum compressive strength, was found to be at 7.5%.
4. It is necessary to achieve a better understanding for the use of SF with HPC in concrete applications.
5. Mechanical properties of HPC are sensitive to the type of additive materials used and curing techniques adopted.
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High Performance Foam Concrete Produced in Turbulence Mixers

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Abstract. Foam concrete is modern building material which combines satisfactory mechanical and heat insulating properties. The developed technology of foam concrete allows obtaining material, which is characterised by wide range of density, without additional thermal treatment. The aim of this research is to elaborate foam concrete compositions with the best rate between mechanical strength and volume weight by controlling technological parameters of producing process as well as adding non-metallic fibers and using pozzolanic admixtures, such as silica fume. In the framework of research basic properties of foam concrete, such as water absorption, compressive strength, density, have been investigated for different mix compositions. The obtained properties of foam concrete have been compared with properties of traditionally used autoclaved aerated gas silicate concrete. Experimental results showed that use of intensive mixers makes it possible to achieve stable mix and high performance properties of hardened material. Density of the material ranged from 400 to 1100 kg/m³ and compressive strength 1–10 MPa. Use of pozzolanic micro fillers allows to improve mix stability, compressive strength and decrease value of water absorption of foam concrete.

1. INTRODUCTION. FOAM CONCRETE AND GAS SILICATE CONCRETE

Cellular concrete is modern and popular building material which combines satisfactory constructive properties and low density. Two types of cellular concrete are used in modern building construction, namely autoclaved gas silicate and foam concrete. Traditionally, gas silicate concrete is used for prefabrication of small-sized constructive elements (for example wall blocs). Production of this material requires specific equipment, such as high pressure steam chambers. Technology of foam concrete is more simple and flexible comparing to traditional autoclaved gas silicate concrete. Production process includes preparation of foamed cement mortar mix, casting and hardening in normal conditions, the way it is done when making traditional concrete. Such technology makes it possible to produce monolithic constructive elements in-city. Foam concrete with various densities ranging from 200 kg/m³ to 1600 kg/m³ can be produced depending on its application.

There are two basic problems which restrict wide application of foam concrete nowadays. The first is increased drying shrinkage and risk of cracking. The second problem is decreased strength, comparing to autoclaved gas silicate concrete with the same density. Therefore, development of foam concrete technology should be focused on achieving the best relation between mechanical strength of the material and unit weight (or density). It makes possible to provide higher strength in the same density or reduction of density keeping the same strength. The solution for increasing the competitiveness of the foam concrete is the use of modern concrete technology developments. Two basic directions of increasing effectiveness of foam concrete may be noted, such as modifying concrete mix with active admixtures and the use of intensive mixing technology.

Technology of the traditional foam concrete includes producing of stabile foam and mixing it with the prepared cement slurry using conventional (low speed) mixers. It must be noted that
this type of mixers are not capable of providing homogenous mix and does not allow to split agglomerated cement and fine aggregate particles, as a result, cement is used inefficiently.

The modern technology of foam concrete involves the use intensive high-speed mixers with effects of turbulence and cavitation. The phenomenon of turbulence is defined by Oxford advanced learner's dictionary as violent or unsteady movement of air or water, or of some other fluid. It may include chaotic movement and rapid variation of pressure and flow velocity in foam concrete mix. The use of intensive mixing technology allows to improve homogeneity of the fresh mix and to increase physical and mechanical properties of hardened material. In practice intensive mixing technology has been performed in turbulence mixers equipped with a high speed mixing blades. In the case of ultrahigh mixing rate (up to 15 m/s and more), the effect of cavitation may take place.

Hydrodynamic cavitation is associated with creation of vapor cavities, and the increase of the liquid pressure results in the cavitation bubble collapse (figure 1). Cavitation bubbles collapse violently generating either the micro-jets or shock pressure waves featured by high velocities, pressures and temperatures in the small volume. Commonly, cavitation is harmful phenomenon causing damaging the steel parts of mechanisms. With regard to the mix preparation, the effect of cavitation could be effectively used for preparation of homogeneous mix and considerable increase of reactivity of binding agent.

The additional way to improve microstructure and properties of cement compositions is use of active admixtures and micro-fillers. Technology of modern high performance concrete provides for use of highly reactive pozzolanic admixtures, such as silica fume, metakaolin, fly ash etc. Pozzolanic micro fillers are micro and nano-sized amorphous non-crystalline silicate or alumino-silicate particles. Being added in cement mix, they improve workability of the mix, react with calcium hydroxide (product of cement hydration) and form additional minerals of calcium silicate hydrates (C-S-H), which provides more dense packing of cement matrix. Some researchers has established positive role of nano-silica in improving strength of lightweight concrete.

Technology of foam concrete allows adding micro fibers as a component of mix. Micro fibers usually are used in foam concrete mixes in order to improve bending strength, ductility and increase safety factor of material. Polypropylene fibers usually are used, but some studies proved that specialtypes of glass, PVA fiber and other types of polymer fibers are more effective. Researchers proved that adding supplementary cementitious materials, such as carbonate-containing salt waste, added together with polypropylene fibers also can improve physical and mechanical properties of aerated concrete.

The aim of this study is to evaluate foam concrete mix containing pozzolanic admixtures, non-metallic fibers which are produced applying intensive mixing technology with effect of cavitation.

2. MATERIALS AND METHODS

Experimental foam concrete mixes were produced in a foam concrete plant using different types of mixers: high speed turbulence mixers with effect of cavitation and conventional low speed
mixture. Raw materials were dosed by weight with accuracy ± 2%. In accordance with producer information, the rotation speed of mixing tools > 15 m/s can initiate the effect of cavitation.

- The following raw components were used for experimentally prepared foam concrete mixes:
- Normal type Portland Cement CEM I 42.5 N – main binding agent.
- Natural washed sand with fractions 0/1 mm – filling component, which also promotes foam formation during mixing. Rate of sand to cement is close to 1 by weight.
- Silica fume grade 920D (by Elkem) in amount of 2-3% by cement mass – active pozzolanic admixture and supplementary cementing material, usually used in high performance cement compositions. It is characterized by very fine particles (in the range of 1 μm up to 15 nm) and very high specific surface area.
- Synthetic foaming agent is added during mixing in amount of 0.2 % by cement weight.
- Polypropylene fibers (chopped in length 12 mm) and carbon fibers (diameter 7 μm, length 12 mm).

Three experimental moderate density mixes are produced in high speed turbulence mixer (REF, SF, SF/C). Two experimental mixes are produced in another plant (PB400 and PB700). Low density foam concrete PB400 is produced in ultrahigh turbulence mixer with effect of cavitation. Moderate density foam concrete PB700 is produced using traditional foam concrete technology, which involves preparing foam, cement mortar and mixing them. Two types of commercially available gas silicate concrete are used as reference samples (GB400 with density 400 kg/m³ and GB550 with density 550 kg/m³). Designations of the experimental mixes are presented in the table 1.

Compressive strength was determined in hydraulic testing machine, accuracy ± 1%.
Density was determined by measuring cubic samples with accuracy ± 1 mm and weighting with accuracy ± 1 g.
Capillary water absorption was determined as mass of water capillary penetrated from exposed surface. After drying sides of the specimens were sealed by resin and then samples were immersed in water in a depth of 10 mm to cover only one surface. The mass of water penetrating in the specimens was controlled periodically.

<table>
<thead>
<tr>
<th>Table 1. Designation of experimental mixes.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Portland Cement CEM I 42.5N</td>
</tr>
<tr>
<td>Sand 0/2 mm</td>
</tr>
<tr>
<td>Water</td>
</tr>
<tr>
<td>Foaming agent</td>
</tr>
<tr>
<td>Silica Fume</td>
</tr>
<tr>
<td>PP fiber</td>
</tr>
<tr>
<td>Carbon fiber</td>
</tr>
</tbody>
</table>

3. RESULTS AND DISCUSSIONS

Results of capillary water absorption are presented in the figure 2. Two samples of traditionally used autoclaved aerated gas silicate concrete (GB400 and GB550) are used as reference samples with in order to compare the results. Summarizing the experimental data it may be concluded that the reference samples of gas silicate autoclaved concrete has the highest values of absorption and high rate of water absorption. Foam concrete sample has two times lower water absorption capacity in the case of comparing samples with the same density (PB400 and
GB400). This phenomenon can be explained by type of pore system: non-autoclaved foam concrete is characterized by closed pore system but autoclaved gas silicate has open porosity caused by effect of high temperature and pressure during thermal treatment.

Fig 2. Comparing capillary water absorption for different densities of foam concrete and commercially available gas silicate concrete.

Results of compressive strength and density are summarized in the figure 3. Analysing the testing results of high density foam concrete (mix compositions REF, SF and SF/C), it can be affirmed that adding silica fume gives strength increase up to 9% comparing to reference mix, when density remain the same. Improving effect can be explained by formation of more compact microstructure of cement matrix thanks to filling gaps between cement grains and pozzolanic reactions. Visual evaluation of the produced fresh mixes also proved beneficial role of silica fume pozzolanic admixtures on the mix workability and producing foamed mix, which is more stable during transportation, filling and hardening processes.
Adding carbon micro-fibers together with silica fume (SF/C) increases compressive strength values up to 45\% comparing to REF mix. At the same time, density of SF/C mix also is increased by 12\%. It must be noted that objective evaluation of foamed concrete properties should include simultaneous estimation of strength and density. One way of assessing these both properties is the use the coefficient of constructive quality or specific strength as relation of strength to density:

$$K = \frac{R}{\rho} \text{ [N/mm}^2\text{]} / \text{[kg/dm}^3\text{]}.$$  

Mix SF/C has the highest value of specific strength (13.4 MPa/(kg/dm$^3$)) but REF mix and SF mix – 10.4 and 11.3 correspondingly.

Microstructures of high density foam concrete compositions REF, SF and SF/CF produced by cavitation mixing technology are investigated by scanning electron microscope (SEM). The obtained images (figures 4–6) show quite homogeneous microstructure and multi-sized air voids distribution ranged 10 – 500 $\mu$m in diameter. Comparing the REF (figure 4) and SF (figure 5) samples, no significant differences are found but SF mix is characterized by more clear shapes of air voids and more defined borders between air voids.

Considerably increased strength and density of mix modified with carbon fibers and silica fume (SF/C) may be explained by strengthening effect based on advanced mechanical properties of carbon fibers. It must be noted, that carbon fibers has 4 times more tensile strength and 40 times more Young’s modulus compared to the traditionally used polypropylene fibers. Additionally, small diameter of carbon fiber (7 $\mu$m) ensures higher amount of fibers in volume unit of the material. It is evident, that the carbon fibers are not flexible (see figure 6); it crosses cement matrix as well as air cells and in this way can reduce the amount of air cells and increases the density of material.

Analysing the results of low and average density concrete, it may be claimed that autoclaved gas silicate concrete has approximately two times higher strength, comparing to the foamed concrete for materials having the same density. For example, compressive strength of the sample GB400 is two times higher than the strength of foam concrete mix PB400. It means that challenges for the future development of technology and composition of foam concrete should be focused on achieving higher strength level of autoclaved gas silicate concrete.
Fig 4. Microstructure of reference mix foam concrete (REF).

Fig 5. Microstructure of foam concrete sample with silica fume (SF).

Fig 6. Microstructure of mix with silica fume and carbon micro-fiber (SF/C). Carbon fibers crossed cement matrix and air cells.
The problem of improving properties of foam concrete is related to finding an appropriate approach for evaluating the efficiency of the given composition. The different behavior of foam concrete and traditional concrete during experimental work must be noted. In the case of foam concrete changing one parameter usually is related to changes in density and strength. Considerable effect of porosity on the strength of foamed concrete is emphasized. Specific strength (value of relation strength/density) may be used as a simple criterion to compare different mix compositions.

Taking into account the results of strength and densities, the corresponding correlation diagram is built up for different foam concrete compositions (figure 7). Normally, the relation close to parabolic is obtained. The authors suggest to consider as an effective compositions, which are situated above the averaged curves, and to consider as non-effective compositions, which are situated below the averaged curves.

Consequently, mix compositions REF, SF, SF/C and PB400 may be considered as effective, but composition PB600 (produced using the ordinary mixer) - as non-effective.

Therefore, in the further development of foam concrete technology attention should be focused on use of turbulence mixers with cavitation effect and simultaneous use of reactive pozzolanic admixtures, shrinkage reducing components and other possible improving components, such as carbon nano-tubes. With regard to the mixing process, mixing with cavitation effect allows to achieve more homogenous mix and destroys cement and silica fume agglomerated particles.

\[
y = 0.0000001179x^{2.5698852669} \\
R^2 = 0.9294535114
\]

![Figure 7. Relation between material density (in air dry condition) and compressive strength.](image)

4. CONCLUSIONS

- Foam concrete is material of multi-functional application. Technology of foam concrete allows obtaining wide range of density without additional thermal treatment.
- Foam concrete produced by turbulence mixing technology has 2 times lower values of capillary water absorption, comparing to commercially used gas silicate concrete. It may be explained by closed pore system of foam concrete and microstructure of pore system.
• Insufficient strength is basic lack of traditionally mixed foam concrete. The use of turbulence mixers with effect of cavitation makes possible to improve physical, mechanical properties of foam concrete and mix homogeneity.
• Adding silica fume gives strength increase up to 9% comparing to reference mix, when density remain the same. Silica fume also allows to obtain more workable and stable mix and decrease capillary water absorption by 20%.
• Adding carbon micro-fibers increase strength and density. Simultaneous use of cavitation mixer, adding silica fume and non-metallic fibers can be regarded as an effective way for achieving high performance foam concrete characterized by the best relation between strength and density.

5. ACKNOWLEDGEMENT

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REFERENCES

Effect of fine clay brick waste on the properties of self compacting concrete.

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**Abstract.** Waste is generated in the production of clay bricks leading to negative impacts on the environment. The waste generated in the form of fine ground material can however be used as a partial replacement to cement in the production of self-compacting concrete, resulting also to improved characteristics of the concrete. The aim of paper is to investigate the influence of ground clay brick as a cement replacement, on the fresh and hardened properties of self compacting concrete. For this purpose, several mixes of self-compacting concrete were prepared and tested. The properties of fresh self compacting concrete were determined through the slump flow method, visual assessment of stability, T50 time, V-funnel method, L-box method and J-ring method. In addition the mechanical properties of hardened self compacting concrete, in particular the compressive strength and splitting tensile strength after 28 days were determined. The results showed an improvement of concrete flow and reduction in the segregation of constituent materials. A significant relationship between compressive strength and splitting tensile strength was also noted.

**Keywords:** clay brick waste, self compacting concrete, compressive & tensile strength

1 INTRODUCTION

Self compacting concrete (SCC) is an innovative concrete with significant improved characteristics when compared to conventional vibrated concrete (VC). Many studies have been conducted on the effects of the addition of mineral admixtures and industrial by-products or waste materials on the properties of SCC. SCC is characterized by an increased powder content when compared to conventional vibrated concrete and mineral additions incorporated in SCC as part of the powder content of the material, are reported to lead to significant advantages. The recycling of waste and industrial by products from different industrial sectors leads to environmental and economic benefits. This results from the production and disposal of less waste and also in a reduced consumption of raw materials. Clay brick waste originating from the demolition of existing buildings leads to large volumes of waste to be disposed of with resulting negative impacts on the environment. On the other hand, clay brick waste is increasingly seen as a valuable resource if transformed into a product and an engineering material for the construction industry. The waste generated from the brick production can be classified as: brick dust, deformed bricks, over burnt bricks and broken bricks. This work was aimed at investigating the properties of SCC produced using the ground clay brick (GCB) as a partially cement replacement. Several mixes of SCC with different percentages of GCB, (0 to 50%), were produced. Fresh and hardened concrete tests were conducted. The results showed that GCB can be used as a partial cement replacement to produce SCC. The chemical analysis of GCB showed that it contains a significant percentage of silica and Alumina. Hence the material can potentially be categorized in the alumino-silicate category.
Mixes with GCB showed a good rheological performance. When analysed up to 28 days, SCC containing GCB as a cement replacement did not present mechanical properties when compared to SCC without GCB. The relationship between compressive strength and split tensile strength was noted to follow power’s law.

2 MATERIALS

2.1. Cement

The ordinary Portland cement type 42.5N conforming to the Libyan standards 340-2009 was used to prepare the concrete specimens.

2.2. Water

A clean drinking quality water was used as mixing water for the production of concrete specimens.

2.3. Fine Aggregate

Two types of fine aggregate were used in this study; the local natural sand and the ground sand. The Specific Gravity, the moisture absorption and the Unit Wight were reported to be 2.5, 1.2% and 1720 kg/m³ respectively. Figure 1 shows the sieve analysis of the fine aggregate.

![Sieve analysis for mixed fine aggregate](image)

2.4. Coarse Aggregate

Two different sizes of coarse aggregate were adopted, 10mm and 14 mm. Properties of the coarse aggregate used are given in table 1. Figure 2 shows the sieve analysis of the coarse aggregate.
Table 1. Properties of the used course aggregate.

<table>
<thead>
<tr>
<th>Test</th>
<th>Size 10 mm</th>
<th>Size 14 mm</th>
<th>Specification Limits</th>
<th>Reference Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>2.6</td>
<td>2.6</td>
<td>2.5 – 2.7</td>
<td>Libyan Standards 256/2006</td>
</tr>
<tr>
<td>Absorption%</td>
<td>1.3</td>
<td>1.1</td>
<td>≤ 3%</td>
<td>Libyan Standards 256/2006</td>
</tr>
<tr>
<td>Los Angeles %</td>
<td>---</td>
<td>17.24</td>
<td>≤ 45%</td>
<td>Libcan Standards 256/2006</td>
</tr>
<tr>
<td>Crushing Value %</td>
<td>20.26</td>
<td>20.26</td>
<td>≤ 45%</td>
<td>Libyan Standards 256/2006</td>
</tr>
<tr>
<td>Unit Wight(kg/m³)</td>
<td>1720</td>
<td>1720</td>
<td>1400-1800</td>
<td>ASTM C29/129M:2007</td>
</tr>
<tr>
<td>Flakiness %</td>
<td>25.6</td>
<td>15.4</td>
<td>≤ 40%</td>
<td>BS 812</td>
</tr>
<tr>
<td>Elongation %</td>
<td>33.1</td>
<td>19.5</td>
<td>≤ 40%</td>
<td>BS 812</td>
</tr>
</tbody>
</table>

2.5. Clay Powder

The brick waste product used for this work was collected from the Al-Swani clay brick factory, located 25 km south of Tripoli. The brick waste was crushed and ground to a fineness of about 75μm and used as a ground clay brick. Both X-ray diffraction analysis (XRD) and differential thermal chemical analysis (DTA) were performed on the ground clay brick in the Industrial Research Center laboratories – Tripoli, Libya. Furthermore X-ray diffraction analysis (XRD) and Fourier transform Infrared Spectroscopy (FTIR) were carried out at the University of Malta. The chemical composition of ground clay brick is given in table 2.

Table 2. Chemical composition of ground clay brick.

<table>
<thead>
<tr>
<th>Property</th>
<th>% Content</th>
<th>Standard Limits (ASTM C618-89a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicon Dioxide (SiO₂)</td>
<td>76.9</td>
<td>≥ 40%</td>
</tr>
<tr>
<td>Aluminum Oxide (Al₂O₃)</td>
<td>10.0</td>
<td>-----</td>
</tr>
<tr>
<td>Iron Oxide (Fe₂O₃)</td>
<td>5.6</td>
<td>-----</td>
</tr>
<tr>
<td>(SiO₂)+ (Fe₂O₃)+ (Al₂O₃)</td>
<td>92.5</td>
<td>≥ 70%</td>
</tr>
<tr>
<td>Sulphur Trioxide (SO₃)</td>
<td>0.28</td>
<td>≤ 3%</td>
</tr>
<tr>
<td>Calcium Oxide (CaO)</td>
<td>1.96</td>
<td>≤ 10%</td>
</tr>
<tr>
<td>Sodium Oxide (Na₂O)</td>
<td>0.32</td>
<td>-----</td>
</tr>
<tr>
<td>Potassium Oxide (K₂O)</td>
<td>2.94</td>
<td>-----</td>
</tr>
<tr>
<td>Magnesium Oxide (MgO)</td>
<td>0.28</td>
<td>-----</td>
</tr>
<tr>
<td>Loss of ignition (LOI)</td>
<td>0.69</td>
<td>≤ 5%</td>
</tr>
</tbody>
</table>
2.6. Chemical admixtures

Sika Viscocrete Tempo 12 was used in the production of SCC mixes.

3 EXPERIMENTAL WORK

An experimental program to investigate the effect of ground clay brick on SCC properties was conducted. Eleven replacement levels by weight of cement, ranging from 0% to 50%, of GCB to Portland cement were tested. The fresh concrete properties investigated were, Slump flow + T50 test, V-funnel test, J-ring test and L-box test. These tests on fresh concrete were carried out as per EFNARC guidelines. The compressive strength and splitting tensile strength of SCC after 28 days curing were also investigated. For all SCC mixes, the total amount of binder (cement + GCB), the total amount of coarse aggregate, the total amount of fine aggregate and the total amount of the chemical admixture were kept constant. In order to keep the mix consistency, the amount of mixing water was slightly increased when it was necessary. The details of the SCC mixes are given in table 3.

Table 3. Details of constituents for the SCC mixes.

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Total Powder kg/m³</th>
<th>Cement kg/m³</th>
<th>GCB kg/m³</th>
<th>GCB %</th>
<th>Mixing water Total CA (10+14mm) kg</th>
<th>Total FA kg</th>
<th>Viscocrete liter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>500</td>
<td>500</td>
<td>0</td>
<td>0</td>
<td>179.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>2</td>
<td>500</td>
<td>475</td>
<td>25</td>
<td>5</td>
<td>179.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>3</td>
<td>500</td>
<td>450</td>
<td>50</td>
<td>10</td>
<td>181.7</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>4</td>
<td>500</td>
<td>425</td>
<td>75</td>
<td>15</td>
<td>184.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>5</td>
<td>500</td>
<td>400</td>
<td>100</td>
<td>20</td>
<td>186.7</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>6</td>
<td>500</td>
<td>375</td>
<td>125</td>
<td>25</td>
<td>191.7</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>7</td>
<td>500</td>
<td>350</td>
<td>150</td>
<td>30</td>
<td>199.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>8</td>
<td>500</td>
<td>325</td>
<td>175</td>
<td>35</td>
<td>204.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>9</td>
<td>500</td>
<td>300</td>
<td>200</td>
<td>40</td>
<td>204.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>10</td>
<td>500</td>
<td>275</td>
<td>225</td>
<td>45</td>
<td>211.7</td>
<td>911.6</td>
<td>808.4</td>
</tr>
<tr>
<td>11</td>
<td>500</td>
<td>250</td>
<td>250</td>
<td>50</td>
<td>219.2</td>
<td>911.6</td>
<td>808.4</td>
</tr>
</tbody>
</table>

3.1. Preparation and casting of test specimens

The following mixing sequence was considered, after several trials intended to optimize the mix in view of workability requirements. All the ingredients were first mixed in dry condition in the concrete mixer for one minute. Then the amount of mixing water was added, gradually, to the dry mix and the material was mixed thoroughly for one minute. The assigned portion of the viscocrete was poured into the mixer and the concrete was mixed for three minutes.

After the mixing procedure was completed, deformability and viscosity of fresh concrete are evaluated through the measurement of slump flow time and diameter, J-Ring test, L-Box ratio test and V-funnel flow time test.

From each GCB percent mix, nine SCC cylinders 150 mm in diameter and 300 mm in height were cast. All specimens were cast in one layer without any compaction. After demoulding, all specimens were placed in a curing tank at room temperature. The specimen were tested for the compressive strength and for the splitting tensile strength.

4 RESULTS AND DISCUSSION

4.1. Characteristics of Ground Clay Brick

Due to its nature GCB can be defined as calcined natural pozzolan. According to (ASTM C618-89a) a calcined natural pozzolan for use as a supplementary cementitious material, or mineral admixture as referred in the standard, in concrete is required to have minimum 70% of...
[silicon dioxide (SiO2) and aluminum oxide (Al2O3) and iron oxide (Fe2O3)], and maximum 4% of sulfur trioxide (SO3), and loss on ignition needs to reach a maximum of 10%. The ground clay brick used in this study satisfies the chemical requirements of ASTM C618-89a. Its required total oxide content was 92.5%, sulphur trioxide content was 0.28% and the loss on ignition was 0.69%.

4.2. Fresh concrete properties

The details of the fresh properties of SCC mixes are given in table 4. The required W/P (Water/Powder) ratio had to be increased with increasing GCB content. One of the major reasons for this is the water absorption of brick powder which in itself is a very high. This in turn causes an increase in water absorption of the concrete [2]. For the slump flow, one can see that all mixes were within the specifications. The same is observed in the case of the L-Box Test and the V-Funnel Test. The ratio of H2/H1 for L-Box ranges between 0.83 and 0.97. T50 & J-Ring results are reported as minimal to noticeable blocking as per ASTM C29/M129, 2007 [3], and that is probably due to the sequence in testing and delay in testing for some samples.

Table 4. Fresh Properties of SCC mixes.

<table>
<thead>
<tr>
<th>Mix No</th>
<th>GCB %</th>
<th>W/P ratio</th>
<th>Slump flow (mm)</th>
<th>T50 (sec)</th>
<th>J-Ring test (mm)</th>
<th>L-Box test (H2/H1)</th>
<th>V-funnel Test (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.358</td>
<td>780</td>
<td>2.3</td>
<td>28</td>
<td>0.95</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>0.358</td>
<td>760</td>
<td>2.7</td>
<td>25</td>
<td>0.85</td>
<td>7</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>0.363</td>
<td>680</td>
<td>3</td>
<td>20</td>
<td>0.846</td>
<td>6.5</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>0.368</td>
<td>750</td>
<td>2.1</td>
<td>26</td>
<td>0.94</td>
<td>6.3</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>0.373</td>
<td>710</td>
<td>3.2</td>
<td>5</td>
<td>0.93</td>
<td>6.1</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>0.383</td>
<td>745</td>
<td>2.9</td>
<td>7.5</td>
<td>0.93</td>
<td>7</td>
</tr>
<tr>
<td>7</td>
<td>30</td>
<td>0.398</td>
<td>785</td>
<td>2.4</td>
<td>22.5</td>
<td>0.95</td>
<td>5.9</td>
</tr>
<tr>
<td>8</td>
<td>35</td>
<td>0.408</td>
<td>790</td>
<td>1.3</td>
<td>30</td>
<td>0.93</td>
<td>7.2</td>
</tr>
<tr>
<td>9</td>
<td>40</td>
<td>0.408</td>
<td>780</td>
<td>5.6</td>
<td>35</td>
<td>0.97</td>
<td>6.9</td>
</tr>
<tr>
<td>10</td>
<td>45</td>
<td>0.423</td>
<td>756</td>
<td>2.8</td>
<td>10</td>
<td>0.93</td>
<td>8.2</td>
</tr>
<tr>
<td>11</td>
<td>50</td>
<td>0.438</td>
<td>690</td>
<td>6.9</td>
<td>42</td>
<td>0.83</td>
<td>7.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EFNARC Specifications</td>
<td>---</td>
<td>---</td>
<td>650 - 800</td>
<td>2 – 5s</td>
<td>0 - 10</td>
<td>0.8– 1.0</td>
<td>6 – 12s</td>
</tr>
</tbody>
</table>

4.3. Hardened concrete properties

The results of 28-day compressive strength and splitting tensile strength for the examined SCC specimens are presented in table 5. The results indicate that the compressive strength and split tensile strength of SCC samples containing GCB are less than that of the control mix without GCB. This behavior is well expected since the size of the GCB particles, 75μm, is quite large to fully hydrate in a reasonable amount of time. In this case the GCB would act as a filler material rather than a reactive pozzolanic material. Another reason of the strength reduction is the extra water needed for GCB.
Table 5. Mechanical Properties of SCC Samples.

<table>
<thead>
<tr>
<th>Mix No</th>
<th>GCB %</th>
<th>W/P ratio</th>
<th>Compressive Strength $f_c$ (Mpa)</th>
<th>Splitting Tensile Strength $f_{spt}$ (Mpa)</th>
<th>$f_{spt} / f_c$</th>
<th>Factor of $f_{spt}$ to $f_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.358</td>
<td>52.7</td>
<td>5.10</td>
<td>9.67</td>
<td>$0.70\sqrt{f_c}$</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>0.358</td>
<td>49.25</td>
<td>4.95</td>
<td>10.05</td>
<td>$0.70\sqrt{f_c}$</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>0.363</td>
<td>46.3</td>
<td>4.64</td>
<td>10.02</td>
<td>$0.68\sqrt{f_c}$</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>0.368</td>
<td>40.4</td>
<td>3.91</td>
<td>9.68</td>
<td>$0.61\sqrt{f_c}$</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>0.373</td>
<td>38.5</td>
<td>3.64</td>
<td>9.45</td>
<td>$0.58\sqrt{f_c}$</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>0.383</td>
<td>37.4</td>
<td>3.68</td>
<td>9.84</td>
<td>$0.60\sqrt{f_c}$</td>
</tr>
<tr>
<td>7</td>
<td>30</td>
<td>0.398</td>
<td>32.95</td>
<td>3.39</td>
<td>10.29</td>
<td>$0.59\sqrt{f_c}$</td>
</tr>
<tr>
<td>8</td>
<td>35</td>
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<td>3.25</td>
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<td>$0.57\sqrt{f_c}$</td>
</tr>
<tr>
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<td>40</td>
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<td>33.2</td>
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</tr>
<tr>
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<td>26.6</td>
<td>3.02</td>
<td>11.35</td>
<td>$0.58\sqrt{f_c}$</td>
</tr>
<tr>
<td>11</td>
<td>50</td>
<td>0.438</td>
<td>25.65</td>
<td>2.82</td>
<td>11</td>
<td>$0.55\sqrt{f_c}$</td>
</tr>
</tbody>
</table>

4.3.1 Relation Between Compressive & Tensile Strength.

Compressive strength ($f_c$) and splitting tensile strength ($f_{spt}$) are two significant indexes in the design of concrete structure. Ideally, the splitting tensile strength is measured directly on concrete samples under uniform stresses. However, this is not always easy from an experimental point of view. To avoid the demanding and time-consuming direct measurements of the splitting tensile strength, engineers and researchers have tried to predict the splitting tensile using theoretical and empirical approaches based on compressive strength. Generally, the tensile strength of concrete is often assumed to be proportional to the square root of its compressive strength. There is no agreement on the precise form of the relationship between ($f_{spt}$) and ($f_c$). However, National building Codes propose various formulas for prediction tensile strength of concrete from compressive strength.

Based on the results in table 5, a mathematical equation (equation 1), with a coefficient of variation $R^2=0.9703$, was obtained expressing the relationship between compressive strength and split tensile strength for SCC mixes with GCB. Figure 3 shows the relationship between $f_c$ and $f_{spt}$. The mathematical relationship indicates that the relationship follows Power’s Law with the expression ( $f_{spt} \approx 10\%$ of $f_c$ ).

$$f_{spt} = 0.0875f_c + 0.4809 \quad (1)$$

![Fig 3. Relationship between tensile and compressive strength](image-url)
Table 5 shows also that the splitting tensile strength is in the range of $0.55\sqrt{f_c}$ to $0.70\sqrt{f_c}$ of the compressive strength with an average of $0.61\sqrt{f_c}$. The equation is in a good agreement with the relationship provided by the ACI code, $f_{sp} = (0.4 - 0.7)\sqrt{f_c}$.

5 CONCLUSION

Based on the experimental study investigating the use of ground clay brick in self compacting concrete, the following conclusions can be drawn:

4. The chemical analysis of the waste crushed bricks showed that it contains a significant percentage of silica and Alumina. Hence it is categorized as alumino-silicate.
5. Clay brick waste can be recycled as a cement replacement material in SCC where large amount of powder is used when compared to conventional concrete.
6. The use of GCB as a cement replacement material in SCC improves the rheological performance (workability and stability of the mixture).
7. Up to 28 days, SCC with GCB as a cement replacement did not present improved mechanical properties when compared to SCC without GCB.
8. The relationship between compressive strength and split tensile strength follows power’s law.
9. It is necessary to carry out further investigation to better understand the potential use of ground clay brick as a cement replacement in SCC applications.

REFERENCES

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Possibilities of Redevelopment of Embankment Dams by Suitable Grouting Mixtures

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Abstract. Suitable material composition of grout is important for avoiding or reduction leakage of dams. Clay-cement mixture is usually chosen for remediation of embankment dams. Illite is usually suitable for its preparation. Clay-cement mixtures are used because they are better and have a better tendency to penetrate into incoherent sediments. To reduce costs the use of power industry by-products and secondary raw materials is required. Above all, it is fly ash. It has been found, that the use of fly ash leads to improvement of consistency of the fresh mixture, reducing the water-cement ratio and shrinkage of the mixture. The goal was to find the optimal selection of suitable materials for remediation of embankment dams.

1. INTRODUCTION

The paper with the suitability of material composition and technology of remediation of embankment dams for in order to reduce or prevent leakages. Embankment dams are among the simplest types of dams. They are the most frequent type in the Czech Republic. In terms of the choice of technology, it is appropriate to choose one whose technical implementation of remediation does not cause further damage to the dam. Given the fact that they are relatively small pieces of water infrastructure, the optimal remedial technology is chosen to be traditional injection.

Due to uneven subsidence, tension cracks may occur in the core of the dam and cause leaks. Leaking embankment dams can have devastating consequences. For this reason, the dams must be examined and meticulously secured. To reconstruct a dam would be demanding in terms of time, finances and technology; for this reason it is necessary to design an optimal technology of remediation.

Clay-cement grouts are some of the most suitable mixtures for injection grouting. Water permeates through clay soils very slowly. Thus, clay is chosen as the main grouting material. The appropriate choice of materials is an important aspect during mixture design. The main function of the materials is to contribute to improving water tightness.

The use of secondary products from the power industry is expected in grouting. These materials are cheaper and have such properties as to be a feasible substitute for the main raw material, i.e. clay.

Optimizing of consumption of construction materials aimed at reducing the use of primary non-renewable raw materials is one of the basic requirements for development of new remediation mixtures. It is about respecting the requirements of sustainable construction, i.e. environmental, economic and social requirements and criteria. Environmental and economic
benefits primarily related to the decrease in the consumption of primary non-renewable raw materials, reducing the demands on the transport and manipulation of materials, with less amount of waste and materials for recycling after the life expectancy of the structure.

Clay is the primary sealing component in the mixture. Cement is used to provide high compressive and flexural strength. It also improves pumpability. Another important component are fillers which ensure the required bulk density at a given water content. Mutual compatibility of the components results in a good grouting mixture. The goal of the paper is to design an optimal grouting mixture for remedial sealing of embankment dams.

2. FUNDAMENTAL CHARACTERISTICS OF GROUTING MIXES

The most technologically and financially suitable means of dam remediation is traditional injection. This method is very common. The reason being its simplicity and effectiveness. A correctly performed grout can improve the properties of foundation soils. However, what is also important is an appropriate choice of materials suitable for the technology. The grouting mixtures used can be both only suspensions as well as stabilised cement mixtures. For the purposes of sealing, clay-cement mixtures are used.

According to, the grain size of the grouting mixture should be 3 times smaller than the pores of the body being grouted. The reason being the ease of entry into voids, caverns, etc. Larger grains may create an obstruction which prevents the mixture to penetrate into places where it is needed. According to, a correct economical design and implementation of an injection grout requires perfect knowledge of the body being grouted (Figure 1). Inserting cuff tubes during traditional injections, according to, enables an ease of pumping of a grouting mixture into alluvia and grout in short segments.

Unstable suspensions are used for sealing cohesive rocks. Excess water from these mixtures is filtered out. For non-cohesive soils, such mixtures are designed which can be pumped in the form they are made and are able to harden in that form as well. Thus, there must be no grain sedimentation during injection and curing. These mixtures are called stable. They are clay-cement and clay-lime suspensions. These mixtures have sufficient viscosity and after being pumped into pores are sufficiently resistant to wash-out. However, according to, attaining such conditions is not always simple. Binders appear to be suitable for the stabilisation of these mixtures. The approximate limits of application of grouting mixtures are in Figure 2.
Figure 2 shows that certain materials are suitable for certain technologies. These materials must meet the given requirements, especially granulometry. If an embankment dam is to be well sealed, according to, the grouting mix must be adjusted for the type of dam and be compatible with the local conditions. A mixture is always designed for the specific natural conditions. An important parameter in grouting mixture design is decantation. No decantation occurs in stable grouting mixtures.

3. METHODS

In order to design grouting mixtures properly a number of laboratory tests need to be performed. The tests determine the content of the individual components compatible with one another. The tests must be performed for each dam individually. The properties are determined both in fresh and hardened state. The properties of a mixture in fresh state are yield value, consistency, viscosity, decantation and bulk density. Properties of a mixture in hardened state are compressive strength, bulk density, contraction, permeability – i.e. filtration coefficient, ekotoxicity and leachability.

Leachability tests are one of the basic tests, creating criteria for further usability of industrial waste. When exceeding the legislative criteria, it is not a given waste material treated as secondary raw material, but as hazardous waste. The test is carried out according to EN 12457-4 (838005) Characterization of waste - Leaching.

4. GROUTED MATERIALS

The main raw material for grouting is clay. Mixture consumption during grouting is high and therefore it should be made from available materials and should meet the following requirement:

- low purchasing cost,
- good workability,
- good injectability,
- good pumpability,
- stable volume and high erosion resistance,
- adequate compressive strength.

It is also important for the mixture to have some plasticity. It is especially due to its still being able to function even during movement of subsoil. According to, it is necessary for ease of penetration of the mixture to maintain a certain relation between the grains of the environment and the mixture. According to French experts (Cambefort, Ischy, Caron), a mixture
is satisfactory if the raw materials have yield point of 60%. The suitability of the use of clay depends on its grain size, consistency limits and mineralogical composition.

4.1. Ge clay

GE clay (Grüne Erde, i.e. green earth) was chosen for the design of the grouting mixture. It is a montmorillonite-illite-kaolinite clay with substantial content of montmorillonite, illite and kaolinite. Compared with traditional bentonite, it differs primarily in the amount of fixed potassium in between the layers of montmorillonite. This reduces the sorption properties of GE clay. Despite lower content of montmorillonite, clay has good sorption properties and ability of ion exchange. Due to this, it was chosen as the basic raw material for the grouting mixture design.

4.2. Fly ash

Alternative raw materials appear to be suitable as partial substitute of clay. It is especially fly ash. Its use requires knowledge of its physical, chemical and mineralogical properties. There are two basic kinds; fly ash (characteristic mainly by β-quartz content) and FBC ash (characteristic by a higher CaO content). During the research, fly ash was included in the grouting mixture design.

4.3. Binders

For correct mixture design, its stabilisation is especially important. This can be achieved by the use of binders. In financial terms, there is an emphasis to use a low amount of stabiliser in a mixture. As opposed to cement, lime chemically attacks the clay component of soil, for which reason lime was chosen for the grouting mixture design. Stabilisation of the mixture using lime brought a slight increase in strength, permeability and erosion resistance. It also increased the volume stability of the mixture. Despite the above-mentioned advantages of clay, it must be said that lime never brings such strength as cement. However, content of cement caused a decrease of pumpability and void filling. A cement mixture reacts with the surface during injection which prevents further spread of the mixture. The choice of binders must always be assessed in terms of the requirements for the mixture. Lime was used as binder in this experiment.

5. RESULTS AND DISCUSSION

The following figures show the results of tests performed with grouting mixtures containing Ge clay, fly ash and lime. The ratio of the individual phases and their co-action have an influence on the behaviour of the mixture. During the research, a number of tests were performed, out of which the most important ones are in the charts. They are the determination of consistency by means of Marsh cone, water content and compressive strength after 28 days of curing.

Fig 3. Dependence of viscosity on water content of mixtures
Figure 3 shows water content dependence of viscosity for GE clay, fly ash and lime. The figure shows that as fly ash content increases, the amount of water necessary for the determination of consistency decreased. Thanks to its mineralogical composition, clay has a tendency to bind a greater amount of water. This showed in its consistency.

Generally speaking, fly ash improved rheological properties. However, an increased amount of fly ash resulted in higher decantation. In order to design a GE clay and fly ash based mixture, the content of fly ash must be such as not to increase decantation. The optimal fly ash content in a mixture is up to 25%.

In mixtures where lime was used as stabiliser, there was a slight deterioration of rheological properties. The overall workability of the mixture was worse. At the same time, however, there was a decrease of decantation. Lime has a tendency to immediately react upon contact with water. The amount of added lime influenced the amount of necessary water. The presence of lime increased the value of water content. On the other hand, as the amount of fly ash increased, this value decreased slightly. The optimal amount of used lime was 2%.

In mixtures where lime was used as stabiliser, there was a slight deterioration of rheological properties. The overall workability of the mixture was worse. At the same time, however, there was a decrease of decantation. Lime has a tendency to immediately react upon contact with water. The amount of added lime influenced the amount of necessary water. The presence of lime increased the value of water content. On the other hand, as the amount of fly ash increased, this value decreased slightly. The optimal amount of used lime was 2%.

Figure 4 shows test results of compressive strength for specimens after 28 days of curing. Compressive strength decreased as the amount of fly ash increased. Clay together with fly ash forms less strong bonds which results in lower strength. In terms of strength, the optimal amount of fly ash is about 25%.

It can be seen in mixtures containing lime that lime chemically attacks clay. In its presence, clay loses its properties. There was especially a decrease in strength. In this case, lime is an effective stabiliser; however, it is not a binder which would contribute to increasing strength.

Water leaches of injecting mixtures pointed to frequent problems with pH. First of all it was a mixture containing fly ash. When evaluating fulfilment of the requirements of Decree no. 294/2005 Coll., all parameters were practically fulfilled. The only above-limit was arsenic and nickel. Generally, it is favorable in terms of environmental impact of the maximum proportion of fly ash in a mixture to 50%, without admixture of a binder. These mixtures are suitable from the viewpoint of evaluation of physical-mechanical parameters.

6. CONCLUSION

The primary raw material for the remediation of embankment dams is clay. As far as mixtures are concerned, it is clay suspensions. They are mixtures made with clay, water, suitable alternative materials and other prospective additives. The mixtures pumped into dams (the pore system) have almost no strength. However, they are able to resist great water pressure. In order to design a mixture correctly, laboratory tests must be performed whose purpose is to determine the optimal content and suitability of use for each component. A partial substitute for the primary raw material, i.e. clay, are alternative raw materials. One such material is especially fly
ash. For the stabilisation of the mixture, appropriate binders may be chosen. Nevertheless, their effect in the mixture must be determined beforehand. Its determination is in accordance with the requirements the mixture must meet.

The results indicate that fly ashes reduce yield value. At the same time, however, they increase decantation and reduce the amount of water required for the mixture.Binders increase the necessary water content during consistency testing. However, they slightly improve strength and adhesion.

An important property of a mixture is its stabilisation, i.e. uniform particle distribution. The mixture must maintain this property until the hardened state. Improving stabilisation of the mixture can be done by means of binders. However, it is important to prevent particle aggregation. Together with other components, they must be uniformly scattered throughout the whole water environment. This can be achieved by intensive mixing using disintegrators. Meeting all requirements which are set for the mixture can produce a compact mixture which prevents further leakages of embankment dams.

The parameters studied from the effect on the environment were ekotoxicity and leachability. In evaluating results of analysis water leaches were detected a partially over-limit values of mixtures with a higher content of fly ash. When comparing the requirements of decree no. 296/2005 Coll., It was found above the limit values significantly less. Rating ekotoxicity grout mixture showed that the test mixture based on GE clay and fly ash satisfies the requirements of the decree in its entirety.

7. ACKNOWLEDGEMENTS

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Steel Jacketing of RC Columns: Reliability of Capacity Laws for Concrete

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**Abstract.** In the seismic assessment of framed R.C. structures reinforced by steel angles and battens (steel jacketing) the use of proper confinement models for concrete is still a main topic. Really, the capacity of reinforced concrete columns under concentric and eccentric loads strongly depends on confinement exerted by the external reinforcement. Further a proper attribution of concrete stress-strain laws allows obtaining reliable estimations of inelastic capacity of reinforced elements. Moreover the presence (or not) of connections of the angles with the end joints influences their capacity to support vertical loads. In the two cases the mechanical models have to account for buckling or frictional effects. In this frame this paper presents a selected review of literature confinement models for steel jacketed concrete elements, presenting a parametric study in which the main confinement parameters predictable by each of them are compared. The reliability of the models in object is finally tested comparing analytical predictions of the ultimate vertical capacity with experimental results coming from several literature compressive tests of reinforced column specimens.

1. INTRODUCTION

Steel Jacketing of RC columns is a strengthening system adopted to improve the deformation and strength capacity of existing buildings not seismically designed. This technique is widely used thanks to the practicality of its application to structural frames. Reliable predictions of the capacity of reinforced elements are basic for the design of this kind of interventions. For this reason the use of suitable confined concrete laws is fundamental to the accuracy of results.

One of the most common issues of this strengthening system regards the technological aspect related to the possible arrangement of the end connections. Two types of end connections of the corner steel angles are possible: steel angles well connected to the end of the columns by specific devices and steel angles not presenting mechanical connections. In this last case an interface layer of cementitious mortar is placed between the steel cage and the sides of the column. For both the cases, several studies have shown that angles provide significant contribution in terms of load carrying capacity. Despite this the current Italian Code (NTC 2008) and Eurocode 8 provide in the case of wrapping steel (with angles well connected or not) to entrust the assessment of the performance increase only to the confinement action, neglecting the direct contribution of the angles to the flexural and bearing capacity. Eurocode 4 approach considers instead the contribution of the angles only for the case of full connection, providing rules for the design and verification of the hybrid cross-section.

In the current paper, a selected review of the most popular models for confined concrete by steel jacketing available in literature is presented and discussed together with the respective
analytical formulations. Furthermore, a discussion on their reliability is carried out by means of a parametric study on the main parameters resulting from the implementation of each model. Finally the models accuracy in predicting the ultimate bearing capacity of the reinforced columns is tested against the results of real experimental tests providing final comments and revealing their limits of validity.

The parameters investigated are those which mainly affect the prediction of compressive capacity of columns. However the same parameters play a relevant role even in assessment of the cross-section ductility and definition mechanical nonlinearities in computational models. In particular, the models by Montuori and Piluso (2009), Nagaprasad et al. (2010) and Badalamenti et al. (2010) are compared. In addition to these, the capacity model provided by Italian Code (NTC 2008) is also analyzed. The comparison with the experimental tests shows the cases in which the above models are more or less reliable in predicting the ultimate strength capacity.

2. STEEL JACKETING CONFINEMENT MODELS: ANALYTICAL EXPRESSIONS AND COMPARISONS OF RESULTS

The above mentioned models are described and compared in this section. In the formulation proposed by Montuori and Piluso, the authors define a relationship for the determination of the lateral confinement pressure of columns strengthened by the internal and external reinforcement starting from model by Mander et al.. They highlighted the fact that actually in the case of cross sections reinforced by steel angles and battens, four different concrete laws should be considered to rigorously analyse the cross-section: unconfined concrete; concrete confined only by internal hoops; concrete confined only by steel angles and battens and concrete confined by internal hoops, steel angles and battens respectively. In this condition a univocal definition of the strength increment factor \( k \) results really difficult. To solve the problem the authors proposed to calculate the volumetric reinforcement ratio along \( x \) and \( y \) directions by homogenizing the area of the battens as function of the internal reinforcement using the ratio \( (f_{yb}/f_{yk}) \) of the yielding stresses of battens \( f_{yb} \) and hoops \( f_{yk} \). In this way it is possible to consider the entire cross-section as strengthened by an equivalent reinforcement. Moreover the confinement effectiveness factor \( k_e \) (that considers arching action of the confined concrete) is assumed to be almost equal to the one adopted for the case simple internal reinforcement. The strain limits for the confined concrete (strain at peak of strength \( \varepsilon_{cc}\) and ultimate strain \( \varepsilon_{cu} \)) are determined according to Mander et al. rules.

The model proposed by Nagaprasad et al., once again is an extension of Mander’s model referred to rectangular cross-sections. Differently from Montuori and Piluso the authors take into account only the confinement effect exerted by the steel angles and battens, neglecting the confinement contribution of the internal reinforcement, and proposing a methodology to determine the lateral confinement pressure along the principal directions of inertia. The latter is evaluated considering the effectively confined area in plan and elevation, assuming the arching lines of confining stresses between the steel angles. Strain limits are determined also in this case according to Mander’s model.

In Badalamenti et al. is proposed a model for square columns reinforced with steel angles and battens, arranged in such a way that the angles are not directly loaded along their vertical axes. The analytical formulation considers the confinement action provided by the reinforcement as a function of the friction coefficient between angles and concrete members. The determination of the lateral confinement pressures is carried out simplifying the actual system into a one-dimensional Winkler’s problem model considering two elastic beams (battens) on a bed of springs. The authors assume that cohesion between mortar cementitious and steel angles \( (c_0) \) is negligible and suggest a friction coefficient value \( \mu = 0.5 \). Even in this case the confined limit strains are determined according to Mander determinations.

Finally the Italian Technical Code (DM 14/01/2008) does not define a methodology for the determination of the lateral confinement pressure provided by the steel jacketing system, but provides an analytical relationship to obtain the strength of the confined concrete. It is implicitly
taken into account that the maximum lateral confinement pressure is calculated by multiplying volumetric ratio of transverse reinforcement $\rho_v$ along each of the transverse direction with the yielding stress of battens $f_{yb}$. Even for the confined strain at peak of strength $\varepsilon_{cc}$, NTC does not provide prescriptions, hence in agreement with the previously discussed models this was here evaluated by Mander's expression.

A summary of the expressions for the determination of the confinement parameters of the above discussed models is provided in Table 1. For every model Mander's relationship is also used to determine the stress-strain law.

<table>
<thead>
<tr>
<th>Models</th>
<th>Confinement pressure $f_{l,max}$</th>
<th>Effective confinement factor $k_e$</th>
<th>Strength of confined concrete $f_{cc}$</th>
<th>Strain limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montuori and Piluso (2009) [10]</td>
<td>$f_{l,i} = \rho_{ul} f_{yb}$, $f_{l,v} = \rho_{vl} f_{yb}$</td>
<td>$\left(1 - \frac{s_b}{h} - \frac{f_{cc}}{2(b-2c)}\right) \left(1 - \frac{s_b}{h} + \frac{f_{cc}}{2(h-2c)}\right)$</td>
<td>$f_{cc}$</td>
<td>$\varepsilon_c = \varepsilon_{cc} + 1.5 \left( \frac{f_{cc}}{f_{c}} - 1 \right)$</td>
</tr>
<tr>
<td>Nagaprasad et al. (2010) [11]</td>
<td>$f_{l,i} = \frac{2Lx(h-L_y)}{s_h f_{yb}} f_{yb}$, $f_{l,v} = \frac{b}{h} f_{yi}$</td>
<td>$\left(1 - \frac{b-L_x}{2b} \frac{L_x}{h} \frac{L_y}{h} \frac{1}{1-\rho_{l,eff}}\right)$</td>
<td>$f_{cc}$</td>
<td>$\varepsilon_{cc} = \varepsilon_{cc} + 1.4 \rho_{ul} f_{yb} f_{cc}$</td>
</tr>
<tr>
<td>Badalamenti et al. (2010) [3]</td>
<td>$f_{l,i} = \frac{1.33 \cdot f_{yb} b}{s_b (L_x + L_y) + s_h \cdot s_v} \left(1 - \frac{s_v}{s_h - s_v} \frac{L_x}{s_v - s_v} \right)$</td>
<td>$e^{-\frac{(\varepsilon_{cc})^2}{\sigma^2}}$</td>
<td>$f_{cc}$</td>
<td>$\varepsilon_{cc} = \varepsilon_{cc} + 3.7 (f_{l,eff})^{0.5}$</td>
</tr>
<tr>
<td>NTC 2008 [7]</td>
<td>$f_{l,max} = \rho_v f_{yb}$, $\rho_v = \frac{2(b+h)A_{bh}}{s_h f_{yb}}$</td>
<td>$\left(1 - \frac{b-2R}{3bh} \right) \left(1 - \frac{s_b}{h} + \frac{f_{cc}}{2(b-2c)}\right)$</td>
<td>$f_{cc}$</td>
<td>$\varepsilon_{cc} = \varepsilon_{cc} + \frac{0.25 \sigma^{\alpha} \rho_v f_{cc}}{f_{c}}$</td>
</tr>
</tbody>
</table>

### 2.1. Comparative analysis of the confinement parameters by the different models

With reference to the reinforced cross-sections having the characteristics reported in Fig. 1 and Table 2 is carried out a comparative analysis of the main direct and indirect confinement parameters resulting by the different models. The direct parameters investigated are the lateral confinement pressure $f_{l,eff}$, the strength of confined concrete $f_{cc}$, the strain at the peak-strength $\varepsilon_{cc}$, the ultimate strain of confined concrete $\varepsilon_{cc}$ and the strength increment factor $k=f_{cc}/f_{c}$. The latter are reported as function of the dimensionless spacing ratio ($s_b/b$), $s_b$ and $b$ being the battens interaxis and the largest dimension of cross-section respectively. The comparisons are shown in Figs. 2-3.
Fig 1. Arrangement of internal and external reinforcement on the columns.

Table 2. Section geometrical and mechanical properties of the reference cross-sections.

<table>
<thead>
<tr>
<th>(b x h)</th>
<th>c</th>
<th>f_c</th>
<th>s_i</th>
<th>( \varphi_d )</th>
<th>( \varphi_l )</th>
<th>f_{ik}</th>
<th>L_1</th>
<th>t_1</th>
<th>t_2</th>
<th>s_2</th>
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<td>(mm)</td>
<td>(mm)</td>
<td>(MPa)</td>
<td>(mm)</td>
<td>(mm)</td>
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<tr>
<td>(A)200x200</td>
<td>25</td>
<td>15</td>
<td>200</td>
<td>8</td>
<td>12</td>
<td>450</td>
<td>50</td>
<td>5</td>
<td>5</td>
<td>50</td>
<td>275</td>
</tr>
<tr>
<td>(B)200x300</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</table>

All the parameters investigated strongly decrease with the increase of the ratio \( s_b/b \). Moreover, the predictions coming from the different models result largely scattered. For both square and rectangular cross-sections, the trend of the confined strain at peak-strength \( \varepsilon_{cc} \) results enclosed between the predictions given by the Nagaprasad et al. model and values obtainable by using NTC 2008 Code. Even for the estimation of the ultimate confined strain \( \varepsilon_{ccu} \), NTC 2008 returns the lowest values, while in this case the model by Badalamenti et al. constitutes the upper bound. A trend similar to that of \( \varepsilon_{cc} \) is observed for the strength increment factor \( k = f_{cc}/f_c \). Also here the highest and lowest estimation are those provided by Nagaprasad and NTC 2008 models respectively.

Finally, the prediction of the effective lateral confinement pressure shows scattered results only for the case of square cross sections, where again Nagaprasad model provides the highest estimations. On the other hand, one can observe that predictions made for the rectangular cross-section lead to similar resets using the different models. Moreover, it can be noted that according to the majority of the models, the effective lateral confinement pressure of the reinforced square cross-section reaches negligible values in correspondence of a \( s_b/b \) value of approximately 1.5. This value tends to reduce when moving to cross-sections having larger aspect ratios.
Fig 2. Comparison between strain parameters.

Fig 3. Confinement parameters from the different models against sb/b ratio.
3. LOAD CARRYING CAPACITY OF ANGLES

Steel angles provide a significant contribution to the ultimate vertical load flexural capacity. Their mechanical behaviour however depends on their arrangement. If angles are connected to the joints they can be considered as directly loaded. On the other hand if no mechanical connections are used, they can indirectly absorb load because of the frictional action occurring. In the case of angles directly loaded the contribution that they can provide in terms of flexural strength and bearing capacity is maximum. However the experimental evidence has shown that in this case buckling may occur as function of the distance between two consecutive battens. This reduces their post yielding bearing capacity and consequently the overall ductility achievable. In the second case (angles indirectly loaded) the bearing capacity is due to frictional and cohesive forces arising in the interface between steel and concrete. In particular cohesion is considered when the interface is constituted by a layer cementitious mortar, typically used for the bonding of the angles to the corners of the columns. This typology of arrangement of angles provides a lower bearing capacity with respect to the first case because of a reduced vertical stress \( f_{yb} \) that can be transferred to the angles as a function of the frictional forces. On the other hand, buckling is avoided in this case. The two possible stress-strain responses of the angles are schematized in Fig. 4.

![Stress-strain response of directly and indirectly loaded steel angles in compression.](image)

**Fig 4.** Stress-strain response of directly and indirectly loaded steel angles in compression.

### 3.1. Definition of stress-strain law for angles directly loaded

The behaviour in compression of the angles including the buckling effect can be carried out using the analytical formulation proposed by Badalamenti et al. (2010), based on the results from Gomes and Appleton (1997). With reference to Fig. 5, where \( w_h \) is the lateral displacements, \( \delta \) the axial shortening, \( \theta \) the angle defining the buckled position, the buckling load is evaluated imposing a buckled limit equilibrium condition as:

\[
N_c \cdot w_h - 2 \cdot M^* + \frac{q \cdot L_1 \cdot s_b^2}{8} = 0
\]  

\( M^* \) being the ultimate bending moment of the angle subjected to axial force, axial bending and lateral loads \( q \), the latter assumed equivalent to the lateral confinement pressure \( f_{lb, \text{max}} \). The critical stress \( \sigma_c \) is determined by varying the axial strain of the columns and dividing the resulting critical load \( N_c \) by the area of the angle. In particular the critical stress \( \sigma_c \) assumes the following expression:

\[
\sigma_c = \frac{1}{s_b \cdot L_1 \cdot t_1 \cdot \sqrt{2\varepsilon_x - \varepsilon_x^*}} \left\{ 2 \left[ \frac{L_2^2 \cdot t_1 \cdot f_{yb}}{4} - \frac{\left(N_c^*\right)^2}{16 \cdot f_{yb} \cdot t_1} \right] - \sqrt{2}f_{lb, \text{max}} \cdot L_1 \cdot \frac{s_b^3}{4} \right\}
\]
In the previous expression $N_u^*$ is depends on the axial strain being $N_u^* = \varepsilon_a \cdot 2 \cdot L_a \cdot t_1 \cdot E_s \leq 2 \cdot L_a \cdot t_1 \cdot f_{yb}$.

Finally, stress-strain curve for steel angles in compression taking into account bucking effects can be obtained by assuming the following limits for the compressive stress:

$$\sigma = \begin{cases} \min\left(E_s \varepsilon_s ; \sigma_c^*\right) & \text{if } \varepsilon_s \leq \varepsilon_{yb} \\ \min\left(f_{yb} ; \sigma_c^*\right) & \text{if } \varepsilon_s > \varepsilon_{yb} \end{cases}$$

where $\varepsilon_{yb}$ is the yielding strain of steel angles.

3.2. Definition of stress-strain law for angles indirectly loaded

The definition of the stress-strain curve in the case of angles indirectly loaded can be carried out using the formulation provided by Campione et al (2015). The load carrying capacity of the angles is a function of the tangential stresses developed along the contact surface during a sliding process. The two following assumptions are considered: a) the interface mortar between steel and concrete corners provides a cohesive strength $c_0$ along the contact surface; b) frictional action is also present and depends on the lateral confinement pressure $f_{le}$ (exerted by the battens because of the core expansion) through the friction coefficient $\mu$.

Because of this, the stress path followed by the steel angles interface may not exceed the stress value $\tau_{max}$, calculated in analogy to the Coulomb law as follows:

$$\tau_{max} = c_0 + \mu \cdot f_{le, max}$$

The maximum load $P_A^*$ that can be supported by the angles is thus evaluable by an equilibrium equation along the direction of the contact surface of the angles as follows:

$$P_A^* = 2 \cdot n_a \cdot l_{ia} \cdot l_o \cdot (c_0 + \mu \cdot f_{le, max})$$

where $n_a$ is the number of angles (equal to 4 for square and rectangular cross sections), $l_{ia}$ the width of each of their sides and $l_o$ their total length. When the maximum load $P_A^*$ is reached, the following reduced maximum normal stress $f_{yb}^*$ is developed:

$$f_{yb}^* = \frac{P_A^*}{A_a}$$

where $A_a$ is the total area of the angles.

Elastic perfectly-plastic behaviour for the angle subject to sliding can be assumed. The fictitious elastic modulus $E_s^*$ accounting the reduced stiffness governed by the fictional phenomena is calculated as:
Experimental observations show that the strain \( \varepsilon_{yb} \) can be assumed with good approximation equal to the strain at peak-strength of unconfined concrete, thus \( \varepsilon_{yb} = \varepsilon_{eb} \). Beyond this strain limit the angles no longer undergo load increments and start to slide. A schematization of this mechanical model is proposed in Fig. 6.

The authors suggest the values \( c_0 = 0.10 \text{ MPa} \) for the cohesion (based on the experimental and numerical determinations reported by Baltay and Gjelsvik (1990)) and \( \mu = 0.4 \) (based on the best fitting of the experimental data).

![Frictional model schematization for indirectly loaded angles.](image)

4. **COMPARISONS BETWEEN ANALYTICAL PREDICTIONS AND EXPERIMENTAL DATA**

The reliability of the above discussed confinement models is tested against a wide dataset of experimental compressive tests on RC columns reinforced by steel jacketing available in the literature. The experimental tests comprised both specimens with and without end connections (directly and indirectly loaded). The prediction of ultimate load capacity was carried out differentiating the analytical formulations for these two cases and in particular assuming the above described approaches to account for buckling effects or frictional effects for the cases of angles fully loaded or not respectively. Internal reinforcement was modelled assuming elastic-plastic law while concrete stress-strain law were each time differentiated according to confinement models considered.

Geometrical and mechanical parameters of the specimens are reported in Tables 3-4. The symbols assumed for specimens are referred to the respective authors. In particular symbols C1-C15 correspond to Cirtek et al., A1-2 to Adam et al, M1-2 to Mohamed et al., T1-8 to Tarabia et al., Ca1-3 to Campione et al.
The comparisons between analytical and experimental results are proposed in Figs. 7-8 in terms of ultimate vertical load. For the case of specimens with angles not directly connected the best correspondence was found using the confinement model by Montuori and Piluso. A good average correspondence was also found observing the results obtained from Nagaprasad model. For this latter case a large variance was however recognized. Similar and more conservative results are instead obtained from Badalamenti et al. and NTC 2008 models. For the case of angles fully loaded a really good correspondence was found from Nagaprasad model. The models by Montuori and Piluso and Badalamenti et al. have also shown a good agreement with experimental results but with a larger margin of uncertainty. Finally also in this case the use of NTC 2008 model resulted significantly conservative.
5. CONCLUSIONS

The use of steel jacketing of RC columns is a very common practice for the retrofitting of existing buildings. The increase of bearing capacity that can be achieved is however sensitive to different aspects that have to be taken into account in computational models. The use of suitable
confinement models for concrete makes it possible to evaluate the effective lateral confinement pressures exerted by the reinforcing system and provides more accurate results. On the other hand the mechanical aspects related to the technological arrangement of the connections of the angles with the end joints have to be included in computational models. In particular buckling has to be considered for angles directly connected and fully loaded while effects of friction and cohesion have to be included in the case of angles not directly connected. The paper presented a comparative study about the main confinement models for steel jacketed columns, highlighting the related differences in predicting the confinement parameters. The models reliability has been also tested comparing analytical results with those of experimental compressive tests carried out by different authors for reinforced column specimens, both for the cases of presence and absence of end connections. For all the cases considered the Italian Code (NTC 2008) resulted to be conservative with respect to the prediction of the actual capacity. The other models had a good agreement (more or less scattered) with respect to experimental results. Montuori and Piluso model resulted better than the other models in predicting the capacity of columns with angles not presenting end connections while Nagaprasad et al. model was more reliable for the case of angles well connected.

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Innovative and Traditional Techniques for Seismic Retrofitting of an Existing RC School Building: Life Cycle Assessment and Performance Ranking through MCDM Methods

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Abstract. In the paper the seismic retrofitting of an existing RC school building located in the district of Naples has been faced. The school, which was designed to sustain gravity load only, is composed of seven constructions seismically jointed each to other. One of these constructions has been retrofitted with traditional (RC shear walls) and innovative (concentric braces, eccentric braces, buckling restrained braces and steel plate shear walls) intervention techniques, whose effectiveness has been evaluated in the non-linear field. Moreover, the environmental impact of these interventions has been assessed by means of an appropriate analysis program. Finally, the choice of the best intervention from economic, structural and environmental points of view has been done by using the MCDM TOPSIS method.

1. INTRODUCTION

The disastrous effects of the earthquakes occurred in Italy during the past few years do not depend on their significant seismic intensity only, but also on both the high population density of the different territory areas and the poor attention paid to build seismic-resistant buildings. In particular, more than 30% of the reinforced concrete buildings in Italy are inadequate to withstand design seismic loads prescribed by Italian current regulations. The seismic behaviour and the consolidation of existing RC buildings, with particular reference to those with public functions, is therefore an extremely important topic in the field of Earthquake Engineering. The present work fits perfectly within this context, it being developed as part of the European research project COST Action C26 "Urban Habitat Constructions under Catastrophic Events" with the aim of assessing the vulnerability of buildings with respect to catastrophic, both natural and artificial, actions. In particular, it was made reference to the risk scenario deriving from a possible eruption of Vesuvius. In-situ surveys were conducted for the seismic-volcanic vulnerability analysis of private, monumental and public buildings of the most populous city in the area around Vesuvius, Torre del Greco. With reference to the school buildings, ten masonry buildings (primary schools) and five reinforced concrete buildings (secondary schools) were examined. The majority of such buildings was erected without seismic requirements and, therefore, requires a seismic retrofit. In this paper the attention is focused on the "d’Assisi"
secondary school, which consists of seven reinforced concrete constructions seismically-joined each to other. The case study is one of these constructions, which is developed on two-story and is herein retrofitted with both traditional (RC shear walls) and metal-based innovative (concentric braces, eccentric braces, buckling restrained braces and steel plate shear walls) techniques. The comparison among these interventions has been faced in terms of structural, economic and environmental points of views by using the Multi-Criteria Decision Making (MCDM) TOPSIS method.

2. THE CASE STUDY: THE “SAN FRANCESCO D’ASSISI” SCHOOL IN TORRE DEL GRECO (NA)

The structure under study is part of the school complex "San Francesco d'Assisi" located in Torre del Greco, district of Naples. The school, which was erected in the late eighties, is divided into seven RC buildings (two used as gyms), independent from each other by seismic joints (Fig 1a). The structural unit object of the research is the construction representing three of the seven buildings that constitute the school complex. The interiors of the modular RC structure are used as classrooms and teaching laboratories (Fig 1b). The selected structural unit has almost rectangular shape with plan dimensions of 19.70m x 23.00m and develops on two levels. The structural organization shows an eccentric arrangement of the staircase that confers to the building a plan irregularity. The seismic-resistant vertical structures are RC frames placed in the vertical direction (y), which withstand the loads deriving from floors (Fig 1c).

Due to the absence of the original technical drawings, the design of the structural elements in terms of geometrical dimensions and bars (longitudinal and stirrups) have been done by means of the simulated building project, which was executed under the rules used at that time of construction. The mechanical properties of the concrete were determined using the results of laboratory tests performed on buildings built in the same period within the same territorial region of the investigated construction. From the results of the experiments, it was found a C20/25 type for the concrete. Instead, for reinforcing bars, considering the time of construction and the intended use of the structural module investigated (strategic building), a steel type FeB38k was considered.

![Fig 1: The school complex “D’Assisi” (a) and the structural unit under investigation: the architectural layout (b) and the structural scheme (c).](image-url)
The building under investigation has been modelled by using the finite element software SAP2000 V.14.2.4 [3] (Figure 2a). It has an irregular seismic behaviour, as it appears from results of modal analysis (Figures 2b, c, d), where it is evident that the second mode is of torsion type.

Figure 2: 3D model of the school unit under investigation (a) and modal analysis vibration shapes: first mode (T = 0.61) (b); second mode (T=0.43) (c) and third mode (T=0.34) (d).

3. DESIGN AND APPLICATION OF THE PROPOSED REHABILITATION SYSTEMS

The upgrading design herein proposed is finalized both at increasing strength and stiffness of the examined construction under seismic actions. Various upgrading systems have been applied to the proposed case study to achieve the proposed targets: Concentric Bracing Frames (CBF), Eccentric Bracing Frames (EBF), Buckling Restrained Braces (BRB), Steel Plate Shear Walls (SPSW) and seismic-resistant RC Shear Walls (RCSW). Taking into account the location of the staircase, the upgrading systems have been placed in an eccentric manner with respect to the school barycentre so as to guarantee a good regularization of its seismic behaviour. Therefore, the existing structural parts, that is the RC frames hosting the considered upgrading systems and foundations, have been verified under the new stress state deriving from insertion of new devices.

3.1. Analysis and comparison of results

The application of the five seismic upgrading systems has been done to improve the dynamic behaviour of the existing structure, affected by problems of torsion rotation of the floors caused by the inhomogeneous location in the plane of seismic-resistant systems. Table 1 shows values
and directions of the main vibration periods for the original building and the same building retrofitted with the above mentioned techniques. From this table it is noticed that the structural performances of the retrofitted building improve in all cases. In fact, unlike the case of the bare structure, with all upgrading systems the first two modes are translational, while the third is of torsion type.

Table 1: Comparison among different retrofitting techniques

<table>
<thead>
<tr>
<th></th>
<th>Existing structure</th>
<th>CBF</th>
<th>EBF</th>
<th>BRB</th>
<th>SPSW</th>
<th>RCSW</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔF (%)</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>ΔV (%)</td>
<td></td>
<td></td>
<td>106</td>
<td>39</td>
<td>170</td>
<td>72</td>
</tr>
<tr>
<td>Ductility</td>
<td></td>
<td></td>
<td>1.78</td>
<td>1.75</td>
<td>1.90</td>
<td>1.72</td>
</tr>
<tr>
<td>T1 [sec]</td>
<td>0.61 (x)</td>
<td>0.43 (x)</td>
<td>0.39 (x)</td>
<td>0.45 (x)</td>
<td>0.44 (x)</td>
<td>0.50 (x)</td>
</tr>
<tr>
<td>T2 [sec]</td>
<td>0.43 (μ)</td>
<td>0.33 (y)</td>
<td>0.33 (y)</td>
<td>0.32 (y)</td>
<td>0.32 (y)</td>
<td>0.33 (y)</td>
</tr>
<tr>
<td>T3 [sec]</td>
<td>0.34 (y)</td>
<td>0.31 (μ)</td>
<td>0.31 (μ)</td>
<td>0.31 (μ)</td>
<td>0.32 (μ)</td>
<td>0.31 (μ)</td>
</tr>
</tbody>
</table>

Figures 3 and 4 show pushover curves of the structure upgraded with the different techniques used considering the distributions of forces proportional to the first vibration mode and those related to the structural masses, respectively.

Fig 3: Pushover curves with forces proportional to the first vibration mode in directions x (a) and y (b)

Fig 4: Pushover curves with forces proportional to the masses in directions x (a) and y (b)

Moreover, in all cases, the energy dissipation was concentrated in the sacrificial elements designed with dissipative function that, exhibiting extensive damage in the plastic field, preserve the existing structure from brittle collapse. Referring to the examined innovative
retrofitting systems, the greatest increases of performance in terms of stiffness and strength in comparison to the behaviour of the bare structure are obtained respectively with EBF and SPSW. Even in terms of ductility substantial performance improvements are found, with values ranging between 1.72 (SPSW) and 1.90 (BRB).

4. LIFE CYCLE ASSESSMENT

4.1. Foreword

The Life Cycle Assessment is an analysis method evaluating a group of iterations that a product or service has with the environment during its entire life cycle. It includes the steps of pre-production, considering also the extraction and the production of primary materials, production of the finite element, distribution, use, taking into account also any reuse and materials used for normal maintenance, recycling and final disposal.

Therefore, LCA is an objective evaluation and quantification method of energy, environment loads and potential impacts associated with a product, considering both its fabrication process and activity along the whole life cycle, from raw material acquisition to the end of life. The criterion under question is considered as a fundamental support to the development of environmental labelling schemes with the purpose either to define reference environmental criteria for a given group of products or to represent the main tool to obtain an Environmental Statement of products. It considers the environmental impacts of the examined case in relation to the human health, the quality of the ecosystem and the impoverishment of resources, considering also the economic and social impacts. In the current study only general information on the life cycle of each product are given, that is only a partial assessment of the processes of pre-production and production of the products has been done, whereas a comprehensive study of all the processes occurred during their whole life cycle has not been performed.

4.2. LCA of retrofitting strategies

After the structure has been updated through different retrofitting techniques, the problem of evaluating the environmental impact of each of them arises. The main objective is not just going to assess the environmental performance of the above techniques, but also to compare them in order to evaluate that with the lowest environmental impact.

In order to perform a LCA analysis of comparative type it has been made the hypothesis that the different intervention techniques are designed to achieve the same structural performance, thus defining the functional unit of the analysis. In particular, the techniques are applied to obtain the same increase of the structural capacity, such that the retrofitted structure can sustain seismic actions corresponding to a risk index equal to 100%. In addition, for each technique used, the LCA is conducted by referring to the steps from the “cradle to gate”: extraction of raw materials, production, preparation of the substrate to host the reinforcement installation and seismic upgrading. For the analysis the following steps have been considered:

- Production of the material: this stage includes the extraction of raw materials and the production process of the materials used in the upgrading techniques;
- Preparation of the substrate where the reinforcement will be applied;
- Installation of the reinforcement.

About inventory analysis, both primary data and secondary ones have been used. In particular the main data have been used for modeling steel and concrete. Instead, secondary data were taken from the databases Ecoinvent and Idemat available in the program Simapro 7.

The environmental impact analysis is conducted by the method Impact 2002+, whose results are presented in terms of "End point category" or categories of damage (Human Health, Ecosystem quality, Climate Change and Resources) (Figure 5).

The environmental impact assessment of each intervention on the whole structure, once the environmental impacts of each reinforcement per square meter are known, is achieved with a simple multiplication operation. After conducting the LCA for the individual strategies of seismic upgrading, the next step is to perform a LCA comparison between the different strategies in order to evaluate which of those used has the best environmental performance. The
results of the analysis are presented with normalized values relative to the value of maximum impact for each category of damage. Once these normalized values are known, a multi-criteria analysis considering as LCA criteria the "Human health" has been performed, as it will be shown in the next Section.

Fig 5: Results of the LCA analysis

5. MULTI-CRITERIA DECISION ANALYSIS: THE TOPSIS METHOD

The MCDM analysis methods are comparison procedures based on multiple criteria aiming at contributing to the development of a learning process which feeds the same decision-making process. In particular, they can be considered mathematical tools allowing to solve a decision problem by identifying the best alternative meeting a given number of criteria. All multi-criteria problems, regardless of their different nature, have common features, which can be summarized as follows:

- Multiple goals/attributes with the purpose to identify objectives and/or attributes relevant to the focus of the problem;
- Conflicts between criteria;
- Immeasurable measurement units;
- Selection of the most satisfying alternative.

All multi-criteria decision problems are analysed by considering the following elements:

- A "goal" or a set of "goals", which represents the general aim to be achieved.
- A Decision Maker (DM) or a group of decision makers (DMs) involved in the selection process, who are responsible of the evaluation procedure.
- A set of decisional alternatives, which are the fundamental elements of the evaluation and selection process.
- A set of evaluation criteria, used by DMs to evaluate the performance of alternatives.
- The preferences of DMs, which are typically expressed in terms of weights assigned to the evaluation criteria.
- A set of scores, expressing the value of the alternative i with respect to the criterion j.
The TOPSIS (Technique for Order Preference by Similarity to Ideal Solution) method is an easy MCDM technique used by a DM to find the best solution among a number of alternatives or various options considered. This method allows to represent the various alternatives as points of a vector space having dimensions equal to the number of criteria considered, so that the performances of the different solutions become the coordinates in the vector space assumed. The TOPSIS method identifies two ideal alternatives, the worst (A-) and the best (A+), with reference to the criteria investigated, so that the optimal solution of the decision problem is the alternative having the shortest distance from A+ and the maximum distance from A-. This method has been already applied from several researchers to some cases of structural modification interventions, namely vertical addition and seismic retrofitting, of existing buildings.

In this case, the "alternatives" are the various retrofitting techniques applied to the structure under study (CBF, EBF, BRB, SPWS and RCSW), while the evaluation "criteria" are the vulnerability index, the continuation of the educational activity, the reversibility of the intervention, the human health (LCA) and the intervention cost. The criteria under consideration can be identified as of benefit (B) type or the cost (C) one, with the former and the latter that must be maximized and minimized, respectively. At the end of the decision-making procedure a sensitivity analysis of the solution is conducted for evaluating the reliability of values assigned to the weights of the judgment criteria. This analysis assesses the stability of the optimal solution, ensuring that it does not change when the values of the weights are modified. The stability of the results obtained is evaluated by varying the weight from 0 to 1 and checking that the final solution of the decision-making process does not modify.

In the examined case, first, among the five criteria considered, major attention has been dedicated to the structural (vulnerability index), environmental (LCA) and economic (cost) parameters, which have assumed weights greater than the others. Afterwards, three different analyses have been performed with the TOPSIS method. In the first analysis, the greatest weight has been assigned to the Vulnerability index ($I_v$), while in the second and third analyses the highest value of the weight has been given to the Human Health (LCA) and to the cost of the intervention (C), respectively. The weights assigned to all criteria are depicted in Table 2.

For each of the analyses carried out, the ranking of the alternatives considered are plotted under form of histograms in Figure 6, where the best consolidation technique to be used is identified.

From the analyses performed it is apparent that the best intervention for retrofitting the school structural unit under study is represented by CBF, which are immediately followed by the use of BRB. On the other side, from seismic and environmental points of views RCSW represent the worst intervention, whereas SPWS are the most expensive technique.

Table 2: Weights assigned to the criteria in the three MCDM analyses performed

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Weight</th>
<th>Max C1</th>
<th>Max C4</th>
<th>Max C5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_1$ Vulnerability Index ($I_v$)</td>
<td>$w_1$</td>
<td>0,36</td>
<td>0,25</td>
<td>0,24</td>
</tr>
<tr>
<td>$C_2$ Continuation of the Educational Activity</td>
<td>$w_2$</td>
<td>0,05</td>
<td>0,05</td>
<td>0,05</td>
</tr>
<tr>
<td>$C_3$ Reversibility of the Intervention</td>
<td>$w_3$</td>
<td>0,10</td>
<td>0,10</td>
<td>0,10</td>
</tr>
<tr>
<td>$C_4$ Human Health (LCA)</td>
<td>$w_4$</td>
<td>0,24</td>
<td>0,36</td>
<td>0,25</td>
</tr>
<tr>
<td>$C_5$ Cost (C)</td>
<td>$w_5$</td>
<td>0,25</td>
<td>0,24</td>
<td>0,36</td>
</tr>
<tr>
<td>- Wtot</td>
<td>$w_{tot}$</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
</tr>
</tbody>
</table>
6. CONCLUDING REMARKS

In the present paper the problem of seismic upgrading of a RC school building by means of innovative and traditional techniques has been treated. Nonlinear static analyses have shown that the seismic upgrading systems designed allow to increase stiffness and strength of the existing building, providing also an improvement of its dynamic behaviour. These purposes have been achieved by recording a decrease of periods of vibration and a regularization of the structure dynamic behaviour, with a third vibration period of torsion type.

In all analysis cases, the energy dissipation has been always concentrated in the upgrading dissipative systems, which have preserved the existing structure from damage. The comparison between the bare structure behaviour and the upgraded structures one has shown that the greatest performance increases in terms of stiffness and strength have been achieved respectively with EBF and SPSW. Noteworthy performance improvements have been found even in terms of ductility, with values ranging between 1.72 (SPSW) and 1.90 (BRB). As a conclusion, the results obtained from the analyses conducted show the effectiveness of all the devices tested for the upgrading of RC school building investigated.

Moreover, a LCA analysis has been performed with the method Impact 2002+, implemented within the Simapro 7 software, to evaluate the environmental impact analysis of different techniques used.

Finally, in order to detect the best retrofitting solution, the MCDM TOPSIS (Technique for Order Preference by Similarity to Ideal Solution) method has been used. Three different analyses have been performed by assigning the highest weight value before to the seismic (vulnerability index) parameter and after to the environmental (LCA) parameter and to the economic (cost) one.

From these analyses it is apparent that the best intervention for retrofitting the school structural unit under study is represented by CBF, which are immediately followed by the use of BRB. On the other side, from seismic and environmental points of views, RC shear walls represent the worst intervention, whereas SPSW are the most expensive technique.
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The Hidden Value of Stone: Life Cycle Assessment of the Construction and Refurbishment of a 60-year-old Residential Stone Building

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Abstract. In many certification labels, the environmental performance of a building is evaluated in terms of embodied energy and energy consumption. These indicators are calculated through a Life Cycle Assessment of the construction and maintenance/use phase of the building respectively. The objective of this research is to study how stone as a material performs in these two categories. We selected as a case study the 60-year-old residential stone building “Résidence-le-Parc” in Paris and performed a LCA based on historic data from its life cycle, both its construction and maintenance. Different what-if scenarios for the refurbishment served to examine what impact various interventions in the cell of the building can have on the embodied energy and the energy consumption. It is observed that although the building is a massive stone structure, the contribution of stone to the life cycle assessment and the thermal performance is minimal. The study shows that apart from the environmental and economic aspect, there is some hidden value in the use of stone in multi-ownership residential buildings, related to the social dimension.

1. INTRODUCTION

Buildings are one of the main consumers of energy and resources. Around one third of the global final energy consumption is attributed to the building sector. In some countries, such as the United States, more than half of the energy consumption is directly or indirectly related to building construction and operations. This increased consumption of energy as a result of the activities of the construction sector produces high levels of greenhouse gas emissions.

The European Commission, in an effort to promote reduced energy dependency, has prescribed some targets for greenhouse gas emissions reduction. In addition, many certification labels place a strong emphasis on supporting a more environmentally friendly design and construction. For example, LEED requires a 5% reduction in comparison to the energy consumption of the baseline building through a whole-building energy simulation. DGNB goes one step further by requiring a Life Cycle Assessment and evaluates the primary energy need of a building during its construction, maintenance, demolition and recycling and the environmental impacts during the use phase.
Therefore, it is clear that in order to attain such targets, it is important to reduce the energy consumed by the building sector, both during the construction and the use phase. Overall, there is a need to optimize the environmental performance of a building throughout its life cycle. The objective of the current paper is to evaluate the environmental performance of stone in a residential building both in terms of embodied energy and energy consumption during its use phase. For this reason, a 60-year-old residential stone building in Paris was chosen as a case study. First, the environmental impact of its construction and use was assessed with the aid of a Life Cycle Assessment and a thermal simulation of the current state of the building. Furthermore, we evaluated specific thermal interventions with respect to their environmental and economic efficiency. The results allowed us to derive some observations regarding the use of stone in residential buildings and to reflect on additional aspects that a LCA cannot capture.

2. METHOD

The goal of this research is twofold: on the one hand, to study the contribution of every element of the building to the overall energy (embodied and operational) and on the other hand, to improve the thermal and economic performance of the building through different intervention scenarios.

Regarding the calculation of the embodied energy of the construction and maintenance of the building, we used Life Cycle Assessment. LCA is a very common methodology used to trace the environmental impacts of a product throughout its life cycle from raw material acquisition to production, use, maintenance and final disposal. The study is based on a cradle-to-gate LCA of the building from construction to use phase and including the maintenance. The end of life of the different building elements is not taken into account, even though its inclusion in this case study would be positive, since the blocks of stone used in the construction have the potential to be reused. The impact assessment method selected to evaluate the environmental performance of the building was the Cumulative Energy Demand. This is a single-score life-cycle impact assessment methodology presenting the renewable and non-renewable energy use throughout the life-cycle of a product.

The energy consumption of the building for the different intervention scenarios was assessed through an energy simulation with Autodesk Ecotect 2011, by using a Building Information Model created with Revit. Regarding the economic evaluation of the interventions, the method of Net Present Value was applied.

2.1. Description of the case study

The building selected for the purposes of this study is one of the buildings of the residential complex Résidence-le-Parc, in Meudon-la-Forêt, Paris. It was constructed by the French architect Fernand Pouillon during the years 1957-1962. The building examined (G3) is 10-storey (plus ground floor) with a total gross floor area of 7509m² and 8 apartments per floor. A typical plan view (where two of the eight apartments per floor are visible) and a view of the façade of the building are presented in figure 1. Two of the façades of the building are massive walls made of limestone and the other two are mainly covered by glass and endowed with colossal stone pilasters that are regularly spaced. The pilasters, according to Pouillon, are an element that expresses the solidity of the building. Stone was extracted in the quarry of Fontvieille and transported by train up to the station of Massy and then by truck to the construction site. The choice of the specific quarry was related to the general cycle of construction operations of the architect, Fernand Pouillon.

2.2. Data Collection

Primary data were extracted from the architectural plans, from various books detailing the work of Fernand Pouillon as well as from the reports of the architectural office Alluin&Mauduit and the engineering group ReeZOME. Secondary data were extracted from the Ecoinvent database v2.2. The environmental impact of the extraction and processing of stone was based on the data from two swiss quarries, as calculated in Ioannidou et al.. Transportation distances of the
materials were assumed 30km and 150km, depending on whether the materials were locally or nationally produced. Figure 2 shows the mass of the various building elements per m² of gross floor area, as well as their use in the building.

![Fig 1. Typical plan view of two of the eight apartments per floor and exterior view of the building G3 of Résidence-le-Parc](image)

![Fig 2. Mass and use of building materials](image)

<table>
<thead>
<tr>
<th>Materials</th>
<th>Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Others</td>
<td>Doors, windows</td>
</tr>
<tr>
<td>Wood</td>
<td>Windows</td>
</tr>
<tr>
<td>Glass</td>
<td>Tiles for staircases</td>
</tr>
<tr>
<td>Granite</td>
<td>Terrace</td>
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<tr>
<td>Bitumen sealing</td>
<td>Reinforced concrete</td>
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<tr>
<td>Reinforcing steel</td>
<td>Exterior &amp; some interior walls</td>
</tr>
<tr>
<td>Brick</td>
<td>Interior walls</td>
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<tr>
<td>Plasterboard</td>
<td>Walls and pilasters</td>
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<tr>
<td>Limestone</td>
<td>Slabs and staircases</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
</tr>
</tbody>
</table>

Normally, maintenance of stone involves cleaning with water. However, for the purposes of this study, maintenance data for stone were based on an extreme case scenario: the maintenance of the Rathaus building in Zurich against graffiti paintings. For the stone protection, an anti-graffiti protection layer of average thickness 20-30 μm is applied every 2-3 years on the façade of the town hall building, which is made out of sandstone.

Regarding the economic part of the study, costs were calculated based on the Swiss market. Data were extracted from EAK and from publicly available data on the internet, when no data was provided in EAK.
3. RESULTS

3.1. Life Cycle Assessment of the current state

A Life Cycle Assessment of the construction phase of the building reveals the significant contribution of the reinforced concrete to the total embodied energy (Figure 3). The impact of stone (the massive limestone blocks as well as the granite tiles in the staircases) is very limited. If we compare the embodied energy of the various materials to their total mass (Figure 2), we observe that even though stone is the second in quantity material used in the building, it is quite “invisible” with respect to its environmental impact (the extraction and processing). On the other hand, the transportation of stone bears a significant impact to the overall environmental behavior of the building. The transportation of stone from a quarry 185km away, even though by train, still requires significant amount of energy. The picture would change if we had chosen as an indicator for the environmental performance the Global Warming Potential. In this case, since the transportation by train produces less CO₂ than by truck, due to the source of energy used, we would observe that the process of transportation of concrete has a worse ecological behavior than the transportation of stone, despite the disparity between the two transportation distances.

Moreover, an energy simulation of the building indicates the enormous contribution of the heating needs to the total energy consumption throughout its life-cycle. This underlines the necessity to renovate the building in order to reduce its heating demands.

![Fig 3. Energy required for the construction, maintenance and heating of the building](image)

3.2. Life Cycle Assessment and thermal evaluation of various interventions

In order to improve the energy performance of the specific building, we evaluated various interventions, namely:

- The replacement of single with double glazing windows (new U value = 2.74 W/m²K)
- The insulation of the ceiling of the upper floor (terrace)(new U value = 0.12 W/m²K)
- The insulation of the massive stone walls (new U value = 0.38 W/m²K)

In addition, we examined the impact that an increased percentage of stone in the façade (40%, which is around twice the current percentage) would have on the thermal consumption. Although this is not a possible a posteriori intervention, it serves to evaluate different parameters that can affect the thermal performance in this building. In this case, only the width of the windows changes, while position and height remain the same. The results of the simulations, executed with the Ecotect software, are presented in figure 4.
It is observed that only the replacement of the single with double glazing windows and the increase of the surface of stone in the façade have a significant contribution to the reduction of the energy consumption of the building. The insulation of the terrace does not change the thermal performance and it seems that the current massive stone walls provide an adequate level of insulation.

If we further examine the total environmental impact of changing the window glazing, which proved to be the most efficient intervention, we observe that the additional savings in energy if we install triple-glazing windows (U value = 1.82W/m²K) are quite marginal compared to the savings from replacing the single with double-glazing windows (Figure 5).

3.3. Economical Assessment of the interventions

As a last step, we evaluated the economic feasibility of the following interventions:
- replacement of the windows (both frame and glazing) by installing double-glazing windows
- installation of shutters to all the windows
- the repair of the terrace through improvement of its insulation
- insulation of the ceiling of ground floor (where the ground floor is not heated)
- insulation of thermal bridges
- insulation of the pipes of sanitary hot water network
- installation of a ventilation system (single or double-flow)

We compared the performance of the current building for the next 60 years, without any upgrade, just the regular maintenance, to the case where each one of the above interventions was applied (maintenance also included). We assumed that the interventions are performed at the current point in time, when the building has completed almost 60 years of existence, it is therefore at the end of its life-cycle. Normally, this means that a major renovation with
replacement of many components has to be performed. For the purposes of this comparative study, however, we take into account only the additional material required for the specific renovation. Therefore, we calculated the embodied energy required for the installation of the additional materials in the building, which we subtracted from the anticipated energy savings during the life-cycle of the building (60 years). Regarding the economic calculations, we determined both the investment needed for every intervention (Figure 6) and the Net Present Value of every intervention (Figure 7). Both economic numbers were normalized by the amount of energy savings that this intervention entailed. The graphs show the additive impact of the interventions, assuming the independency of all interventions.

Therefore, the graphs show the evolution of the price-benefit ratio, as the energy savings increase. The most beneficial interventions, from a financial point of view, are the replacement of the windows, the installation of a (single-flow) ventilation system and the insulation of the pipes for the sanitary hot water. As expected, the amount of investment required for the last interventions is very high in comparison to the additional savings that these interventions would result in (Figure 6). Commensurately, the Net Present Value declines with an increasing rate, as the energy savings increase (Figure 7).

![Fig 6. Relationship between the amount in CHF spent per MJ of energy saved to the total avoided consumption of energy](image1)

![Fig 7. Relationship between the Net Present Value in CHF of every intervention per MJ of energy saved to the total avoided consumption of energy](image2)

4. DISCUSSION

The study performed on the residential stone structure Résidence-le-Parc revealed the main elements that affect the environmental and thermal performance of the building. With respect to the environmental performance, when the embodied energy of the construction of the building is considered, it is concrete and steel that have the main contribution to the overall impact. Even
though stone is the second largest by mass material, its environmental impact is invisible in comparison to the other main materials of the building.

Furthermore, the energy analysis of the base building and its comparison to some what-if scenarios reveals that the thermal performance is mainly driven by the windows. Single-glazing windows are the main source of heating losses in the building and should be replaced by double or triple glazing windows. A fictitious scenario where we increased the surface of stone in two of the four façades indicated that stone has a positive contribution to the thermal performance. On the other hand, the two other massive stone façades seem to offer an adequate insulation and a further increase of the insulation does not improve considerably the energy performance.

It should be mentioned here that the current study did not account for the energy consumption due to the lighting and the electrical appliances during the life-cycle of the building. However, for the comparative assessments conducted above, the conclusions remain the same. With respect to the lighting in the case of the increase of the surface of stone, a decrease of the window-to-wall ratio (WWR) from 73% (existing) to 60% (scenario) is not anticipated to induce major changes in the lighting consumption, as according to Ochoa et al., when WWR is greater than 60% in the east, west and south façade and greater than 70% in the north façade, there is no big differentiation in the lighting consumption. It is expected that if we further decrease the WWR in our case study, the lighting consumption will increase, therefore a full lighting simulation would be necessary if we would like to identify the optimal WWR for the specific building.

Summarizing, it is clear that stone in the specific building has no adverse environmental consequences. Its behaviour is “invisible” when considering the whole building, since it has a reduced embodied energy and does not negatively affect the thermal performance of the building. In addition, its use as a construction material for this complex was quite economical, because of the innovative method to saw the stone to the required dimensions at the quarry. Furthermore, the use of large stone blocks in the quarry as well as in the construction site increased the efficiency of the operations by reducing the material handling.

But apart from the above qualities of stone, there is another “hidden” value of using this material for multi-proprietorial residential buildings. Even with reduced maintenance, stone can retain a good level of condition, in contrast to reinforced concrete, which necessitates regular maintenance. This is proven in the specific building, which throughout its existence has received very limited maintenance because of the proprietary structure; the whole complex comprises 2635 apartments and is administered as a co-propriety. The syndicate council, which is in charge of the operation of the building, has often presented renovation scenarios to the general assembly. However, since a majority of votes has to be attained for any renovation to be applied, the maintenance works performed till now in the building have been minimal. Nevertheless, it still remains a fully operational building. Therefore, the use of a material that can still provide its main functions, even with reduced maintenance, seems to have offered an additional value to this building.

5. CONCLUSION

This study evaluated the environmental performance of the building Résidence-le-Parc both in terms of embodied energy and energy consumption. It is observed that even though stone is one of the main elements of the building with respect to mass, its contribution to the embodied energy of the building is limited and it is the reinforced concrete that drives the environmental performance of the structure. Regarding the energy consumption, by studying different what-if scenarios, it was noted that the determining façade element that affects the environmental performance are the single-glazing windows and a replacement with double or triple-glazing windows seems necessary. This is also the most beneficial intervention from an economic point of view. Moreover, an increase in the surface of stone improves significantly the overall energy consumption of the building. This shows that the stone blocks bear a reduced environmental impact and at the same time they have a positive contribution to the energy performance of the building. If the social aspect of stone is also taken into account through its reduced need for maintenance, then it is evident that the value of stone in the specific building is multi-fold.
6. ACKNOWLEDGMENTS

We would like to thank the architectural office Alluin&Mauduit, Groupe A&M, and the engineering group ReeZOME, Réseau d'ingénieries pour l'architecture et le développement durable for their reports on the renovation of the building as well as for the designs that they kindly provided to us. We would also like to thank Ms Catherine Sayen, President of the Association “Les Pierres Sauvages de Belcastel” for the provision of information on the project.

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Structural Optimization Including Whole Life Cost of a Timber Building using Evolutionary Algorithms

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Abstract: The present study seeks to couple the problem of the structural optimization of buildings, with that of their energy performance optimization. The objective function of the problem is a cost function, which takes into account the following parameters: heating and cooling costs, wall and window insulation profile, window sizes, photovoltaic array sizing, air-conditioning system type and number of units, lighting control system, frame type (frame bay length optimality), spacing among frames, sizing of cross sections as well as parameters related to the life cycle of the building. Modeling is based on the Eurocodes and KENAK (that is the Greek national interpretation of European standard EN ISO 13790). Optimization is solved using evolutionary algorithms. The optimization problem is implemented on a two storey office building (35x30 m) in Athens Greece, whose frames are made of timber.

Keywords: structural optimization; life cycle analysis; energy cost; timber frames; evolutionary algorithms

1. INTRODUCTION

In order to test the capacity of an automatic method to optimize within one step the envelope, energy and structural subsystems an algorithm in Matlab unifying the structural and energy performance optimization of a building including parameters from its life cycle has been developed. The building that was decided to constitute the model for that attempt would be two-storey, would have a 35x30 m rectangular shape, (see subchapter 6 for detailed plans and sections of the building); its frames will be made of timber and would be used as an office building, assumed to have a 5-day working week, a 12-hrs occupancy and a continuous type of heating. Emphasis was placed on mineral wool insulation profiles and A/C HVAC systems, popular in Greece.

At first, a market research provided average, real-life cost figures of the subsystems that would be used in the algorithm. The market research took into consideration the costs of the following building components: Wall and roof inner & outer layers (wall type of the timber building; outer layers made of timber panels, intermediate layers made of mineral wool, inner layers made of gypsum boards). Mineral wool insulation of various thicknesses, Air-conditioning systems with various energy parameters, Double and triple-glazed aluminum windows (with regular or low-e values), Photovoltaic system cost per kWp, Structural timber cost per kg, Cost of the lighting control system.

In order to save computational time and unify parameters that have an impact and correlate the energy performance parameters with the resultant cost, expotential and multiple linear regression has been used. The cost functions below that were used in the algorithms, are
demonstrated in order for the reader to be able to understand the logic behind that idea (Bekas 2015):

costwindowseast = (171.1422 - 25.3385*Uwineast + 57.63547*ggl)*Awineast (Function that associates the east window profile cost with its energy parameters). Where: U-value is the heat transfer coefficient, ggl value is a coefficient measuring the solar energy transmittance of a window glass.

costinsulationwallnorth = (1.309*Uwallnorth^-0.672)*Awnorth (Function that associates the timber wall profile cost with its U value and depends on the thickness of mineral wool insulation. The same function has been used for the south, the west and the east wall).

Similarly, the following cost functions were also used in the modeling: cost insulation roof = (1.3429*Uroof^-0.679)*Aroof, cost insulation floor = (1.17137*Ufloor^-1.418)*Afloor

The use of functions (or metamodels) for the purpose of correlating particular building components with their energy performance characteristics was preferred as a quick way to model a large number of alternatives. In all cases the computed multiple-R and R-squared values were very high. Therefore, the following scenarios are examined, for three different life cycle periods: Scenario 1: 15 years, Scenario 2: 25 years, Scenario 3: 35 years.

2. LIFE CYCLE CONSIDERATIONS CONSIDERED IN THE OPTIMIZATION PROBLEM

A literature review has taken place in order to study details about the life cycle of the subsystems. In order to predict potential replacements during the life cycle period, the service lives of the subsystems that will be used are as follows (Stanford University 2005): Building Exteriors, Doors, and Windows: 80 years (lifetime), Mineral wool insulation profiles: 50 years, Photovoltaic panels: 25 years, HVAC systems: 15 years, Structural timber: 50 years (lifetime), Lighting control systems: 15 years.

The main considerations that were introduced in the optimization algorithms regarding the life cycle costs of the examined subsystems that also reflect a score of 50% economic efficiency and 50% environmental friendliness (BEES 2014), are the following:

- The mineral wool wall insulation profile is expected to end up in a landfill despite the fact that most of it can be recycled at the end of its useful life (BEES 2014).
- The PV array requires (usually twice a year) periodic removal of the dust concentrated on the panels that affects its performance, but this cost is negligible. Another cost that needs to be taken into account is the replacement of the inverter (indicative useful life of the inverter from the guarantee, however replacement every 5-10 years is a legitimate assumption) (Perdios 2011).
- The maintenance rates for the structural frames are considered to be equal to 4% of their initial value per year, with a start point five years after its construction (G. Bekas, D. N. Kaziolas, G. Stavroulakis, I. Zygomalas 2015). The maintenance rates for the building (excluding the structural frames) are considered to be equal to 1% of its initial value (therefore, unaffected by inflation rates) per year, with a start point five years after its construction (Stanford University 2005).
- For the HVAC systems, the maintenance rate is considered to be equal to 2% of their initial value (unaffected by inflation rates) per year (Nielsen 2002).
- In all scenarios, the residual NP values of the building components (windows, structural frames, walls, insulation profiles) will be ignored, reflecting a predefined design assumption that the building owner would not be interested to recycle them or reuse them at the end of its life cycle.
3. OPTIMIZATION PROCEDURE AND VARIABLES

After a finite element analysis that is conducted in Matlab, the building components are optimized by taking into account the following variables:

- Building envelope u-value. It is assumed that the walls have mineral wool as an insulation material and its thickness needs to be optimized. The lower and upper limits of the u values are as follows (Technical Chamber of Greece 2010, 2012): $0.20 < U_{\text{walls}} < 0.50$. U-value of the roof (it is assumed that the roof has similar components with the walls): $0.20 < U_{\text{roof}} < 0.45$. U-value of the ground floor (it is assumed that the roof consists of a reinforced concrete slab having a thickness equal to 20 cm and mineral wool as an insulation material whose thickness needs to be optimized): $0.20 < U_{\text{floor}} < 0.90$. U-value of the windows (each orientation was examined separately and the u value co-estimates the influence of the thermal bridges). Where: $2.20 < U_{\text{windows}} < 3.40$.

- Area of windows (south elevation and north elevation). In the simulation, it is assumed that area of windows at the south elevation and north elevation can have a value between 20 and 150 m².

- Area of windows (all other elevations; each orientation was examined separately). In the simulation, it is assumed that area of windows can have a value between 10.50 and 100 m².

- ggl value. The ggl values (hence, g values multiplied by 0.75; therefore reduced due to the contribution of the window frame that was considered to approximately occupy 25% of their total area) of windows should have a value between 0.29 and 0.55.

- Type of air conditioning system based on its energy parameters (These parameters are: SCOP (Seasonal coefficient of performance), SEER (Seasonal energy efficiency ratio), Power in kW). In the simulation, 25 different types of air conditioning systems with various energy parameters were considered.

- Number of A/C units. In the simulation, it is assumed that the number of terminals can have a value between 1 and 15.

- Variable examining whether the existence of lighting control is a cost-effective decision or not.

- Heating energy needs (in kWh) during the day of winter that exhibits the lowest levels of solar irradiation (a day during December which demonstrates very low expected values of solar radiation) (Perdios 2011) that can be covered by a photovoltaic panel array, in a way that a 4-day autonomy is also ensured. The photovoltaic panels were considered to have an optimal inclination (equal to 31° for the examined geographic location (Athens, Greece)). It is assumed that the peak power of the photovoltaic panel array could have a value between 0 and 20 kWp. The present study has used for its calculations merely one type of photovoltaic panels; panels with a known cost, area per kWp, efficiency and power parameters (245 Wp, nstc = 14.9%). This variable that was analyzed above can take a value between 0 and 20 kWp.

- Variable related to the spacing between the structural frames (4 possible choices corresponding to a number of equally spaced frames between 3 (30/3 = 10 m) and 6 (30/6 = 5 m)).

- Variable related to the form of the frames that compose the building whose change influences the number of bays (4 possible choices leading to a total number of beam-column elements between 13 and 19).

- Variables related to the lengths of the front beams. The front elevation is considered to be the one that is equal to 35 m, therefore the front beams are the ones along that direction. Each front beam length is considered to have a value between 3 and 6 meters, with a step size of 0.5 m.

- Variables related to the cross-sections of the ground floor columns that compose the structural frames. The following cross sections were considered in the simulation: $b = 100 \text{ mm}$, $b = 125 \text{ mm}$, $b = 150 \text{ mm}$, $b = 175 \text{ mm}$, $b = 200 \text{ mm}$, $b = 225 \text{ mm}$, $b = 250 \text{ mm}$, $b = 275 \text{ mm}$, $b = 300 \text{ mm}$, $b = 325 \text{ mm}$, $b = 350 \text{ mm}$, $b = 375 \text{ mm}$, $b = 400 \text{ mm}$, $b = 425 \text{ mm}$, $b = 450 \text{ mm}$, $b = 475 \text{ mm}$, $b = 500 \text{ mm}$, $h = 100 \text{ mm}$, $h = 125 \text{ mm}$, $h = 150 \text{ mm}$, $h = 175 \text{ mm}$, $h = 200 \text{ mm}$, $h = 225 \text{ mm}$, $h = 250 \text{ mm}$, $h = 275 \text{ mm}$, $h = 300 \text{ mm}$, $h = 325 \text{ mm}$, $h = 350 \text{ mm}$, $h = 375 \text{ mm}$, $h = 400 \text{ mm}$, $h = 425 \text{ mm}$, $h = 450 \text{ mm}$, $h = 475 \text{ mm}$, $h = 500 \text{ mm}$. (Where: b is the smaller dimension of the cross section, h is the larger dimension of the cross section).
-Variables related to the cross-sections of the upper floor columns that compose the structural frames. The cross sections that were considered, are the same as above.
-Variables related to the cross-sections of the front beams that compose the structural frames. The following cross sections were considered: b = 100 mm h = 200 mm, b = 110 mm h = 210 mm, b = 120 mm h = 220 mm, b = 130 mm h = 230 mm, b = 140 mm h = 240 mm, b = 150 mm h = 250 mm, b = 160 mm h = 260 mm, b = 170 mm h = 270 mm, b = 180 mm h = 280 mm, b = 190 mm h = 290 mm, b = 200 mm h = 300 mm, b = 210 mm h = 310 mm, b = 220 mm h = 320 mm, b = 230 mm h = 330 mm, b = 240 mm h = 340 mm, b = 250 mm h = 350 mm.
-Variables related to the cross sections of the back beams that compose the structural frames. The cross sections that were considered, are the same as above.

4. STRUCTURAL OPTIMIZATION OF THE TIMBER FRAME ELEMENTS

After a structural analysis and a selection of a series of frames based on a discrete optimization philosophy, the beams are checked according to Eurocode 5 (EN 1995-1-1:2004, 2004) for bending and shear. Similarly, each column is checked for compression, buckling and combined compression and bending. If a particular check fails conditional penalties are activated.

5. CONSTRAINTS RELATED TO THE ENERGY PERFORMANCE

The algorithm that was developed in Matlab took into account the following constraints:
- The power of the heating system and the cooling systems are determined and through constraints it is ensured that the heating and the cooling needs are covered for the most adverse days of the winter and the summer respectively.
- The overall average u value of the building should be lower than what is required by the relevant specification (KENAK) (Technical Chamber of Greece 2010, 2012).
- The total area of the building windows should ensure sufficient natural illumination and ventilation. According to the Greek building codes, this area should represent at least 10% of the total area of the building.

6. DETAILED EXAMPLE

6.1. Structural design

The building that was used in the simulation is a two-storey office building located on Athens, Greece. A plan view of the building along with a typical wall cross section are shown below (Figures 1-2):
The model follows a discrete optimization philosophy, where an initial estimation (Bekas 2015) of the number of structural elements needed (beams and columns), spacing among frames, types of cross sections for the beam and the column elements was made according to empirical criteria. In a similar manner, seven scenarios of variable-dependent beam lengths were introduced in the algorithm (all possible beam lengths from 3 to 6 meters for every 0.5 m) through the use of a penalty coefficient by which all the beam elements of which the building dimension that is equal to 35 m consists are multiplied. Its value is equal to: \( \alpha = \frac{35}{(\text{total length})} \), when the total length in a particular iteration is lower than 35 m and \( \alpha = \frac{(\text{total length})}{35} \) when the total length in a particular iteration is greater than 35 m. Furthermore, different scenarios of variable-dependent beam and column cross-sections were also introduced in the algorithm (Bekas 2015).

Other assumptions that were made in the model building are as follows:

Building height = 6 m, Loads on the frames = 2 kN/m², Structural timber cost (timber strength C 30): 350 € per kg. The structural members are considered to be made of natural pine wood with a specific weight equal to 460 kg/m³.

6.2. Energy performance design

At first it was assumed that the building will be used as an office building and the simulation and the calculations are based on KENAK. It assumed that both the wall of the building envelope and the windows are shaded to a known extent, therefore a reduction factor of 50% applies. The color of the walls is assumed to be a nuance of grey (Technical Chamber of Greece 2010, 2012). Furthermore, the following considerations were made: Coefficient accounting for the electricity cost in Euros/kWh = 0.07, Cost of the wall and roof profiles (inner and outer layers, excluding the mineral wool insulation): 25 € per m², Cost of the RC floor slab (inner and outer layer, excluding the mineral wool insulation): 101 € per m², Photovoltaic system cost (€ per kWp): 2933.7.

Heating Degree days (Geographic location: Athens) = 1930 (Base temperature inside the building: 22 °C) (Balaras 2011), (Technical Chamber of Greece 2010, 2012), Cooling degree days (Geographic location: Athens) = 679 (Base temperature inside the building: 24.5 °C) (Balaras 2011), (Technical Chamber of Greece 2010, 2012). The building is assumed to have a lighting system that consists of T5 lamps and its total power is equal to 3.50 kW, Lighting control system cost: 9375 €.
6.3. Objective function

The objective function is the sum of the cost of the following subsystems:

\[
\text{total cost} = \text{cost of insulation} + \text{Heating cost} \times \text{Number of years} + \text{Cooling cost} \times \text{Number of years} + \text{cost of A/C system} + \text{cost of windows} + \text{cost of roof} + \text{cost of walls} + \text{HVAC system maintenance and replacements} + \text{general building maintenance} + \text{cost of the floor slab} + \text{cost of photovoltaic array} + \text{PV array maintenance and replacements costs} + \text{frame costs} + \text{frame maintenance} + \text{illumination costs} + \text{illumination control system cost} + \text{illumination control system replacement cost} + \sum_{i=1}^{\sigma} p_i
\]

The terms \( p_i \) represent all the aforementioned constraints that concern the structural and energy design problem. The value of the factors \( p_i \) is conditional; either equal to very high values exceeding the highest possible cost of the building or equal to zero in all other cases.

7. OPTIMIZATION PROCESS AND RESULTS

For the optimization calculations both the methods of simulated annealing as well as genetic algorithms have been. The second method in all scenarios was found to be faster. Nevertheless, the first method is more suitable after some relaxation of the optimization search space (when several initial optima have been found). The approach that has been followed made use of the optimization toolbox of Matlab. The authors have generally preferred the preset options of the optimization toolbox and some critical details about the optimization procedure are as follows:

- SIMULATED ANNEALING: The temperature function updates itself exponentially, the maximum number of function evaluations was set as equal to 150000 and a fast annealing function was used.

- GENETIC ALGORITHMS: Fitness scaling is based on rank, the initial population can have iteratively any size from 100 to 1000, the function selection is stochastic and uniform, a scattered crossover function is used, the mutation function is constraint dependent. The maximum number of iterations was set equal to 1000.

Each optimization scenario necessitates at least 10 trials to ensure that the optimum found at each trial constitutes a good global solution. The results of the trials were compared and the best solution among the optima generated by each trial is presented below. The optimization results (Table 1, Table 2, Table 3, Table 4) that were produced by running the scenarios 1, 2 and 3, are shown in the appendix A1. An interpretation of the results leads to the following conclusions:

- The total optimal cost for the structural frames is 10,097,325 € (initial construction cost). By taking the maintenance rates into account the final cost for the structural subproblem escalates to: 14,136,255 € (Scenario 1), 18,175,185 € (Scenario 2), 22,214,115 € (Scenario 3) (construction cost of structural frames and maintenance costs). No optimal front beam length was less than 4.62 m. Moreover, the optimal total number of elements in the building elevation that is equal to 35 m, is 13 and the optimal spacing between frames -in the building elevation that is equal to 30 m- is 10 m.

- For all scenarios the most cost-effective combination for the HVAC system is that of 3 units composed of air-conditioning systems that fall into the A-energy class category. Specifically, it has the following characteristics: SEER = 5.50, SCOP = 3.86, Power = 6.80 kW, cost = 2566 €. Other buildings would possibly require a different optimal combination, however, it seems unlikely that a combination of low energy class A/C systems (below the A-energy class category) could be the optimal solution.

- For all scenarios it seems more preferable to primarily place the larger total area of windows on the north elevation (hence the elevation with the least amount of solar gains) and secondarily on the south elevation (hence the elevation with the highest amount of solar gains). This is related to the chosen geographic location. For both scenarios the most cost-effective decision is that of double-glazed windows with high ggl values. This assumption was made after applying a nearest neighbor classification to the sample of windows that was subject to multiple linear regression. Using windows with low ggl window coefficients in other cases of smaller buildings could possibly aid cost effectiveness, since the results demonstrate a preference for the lowest possible ggl values. Due to the occupancy profile of the building it seems that for the examined rates and periods of times the optimal mechanical equipment does not change during the
examined life cycle periods. This is not necessarily the truth for other types of buildings, however, it seems that due to the type and occupancy levels of the building considered in the simulation, a fast pay-off is attained that excludes other types of photovoltaic arrays and combinations of A/C systems from being the optimal solution.

-For all the examined scenarios using a lighting control system is not a cost-effective decision.

8. CONCLUSIONS

Modern computational optimization allows us perform structural optimization including the entire life cost of a structure. It is expected that this trend will become more popular in future and will allow for more efficiency in the usage of resources.

9. APPENDIX A1

Table 1: Results of the optimization calculations (Energy/LCC design subproblem).

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>A/C terminals</th>
<th>Illumination control</th>
<th>South (m)</th>
<th>South (m)</th>
<th>South (m)</th>
<th>East (m)</th>
<th>East (m)</th>
<th>North (m)</th>
<th>North (m)</th>
<th>West (m)</th>
<th>West (m)</th>
<th>PV Array (kWp)</th>
<th>Cost €</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>Not needed</td>
<td>0.292</td>
<td>31.808</td>
<td>149.926</td>
<td>10.121</td>
<td>18.232</td>
<td>0.440</td>
<td>0.496</td>
<td>0.366</td>
<td>0.412</td>
<td>0.492</td>
<td>3.395</td>
<td>7199.98</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>Not needed</td>
<td>0.293</td>
<td>40.127</td>
<td>149.943</td>
<td>10.057</td>
<td>10.022</td>
<td>0.438</td>
<td>0.447</td>
<td>0.428</td>
<td>0.497</td>
<td>0.443</td>
<td>3.393</td>
<td>1.160.157</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>Not needed</td>
<td>0.291</td>
<td>40.044</td>
<td>149.916</td>
<td>10.089</td>
<td>10.088</td>
<td>0.425</td>
<td>0.494</td>
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<td>0.459</td>
<td>0.459</td>
<td>3.394</td>
<td>1.589.532</td>
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</table>

Table 2: Results of the optimization calculations (Structural design subproblem, ground floor).

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Number of intermediate frames</th>
<th>Column 1</th>
<th>Beam 1</th>
<th>Column 2</th>
<th>Beam 2</th>
<th>Column 3</th>
<th>Beam 3</th>
<th>Column 4</th>
<th>Beam 4</th>
<th>Column 5</th>
<th>Beam 5</th>
<th>Column 6</th>
<th>Beam 6</th>
<th>Column 7</th>
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</thead>
<tbody>
<tr>
<td>All</td>
<td>2</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
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<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td></td>
</tr>
</tbody>
</table>

Table 3: Results of the optimization calculations (Structural design subproblem, first floor).

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Column 1</th>
<th>Beam 1</th>
<th>Column 2</th>
<th>Beam 2</th>
<th>Column 3</th>
<th>Beam 3</th>
<th>Column 4</th>
<th>Beam 4</th>
<th>Column 5</th>
<th>Beam 5</th>
<th>Column 6</th>
<th>Beam 6</th>
<th>Column 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>All</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
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<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
<td>125 mm</td>
</tr>
</tbody>
</table>

129
Table 4: Results of the optimization calculations (Structural design subproblem, cross sections of the back beams' and lengths of the front beams).

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Back beam 1</th>
<th>Back beam 2</th>
<th>Back beam 3</th>
<th>Back beam 4</th>
<th>Back beam 5</th>
<th>Back beam 6</th>
<th>Front beam length 1</th>
<th>Front beam length 2</th>
<th>Front beam length 3</th>
<th>Front beam length 4</th>
<th>Front beam length 5</th>
<th>Front beam length 6</th>
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<tbody>
<tr>
<td></td>
<td>b = 130 mm, h = 230 mm</td>
<td>b = 130 mm, h = 230 mm</td>
<td>b = 130 mm, h = 230 mm</td>
<td>b = 130 mm, h = 230 mm</td>
<td>b = 130 mm, h = 230 mm</td>
<td>b = 130 mm, h = 230 mm</td>
<td>5.28</td>
<td>7.93</td>
<td>4.62</td>
<td>7.27</td>
<td>5.28</td>
<td>4.62</td>
</tr>
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</table>

REFERENCES

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Seismic Vulnerability and Fragility of Industrial Steel Buildings Affected by the Emilia-Romagna Earthquake

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Abstract. The recent L’Aquila (2009) and Emilia-Romagna (2012) Italian earthquakes have highlighted the vulnerability of recently erected buildings, with particular reference to industrial ones. Although steel buildings have usually demonstrated a good behaviour under earthquakes, with limited damages and rare cases of collapse, they still represent a structural typology at risk due to the significant exposure connected to the importance of the resources they host. In the paper a parametric study on several industrial steel buildings, different for typology, geometrical dimensions, seismic zone and snow geographic area, has been done through pushover analyses. The results have allowed to plot vulnerability curves, which have been compared to seismic fragility curves derived from literature studies. The comparison among curves have allowed to estimate the effectiveness of the theoretical relationships, as well as to evaluate the seismic damages suffered by investigated structures under dissimilar grade earthquakes.

1. INTRODUCTION

The most of the Italian built heritage consists of buildings constructed without appropriate anti-seismic design rules, as they were erected in those regions that, before of the new technical legislative measures, were not considered as seismic zones. The recent L’Aquila (2009) and Emilia-Romagna (2012) Italian earthquakes have highlighted the vulnerability of the Italian built heritage, with reference not only to the historical buildings, but also to the recently erected industrial constructions. In this direction, an example is the town of Mirandola (district of Modena), where 80% of industrial buildings, mostly made of pre-stressed reinforced concrete, was destroyed or considered to be unfit for use after seismic events occurred on 2009 May 20th and 29th.

The constructive peculiarity of these buildings, that are much widespread all over the Italian country, is the easy erection process based on hinged beam-to-column joints. Contrary, this building typology is particularly sensible to horizontal actions, especially when additional structural systems, such as cranes and pallet racks, are placed inside them. In this paper the attention is dedicated to the industrial steel buildings that, despite they suffered limited damages compared to those of pre-stressed r.c. structures, were designed without suitable seismic rules introduced in Italy only since 2003. Generally, the industrial steel frame buildings assure high levels of reliability in case of earthquake, considering that many of them, although they were not designed to resist seismic actions, remained either unharmed or suffered limited damage. The only cases of collapse are mostly conditioned by the failure of pallet racks that, often reaching considerable heights, represent real structures inside the industrial building and, therefore, they need an appropriate seismic design. Therefore, in the case of industrial buildings, the issue of the life safety is associated to the theme of the safeguard of the values exposed at
risk and, above all, to the continuity of business activities after the earthquake. In the paper the seismic behaviour of some industrial steel buildings has been assessed through non-linear static analyses which allowed to plot, starting from pushover curves, their vulnerability curves, used to know exhaustively the expected seismic damages suffered under earthquakes of different intensity with reference to different limit states. Considering the difficulty to investigate all varieties of existing industrial buildings, only some of the most common types detected in Italy, representative of the industrial steel buildings heritage, have been examined. In particular, identification and analysis of a number of typical buildings, different each other for geometric dimensions and constraint conditions, have been done, as it will be shown in the next Sections.

2. SELECTION OF STRUCTURAL TYPOLOGIES

Generally, one-story buildings for industrial use are characterised by regular plan layouts having large spans with minimum encumbrance of structural elements. Usually, the longitudinal distance among columns ranges from 5 to 15 m, while the transversal one varies from 15 to 30 m. The inner height between the work plane (about 1 m far from the floor) and the lowest point of the roof intrados is often contained between 5 and 15 m. Obviously, these dimensional values are only for guidance and they are usually variable depending on the types and structural elements employed. In order to be able to assess more accurately the variability fields of these buildings, a significant number of projects and real case studies, from which the most recurrent in plan and in elevation average sizes for each type are derived, have been collected. Subsequently, the selected types have been divided into classes depending on both the type and the slope of roof beams. Finally, for each of the case studies selected, lattice girders or full web beams have been considered. The structural schemes adopted have been designed on the basis of the regulations at the time of their realization through a simulated design process. After defining the individual sub-models (geometrical, mechanical and loading), which together contribute to define the structural numerical model, the simulated design has been carried out considering the variability of various parameters associated to the constraint conditions, the dimensional aspects and the geographical area where structures are located.

The geometrical model has been defined considering the variability of the most common structural schemes symbolising the industrial steel buildings. After identifying the more representative model of each investigated structural type, having given average dimensions, different schemes of the same structural system, but with different sizes (columns (h) and roof beams (h') depths) and constraint conditions (hinge(H) or encastre(E)), have been numerically examined. All schemes subjected to the seismic vulnerability assessment are shown in Figure 1, where the investigated frames are identified with acronyms according to both the type of structural elements used for roofing systems (plane lattice beams (PLB), plane beams (PB), double slope lattice beams (DSLB) and double slope beams (DSB)) and the constraint conditions (H - E).

Then, such schemes have been respectively identified by the letters A, A’, B and B’ and, for each of them, two different constraint conditions have been contemplated, leading to the definition of the following eight structural systems: A_H, A_E, A’_H, A’_E, B_H, B_E, B’_H and B’_E. For the first two structural patterns (A_H and A_E), it has been also hypothesised a variability of the dimensions h and h’ for the execution of a more wide parametric analysis. In such a case, starting from a reference case study having assigned average dimensions (l = 20 m, h = 9 m and h’ = 2 m), it has been expanded the field of investigation considering possible variations of the geometric parameters h and h’; which gave rise to thirty cases of analysis (Figure 1). Therefore, considering also other six cases where any parametric analysis has been performed, a total of 36 structural models have been analysed.

The mechanical model has been implemented by defining the nature of the materials used. In the case in question it has been used a S275 steel type with characteristics intermediate between the mild steels commonly used in the constructive practice. Finally, with regard to the loads acting on the structures and, therefore, to the loading model, gravitational actions linked to the masses of structural and non-structural elements have been defined and the characteristic values of the variable actions due to snow, wind and earthquake have been calculated. Actually, all the
parameters to be analysed for an extensive parametric analysis would be significantly numerous, but this paper refers to a particular selection of some typological, dimensional and geographical variables only. Geographic variability is linked to the location of the structural system and it influences the entity of seismic and variable (wind and snow) loads considered. With reference to these former loads, by taking into account the three climatic zones representative of the Northern, Central and Southern regions of the Italian country, the number of analyses have been increased from 36 to 108.

After defining the geometry and global dimensions of the various structural systems to be investigated, considering all possible loads applied, the design of individual profiles to be employed have been done. The structures of the lattice girders have been obtained by coupling two profiles with C or L cross-sections: the former (UPN) has been assumed for the upper (U.B.) and lower (L.B.) beams, while the latter has been chosen for the diagonal members (D.M.). For the structural type with plane beam (P.B.) HEB700/800/900 profiles, with a variability essentially conditioned by the geographical location assumed for each structural scheme, have been used.

As a first step, the values of the more heavy stresses related to the different load combinations for each structural element have been collected, so to identify the most suitable profiles to be adopted for each individual component of the analysed systems. This design phase has been performed through a simulated design according to the prevailing regulations at the construction time of these structures, essentially based on the Allowable Stress method (Table 1). The structure check has been done by means of the actual code, allowing to confirm or not
the dimensions originally assigned to frame members (Table 2). This check phase has been performed through the finite element analysis program SAP 2000.

From the comparison between the frames dimensioned according to the two different code approaches, a minimum difference of weight, with an average percentage difference equal to 1.26%, emerges. This means that the seismic actions do not affect the design of these structures, whose design is essentially dictated by the wind loads only. As a consequence, high levels of structural reliability of these structural types also according to the new seismic code are assured.

### Table 1: Weights of AH structures designed according to the old Italian technical code (CNR 10011)

<table>
<thead>
<tr>
<th>Frames</th>
<th>L.B.</th>
<th>U.B.</th>
<th>D.M.</th>
<th>Columns</th>
<th>Weights [KN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>AH121 (l=20, h=6, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 200</td>
<td>90x10</td>
<td>HEB 220</td>
<td>80.1</td>
</tr>
<tr>
<td>AH122 (l=20, h=6, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 220</td>
<td>90x12</td>
<td>HEB 260</td>
<td>85.3</td>
</tr>
<tr>
<td>AH123 (l=20, h=6, h’=2.5)</td>
<td>UPN 100</td>
<td>UPN 180</td>
<td>90x7</td>
<td>HEB 280</td>
<td>78.5</td>
</tr>
<tr>
<td>AH221 (l=20, h=9, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 200</td>
<td>90x10</td>
<td>HEB 280</td>
<td>91.4</td>
</tr>
<tr>
<td>AH222 (l=20, h=9, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 220</td>
<td>90x12</td>
<td>HEB 300</td>
<td>98.4</td>
</tr>
<tr>
<td>AH223 (l=20, h=9, h’=2.5)</td>
<td>UPN 100</td>
<td>UPN 180</td>
<td>90x7</td>
<td>HEB 360</td>
<td>93.3</td>
</tr>
<tr>
<td>AH321 (l=20, h=12, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 200</td>
<td>90x10</td>
<td>HEB 320</td>
<td>104.1</td>
</tr>
<tr>
<td>AH322 (l=20, h=12, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 220</td>
<td>90x12</td>
<td>HEB 360</td>
<td>112.5</td>
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<tr>
<td>AH323 (l=20, h=12, h’=2.5)</td>
<td>UPN 100</td>
<td>UPN 180</td>
<td>90x7</td>
<td>HEM 320</td>
<td>130.9</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>874.5</td>
</tr>
</tbody>
</table>

### Table 2: Weights of AH structures designed according to the actual Italian technical code (M.D. 08)

<table>
<thead>
<tr>
<th>Frames</th>
<th>L.B.</th>
<th>U.B.</th>
<th>D.M.</th>
<th>Columns</th>
<th>Weights [KN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>AH121 (l=20, h=6, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 180</td>
<td>90x9</td>
<td>HEB 220</td>
<td>77.6</td>
</tr>
<tr>
<td>AH122 (l=20, h=6, h’=2.5)</td>
<td>UPN 160</td>
<td>UPN 200</td>
<td>90x10</td>
<td>HEB 260</td>
<td>84.7</td>
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<tr>
<td>AH123 (l=20, h=6, h’=2.5)</td>
<td>UPN 100</td>
<td>UPN 140</td>
<td>80x10</td>
<td>HEB 280</td>
<td>78.1</td>
</tr>
<tr>
<td>AH221 (l=20, h=9, h’=2.5)</td>
<td>UPN 140</td>
<td>UPN 180</td>
<td>90x9</td>
<td>HEB 280</td>
<td>89.9</td>
</tr>
<tr>
<td>AH222 (l=20, h=9, h’=2.5)</td>
<td>UPN 160</td>
<td>UPN 200</td>
<td>90x10</td>
<td>HEB 300</td>
<td>93.7</td>
</tr>
<tr>
<td>AH223 (l=20, h=9, h’=2.5)</td>
<td>UPN 100</td>
<td>UPN 140</td>
<td>80x10</td>
<td>HEB 400</td>
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<tr>
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<td>UPN 140</td>
<td>UPN 180</td>
<td>90x9</td>
<td>HEB 400</td>
<td>109.9</td>
</tr>
<tr>
<td>AH322 (l=20, h=12, h’=2.5)</td>
<td>UPN 160</td>
<td>UPN 200</td>
<td>90x10</td>
<td>HEM 320</td>
<td>122.9</td>
</tr>
<tr>
<td>AH323 (l=20, h=12, h’=2.5)</td>
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<td>UPN 140</td>
<td>80x10</td>
<td>HEM 320</td>
<td>130.6</td>
</tr>
<tr>
<td>Total</td>
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<td></td>
<td></td>
<td>889.5</td>
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</tbody>
</table>

3. **NON-LINEAR ANALYSIS AND CAPACITY CURVES**

The seismic response of the structures under study has been evaluated by non-linear static analyses carried out by using the calculation program SAP2000. A lumped plasticity modelling for structural elements has been adopted by identifying areas of plasticity and defining the behaviour of the plastic hinges in terms of generalized force-displacement curves. The non-linear behaviour of beams has been taken of elastic-plastic type with an ultimate limit of rotation $\theta_u = \theta_y$. For columns, instead, it has been defined a domain of resistance considering the simultaneous application of compressive and bending stress. The analysis has been conducted under displacement control, assuming as control point the geometric centre of gravity of the roofing.

Given the considerable number of frames analysed, their subdivision into classes has been done and a capacity curve representative for each of them has been plotted. The cases presented have been divided according to the types and the constraint conditions, also considering further variables, such as the geometric dimensions and the geographical location. For the early two
cases $A_H$ and $A_E$, indicative of the type with plane lattice beams (PLB), a parametric analysis has been conducted considering the height of columns (6m, 9m, 12m) and the roofing systems one (1.5m, 2.0m, 2.5m, 3.0m, 3.5m) as variable parameters. Therefore, a further subdivision of each of the above-mentioned schemes in the three subcases ($A_{H1}$, $A_{H2}$, $A_{H3}$ and $A_{E1}$, $A_{E2}$, $A_{E3}$) has been made (see Tables 1 and 2). For all the other cases ($A'$, $B$ and $B'$), where other additional variabilities, other than the different constraint conditions and the different geographical location, have not been considered, three representative capacity curves, one for each structural type, have been derived. Therefore, in accordance with the preceding subdivision, nine capacity curves representative of the different frames analysed, have been obtained and compared each to other, so to grasp typical behaviour and peculiarities of the different classes of structures examined.

The analysis results have been represented in terms of base shear, normalised to the total weight of the structural system, versus the displacement of a control point coincident with the gravity centre of the roofing. The representative capacity curve, characterised by a bilinear shape (typical of a SDOF system), corresponds to the average capacity curve of the family of curves obtained for each class of frames (Figure 2).

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4. FRAGILITY AND VULNERABILITY CURVES

In case of earthquake, each structural system is exposed to a risk correlated to both losses and the degree of damage that it can exhibit. In the seismic risk analysis, in fact, it is necessary to translate the knowledge of the built vulnerability in the damage that can occur as a result of earthquakes having different magnitudes. The seismic risk parameter of a system $R$ can be expressed as a function of its vulnerability $V$, of the parameter $s$, related to the severity of the...
earthquake, and of the parameter \( d \), intended as measure of the damage, by means of a correlation law \( R = R(V, d, s) \).

One of the tools for the determination of the structure seismic risk and, therefore, of the above-mentioned functional link, is represented by the fragility curves. They provide the probability of a structural system exposed to a seismic input assigned to overcome certain levels of damage. In this paper two procedures have been used for deriving fragility curves of investigated structures. The first is a *discrete / manual* procedure, punctually derived from the curves of capacity, which gives rise to those that will be defined as vulnerability curves, whereas the second procedure is an *analytical* method based on some literature indications. For the seismic reliability assessment of the structures, the current research trends are directed towards rigorous probabilistic approaches, involving both random and deterministic variables that are often difficult to be considered at all in the project. For this reason, it has been herein proposed and developed a procedure simpler than that based on analytical formulations. The procedure presented allows to validate, through appropriate comparisons, the effectiveness of the fragility curves to estimate the seismic damage of structures subjected to earthquakes of different intensity.

The *discrete / manual procedure* evaluates the structural capacity and compares it directly with the demand related to the particular seismic event on the basis of a limit state considered. In this paper, based on the FEMA 356 guidelines, three performance levels (IO, LS, NC), characterised by appropriate values of inter-story drifts, have been taken into account. Starting from the structural behaviour in the non-linear static field, some damage levels of the structure, corresponding to the limits above defined by FEMA 356 provisions, have been defined and, for each earthquake with a given hazard level, the expected building damage can be estimated by simply correlating the capacity displacement (or inter-story drift) with the demand one. The ratio between the demand parameter and the capacity one is then correlated to the damage levels of the EMS 98 scale normalised in the range [0-1]. By varying the earthquake intensity and, therefore, the seismic demand, for each of the three limit states considered, the above ratio is calculated, allowing to plot step-by-step the structure fragility in a simple way, which is herein called vulnerability curve (Figure 3).

![Fig 3: Vulnerability curves of the structural system AE2](image-url)

Wanting to evaluate the propensity at damage of the examined industrial steel buildings considering the random nature of the earthquake, a peak ground acceleration of the demand spectrum variable between 0.01g and 1ghas been considered. It is interesting to note that, as for this procedure, the first examples of fragility or *vulnerability* curves were referred to a conventional scheme that simplified the assessment procedure of the seismic vulnerability [6].

So, the general procedure involving a number of points of the curve can be replaced by a simpler method, which is based, in absence of additional information, on two parameters only: the collapse point (acceleration \( y_c \)) and the damage starting point (acceleration \( y_i \)) of the building (Figure 4). Thus, the curve was obviously undetermined, but it could be adequately represented by a linear conventional trend.
The *analytical* procedure defines, according to the variation of seismic intensity, the structure probability of reaching or exceeding a particular limit state. In mathematical terms, this is expressed by the function of conditional probability $P[SL|I]$, where $SL|I$ is a symbol indicative of achieving or exceeding the assigned limit state when the seismic intensity value ($I$), which can be represented under form of PGA, spectral acceleration, etc, is fixed.

The ways to define the damage thresholds are numerous: one of these is defined as a function of the two points representative of the push-over curve, that is the yielding displacement $D_y$ and the ultimate one $D_u$. The approach proposed in this paper, instead, correlates the limit states at appropriate drift values, in line with the provisions of other scientific researches on the subject [5]. As a seismic parameter (measure of the intensity $I$) the spectral displacement $S_d$ has been adopted, because the capacity curves have been converted into the ADRS format in order to be able to compare in an easy manner the capacity values with those of the seismic demand represented by the response spectrum. As a result, the fragility curves are obtained mathematically using the following equation:

$$P[SL/I] = \Phi \left[ \frac{1}{\beta} \cdot \ln \left( \frac{I}{I_{SL}} \right) \right] \quad (1)$$

The equation (1) defines the probability of occurrence or exceeding the state limit considered by means of a log-normal cumulative distribution, where:
- $\Phi$ is the standard normal distribution function;
- $I$ is the measurement unit of the intensity (or intensity measure);
- $I_{SL}$ is the median of the intensity measure for which the building reaches a given limit state;
- $\beta$ is the standard deviation of the intensity natural logarithm for the limit state considered, assumed equal to 0.6 according to the indications reported in.

According to this method, each fragility curve is characterized by two parameters: the first is the average value of the intensity measure responsible of reaching the limit state threshold and the second parameter is the relative standard deviation. For each structural system it is possible to trace more fragility curves, each of them associated to a predetermined limit state. An example of fragility curves constructed according to the previous analytical procedure is reported in Figure 5, where for the same structural system (case $A_{E2}$) three curves obtained for three different limit states (IO, LS and NC) are simultaneously reported. Later on, the seismic safety of structures examined, placed for the sake of example in Mirandola, one of the sites most affected by the Emilia-Romagna earthquake, has been assessed and, finally, the reliability of the fragility curves deriving from literature analytical formulations has been proved.
As regards to the first aspect, it is noticed that, at the life safety limit state, the structures, although designed for vertical loads, meet the safety requirements of the current regulations for seismic-resistant structures. In fact, they reach the collapse under an acceleration value greater than that of the seismic zone with the greatest intensity, that is characterized by a peak ground acceleration of 0.35g.

On the other hand, with reference to the second key question, the discrete fragility curves are manually defined according to a procedure much more laborious than the analytical one, the latter requiring the knowledge of a smaller number of factors to more practically assess the safety of structures. The comparisons highlight that the analytical curves show values of expected damage greater than the values obtained by discrete curves. Therefore, the literature fragility curves represent a conservative prediction method of the structural safety of steel industrial buildings.

5. CONCLUSIONS

The non-linear analyses carried out in order to determine the capacity and the fragility curves of a set of steel one-storey building typologies, that are very spread on the Italian country, have led towards the following conclusions:
1. The seismic action has a little influence on design of industrial steel structures: they are more influenced by the action of wind loads rather than those of the earthquake.
2. The structures, although are designed only for gravity loads, have demonstrated a good behaviour under earthquake. In fact, they reach the collapse for an acceleration value higher than 0.35g, that is the maximum value of the PGA for the highest Italian seismic hazard area.
3. The analytical fragility curves overestimate the damage predicted by the discrete vulnerability curves. Thus, they are a method on the safe side in forecasting the steel industrial building collapse under seismic actions.
4. The performed analyses, although they have provided interesting considerations about the seismic hazard of the steel one-storey buildings for industrial use investigated, represent only the first step towards the characterization of all types of this structural typology. As a consequence, additional analyses could be carried out by taking into account the variability of both the gravity loads and the steel grades used, as well as of the geometric parameters defining the structural schemes of the inspected typologies.
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**Abstract.** Within the activity program of the Zero Emission Buildings (ZEB) research centre in Trondheim, two new laboratories have been built at the NTNU campus of Gløshaugen in Trondheim. In the Living-LAB materials and components for a low carbon built environment will be tested in relation to different users’ behaviour. The LivingLAB was designed in order to be representative, for dimension and construction, of a regular size Norwegian detached house built according to ongoing passive house standards. Specific solutions adopted in the house can be easily replicated in Norwegian detached houses, representing 29% of the total building stock. High performance windows, equipped with VIP insulation, have been coupled with automated control systems and PCM panels able to stabilize temperature fluctuations when intermittently heating is used. This article will investigate the environmental performance of the house with specific focus on the use of advanced materials such as VIP, PCM and Aerogel. Dynamic simulations will be carried out in order to analyze the environmental performance of the house towards energy efficiency. Results will be discussed considering the environmental impact deriving from the use of the same materials in a life cycle perspective.

1. INTRODUCTION

The ZEB Living LAB (figure 1) is the result of a complex multidisciplinary design process in which students, researchers and industrial partners have been called to collaborate in the design of a solar powered house able to produce more energy than it consumes. The development of the project at NTNU has been integrated within the activities of the Master of Science in Sustainable Architecture in a sort of competition internal to the master program. The winning concept was further developed inside the ZEB research centre during twelve months of design in which flexibility was further enhanced and addressed to solve many other issues. The LivingLAB was originally conceived as an off-grid construction module that can be located in rough nature or also serve as extension of detached family houses able to compensate their lacks in energy efficiency (figure1) (e.g. detached wooden houses represent the most energy demanding typology in Norway).
Fig 1. The ZEB LivingLAB at NTNU.

As one of the pilot buildings built inside ZEB, the LivingLAB was designed in order to accomplish ZEB targets generating enough renewable energy to balance the whole environmental footprint of the building[3]. Four different levels of Zero Emission Buildings have been identified at the ZEB centre, each of them aiming, with increasing ambition, at balancing emissions due to:

- O/Eq. – the use of the technical equipment.
- – the whole building operation, including appliances.
- O+M - embodied in the used material.
- O+M+C – due to the construction and demolition of the building.

Balance between energy needed throughout the whole life cycle of the building and energy generated from the integrated renewable energy systems has been estimated using SIMIEN and PVSyst respectively. Simulations conducted by I. Graabak, N. Feilberg collected in the document "CO₂ Emissions in different scenarios of electricity generation in Europe" showed that it is possible to achieve a 90% reduction of CO₂ emissions by 2050. If this trend would be true for the whole life cycle of the building (60 years), the generation of 1 KWh of electricity within the European Energy network would have an impact of 132g of CO₂eq (this represents the stationary factor commonly used at the ZEB research centre at NTNU). Under this hypothesis, the operational carbon footprint of the LivingLAB was calculated as 522.08 KgCO₂eq, while the total carbon footprint, including embodied emissions in materials and construction, was calculated as 1054 KgCO₂eq. Simulations run in PVSyst showed that the combination of a 12Kwp photovoltaic system, and a ground heat pump connected to a 150 m heat exchanger in the ground and a 4 m² solar thermal system should have been sufficient to reach the forth level of ZEB, OMC.

Information extracted throughout the whole design process from simulation software represented a meaningful platform for the architectural design of the LivingLAB. In this work main attention is addressed towards the building construction and more specifically into the use of advanced materials – such as PCM, VIP and Aerogels - introduced during the later stage of the design process in order to optimize the building environmental performance and reduce the operational carbon footprint of the building. Analyses were conducted in order to verify how the use those materials impacted the balance between operation and embodied emissions of the LivingLAB.
2. THE ZEB LIVINGLAB.

The ZEB Living LAB is a single family house with a gross volume of approximately 500 m$^3$ and a heated floor area of approximately 100 m$^2$. Flexibility of the plan (Figure 2) was particularly addressed towards the possibility of allocating many different programs within the building surface (young/old couples, families or housing for students). The plan of the Living Lab is organized in two main zones: a living area facing south and a working/sleeping area towards north. The entrance is located in the south west corner, and through a filter space that hosts a wardrobe, the user gets access to the living room. The kitchen is located at the opposite end of the living room. An automated double skin (ventilated) window is installed in the living room, covering the largest part of the south facade. At the center of the north zone, there is a shared studio area, equipped with a long writing desk and with an automated window. The double skin window facing south and the north-facing single skin window are equipped with motor that can open them when cross ventilation is needed in order to avoid overheating. Two couples of roof windows are installed above the kitchen and the mezzanine (sitting above one of the bedrooms); these are also equipped with electric motors for automated/manual operation for extracting exhaust air.

![Fig 2. Plan of the LivingLAB.](image)

Two bedrooms (one facing east and one facing west) are located at the two sides of the studio room. The technical room (accessible from outside the building), bathroom (accessible from the studio room) and the kitchen have been placed all along the central spine of the building in order to optimize the distribution of the technical equipment. A small mezzanine is placed above the west bedroom and it is equipped as sleeping/working area for guests or as play area for children.

2.1. Building construction.

The LivingLAB construction has been optimized through a set of simulations and resulted into a highly insulated envelope characterized by a glass ratio of around 20%. Walls, floors and roofs are made out of a conventional wooden-frame structure with a double layer of rock wool insulation for a total of 40, 40 and 45 cm respectively (and a U-value of 0.11, 0.10 and 0.11 W/m$^2$K, respectively). Paper in between the two Rockwool layers aim at minimizing convection phenomena that would limit the envelope thermal resistance.

Ninety squared meters of PCM-based boards have been installed at the indoor interface of the roof slopes (just behind the finishing, wooden cladding)(Figure 3). The use of phase changing materials, with a melting point of 22°C, aims at minimizing the risk of overheating due to the lightweight construction of the building.
Windows used in the Living LAB are characterized by markedly low u-values (0.65 to 0.69 W/m²K for the south window, depending on the ventilation feature; 0.97 for the north window; 0.80 W/m²K for the windows towards east and west; 1.00 W/m²K for the roof windows). The total glass ratio of the building was reduced in a later stage of the design process from 40 to 20% of the total heated area, fulfilling ongoing Norwegian Passive house standards NS3700. Glazed area was reduced varying the height of the north window and replacing one side of the sliding doors glasses towards east and west with vacuum insulation panels.

Originally the Living LAB should have been prefabricated in volumetric components (Figure 4) limiting construction time and related environmental impact. The geometry of the two roof components was optimized in order to limit self-shading and maximise renewable energy generated from the integrated photovoltaic system towards south. On the north side of the roof components two large surfaces of translucent aerogels aimed at catching and distributing diffused light from north. Aerogel surfaces were cut when transparent areas were reduced in order to limit heat losses. The respect of minimum daylight requirements of 2% under standard CIE sky condition were in the meanwhile verified through Radiance.
3. ENVIRONMENTAL PERFORMANCE VS ENVIRONMENTAL IMPACT ANALYSES. METHODOLOGY.

In this work environmental impact related to the use of advanced material, such as VIP, PCM and Aerogels, in the LivingLAB is calculated taking into account both their influence on the building environmental performance and the embodied emissions in materials used. Their ability to optimize the building environmental performance, reducing the operational carbon footprint, has been quantified modelling the building in DesignBuilder, a dynamic simulation software using the EnergyPlus thermal engine. Data related to embodied emissions in materials have been collected from a report developed by Inman and Wiberg at the ZEB research centre in Trondheim using the Ecoinvent processes through SimaPro Analyst version 8.0.5; transportation mode and distance travelled have been calculated specifically for the LivingLAB project assuming EURO 5 transport means. Data relative to the eventual use of Aerogel windows have been instead extracted from the work of N. Lolli titled “Life cycle analyses of CO2 emissions of alternative retrofitting measures”. According to analyses conducted by Inman and Wiberg, VIP and PCM in the LivingLAB account respectively for 2.2 and 0.7 % of the whole LivingLAB environmental footprint. Data collected from their work have been elaborated through an excel sheet appositively conceived for defining alternative scenarios related to the use of advanced materials in the LivingLAB project.

3.1. Model settings.

Environmental performance analyses have been run in DesignBuilder, where operation times and internal loads of the building have been fixed according to Norwegian Passive house standards NS 3700. Envelope characteristics have been adjusted in accordance with u-values and layering reported above while airtightness has been set as 0,5 in accordance to NS3700. Internal comfort conditions assumed as environmental performance target are summarized in table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maxair speedwinter</td>
<td>cm/s</td>
<td>15</td>
</tr>
<tr>
<td>Maximum operating temperature during summer</td>
<td>°C</td>
<td>26</td>
</tr>
<tr>
<td>Minimum operating temperature during winter</td>
<td>°C</td>
<td>21</td>
</tr>
<tr>
<td>Minimum floor temperature</td>
<td>°C</td>
<td>19</td>
</tr>
<tr>
<td>Minimum daylight factor</td>
<td>%</td>
<td>2</td>
</tr>
<tr>
<td>Heating set point (setback)</td>
<td>°C</td>
<td>21 (19)</td>
</tr>
<tr>
<td>Minimum fresh air changes for indoor air quality</td>
<td>l/(s m²)</td>
<td>0,4a</td>
</tr>
</tbody>
</table>

*aCorresponding to circa 0,5 ach.

Delivered energy has been calculated assuming the use of an air-to-water heat pump unit with a COP factor of 3.0 serving a water based radiator and a ventilation air-handling unit with a heat recovery of 85% (running in the heating season only with a specific fan power of ca. 1.0 kW/m³/s). A generic Ideal Loads System template was set in the HVAC tab. Energy system mostly dedicated to heating is expected to be ON from the 1st of January to the 30th of April and from 1st September to 31st of December.

4. RESULTS

Environmental performance analyses showed that only during the months of January and December the indoor temperature goes below 21°C because of high heat losses through the building envelope. During summertime indoor temperatures reach instead the peak of 29°C for a few days. Energy modelling analyses showed that expected electricity consumption of the LivingLAB for how it has effectively been built (titled as M-0) is equal to 32.9 KWh/m²y,
corresponding to an operational carbon footprint of 26057 KgCO$_2$eq assuming the mentioned conversion factor of 132 gCO$_2$eq/KWh.

Model M-0 has been used as comparative term for the analysis of three alternative scenarios characterized the first by a larger glass area (M-1_noVIP), the second by the absence of PCM panels in the ceiling (M-2_noPCM) and the third by the use of Aerogel windows on the north side of the roof components (M-3_Aerogel) as described above.

Table 2. Energy modelling results

<table>
<thead>
<tr>
<th>Thermal analysis</th>
<th>M-0</th>
<th>M-1</th>
<th>M-2</th>
<th>M-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heat losses through transparent areas, KWh/m$^2$</td>
<td>-26.3</td>
<td>-37.4</td>
<td>-26.0</td>
<td>-38.9</td>
</tr>
<tr>
<td>Heat losses through opaque elements, KWh/m$^2$</td>
<td>-32.4</td>
<td>-35.2</td>
<td>-32.0</td>
<td>-30.4</td>
</tr>
<tr>
<td>Solar gains, KWh/m$^2$</td>
<td>48.0</td>
<td>62.9</td>
<td>47.6</td>
<td>49.8</td>
</tr>
</tbody>
</table>

Delivered Energy

<table>
<thead>
<tr>
<th></th>
<th>M-0</th>
<th>M-1</th>
<th>M-2</th>
<th>M-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Electricity for heating, KWh/m$^2$</td>
<td>15.1</td>
<td>15.0</td>
<td>15.4</td>
<td>16.6</td>
</tr>
<tr>
<td>Lighting, KWh/m$^2$</td>
<td>11.0</td>
<td>11.0</td>
<td>11.0</td>
<td>11.0</td>
</tr>
<tr>
<td>Appliances, KWh/m$^2$</td>
<td>6.8</td>
<td>6.8</td>
<td>6.8</td>
<td>6.8</td>
</tr>
</tbody>
</table>

4.1. Vacuum insulation panels.

As already stated, vacuum insulation panels have been used as replacement for 50% of the glazed area of the sliding doors towards East and West (figure 5). According to simulation results reported in Table 2 their use made possible to reduce yearly heat losses through the glazed areas from 37.4 to 26.3 KWh/m$^2$. Solar gains resulted anyway also markedly reduced moving from 62.9 to 48.0 KWh/m$^2$. For this reason, delivered energy reduction thanks to the use of VIP panels resulted to be at least 0.1 KWh/m$^2$y. This value corresponds in a 60 years lifetime to an carbon footprint reduction of 64 KgCO$_2$eq.

Environmental impact due to the replacement of glazed areas with vacuum insulation panels has been calculated taking into account embodied emissions in VIP, 514 KgCO$_2$eq together with internal and external cladding, for a total of 801 KgCO$_2$eq. The cladded VIP panel replaced a triple glazing with Argon that would have accounted for 762 KgCO$_2$eq. This means that additional environmental impact due to the use of VIP is equal to 39 KgCO$_2$eq.

The replacement of glasses towards east and west with VIP panels should thus be able to mitigate environmental impact of the LivingLAB throughout its all life cycle of 25 KgCO$_2$eq (corresponding to the difference between reduction in operation and increased environmental impact because of the higher embodied emissions in the material).

![Fig 5. VIP used in the LivingLAB project and environmental impact analysis.](image)
4.2. Phase changing materials

Environmental impact related to the use of PCM panels was estimated as 1144 KgCO$_{2eq}$, of which 961 due to its initial production. According to simulations run in DesignBuilder, the use of PCM panels in the ceiling, is able to stabilize temperature fluctuations towards comfort reducing the building cooling demand. Since cooling is solved through free-cooling and NatVent no real impact on the operational carbon footprint of the building was estimated during the warm season. Operational footprint of the building is instead reduced of 345KgCO$_{2eq}$in the whole building life cycle.

![Fig 6. PCM used in the LivingLAB project and environmental impact analysis.](image)

4.3. Aerogel windows.

According to simulation results, when Aerogel windows are used on the north side of the roof, instead of a Rockwool wall equipped with skylights, heat losses through transparent areas increase from 26.3 to 38.9 KWh/m$^2$ per year. Yearly electricity demand increases as a consequence of 1.6 KWh/m$^2$. This corresponds to an environmental impact surplus of 1227 KgCO$_{2eq}$ over 60 years.

Environmental impact due to the use of aerogel windows was calculated using data elaborated by Lolli and reelaborated into two large aerogel windows whose environmental impact would account for 6966.64 KgCO$_{2eq}$. The adopted solution, made of a rockwool wall with automated skylights, also because of a more complex layering and construction system, account instead for 14927.94 KgCO$_{2eq}$ (11676 are attributed to the used skylights), almost 8 tons more than aerogel windows.

![Fig 7. Aerogel hypothetically used in the LivingLAB project and environmental impact analysis.](image)
5. DISCUSSION.

Architectural design of Zero Emission buildings is based on a reasoned use of materials that goes beyond mere optimization for energy efficiency. The effort into reducing the building energy demand through the adoption of energy efficiency measures should always be evaluated at the light of their impact on the whole building carbon footprint. This scenario is even more complex if we consider that not everything can be easily measured in a project and implications of the use of specific solution, such as the reduction of the glass ratio in a building, are by far larger than reducing heat losses; affecting not only comfort in the building but also modern architecture dogmas such as the indoor-outdoor connection. According to the report developed by Inman and Wiberg, embodied emissions in the LivingLAB materials account for over 140000 KgCO₂eq if we assume a building life cycle of 60 years. The minimization of the building carbon footprint limiting heat losses through the use of VIP showed to have a marginal impact on the building footprint mitigation but a rather strong impact on the building architecture. Solutions that might result into simpler construction systems, such as the use of aerogel windows on the north side of the building roof, although energy losers might have a positive impact on the carbon footprint when they result in a simpler construction system. The use of phase changing materials might have positive impacts on indoor comfort beyond those numerically recorded within the operational carbon footprint of the building.

6. ACKNOWLEDGEMENTS

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Fiber Reinforced Polymers Textiles for Strengthening of Historical Buildings and the Examples for Use in Italian and Maltese Heritage

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Abstract. In the building industry, structural restoration and reinforcement by means of fiber reinforced products (FRP – Fiber Reinforced Polymers) are taking up a more and more important position against classical reinforcements with steel and reinforced concrete. The FRP fabrics laid with resins on the concrete structures, allow builders to replace the steel bars in a very quick and effective way. Using special devices, it is also possible to reinforce masonry structures with FRP layed on vaults, masonry panels and columns. The possibility to restore damaged structures and to strength existing structures in a simple and fast way makes this product very suitable for use on existing structures. The technical characteristics of the Carbon Fiber materials and the technology for structural repairs and strengthening with C-FRP will be illustrated. All information to the proper choice of the right kind of materials in accordance with the standards will be given. The common use of C-FRP reinforcements usually involves reinforced concrete structures, but it is very interesting to describe how this new technology can be adapted also to historical structures, respecting the standard of restoration.

1. FIBER REINFORCED POLYMERS TEXTILES FOR STRENGTHENING OF HISTORICAL BUILDINGS

In the building industry, structural restoration and reinforcement by means of fiber reinforced products (FRP – Fiber Reinforced Polymers) are taking up a more and more important position against classical reinforcements with steel and reinforced concrete. The FRP fabrics laid with resins on the concrete structures, allow builders to replace the steel bars in a very quick and effective way. Using special devices, it is also possible to reinforce masonry structures with FRP layed on vaults, masonry panels and columns. The possibility to restore damaged structures and to strength existing structures in a simple and fast way makes this product very suitable for use on existing structures.

Many historical buildings of the world heritage can be restored with little invasiveness and respecting the standards of restoration, as happened in many cases in Italy and, recently, in Malta for St. Francis Church of Valletta, where the repairing and strengthening of the choir vault was performed by stitching carbon FRP stripes.

1.1 FRP characteristics and advantages

Continuous fiber-reinforced materials with polymeric matrix (FRP) can be considered as composite, heterogeneous and anisotropic materials with a prevalent linear elastic behavior up to failure. Composite for structural strengthening are available in several geometries and compositions, but the most appropriate products for the reinforcement of historical structures are the carbon fiber textiles impregnated with epoxy resins. In fact, the dry unidirectional carbon
fiber textiles can be easily adapted and layed on the existing structures with epoxy resins using the wet lay-up technology.

The main advantages of these wet lay-up FRP systems are:
- High mechanical properties associated with low weight and dimensions
- Durability of the material
- High corrosion resistance
- Little invasiveness and reversibility of the work
- Speed in execution of works
- Freedom of design thanks to the adaptability of the shape and thanks to the adhesion on various types of structural elements and building materials.

1.2 The use of C-FRP materials

The common use of C-FRP reinforcements usually involves reinforced concrete structures, but it is very interesting to describe how this new technology can be adapted also to historical structures, respecting the standard of restoration.

The proper choice of the right kind of materials and technology in accordance with the standards can permit to use these new materials to reinforce and restore heritage structures without renouncing to the principles of restoration.

In fact, the structural reinforcement with C-FRP materials are always in observance of the prudential criteria encoded by the discipline of the restoration of cultural heritage: minimum intervention, non-invasiveness, potential reversibility, compatibility of old and new materials, recognizability of new works.

The method of statement to work on historical buildings with C-FRP involves the following phases.
- Survey on the historical structure
- Historic research
- Knowledge of existing materials through literature data and laboratory and site tests
- Identify the state of conservation of the existing structures and any structural problems
- Identify the correct structural strengthening method
- Choosing of the correct codes and regulations
- Design of the reinforcements with carbon fiber textiles to obtain appropriate strength and meet serviceability and durability requirements
- Technical drawing and specifications
- Execution of works with proper supervision and quality control
- Site tests and final check

The previous method was tested during the design process and the work phases for the strengthening of many historical building in Italy and recently also in Malta, as shown in the following.

2. THE EXPERIENCE OF STRUCTURAL REINFORCE OF EXISTING STRUCTURES IN ITALY

Many historical buildings of the world heritage can be restored with little invasiveness and respecting the standards of restoration, as happened in many cases in Italy.

Vaults, masonry walls and columns can be strengthened to accomplish the requests for safety against imposed and live vertical loads and seismic horizontal loads.

An example of reinforcement of historic structures, within the context of a restoration, is the strengthening of the brick vaults of San Martino Church, a XII century building located in Schignano (Prato- Italy), shown in the following figures 1.

Moreover, in the following figures 2, it is shown an example of the structural reinforcement of vertical masonry stone walls and cross vaults owing to the works of conservative consolidation of a XVI century building located in Cles (Italy).
3. THE DESIGN AND WORKS ON VAULTS OF ST. FRANCIS CHURCH IN REPUBLIC STREET, VALLETTA

During World War II an anti-personelle bomb was thrown by the enemy onto the friary and besides leaving considerable damage within that area, the blast also caused severe damage in the rear part of the church, particularly the arched part behind the altar and lantern above (as indicated in figure 3 and 4).

During the years, the vault has maintained a precarious equilibrium, since some pieces of stone fall down. Consequently, during summer 2014, the St. Francis Friary decide to repair and strengthen the structure to achieve a higher safety. There were two possibilities: dismantle and rebuild the stone vault or reinforce the existing structure with the application of carbon fiber fabrics.

The disadvantages to dismantle and rebuild the vault were:
- Dismantle and rebuilt the vault is a very long and dangerous work
- There is no guarantee to rebuild the exact shape of the vault and it is possible that the paintings do not fit in their previous position
- We would have lost an historic structure.

The advantages to use carbon fiber textiles to reinforce the vault were:
- Fast and safe work
- The carbon fiber textiles are few millimeters thick and it possible to put in place the paintings in their exact position
- The work is potentially reversible

For these reasons the works observe the criteria encoded by the discipline of the restoration of cultural heritage: minimum intervention, non-invasiveness, potential reversibility, compatibility of old and new materials.
The method for the repairing and strengthening of the choir vault involves the stitching of carbon FRP stripes at superior and inferior faces of the vault.

To perform the design of reinforcements, it was very important to determine the exact geometry of the vault and the characteristics of the materials. So, a detailed measurement study was performed not only with standard tools, but also with high technology laser and digital image scanner. In addition to the geometrical surveys, compression and pull off tests were performed on samples of the stone, to determine the mechanical properties of existing materials.
With these information, the structural calculations and the design were performed to
determinate the exact layout and the quantity of FRP strips and the level of safety of the
structure.

3.1 Existing materials of masonry vault structure.

The existing masonry vault was built using Maltese Globigerina Limestone and the mechanical
characteristics of this stone and mortar are analyzed to design the structural reinforcement.

The study starts using the available information from the existing literature, which during the
years characterizes the mechanical behavior of the Maltese Limestone. Researchers, working in
this field, have conducted a series of destructive and non-destructive tests on the stones used in
the construction of historic structures.

For example, for the design of Mosta Rotunda, the Maltese architect Giorgio Grognet de
Vassé referred to Milizia and his “Principi di ’Architettura Civile” and he evaluated the
compressive strength of his masonry structure by comparing Maltese Globigerina limestone to
the "pietratenera" in Milizia’s document, which the latter described as having a bearing capacity
of 2488321b/square palmi, equivalent to 19.595 N/mm². As Grognet, many other researchers
have conducted a lot of studies on Maltese Limestone, until to define a set of almost secure
parameters.

In order to better investigate the exact characteristics of the stone that forms the vault of St.
Francis Church, a series of laboratory and site tests are performed. So, a series of uniaxial
compression tests are performed on same sample of stone taken from the existing structures of
the St. Francis Church.

In addition to the compressive strength, another very important parameter to be considered is
the tensile strength of the stone and the cohesion resistance with the stitching of carbon FRP
stripes. To determine this data a series of pull off tests are performed in laboratory and on site.

The pull off tests are performed with different kind of adhesives to identify the best kind of
layering of reinforcement for Maltese Globigerina Limestone. So at the end we identify a
specific set of carbon fiber textiles and epoxy resins, that give the best results with Maltese
stone.

It is very important to underline that the use of carbon fiber textiles on Maltese Globigerina
Limestone involves a specific kind of epoxy resins, because the standard materials do not give
the best results.

To perform the calculation of the vault, in addition to the mechanical properties of the stone,
it is important the evaluation of the properties of the masonry. So, with the results of laboratory
and site tests and with literature data it was possible to obtain the values of the masonry
mechanical properties of the vault, considered for the structural calculations (table 1).

<table>
<thead>
<tr>
<th>Table 1. Properties of masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression strength - $$f_m$$</td>
</tr>
<tr>
<td>Average tensile strength - $$f_{mm}$$</td>
</tr>
<tr>
<td>Modulus of Elasticity - E</td>
</tr>
<tr>
<td>Joint friction coefficient</td>
</tr>
<tr>
<td>Unit Weight - $$w$$</td>
</tr>
</tbody>
</table>
3.2 New materials used to reinforce the existing structure.

The C-FRP materials used in structural design are represented from the following mechanical properties according to Italian Code CNR-DT 200 (table 2).

Table 2. Properties of carbon fiber textiles

<table>
<thead>
<tr>
<th>Code of carbon fiber textile</th>
<th>C-Tex 1000 LMN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of carbon fiber textile</td>
<td>Unidirectional carbon fibre non-impregnated fabric</td>
</tr>
<tr>
<td>Weight of carbon fiber</td>
<td>1000 g/m²</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>4000 MPa</td>
</tr>
<tr>
<td>Tensile modulus of elasticity</td>
<td>240 GPa</td>
</tr>
<tr>
<td>Density</td>
<td>1.8 g/m³</td>
</tr>
<tr>
<td>Strain at failure</td>
<td>1.5 %</td>
</tr>
<tr>
<td>Equivalent design thickness</td>
<td>0.546 mm</td>
</tr>
</tbody>
</table>

Considering the formulation 5.6, proposed by CNR-DT 200R1/2013, for the ultimate design strength for debonding we obtain the value $f_{ud} = 112$ N/mm².

3.3 Seismic hazard

The gravity loads due to self-weight of structural elements and permanent weight of finishes are added to the live loads to obtain the total vertical loads on the vault.

The heritage value of the church imposed to consider also the horizontal loads due to a possible seismic hazard. The design for earthquake resistance is performed in accordance with Italian code NTC2008 (D.M. 14.01.2008) and European code Eurocode 8 (EN 1998-1). Even if Maltese National Authorities do not identify a seismic zonation map, there are several preliminary studies regarding a possible seismic hazard map for the Maltese archipelago. In particular, the recent paper “Seismic Hazard Maps for the Maltese Archipelago: Preliminary Results” describe a preliminary map for PGA distribution on rock site obtained for a 10% probability of exceedance, that shows values ranging between 0.09-0.18 g. On behalf of safety, it is possible to assume a value of $a_g = 0.18g$ for the Peak Ground Acceleration for a 10% probability of exceedance with a reference period of 50 years. This value corresponds to a medium-high seismic zone in Italian National code.

Considering the importance of this heritage building we also consider a “reference period” for the seismic design strategy of 150 years in accordance with Italian code NTC2008.

3.4 Structural calculations

Some finite element models are used to estimate the structural behavior and to solve the problem of distribution of loads and calculation of stresses for vertical and horizontal loads.

In detail are used a three dimensional finite element model to evaluate the overall behavior of the structure and many two dimensional models of single arches to check the local behavior of the structure.

The main three dimensional finite element model starts from the detailed measurement performed during survey and considered all the components of the vault.

After the study of global model, the vault is studied through a discretization of the most significant arches and performing a stability check of these arches. Then, it is developed a finite element model of each arch using rigid elements, like “blocks”, which are only capable of transmitting compressive forces through the interfaces. Thus, the generic arch is modelled with a system of "discrete" substructures (segments) connected by a mechanical system interface (joints), constituted by rods perpendicular to the interface and in the tangent direction. This interface model corresponds to consider a hard-brittle behavior of the joints.
With this model it is possible to perform the analysis of stability of the arc using the theory of J. Heyman, which assumes for the arch the inability of tensile strength, the infinite resistance to compression and the infinite shear strength.

In this context, the analysis will be conducted with linear kinematics of the failure mechanisms in accordance with the provisions of NTC-2008 code.

These analyses, as well as the theory of Heyman, do not allow to take into account the compressive strength of the masonry.

However, due to the potential of the model developed, it is possible to perform additional analyses than the minimum required by the NTC-2008 code and read, in the interface connecting rods, the forces that will result as the normal stress in the interface. The point of application of these forces will allow to calculate the resulting bending moment in the section, while the forces associated to the connecting tangent rod constitute the shear forces. This additional analysis performed with the same model is a non-linear static analysis due to the nature of the interface considered.

So, for each arc will be performed a linear kinematic analysis, required by codes, and in addition, a non-linear static analysis for the evaluation of stresses inside the arc.

With the finite element models described above, three different structural verifications are performed: the structural check of the existing structure, the structural check of the vault reinforced on the lower face and the structural check of the vault reinforced on the upper face.

Regarding the structural check of the existing structure, the arc is stable, but the compression limit for the masonry is not verified and the sections on the lower part of the vault are overstressed. Moreover, the existing vault, without reinforcements, is capable to sustain only the 10% of the seismic actions, described in previous paragraph 3.3.

Regarding the structural check of the existing structure reinforced with carbon fiber textiles on lower and upper faces, the vault is stable and also all the verifications for the existing and new materials are satisfied, as shown by the following safety factors.

- Compression of masonry: \(2.8 > 1.0\) verified
- FRP reinforcements stress: \(6.6 > 1.0\) verified

Moreover, the arc with the carbon fiber reinforcements is capable to resist to 115% of the design seismic actions with a return period of 1945 years for the considered seismic event.

3.5 Execution of works

The works were executed during summer 2014 with proper supervision and quality control. The laying of carbon fiber textiles with epoxy resin took about 8 days.

Fig 5. C-FRP reinforcement on the lower surface of the vault of Francis Church, Valletta
The first works were the removal of the paintings from the lower surface of the vault and the removal of the covering material of the vault.

Once the stone structure of the vault was perfectly uncovered, it has been possible to apply the carbon fiber reinforcements on both upper and lower surfaces (figure 5).

At the end of the work a complete set of site tests (figure 6) on the carbon fiber reinforcements was performed by a Maltase authorized laboratory.

The results of tests were completely satisfactory and demonstrate as the adhesion between carbon fiber textiles and the existing stone structure fully complies with the requirements of the project.

![Pull-off tests on the lower surface of the vault of Francis Church, Valletta](image)

As before the reinforcement works were carried out, the historical paintings of the church were again put in their original place and the image of the church is the same as before it was carried out the reinforcement work.

In this case the C-FRP materials will guarantee the safety of the church that holds very important artworks like the paintings of Mattia Preti.

4. THE FUTURE TECHNOLOGY OF ENVIRONMENTALLY FRIENDLY CARBON FIBER TEXTILES

As time goes on, the carbon fiber textiles are being improved, in order to obtain an even more efficient and less invasive product.

Research conducted are going in three directions parallel to each other:

- Obtain carbon fiber textiles, which minimize the deviations of the carbon fibers so that they remain perfectly parallel to each other before and after the impregnation with resin.
- Obtain carbon fiber textiles of high weight, which guarantee an optimal impregnation with the resin.
- Obtain carbon fiber textiles, where the carbon fibers are held together without the use of synthetic adhesives or plastic materials, but with the use of a natural fiber net.

Recently C-Six, an Italian carbon fiber textiles producer, has introduced on the market a particular type of eco-compatible textile, made with a cellulose fiber net that holds together the carbon fiber tows.
Fig 7. Carbon fiber textiles, where the carbon tows are held together with a cellulose fiber net.

Fig 8. Carbon fiber textiles scheme, where the number 1 is the cellulose fiber net and number 2 is the carbon tows.

As detailed described in C-Six technical bulletin TN-110, this environmentally friendly textiles use only natural materials to keep together the carbon fiber tows. This means the following advantages:

- The textile is composed only by carbon and natural fibers without synthetic adhesives or plastic materials and for this reason it is eco-compatible and therefore suitable to be used in the restoration of historic buildings.
- The cellulose net is very soft and it does not deform the carbon fibers, keeping them straight, with a mechanical resistance improving.
- The cellulose net guarantees an optimal fiber impregnation with resin. This natural fiber net has solved the problem of impregnation of fabrics of high weight, which was hindered by the presence of synthetic adhesives or plastic materials used in the common kind of textiles.

REFERENCES


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C-Six technical bulletin TN-110, January 2016. www.c-six.it
A Study on Moisture Movement within Traditional and Contemporary Building Materials

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Abstract. Relatively recent studies have proved that the hygric properties of building materials can be put to advantage to improve the indoor air quality of a space. Theoretically-derived and practical results on the ability of materials to 'regulate' indoor relative humidity are being compared. Laboratory testing on typical building components of the Maltese built environment was carried out by following the NordTest protocol of attributing a 'Moisture Buffer Value' to materials, in order to classify their performance with respect to the amount and rate of sorption at cyclic RH variations. The local Globigerina Limestone, two traditional plasters and a limewash coating were tested as single-layered material samples and as systems of materials combined. Contemporary materials consisting of the hollow concrete block and a commercial two-component gypsum plaster were also investigated. The application of additional layers of coats and finishes to the substrate of a building component reduced the latter's performance in buffering indoor relative humidity. The theoretical moisture buffering performance of material samples was calculated from the materials' density, water vapour permeability and moisture capacity results, using the equations devised by NordTest.

1. INTRODUCTION

Vapour permeability is generally the least considered material function during the construction of new buildings and renovation of existing structures, even though this function highly influences the performance of a wall system. Research has proved that the use of 'breathable' building materials promotes the long-term durability of the building. In addition, these materials are capable of altering the indoor RH by vapour diffusion occurring at their indoor surface.

It is known that a high indoor relative humidity has the capacity of inducing microbial growth and enhances the propagation of dust mites. On the other hand, low RH levels also affect the occupants' health, including eyes' irritability and electrostatic discharges. Closed controlled ventilation systems may induce rapid oscillations of the internal relative humidity between the optimum of 40% - 60% RH. However, rapid fluctuations in RH by active systems, such as dehumidifiers, can create the worst of situations by generating all of the conditions necessary in which health problems arise. The use of building materials to passively 'regulate' the indoor climate allows for gradual variations of the indoor RH, thus improving the occupants' thermal comfort whilst avoiding the problems derived from the use of active systems. At the same time, gradual variations of the indoor RH have the added advantage of allowing sensitive heritage objects and materials to acclimatise and 'respond' to the change in humidity variations without undergoing abrupt high stresses within the materials.

The performance of vapour permeable materials depends on their 'hygroscopicity' which can reduce ventilation requirements and increase potential energy savings. These materials maintain the moisture equilibrium of the indoor environment through the storing or releasing of water vapour. Thus the indoor RH is slightly increased when humidity levels decline and alternatively it is slightly decreased when humidity reaches its annual high peaks. As a result of varying loads of an interior space, the indoor RH also significantly varies during the day. It is also important
to note that the faster the rate at which a wall surface adsorbs moisture from the air, the lower
the risk of surface condensation and surface relative humidities required to support fungal
growth.

2. THE MOISTURE BUFFER VALUE STUDIED AT MATERIAL AND SYSTEM LEVEL

The phenomenon of moisture adsorption and desorption in a hygroscopic building material
occurs by the exchange of moisture on its surface area and moisture flow in the material [6].
The practical Moisture Buffer Value (MBV_{PRACTICAL}) has been defined by the NordTest Project
as "the amount of water that is transported in or out of a material per open surface area, during a
certain period of time, when it is subjected to variations in relative humidity of the surrounding
air". This property can be evaluated by measuring the weight of material samples when the
relative humidity is varied in cyclical step-changes between 75% RH and 33% RH for 8 hours
and 16 hours respectively, at constant temperature. The change in weight of the material per
exposed surface area and per % RH variation gives the practical MBV in kg m^{-2} %RH^{-1}. Ramos
and de Freitas (2004) suggest that this simple experimental procedure may be compared to the
periodic changes in RH levels that occur in bedrooms "where during the night there will be an
increase in relative humidity due to vapour production by occupants".

2.1. MBV\_IDEAL vs MBV\_PRACTICAL

Alternatively to the test procedure to measure the MBV_{PRACTICAL}, the NordTest Project has also
proposed a theoretical MBV which is referred to as MBV_{IDEAL}. The theoretical or 'ideal' MBV
enables the calculation of the moisture buffering effect of material samples from known
standard material hygric properties. The ideal MBV is defined by the Equation:-

$$MBV_{IDEAL} = 0.00568 \ p_{sat} \ b_m \ \sqrt{\tau}$$

Where $p_{sat}$ is the saturation vapour pressure measured in Pa
$b_m$ is the moisture effusivity measured in kg/(mPas^{1/2})
$\tau$ is the time period measured in s.

The ideal MBV gives a quantity of vapour adsorption and desorption by the material when a
cyclical variation in RH is applied to the material's surface. However there are primary
differences between results obtained for the ideal and the practical MBV. The prior gives a
material property applicable to a single homogenous material, while the latter gives a value to a
system property, the simplest of which is a homogenous material above which is a layer of
stagnant air. This is because the practical MBV gives consideration to the air surface resistance
on the material surface and hence characterises the system property of the material in relation to
its environment. On the other hand MBV_{IDEAL} neglects resistance to the flow of moisture in or
out of the material.

For the calculation of MBV_{IDEAL} it is assumed that the thickness of the material under
investigation exceeds the moisture penetration depth for the material. In contrast, the
MBV_{PRACTICAL} can also be established for materials which are thinner than the moisture
penetration depth. Therefore direct comparison between MBV_{IDEAL} and MBV_{PRACTICAL} is no
longer possible if in practice the material thickness is not greater or equal to the moisture
penetration depth of the material.

2.2. Moisture Effusivity

Although the Moisture Buffer Value is a dynamic material property, the moisture effusivity $b_m$
is based on standard hygrothermal material properties which are established under equilibrium
conditions. This is reflected in discrepancies between values of MBV_{PRACTICAL} and MBV_{IDEAL}
since the latter is defined by material properties determined under steady state conditions.
The moisture effusivity $b_m$ is given by:

$$b_m = \sqrt{\left( \delta_p \rho_0 \xi \right) / p_{\text{sat}}}$$  \hspace{1cm} (2)

where  
- $\delta_p$ is the water vapour permeability measured in kg/(msPa)
- $\rho_0$ is the dry density of the material measured in kg/m$^3$
- $\xi$ is the moisture capacity of the material measured in kg/kg
- $p_{\text{sat}}$ is the saturation vapour pressure given by the test conditions measured in Pa.

The moisture effusivity $b_m$ expresses "the rate of moisture absorbed by a particular material (i.e. as a material property) when it is subjected to a sudden increase in surface humidity" and is used to give a value for the moisture buffering performance of a material in an ideal situation where the resistance to moisture exchange between the material surface and the surrounding air is equal to zero.

2.3. Estimating the Moisture Penetration Depth from Steady State Conditions

The concentration of vapour at the surface of a material decreases exponentially with depth within the material. Arfvidsson (1999) describes the moisture penetration depth of a material as "the depth where the amplitude of moisture content variation is only 1% of the variation on the material surface".

Following the NordTest protocol to find the Moisture Buffer Value, in which a porous material surface is subjected to a cyclic RH variation, the moisture penetration depth within the region of periodic variation in a material sample can be estimated using the Equation:

$$d_{p,1\%} = 4.61 \sqrt{D_w t_p / \pi}$$  \hspace{1cm} (3)

where  
- $d_{p,1\%}$ is the moisture penetration depth measured in m
- $D_w$ is the moisture diffusivity of the material measured in m$^2$/s
- $t_p$ is the time period measured in s.

However, Equation (3) provides only an estimate of the moisture penetration depth within the material and is only valid for a "semi-infinite (or very thick) material". When determining the moisture penetration depth within a material, cyclic changes in RH variation are assumed to occur at the material surface. In practice, changes occur in the surrounding air and "a surface resistance to moisture transfer exists which slows down the moisture exchange". The convective surface coefficient is a function of this resistance to moisture transfer and depends on the air velocity present during the cyclic RH variation in MBV$_{\text{PRACTICAL}}$.

3. TESTING METHODOLOGY

This study was aimed at investigating the hygric material and system properties of selected traditional and contemporary wall substrates to which a one-coat render was applied.

The MBV$_{\text{IDEAL}}$ for stone, concrete, a hydraulic and a non-hydraulic plaster, and gypsum plaster was calculated based on data from laboratory tests. These tests also enabled a comparative analysis between materials with respect to their density, vapour permeability, and hygroscopic sorption. Total open porosity and water absorption by capillarity were also investigated in order to provide a better understanding on the hygric properties of materials and their performance with respect to the movement of liquid moisture.

The MBV$_{\text{PRACTICAL}}$ for the individual materials and for systems of the materials combined were then investigated by following the NordTest protocol. The comparison of the MBV of traditional and contemporary materials suggested which of these promote healthier indoor environments, reduce energy consumption and provide the stable conditions which are sought for in museum environments. Limewash and gypsum finishing coats were applied as a thin layer on a traditional and contemporary base-coat respectively. The Moisture Buffer Value of
limewash and gypsum surface coatings was not determined in isolation but these were studied as forming part of a 'system'.

4. RESULTS AND OBSERVATIONS

The 'breathability' properties of the material and system samples under investigation are dictated by the physical micro-porous structure of the materials. It is also noted that the performance of system samples does not only depend on the material components, but also on the number of interfaces in the system.

It is important to note that in contemporary system samples there were physical, chemical and mechanical differences between the gypsum plaster and the concrete substrate while in the case of traditional samples, the differences between the plaster and stone substrate were mostly mechanical.

4.1. Single Material Samples

As seen in Table 1, materials having a relatively high vapour permeance and moderate moisture capacity were competitors in moisture buffering performance to material samples having a higher moisture capacity but less vapour permeance. This can be particularly seen in the ideal Moisture Buffer Values (MBV_{IDEAL}) of the gypsum base coat and stone material samples in Table 1, which were calculated from the hygric test results of vapour permeability, density and moisture capacity.

Table 1. Results for Density, Total Open Porosity, Water Vapour Permeance, Water Absorption Coefficient, Moisture Capacity and MBV_{IDEAL} and MBV_{PRACTICAL} of material samples.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Density (kg/m³)</th>
<th>Porosity (%)</th>
<th>Permeance Wp (kg/m²sPa)</th>
<th>Absorption Coefficient WAC [kg/(m²s³)]</th>
<th>Moisture Capacity ξ (kg/kg)</th>
<th>MBV_{IDEAL} (g/m²%RH)</th>
<th>MBV_{PRACTICAL} (g/m²%RH) (±AV. Dev)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>1633.61</td>
<td>38.58%</td>
<td>1.08 x 10⁻⁹</td>
<td>0.1431</td>
<td>0.0038</td>
<td>1.014</td>
<td>1.056 (±0.06)</td>
</tr>
<tr>
<td>C</td>
<td>1902.83</td>
<td>27.97%</td>
<td>1.05 x 10⁻⁹</td>
<td>0.0362</td>
<td>0.0066</td>
<td>1.419</td>
<td>0.789 (± 0.10)</td>
</tr>
<tr>
<td>GB</td>
<td>1261.49</td>
<td>46.44%</td>
<td>1.56 x 10⁻⁹</td>
<td>0.1459</td>
<td>0.0026</td>
<td>0.922</td>
<td>0.422 (±0.05)</td>
</tr>
<tr>
<td>M1</td>
<td>1647.68</td>
<td>37.29%</td>
<td>1.07 x 10⁻⁹</td>
<td>0.1552</td>
<td>0.0063</td>
<td>1.345</td>
<td>0.572 (±0.01)</td>
</tr>
<tr>
<td>M2</td>
<td>1613.61</td>
<td>38.34%</td>
<td>1.11 x 10⁻⁹</td>
<td>0.1698</td>
<td>0.0066</td>
<td>1.366</td>
<td>0.554 (±0.02)</td>
</tr>
</tbody>
</table>

S - Stone
C - Concrete
GB - Gypsum Base coat
M1 - Hydraulic plaster consisting of 1 part mature lime putty: 2 sand: 1 crushed pottery proportioned by volume
M2 - Non-hydraulic plaster consisting of 1 part mature lime putty: 2 sand proportioned by volume

Thus the highly vapour permeable gypsum having relatively low moisture capacity, attained an MBV_{IDEAL} result which was similar to that obtained by the less vapour permeable but more adsorptive stone. This is in agreement with the conclusions attained by Ramos and de Freitas (2004) when studying hygric properties of cement and gypsum plaster [4]. Thus at equilibrium conditions, the moisture storage and release in the materials investigated is correlated to the diffusion of vapour through the surface area of the test specimens in contact with the indoor air and to the hygric moisture capacity of the materials.

Under dynamic conditions, relatively high MBV_{PRACTICAL} values were attained by materials having a mixture of high vapour permeability and vapour storage capacity, as can be seen in the case of concrete material samples. However, this is relative to the thickness of the material sample. For the dynamic MBV test carried out, it has been shown that the moisture buffering performance of materials is dependent on how much the actual thickness of the material corresponds to the moisture penetration depth for the material. If the actual thickness of the material is greater or equal to the moisture penetration depth of the material, the vapour permeable and moisture adsorptive material results in a good moisture buffer. However if the
actual thickness is less than the moisture penetration depth of the material, this results in a limited moisture buffering performance depending on the length of time to which the material is exposed to extremes in RH. This is seen in the discrepancy of results between $MBV_{\text{IDEAL}}$ and $MBV_{\text{PRACTICAL}}$ in Table 2 which may be attributed to the thickness of material samples used in real life. Therefore while standard test procedures provide an insight into the material properties and theoretical performance of the material, the practical MBV test gives a more realistic approach towards the material's performance as it is used in practice and also provides information on the actual performance of a building component.

### Table 2. Comparison of Practical MBV, Ideal MBV and the moisture penetration depth (mm).

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>$MBV_{\text{PRACTICAL}}$ (g/m²%RH)</th>
<th>$MBV_{\text{IDEAL}}$ (g/m²%RH)</th>
<th>Material thickness (mm)</th>
<th>$d_{p,1%}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>1.056</td>
<td>1.014</td>
<td>150</td>
<td>75.27</td>
</tr>
<tr>
<td>C</td>
<td>0.789</td>
<td>1.419</td>
<td>50</td>
<td>52.03</td>
</tr>
<tr>
<td>GB</td>
<td>0.422</td>
<td>0.922</td>
<td>20</td>
<td>128.79</td>
</tr>
<tr>
<td>M1</td>
<td>0.572</td>
<td>1.345</td>
<td>10</td>
<td>59.81</td>
</tr>
<tr>
<td>M2</td>
<td>0.554</td>
<td>1.366</td>
<td>10</td>
<td>58.53</td>
</tr>
</tbody>
</table>

The results attained when investigating material samples given in Table 1, also evidence the correlation of vapour permeability and moisture capacity with density and porosity results. All hygric properties investigated were seen to be interlinked such that high porosity results corresponded with lower densities which gave high vapour permeability results and low moisture capacities. This can be particularly seen in the case of gypsum which gave the lowest moisture buffer result both in the theoretical and practical MBV.

Although gypsum has high vapour permeability, its low density allows the material to quickly reach its hygric moisture holding capacity. The density of a material particularly affects its hygroscopic performance when this is exposed to high humidity for more than 24 hours. This is in fact reflected in the sorption isotherm test where the higher the density of the material, the higher its hygric moisture capacity. In compensation, gypsum had a relatively high absorption coefficient indicating a high proportion of capillary pores.

In contrast, concrete has a relatively low vapour permeance and absorption coefficient but a relatively high moisture capacity, resulting in a good moisture buffering performance both in $MBV_{\text{IDEAL}}$ and $MBV_{\text{PRACTICAL}}$. As can be seen in Table 1, results for the hydraulic and non-hydraulic mixes were almost equivalent in all hygric tests, including the practical MBV test, in which both plasters exhibited relatively high moisture buffering results. This indicates that the crushed pottery admixture had negligible influence on the material's performance with regards to moisture movement.

### 4.2. Traditional System Samples

It is seen that the practical MBV of system samples consisting of plaster and substrate is smaller when compared to the MBV of the material individual components. The results attained also indicate that the application of five coats of limewash, drastically reduced the moisture buffering performance of the two plastered stone systems.
Table 3. Results for Density, Total Open Porosity, Water Vapour Permeance, Water Absorption Coefficient, Moisture Capacity and MBVIDEAL and MBVPRACTICAL of traditional system samples.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Density (kg/m³)</th>
<th>Porosity (%)</th>
<th>Permeance $W_p$ (kg/m²sPa)</th>
<th>Absorption Coefficient $WAC$ [kg/(m² s½)]</th>
<th>Moisture Capacity $\xi_m$ (kg/kg)</th>
<th>MBVIDEAL (g/m²%RH)</th>
<th>MBVPRACTICAL (g/m²%RH) (±Av. Dev)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>1633.61</td>
<td>38.58</td>
<td>1.08 x 10^{-9}</td>
<td>0.1431</td>
<td>0.0038</td>
<td>1.014</td>
<td>1.056 (±0.06)</td>
</tr>
<tr>
<td>M1</td>
<td>1647.68</td>
<td>37.29</td>
<td>1.07 x 10^{-9}</td>
<td>0.1552</td>
<td>0.0063</td>
<td>1.345</td>
<td>0.572 (±0.01)</td>
</tr>
<tr>
<td>LM1</td>
<td>--</td>
<td>--</td>
<td>1.55 x 10^{-9}</td>
<td>0.1634</td>
<td>0.0061</td>
<td>--</td>
<td>0.515 (±0.01)</td>
</tr>
<tr>
<td>M2</td>
<td>1613.61</td>
<td>38.34</td>
<td>1.11 x 10^{-9}</td>
<td>0.1698</td>
<td>0.0066</td>
<td>1.366</td>
<td>0.554 (±0.02)</td>
</tr>
<tr>
<td>LM2</td>
<td>--</td>
<td>--</td>
<td>1.96 x 10^{-9}</td>
<td>0.1692</td>
<td>0.0066</td>
<td>--</td>
<td>0.487 (±0.03)</td>
</tr>
<tr>
<td>M1S</td>
<td>--</td>
<td>--</td>
<td>0.36 x 10^{-9}</td>
<td>0.0416</td>
<td>0.0033</td>
<td>--</td>
<td>0.735 (±0.04)</td>
</tr>
<tr>
<td>LM1S</td>
<td>--</td>
<td>--</td>
<td>0.38 x 10^{-9}</td>
<td>0.0525</td>
<td>0.0024</td>
<td>--</td>
<td>0.233 (±0.01)</td>
</tr>
<tr>
<td>M2S</td>
<td>--</td>
<td>--</td>
<td>0.36 x 10^{-9}</td>
<td>0.0315</td>
<td>0.0033</td>
<td>--</td>
<td>0.575 (±0.01)</td>
</tr>
<tr>
<td>LM2S</td>
<td>--</td>
<td>--</td>
<td>0.35 x 10^{-9}</td>
<td>0.0389</td>
<td>0.0024</td>
<td>--</td>
<td>0.277 (±0.01)</td>
</tr>
<tr>
<td>LS</td>
<td>--</td>
<td>--</td>
<td>0.52 x 10^{-9}</td>
<td>0.1373</td>
<td>0.0037</td>
<td>--</td>
<td>0.827 (±0.08)</td>
</tr>
</tbody>
</table>

S - Stone
M1 - Hydraulic plaster consisting of 1 part mature lime putty: 2 sand: 1 crushed pottery proportioned by volume
LM1 - Five coats of limewash applied to M1
M2 - Non-hydraulic plaster consisting of 1 part mature lime putty: 2 sand proportioned by volume
LM2 - Five coats of limewash applied to M2
M1S - One coat of hydraulic plaster M1 applied to Stone
LM1S - Five coats of limewash applied to M1S
M2S - One coat of non-hydraulic plaster applied to Stone
LM2S - Five coats of limewash applied to M2S
LS - Five coats of limewash applied to Stone

As can be seen in table 3, while the limewash coating had negligible effect on the moisture capacity of material samples, it drastically decreased the moisture capacity of plastered substrate systems. Therefore on the introduction of two interfaces in a system, the limewash coating altered the hygroscopic sorption of system samples and almost annulled its moisture buffering performance.

When analysing absorption by capillarity data, the hydraulic plastered stone systems 'delayed' saturation by 22 hours, while the non-hydraulic plaster applied to the stone substrate, delayed saturation by 46 hours. Such plasters may draw moisture and salts away from the valuable masonry. Their similar breathability properties to the stone substrate make them ideal, compatible components which protect the stonework from the deterioration mechanisms related to vapour and liquid movement. Their relatively high moisture capacity and absorption coefficient enable them to protect their underlying building fabric from rapid wetting and drying cycles, and any water (or surface condensation) entering the plaster will not be readily transferred to the stone substrate.

4.3. Contemporary System Samples

When considering building components it is important to note that concrete block work is never left exposed and is always finished with a covering material. Therefore it is advantageous that the application of the gypsum base coat and surface finish on the concrete block did not significantly affect the moisture buffering performance of the system. While in the case of traditional systems, limewash significantly reduced the moisture buffer value of the system, the application of the gypsum surface finish coat had negligible influence on plastered concrete.

It is also noted that the absorption coefficient of the plastered concrete system (GBC) was similar to that of the concrete material sample, indicating that the base coat had a negligible effect on the system. Hence the base coat layer did not provide the 'barrier' effect to liquid moisture entry which was exhibited by the hydraulic and non-hydraulic plasters on stone.
Table 4. Results for Density, Total Open Porosity, Water Vapour Permeance, Water Absorption Coefficient, Moisture Capacity and MBVIDEAL and MBVPRACTICAL of contemporary system samples.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Density (kg/m³)</th>
<th>Porosity (%)</th>
<th>Permeance Wp (kg/m²sPa)</th>
<th>Absorption Coefficient WAC [kg/(m²s½)]</th>
<th>Moisture Capacity w (kg/kg)</th>
<th>MBVIDEAL (g/m²%RH)</th>
<th>MBVPRACTICAL (±Av. Dev)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>1902.83</td>
<td>27.97%</td>
<td>1.05 x 10⁻⁹</td>
<td>0.0362</td>
<td>0.0066</td>
<td>1.419</td>
<td>0.789 (±0.10)</td>
</tr>
<tr>
<td>GB</td>
<td>1261.49</td>
<td>46.44%</td>
<td>1.56 x 10⁻⁹</td>
<td>0.1459</td>
<td>0.0026</td>
<td>0.922</td>
<td>0.422 (±0.05)</td>
</tr>
<tr>
<td>GSB</td>
<td>--</td>
<td>--</td>
<td>1.53 x 10⁻⁹</td>
<td>0.0447</td>
<td>--</td>
<td>--</td>
<td>0.550 (±0.01)</td>
</tr>
<tr>
<td>GBC</td>
<td>--</td>
<td>--</td>
<td>0.30 x 10⁻⁹</td>
<td>0.0168</td>
<td>0.0037</td>
<td>--</td>
<td>0.690 (±0.01)</td>
</tr>
<tr>
<td>GSBC</td>
<td>--</td>
<td>--</td>
<td>0.25 x 10⁻⁹</td>
<td>0.0415</td>
<td>0.0038</td>
<td>--</td>
<td>0.653 (±0.03)</td>
</tr>
<tr>
<td>GSC</td>
<td>--</td>
<td>--</td>
<td>0.26 x 10⁻⁹</td>
<td>0.0106</td>
<td>0.0033</td>
<td>--</td>
<td>0.540 (±0.04)</td>
</tr>
</tbody>
</table>

C - Concrete
GB - Gypsum Base coat
GSB - Gypsum Surface finish coat applied to GB
GBC - Gypsum Base coat applied to Concrete
GSBC - Gypsum Surface finish coat applied to GBC
GSC - Gypsum Surface finish coat applied to Concrete

5. CONCLUSIONS

This study is an investigation on selected building components which are typical for a traditional and contemporary Maltese indoor environment, including surface coatings and plasters. Fig 1 illustrates the MBVPRACTICAL results for the materials and systems investigated.

Following the MBV classification proposed by the NordTest protocol only stone qualified as a 'good' moisture buffer while LM1S, LM2S, GB and LM2 samples are classified as 'limited' moisture buffers. All remaining materials classify as 'moderate' moisture buffers with the lowest MBV obtained by LM1 and the highest MBV reached by LS in the 'moderate' moisture buffering range.

Theoretically, from the MBVPRACTICAL results attained in this study, it may be deduced that a room consisting of uncovered 'bajda' stone, measuring 3m x 4m x 3m providing an area of 42m² of interior walls and having constant climatic conditions of 23°C ±0.5°C and 75% ±5% RH, would result in approximately 1815 g of moisture being adsorbed by the stone material after 8 hours. This is a very significant value, considering that at 20°C the maximum moisture content in air is about 17.3 g/m³. If the stonework was to be plastered by the hydraulic Mix 1, this value would be reduced to 1297 g of moisture. If the plaster is then coated by five coats of limewash, the buffered moisture would drastically fall to 412 g.
If the above-mentioned room were to be constructed out of concrete block work and then plastered with a 10mm gypsum base coat and finished with the gypsum surface coating, the moisture adsorbed by the system at the same climatic conditions would approximate 1127 g of moisture. Therefore when considering three-layered building components of coating, plaster and substrate, the selected contemporary systems of gypsum and concrete absorb more moisture from the indoor air than the three-layered traditional systems of limewash, lime plaster and stone.

From the results attained in this study, it may also be concluded that the application of the vapour adsorptive and liquid absorptive lime plasters on the Lower Globigerina Limestone substrate would significantly protect the more important substrate material. Thus while protecting the building material from intrinsic damage caused by moisture movement, the moisture adsorptive traditional lime plasters would also contribute to the indoor air quality of a space by avoiding surface condensation and by maintaining stable interior environments which are important for thermal comfort.

6. ACKNOWLEDGEMENTS

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Waste Minimisation: a Design Review

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**Abstract.** This document provides guidance and information to Design Teams on Designing Out Waste and opportunities available to minimise waste through the design process. It will provide guidance on how to implement a design review process within an organisation to reduce construction waste on projects designed. The document provides general information on waste and its impact on projects, introduces the five basic principles of Designing Out Waste and key actions for Design Teams.

1. **INTRODUCTION**

The issues surrounding sustainable construction have become ever more important over the past few years. As a result, the emphasis placed on environmental, social and economic aspects of projects is likely to increase in the future. Designers play a key role in ensuring that buildings are designed in a sustainable manner. Design is a complex decision making process with each decision resulting in numerous consequences for the construction and operation of buildings. One of these consequences is waste generation, be that construction or operational waste. This paper focuses on the influence of design decisions on construction waste and how waste can be reduced through design.

The construction industry is Malta’s largest consumer of natural resources. The construction industry is responsible for over 700 Thousand tonnes of Construction and Demolition Waste (CDW)(in 2011). An estimated 422 Thousand tonnes of this waste ends up in landfill/backfill without any form of reuse. More efficient use of materials would make a major contribution to reducing the environmental impacts of construction including reduced demand for landfill and the depletion of finite natural resources. This would also contribute to the economic efficiency of the sector and Malta as a whole.

There is no specific legislation concerning CDW management other than the ‘Waste Regulations’ which transpose the EU Waste Framework Directive (2008/98/EC) and the Legal Notice 168 of 2002, Environment Protection Act (Act No. XX of 2001), Waste Management (Landfill) Regulations. The Waste Management Plan (WMP) for the Maltese Islands includes specific provisions for the development of reuse and recycling of CDW, recognizing the need to move away from backfilling to recovery operations higher in the waste hierarchy. There is no market for recycled materials in Malta and no facilities for recycling other materials from CDW. As a result all materials except mineral CDW, are exported for recycling.

Considering waste in the design process is not traditionally part of designers’ education curriculum or one of the primary concerns of general practice. However, it is an issue which is
increasing on the sustainability agenda and rising within the design community. Research to date has identified the important contribution that design professionals and architects in particular can make in reducing waste through design or ‘Designing Out Waste’ (DOW). A number of exemplar case studies on live project have been developed, working with design teams to identify and build the business case for action around designing out waste. This exercise led to a greater understanding of construction waste and the development of certain principles that architects can employ during the design process to minimise waste. Five such principles have emerged and include:

- design for reuse;
- design for modern methods of construction;
- design for material optimisation;
- design for waste efficient procurement; and
- design for deconstruction and flexibility.

This paper provides a detailed explanation of these principles and how considering the efficient use of materials and waste reduction in the design and procurement of buildings can lead to a reduction of waste arising from the construction process and subsequently further benefits.

2. THE CASE FOR ACTION

2.1. Material Resource Efficiency

The three major areas where a construction project can improve its resource related environmental impact are energy, water and use of materials. Material resource efficiency is a balanced approach to materials, ensuring that at each stage in construction (which includes demolition and excavation) the materials that are used, are used in an efficient manner. The diagram below shows the different aspects of materials efficiency and highlights the areas where designers can have a significant input.

Fig 1. Materials resource efficiency as part of sustainable construction

Adopting the waste hierarchy (figure 2) to reduce, reuse, recycle, recovery and finally dispose of waste makes good business sense for the construction sector. Implementing strategies to reduce waste is the first step towards materials efficiency and is one of the most effective ways to address the waste problem in the construction industry. Efficient use of
materials reduces material purchasing costs and minimising waste eliminates the need for subsequent handling and disposal costs.

Maximising the opportunities for waste reduction ensures that less waste is produced. Any subsequent waste can then be managed and diverted from landfill through recovery of materials for either reuse or recycling. Waste reduction and management should be considered at the outset of a project when there is significant scope to influence the projects outcomes. Potential strategies for waste reduction and recovery should then be considered in subsequent project stages by all parties, and any lessons learnt, developed into established good practice and implemented on future projects.

A critical component of achieving good practice waste reduction and recovery involves the formulation and implementation of a Site Waste Management Plan (SWMP) at the pre-design stage. SWMPs should be used to determine key waste streams and target these for reduction and recovery. It is essential that the appropriate planning for waste occurs upfront to ensure the maximum impact and that these are incorporated and implemented through the SWMP. Key to this is communication through the supply chain, and collaboration between the design team and contractor upfront in the design and planning stage.

Major improvements in materials efficiency are therefore possible, without increasing cost by:

- Reducing the quantity of material sent to landfill during the design and construction process through designing out waste and effective waste management;
- Recycling and recovering waste material;
- Utilising more recycled materials and mainstream products with higher recycled content, including recycled content not necessarily sourced from construction and demolition waste.

2.2. Material Resource Efficiency

Waste management is a pressing issue in Malta with over 700 Thousand tonnes of construction sector waste being sent to landfill/backfilled or disposed of at sea.¹ Landfill capacity is becoming ever more limited. There are a number of Government and construction industry initiatives driving the waste agenda forward, in particular the 2020 target of recovering 70% insert CDW. The substantial financial gains, wider project and environmental benefits further support a case for action. This section provides a brief overview of these key drivers.

¹ Waste Management Plan for the Maltese Islands, A resource Management Approach, 2014 - 2020
2.2.1. Site Waste Management Plan Regulations.

A SWMP is a document that identifies how waste arising during the construction process is to be managed, and ultimately reduced or recycled. However, a SWMP is not just a tool for managing waste on site. As good practice SWMP should be used as a tool during the early phases of a project to inform the development of the design by identifying potential waste streams to target for reduction and recovery.

Focusing upon legal compliance only limits the ability to extract maximum value from the SWMP. Real improvements (including cost reductions) can be achieved when the plan is used to drive change throughout the project. To do this, the SWMP should be discussed right through from project initiation to the design and construction phase. This means ensuring that management actions are set, and implemented.

The SWMP should be developed before starting on site and be regularly updated throughout the project. As a minimum, the SWMP should include:
- Headline information about the project (location, date etc.);
- the name of an individual responsible for waste;
- a forecast of the quantity of waste that will be generated, identified by material type;
- a set of clear actions to reduce waste, and to increase the level of recycling;
- The end destination for each waste stream and the recovery rate that will be achieved.

2.2.2. Landfill Tax and Aggregates Levy.

Limited landfill capacity has led to increased costs of waste disposal. This includes a gate fee. Additionally, the extraction of quarry materials is subject to an aggregates levy that is ultimately passed on as cost in the price of construction materials and products. Although designers are not directly involved in the cost management of projects, it is important that they have awareness of the costs associated with waste in order to provide a robust case for waste reducing design solutions.

There is no Landfill Tax in Malta.
- Currently landfilling of waste (any type of waste) is charged € 20 per tonne at the only engineered non-hazardous waste landfill in operation.
- A lower rate of €3.21 per tonne applies to inactive (or inert) waste.

2.2.3. Financial Drivers

By adopting good and best practice material resource efficiency in the design and construction of a project, it is possible to produce significant financial savings. Less waste generated means that a reduced quantity of materials will be purchased, and less waste taken to landfill will reduce gate fees for disposal. Cost savings may stimulate the adoption of improved recovery practices and motivate a sustained change in waste management practice. Furthermore, increased project efficiency through a reduction in construction costs and programme can lead to increased competitive differentiation. This is particularly so where waste reduction opportunities will help meet prospective client’s sustainability objectives.

The true cost of waste is rarely identified at the project level and does not just include skip hire/disposal costs, but also includes the cost of labour to fill skips and the value of those materials thrown away. Research shows that simple steps to reduce waste can achieve substantial savings in the value of new materials that are wasted, estimated on average at 3% of total construction costs and 20% of material costs on site. Taking action to reduce waste can therefore have a substantial impact on the overall cost of a project. In addition, sending
segregated waste offsite can result in substantially reduced disposal costs. One study has shown an estimated saving of €38.15 per tonne by segregating waste on site.2

This is an area where real cost savings are possible. Reducing construction waste provides a direct reduction of the cost of waste disposal. In addition, finding end destinations other than landfill are also likely to become more cost effective, especially accounting for the disposal rates. Similarly, with the rising cost of materials the value of materials wasted is likely to increase as well. Reducing waste in design and site waste management planning upfront in the design and planning stage are key mechanisms for achieving cost savings.

3. THE FIVE BASIC PRINCIPLES OF DESIGNING OUT WASTE (DOW)

There are numerous opportunities to reduce waste during the design process and, this document provides a systemic approach to address waste reduction. The paper is based on the application of five principles. It is not intended that this paper is rigidly followed by designers but it should be used and adapted to suit the precise requirements of each project.

Research carried out by WRAP (Waste Resource Action Programme, UK) has identified that there are five basic design principles that can be adopted to reduce the waste burdens of projects through design. These principles emerged from working on live projects with designers and provide a practical method to achieve waste reduction whilst undertaking the design process.

Whilst many waste saving opportunities could be associated with a number of DOW principles it is likely that most opportunities will display one dominant characteristic. The intent of formalising the five principles is not to develop a strict system of classification but simply to provide a practical method that relates to the design process of considering waste saving strategies for clients, designers, and contractors. An organisational framework is provided as a starting point and thus a means of systematically considering and assessing waste saving alternatives. The significance of the principles is to provide designers with a means of ordering the Designing Out Waste (DOW) process when the principles are applied to a project. They are also the suggested starting point for architects and building designers to consider DOW as part of a larger practice wide sustainability policy.

3.1. Design by Reuse

It is generally acknowledged that the reuse of material components and or buildings, has considerable if not greater potential to reduce the environmental burdens resultant from construction (including waste) than recycling. Reuse may imply the reuse of existing materials on site (the focus of this section) or the use of new materials with recycled content. Reuse of existing materials should not just be limited to immediate on-site material, components, and buildings but also to the reuse of materials and components that have been salvaged from other sites.

The first consideration of Design by Reuse starts with the site analysis, a site visit being perhaps one of the very first activities carried out on projects. The reuse of buildings (if any on the site, including parts of buildings) and or existing materials to accommodate the client’s requirements should be considered from this very early stage (RIBA Stage 1 and 2). If the decision is made that only a new building will satisfy the client’s requirements, demolition and site clearance consequentially follow. Even in these instances the following questions should be asked:

- Can materials from demolition of the building or other phases be reused in the design?
- Can reclaimed products be reused?
- When materials are reused, can they be reused at their highest value?
- Can any excavation materials be reused?

2 WRAP Simons Construction case study on waste segregation
Can cut and fill balance be achieved? Optimised?

If the response to any of these questions is YES then actions should be taken to ensure that the proposed reused materials or components meet the required functionality of the new building design. If there is no opportunity for reuse of materials and components from demolition then the architect may have the opportunity to influence the demolition contract by advising the client of various good practice demolition protocols.

Whereas reuse of existing buildings (or parts thereof) and materials/components on site is subject to an examination of the site, the reuse of materials and components from other projects is hardly ever considered by designers. Other than when there are specific heritage needs and particularly in projects involving refurbishment or extension. The possibility of design being driven by reuse, so called ‘design for salvage’, marks a real change from established design practice where ‘bespoke design’ is the norm. Whole building systems are regularly reused in the agricultural sector and whilst this is uncommon in other, more mainstream construction markets there is no reason why such design practice could not become more common place. For example there is no reason why a steel portal frame used often in buildings with a relatively short life span (distribution centres and ware houses often have a life span of twenty years or less) could not be consider for reuse in many building designs.

There are a number of barriers to ‘design for salvage’ but many of the limitations imposed by these have recently been relaxed or addressed making design for salvage more practicable. Investigations therefore, should be set in hand to establish the possible extent of reuse (frame, components, materials etc.) and its practicality as early as possible. Whilst this does require a fundamental shift in usual design practice, there are many significant benefits not least, of which might be a significant reduction in costs.

Thereafter, during design development (RIBA Stage 3), the reuse of materials, components and even building elements should be considered in detail as these could have an impact on space planning, structural systems in fact the whole design of the building. Moreover, to assure client, regulatory authorities, financial funds etc. that the proposed reused material/component meets the required functionality then the following may be required:-

- the commissioning test or certification from experts,
- carrying out further consultations with the planning and building regulations authorities,
- Initiating discussions with specialist subcontractors
- Initiating discussions with other consultants etc.

During RIBA Stages 3 and 4 a final assessment confirming that the materials meet the correct quality, adequate technical standard and specifications needs to be produced to ensure that this is delivered.

3.2. Design for Modern Methods of Construction

One of the limitations of in-situ construction is that it is a one-off process carried out on a construction site where the advantages of factory-based manufacturing are difficult to apply to site-based processes. However, the use of construction methods that incorporate components manufactured off-site provides a means of delivering many of the advantages of factory-based manufacturing, including considerable waste minimisation. This is specifically relevant when factory manufactured components are used extensively. Application of off-site manufacturing has the potential to change operations on site into a process of rapid assembly of parts that results in very little onsite waste being produced.

Modern Methods of Construction (MMC) is the term used to describe a range of technologies and processes involving various forms of supply chain specifications, prefabrication and off-site assembly. MMCs include among others off-site manufacture, timber and light gauge steel frame, prefabrication and tunnel form concrete casting and offer some of the most important solutions to the construction problems facing mainland Europe including Malta.
In order to assess the most suitable MMC solution for the project, the following key questions need to be addressed:

- Can the design or any part of the design be manufactured off site?
- Can site activities become a process of assembly rather than construction?

The method of construction should be considered from the early project stages. RIBA Stage 1 may seem too early to assess construction methods but experience shows that it can have a significant influence on initial design considerations. If this aspect of the project is left until later stages then its use could effectively be prohibited as design development advances and may make its inclusion impractical. Therefore, the potential or at least a consideration of using MMC should be made at the earliest stage of design because of its impact upon:

- space planning especially structural and planning grids;
- structural design/system selected;
- project buildability;
- procurement routes; and
- Aesthetics.

For smaller scale items, usually procured through specialist subcontractors it is likely that the design team begins the consideration of elements suitable for MMC during this later stages, possibly having identified the most suitable items to be manufactured off-site during RIBA stage 2.

3.3. Design for Material Optimisation

This principle refers to a number of ‘good practice’ initiatives that designers should consider as part of the design process. Good practice in this context means altering the design so that less material is used in the design (i.e. lean design), and or less waste is produced, such as excavation waste.

The three main areas identified under this principle are:

- Minimisation of excavation
- Simplification and standardisation
- Dimensional coordination

In order to assess the best project opportunities for material optimisation, the following key questions need to be addressed:

- Can the design, form and layout be simplified without compromising the design concept?
- Can the design be coordinated to avoid / minimise excess cutting and jointing of materials that generate waste?
- Is the building designed to standard material dimensions?
- Can the range of materials be standardised to encourage off cut reuse?
- Is there repetition & coordination of the design, to reduce the number of variables and allow for operational refinement (e.g. reusing formwork)?

RIBA Stages 1 & 2 do not offer great opportunities for waste minimisation in respect to material optimisation as the design is not sufficiently advanced. However, it is important at this stage to agree with consultants and the client that material optimisation is part of the overall project waste strategy and that appropriate commitment to its inclusion later in the design process is obtained.

During RIBA Stage 3 the spatial organisation of the building, the correct size and all the building’s internal layouts are finalised, this is where the application of MO principles are most relevant.
3.4. *Design for Waste Efficient Procurement (WEP)*

Whilst efforts can be made during design to minimise construction waste, designer’s need to consider the procurement of these initiatives and how this can impact both on the way waste is produced during the construction process and how it may be avoided. Consideration of WEP coincides to some degree with the requirements of Site Waste Management Plans (SWMP). SWMPs refer largely to putting adequate systems in place to deal with site waste once it has been generated. WEP considers the design team initiatives that affect waste through design, specification, contracts or a combination of these.

Designers have a considerable influence on the construction process itself through specification and setting contractual targets prior to the formal appointment of a main contractor. Designers need to consider how work sequences affect the generation of construction waste and work with the contractor and other specialist subcontractors to understand and minimise these, often by setting clear contractual targets. Moreover, work sequences identified as the cause of site waste can sometimes be, once understood, ‘designed-out’.

In order to assess the best project opportunities for reducing waste at construction stage, the following key questions need to be addressed:

- Can construction methods that reduce waste be devised through liaison with the contractor and specialist sub-contractors?
- Has specialist contractor been consulted on how to reduce waste in the supply chain?
- Has the project specifications been reviewed to select elements / components / materials and construction processes that reduce waste?
- Have the efforts in the design stage contributed to the formulation of SWMP?

During RIBA Stages 1&2, similar to MO above, the building design is not sufficiently advanced to offer many opportunities for waste minimisation in regard of WEP. However, it is important at this stage to agree with all consultants and the client that WEP is part of the overall project waste strategy and that appropriate commitment to inclusion into the design process is obtained from all.

The design team need to understand and identify how their design choices lead to the generation of on-site waste and then undertake either design alternatives, develop specifications, instigate changes to the contract or a combination of these initiatives, in light of this knowledge to reduce site waste.

3.5. *Design for Deconstruction and Flexibility*

Designers need to consider during the design stage how materials can be recovered effectively during the life of the building when refurbishment is undertaken or when the building comes to the end of its life. Much work has been carried out to understand what prohibits the large-scale reuse of materials and components in the construction industry. There are a number of barriers not least of which is detailing building assemblies so that materials/components can be easily disassembled and recovered. Yet another is putting in place adequate information so that future designers have an adequate understanding of the material/component attributes to facilitate their future reuse.

In order to assess the best project opportunities for reducing waste at deconstruction stage, the following key questions need to be addressed:

- Is the design adaptable for a variety of purposes during its life span?
- Does the design incorporate reusable / recyclable components and materials?
- Are the building elements / components / materials easily disassembled?

During RIBA stages 1 &2, similar to MO and WEP above, the building design is not sufficiently advanced to do much in terms of waste minimisation in regard of deconstruction. However, it is important to agree with all consultants and the client that this is part of the overall project waste strategy that appropriate commitment to inclusion into the design process is
obtained from all. The point here also is that need to ascertain for an individual project how important this area is.

This is particularly important for deconstruction, as discussions about appropriate structural systems and materials are likely to occur relatively early on in RIBA stage 2. It is therefore important for the design team to keep in mind the principle of deconstruction before decisions are made concerning major construction elements that preclude its implementation later on in the design process.

It is probable that a range of alternative construction methods is likely to be suitable for deconstruction. Generally, those methods that make disassembly at the end of their service life easy, facilitate their potential to be reuse or enhance their recyclability, should be selected in preference for adoption by the design team rather than contiguous structural systems. Reuse should invariably be chosen as a better end of life scenario rather than recycling. There is an added waste reduction benefit in choice of such structural systems as they are likely to be fabricated using off-site methods and assembled on site.

Nevertheless, the choice of structural system is a major design decision and has a considerable impact on the building design, cost and Programme and although the importance of deconstruction may be accepted by the design team it is unlikely to be the major factor that determines which structural system is eventually selected. It however should be regarded as an important attribute to be taken into account whilst determining the most appropriate structural system.

Apart from the structure, a number of other key elements should be reviewed for their potential for being designed with the principle of deconstruction. Whilst much of the detailing remains to be carried out during RIBA Stages 3&4 for these elements discussions with key subcontractors, regarding the application of deconstruction should be initiated. Deconstruction should be placed on the agenda and options explored with them to assess the practicality of applying deconstruction at the earliest stage.

4. PROJECT APPLICATION OF THE FIVE DOW PRINCIPLES

It is broadly true to say that the first two principles (Design by Reuse and Design for Modern Methods of Construction) are best considered at the initial design stages – RIBA Stages 1,2&3.

After Stage 3 the opportunity to include the DOW initiatives that arise from these principles is much reduced as the design is developed, becomes more complex and interdependent and thus more difficult to change fundamentally.

This is particularly the case with large building elements that have significant potential to reduce the generation of construction waste. An example of this is structural design. After RIBA Stage 3 changes that aim to reduce waste can lead to substantial reworking of the design. After Stage 3 some alterations may still be possible but mainly for smaller building components. By their nature these components are not likely to have the same quantitative waste reduction potential as larger building elements, however they are still worth reviewing from a Designing out Waste perspective especially if these are repetitive components as the value of waste saved rather than quantity can be significant.

Design by Reuse and Design for Modern Methods of Construction principles focus on identifying and implementing site waste minimising initiatives early on in the building design. Design for Materials Optimisation and Design for Waste Efficient Procurement principles focus mainly on refining the building design to minimise waste on site. Consideration and application of these principles therefore, occur later in the design process, typically during RIBA Stages 3 & 4.

Having accepted the need to address waste as part of a larger sustainability agenda and in particular waste reduction through the design process, the application of the five DoW principles, to any project is likely to follow the series of steps. These steps are summarised below:

- Step 1 : Towards an evolving practice waste strategy, client contact and negotiations
- Step 2: Appointments and initiation of the design process
• Step 3: Design development and project review

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1. INTRODUCTION

EcoBuild is a new tool which was developed by the Building Industry Consultative Council, intended to promote Green Building Technology in Malta. It is considered as a first database in Malta for available Green Building Products. The tool is based on an online system and is therefore accessible to a wide audience. The Building Industry Consultative Council embarked on this project with the purpose of increasing the awareness on Green Products and promoting a greater understanding of the use of Green products in interventions in buildings. This is achieved by increasing product visibility and competition, and providing building industry professionals with a wider range of information and tools to implement better performing buildings. The project provides information which will enable the building industry to adopt construction practices which are themselves energy efficient or which lead to an improvement in the energy efficiency of both new buildings and also existing buildings. In this project context, green building technology refers to construction techniques and building materials, products and elements of the building fabric which result in reduced energy consumption in buildings. Furthermore the technology refers to renewable energy sources which can be integrated in the building fabric. In addition the tool is intended to serve as a source of information on best practice examples of Green Product application and to promote education for sustainable construction.

2. PROJECT IMPLEMENTATION

The project’s objective is to promote discussion and disseminate expertise in local applications of green building technology. The project supports the construction industry in achieving the EU 2020 targets for energy efficient buildings. This project has been conducted in collaboration with MRA who is a beneficiary in the ERDF Project (ERDF 088) Renewable Energy Sources in the Domestic Sector. One of the aims of this project is to enhance consumer awareness with respect to energy efficiency and renewable energy sources.
The project responds to the following industry demands:

- The need for an organised database of locally available green building products easily accessed and with the possibility of technical comparison.
- Easily accessible information about best use and correct application of different products and technologies within the local context.
- Studies providing technical information about the performance of green building products under local conditions.
- Increase public awareness about the range of green building products available.
- Encourage more competition in the green building product market.
- Access to advice addressing all aspects of the implementation of green building technologies in both new build and retrofit markets.

The tool is based on an online platform www.ecobuild.gov.mt, providing information on Green Building Technology and technical product information for building professionals, technicians and developers. In line with the list of industry demands presented, the tool provides for the following:

- Database of locally accessible and available green building products and technologies.
- Technical product information for building professionals, technicians and developers.
- Technical advice intended to support cost-effective choice of products.
- Non-technical advice for homeowners investing in energy-saving measures.
- Information on skills and competence in Green Building Technology

EcoBuild targets all stakeholders in the Green Building Products industry, architects and engineers, developers, contractors and homeowners. The immediate target results for EcoBuild include a database of products and collection of green building design data, increased exposure for all approved green building products and direct comparison between products from different suppliers. The tool promotes increased awareness and education in Green Building Technologies. Long term benefits include improvement in building energy efficiency, increase in comfort in buildings through the implementation of green technology and a better understanding of green building products and services. Furthermore the tool supports the Maltese Government’s EU 2020 targets.
3. DISCUSSION

The products were categorized into respective sections, reflecting various components of the building, and the building envelope. The products included in the database accessed on the website therefore fall under respective categories. This permits easy access in the website to find information about a possible product. The categories are also reflected in the building diagram presented for ease of access.

A Products Review Board was set up in order to establish objective criteria for acceptance of products and to determine the technical data required for different products included in the database. As a first step, basic criteria for the inclusion of products were established. In the first stage in setting up the tool, the basic criteria include the following:

- All products must comply with the relevant European Standards and product characteristics specified in order to be eligible for registration. The CE marking of construction products is mandatory for construction products in accordance to the Construction Product Regulations and enables the free movement of products within the European market.
- Eco Label of Construction Products and Environment Product Declaration (EPD)

The above approach supports also initiatives towards education in sustainable construction and Green Building Technology.

The long term objective is to introduce criteria which reflect the environmental impact of products and therefore based on a quantitative assessment for Green Products. This allows for clear comparison of products based on their environmental performance leading to product selection on clearly defined environmental criteria. This requires information on the environmental performance of construction products.

4. BRE GREEN GUIDE TO SPECIFICATIONS

The BRE’s Green Guide to Specification series in 1996 aimed to provide a simple ‘green guide’ to the environmental impacts of building materials which was easy-to-use and soundly based on numerical data. The Green Guide forms part of BREEAM (BRE Environmental Assessment Method) which is an accredited environmental rating scheme for buildings. The Green Guide contains more than 1500 specifications used in various types of building. Updates have been implemented since information on the relative environmental performance of materials and components changes from one period to another, reflecting both changes in manufacturing practices, the use of materials in buildings, and additional environmental knowledge.

The Green Guide allows for the examination of the relative environmental impacts of the construction materials commonly used in six different generic types building including Commercial buildings, Educational, Healthcare, Retail, Domestic and Industrial Buildings. Environmental rankings are based on Life Cycle Assessments (LCA), using BRE’s Environmental Profiles Methodology 2008. Materials and components are arranged on an elemental basis so that designers and specifiers can compare and select from comparable systems or materials as they compile their specification. The elements covered are: external walls, internal walls and partitions, roofs, ground floors, upper floors, windows, insulation, landscaping, floor finishes.
Across these building element categories the Guide provides an extensive, but not complete catalogue of building specifications covering most common building materials. This data is set out as an A+ to E ranking system, where A+ represents the best environmental performance and E the worst environmental performance. BRE has provided a summary environmental rating, The Green Guide rating, which is a measure of overall environmental impacts covering the following issues:

- Climate change
- Water extraction
- Mineral resource extraction
- Stratospheric ozone depletion
- Human toxicity
- Ecotoxicity to Freshwater
- Nuclear waste (higher level)
- Ecotoxicity to land
- Waste disposal
- Fossil fuel depletion
- Eutrophication
- Photochemical ozone creation
- Acidification

By evaluating the performance of materials and building systems against these specific environmental impacts, which have also been ranked on an A+ to E basis, it is possible for the specifier to select specifications on the basis of personal or organisational preferences or priorities. Furthermore the specifier can take decisions based on the performance of a material against a particular environmental impact.

5. CONCLUSION

The EcoBuild online tool presents data for locally available product data intended to promote Green Building Technology. It can be considered as a first database in Malta for available Green Building Products. EcoBuild is also intended as an educational tool, including information on products and technology. It presents project Case Studies promoting best practice, guidelines for new buildings and for the retrofit of existing buildings and Green Building recommendations. The website presents information on incentives which are available and intended to promote green building technology. In the first stage basic criteria for the inclusion of products were established reflecting product performance towards Green Buildings. The long term objective is to introduce criteria which reflect the environmental impact of products, based on a quantitative assessment for Green Products, to allow for comparison of products based on their environmental performance.

BIBLIOGRAPHY